Bridge Load Rating

Prepared for

Maine Department of Transportation

Bridge No. 5969

HAMPDEN

I-95 NORTHBOUND

OVER

B&A RAILROAD & EMERSON MILL ROAD

Date of Inspection: July 19, 2011

Date of Rating: August 26, 2013

Prepared By: Theresa D. McAuliffe, P.E.
Checked By: Keith S. Wood, P.E.

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STATE OF MAINE
THERESA
D. MCAULIFFE
No. 13406
LICENSED
PROFESSIONAL ENGINEER

8/26/13

Bridge No.:	5969	Owner:	MaineDOT
Town / City:	Hampden	Maintainer:	MaineDOT
Route Carried:	I-95 NB	Year Built:	1962
Crosses:	B&A RR & Emerson Mill Rd.	Year(s) Rebuilt / Rehab:	N/A

SUMMARY OF BRIDGE RATING

VEHICLE TYPE		RF	RT (TONS)	POSTING LOAD (TONS)
HL-93	INVENTORY	***1.57***	56.52	
	OPERATING	***2.04***	73.44	
HL-93 Modified	INVENTORY			
	OPERATING			
CONFIGURATION 1				
CONFIGURATION 2				
CONFIGURATION 3				
CONFIGURATION 4				
CONFIGURATION 5				
CONFIGURATION 6				
CONFIGURATION 7				
CONFIGURATION 8				

Group 1 Posting Analysis (Configuration 1)
Governing Posting: N/A
Governing Load Model: N/A

Group 2 Posting Analysis (Configuration 2 - 5)
Governing Posting: N/A
Governing Load Model: N/A

Group 3 Posting Analysis (Configuration 6 - 8)
Governing Posting: N/A
Governing Load Model: N/A

LRFR Evaluation Factors:

Live Load Distribution Factor:	0.698
Live Load DF Routine Commercial:	0.698
Live Load DF Special Hauling:	0.698
Impact Factor:	1.33
Governing Condition Factor, Φ_c:	1.00
System Factor, Φ_s:	1.00
ADTT (one-way):	1341

***Please check all the boxes that apply*:**

- ☐ Bridge Load Rating Governed By Substructure Rating
- ☐ Connections Control Load Rating
- ☐ Exterior Girder Controls Load Rating
- ☒ As-Built Load Rating
- ☒ As-Inspected Load Rating
- ☐ One Lane Loaded
- ☐ Advanced Analysis Used
- ☒ Actual Measurements Taken
- ☒ Finite Fatigue Life 95 years

BREAKDOWN OF BRIDGE RATING

Town / City: Hampden
Bridge No.: 5969

Route Carried: I-95 NB
Crosses: B&A RR & Emerson Mill Rd.

LOAD RATING POINTS OF INTEREST

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv. 72.0k	Oper 72.0k	Inv. 90.0k	Oper 90.0k	1 100.0k	2 94.0k	3 88.0k	4 88.0k	5 88.0k	6 75.9k	7 59.0k	8 37.4k
Span 1 Interior Beam Positive Moment 0.5L of Span	1.58	2.05										
Span 1 Interior Beam Shear 0.0L of Span	1.86	2.41										
Span 1 Interior Beam Moment at Cover Plate Transition (9.38')	2.05	2.65										
Span 1 Exterior Beam Positive Moment 0.5L of Span	1.71	2.21										
Span 1 Exterior Beam Shear 0.0L of Span	2.97	3.86										
Span 1 Exterior Beam Moment at Cover Plate Transition (10.17')	2.17	2.81										
Span 2 Interior S19 Positive Moment 0.5L of Span	***1.57***	***2.04***										
Span 2 Interior S17 Shear 0.0L of Span	1.91	2.47										
Span 2 Interior S18 Moment at Cover Plate Transition (10.63')	1.84	2.39										

BREAKDOWN OF BRIDGE RATING

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Bridge No.: 5969

Route Carried: I-95 NB
Crosses: B&A RR & Emerson Mill Rd.

LOAD RATING POINTS OF INTEREST

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv. 72.0k	Oper 72.0k	Inv. 90.0k	Oper 90.0k	1 100.0k	2 94.0k	3 88.0k	4 88.0k	5 88.0k	6 75.9k	7 59.0k	8 37.4k
Span 2 Exterior S20 Positive Moment 0.5L of Span	1.60	2.07										
Span 2 Exterior S20 Shear 0.0L of Span	3.04	3.95										
Span 2 Exterior S15 Moment at Cover Plate Transition (13.33')	1.91	2.48										
Span 3 Interior Beam Positive Moment 0.5L of Span	1.67	2.16										
Span 3 Interior Beam Shear 0.0L of Span	2.08	2.70										
Span 3 Interior Beam Moment at Cover Plate Transition (10.67')	1.94	2.51										
Span 3 Exterior Beam Positive Moment 0.5L of Span	1.73	2.24										
Span 3 Exterior Beam Shear 0.0L of Span	3.31	4.29										
Span 3 Exterior Beam Moment at Cover Plate Transition (13.71')	1.92	2.49										

BREAKDOWN OF BRIDGE RATING

Town / City: Hampden
Bridge No.: 5969

Route Carried: I-95 NB
Crosses: B&A RR & Emerson Mill Rd.

LOAD RATING POINTS OF INTEREST

Span 4 Interior Beam Positive Moment 0.5L of Span	1.62	2.10										
Span 4 Interior Beam Shear 0.0L of Span	2.02	2.61										
Span 4 Interior Beam Moment at Cover Plate Transition (9.56')	2.01	2.61										
Span 4 Exterior Beam Positive Moment 0.5L of Span	1.75	2.27										
Span 4 Exterior Beam Shear 0.0L of Span	3.21	4.17										
Span 4 Exterior Beam Moment at Cover Plate Transition (11.33')	2.07	2.69										
CONTROLLING RATING FACTORS	***1.57***	***2.04***										

Note: The serviceability rating factors for moment are lower than the factors for strength.

- Span 1 Cover Plate Interior: 1.70 (Inv.), 2.21 (Oper.)
- Span 1 Cover Plate Exterior: 2.01 (Inv.), 2.61 (Oper.)
- Span 2 Moment S19 Interior: 1.52 (Inv.), 1.97 (Oper.)
- Span 2 Cover Plate S18 Interior: 1.52 (Inv.), 1.98 (Oper.)
- Span 2 Cover Plate S15 Exterior: 1.64 (Inv.), 2.14 (Oper.)
- Span 3 Moment Interior: 1.45 (Inv.), 1.89 (Oper.)
- Span 3 Cover Plate Interior: 1.59 (Inv.), 2.06 (Oper.)
- Span 3 Moment Exterior: 1.61 (Inv.), 2.10 (Oper.)
- Span 3 Cover Plate Exterior: 1.67 (Inv.), 2.17 (Oper.)
- Span 4 Moment Interior: 1.44 (Inv.), 1.87 (Oper.)
- Span 4 Cover Plate Interior: 1.65 (Inv.), 2.14 (Oper.)
- Span 4 Moment Exterior: 1.72 (Inv.), 2.23 (Oper.)
- Span 4 Cover Plate Exterior: 1.80 (Inv.), 2.35 (Oper.)

DESCRIPTION OF BRIDGE

Bridge Number:	5969
Owner:	MaineDOT
Maintained By:	MaineDOT
Location:	Hampden
Route Carried:	I-95 NB
Feature Intersected:	B&A RR & Emerson Mill Road
Latest NBI Inspection Date:	July 19, 2011
Field Verification Date:	March 27, 2013
Date of Construction:	1962
Bridge Type:	Four simple span rolled steel beam
Material Properties:	Steel: ASTM A-7 (f_y = 33 ksi) Concrete: Unknown, based on year of construction and MBE Table 6A.5.2.1-1 (f'_c = 3.0 ksi)
Original Design Loading:	H20-S16-44 as modified for interstate
Date(s) of Rebuild/Rehab:	None
Description of Rebuild/Rehab:	N/A
Posting:	Open, no restrictions
Superstructure:	Four spans: Each span has six steel beams (W33x118 or W33x130) with bottom flange cover plates and a composite cast in place concrete deck.
Substructure:	Cast in place concrete abutments on piles and piers on footings.
Bearings:	Steel plates
Bridge Spans:	Total Span: 219’-11 $^{11}/_{16}$” Bearing to Bearing: 61’-0”; 55’-7” (varies); 48’-0”; 51’-0”
Bridge Skew:	32°-48’-00”: Abutment No. 1 & Pier No. 1 18°-53’-00”: Pier No. 2, Pier No. 3 & Abutment No. 2
Bridge Width:	43’-2” out-to-out
Roadway Width:	40’-0” curb-to-curb
Roadway Surface:	Concrete
Curbs:	Granite
Sidewalk/Walkway/Median:	None
Utilities:	None
Bridge Railing:	Two bar steel railing with concrete parapet
Approach Railing:	Bridge Transition Type I / Type II (leading/trailing ends) with guardrail
Wearing Surface Condition:	Satisfactory (Field Observation)
Bridge Railing Condition:	Satisfactory (Field Observation)
Deck Condition:	Satisfactory (SI&A)
Beam Condition:	Good (SI&A)
Bearing Condition:	Satisfactory (Field Observation)
Abutment Condition:	Satisfactory (SI&A)
Pier Condition:	Satisfactory (SI&A)

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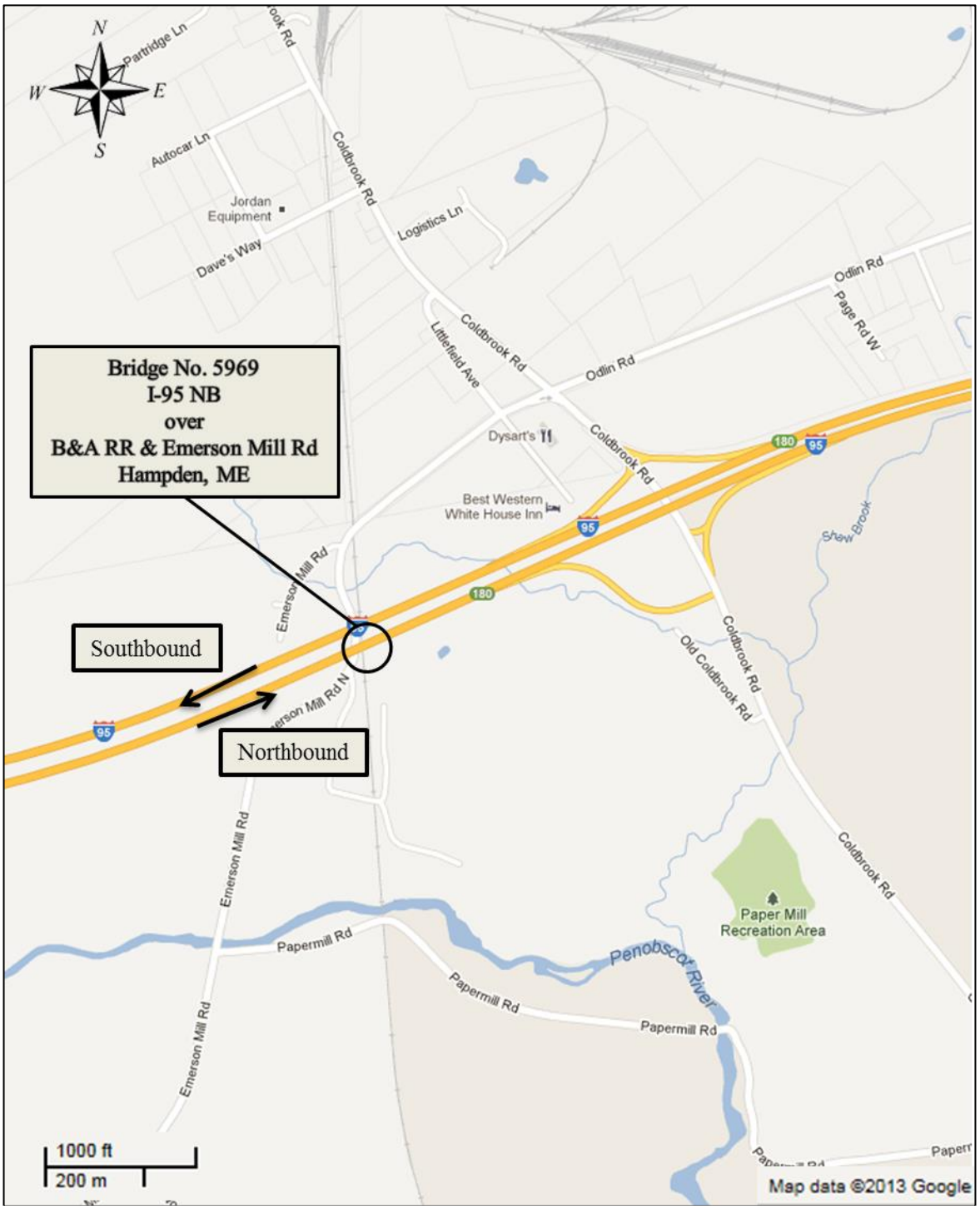
APPENDIX C – SECTION LOSS SKETCHES	

LOCATION MAP



EXISTING BRIDGE CONDITION

The following information was obtained from the SI&A and observations at the site. The SI&A lists the bridge superstructure in good condition. The substructure and deck are listed in satisfactory condition. This information provided the basis for the bridge load rating summarized in this report.

For the purposes of this report, please refer to Figure 1 for the terminology used.

Site Observations

The overall geometry of Spans 1, 2, 3, and 4 was measured. Dimensions were recorded to verify the 1961 bridge construction plans. The bridge substructure is in satisfactory condition with map cracking, delamination, and spalling with exposed reinforcing steel observed at the piers. The bridge deck is in satisfactory condition. The bridge wearing surface is concrete, which replaced the bituminous concrete wearing surface shown on the 1961 bridge construction plans. The aluminum bridge railing with concrete parapet and granite curb appears to be original, as shown on the 1961 bridge construction plans.

The bridge superstructure is in good condition. The beams were noticeably cambered. The superstructure has minor rust and paint failure on the primary members; mostly on the exterior beams. The bearings and beam ends have corroded at the abutment and piers, which appears to be the result of joint failure. This does not impact the member capacity for purposes of this load rating.

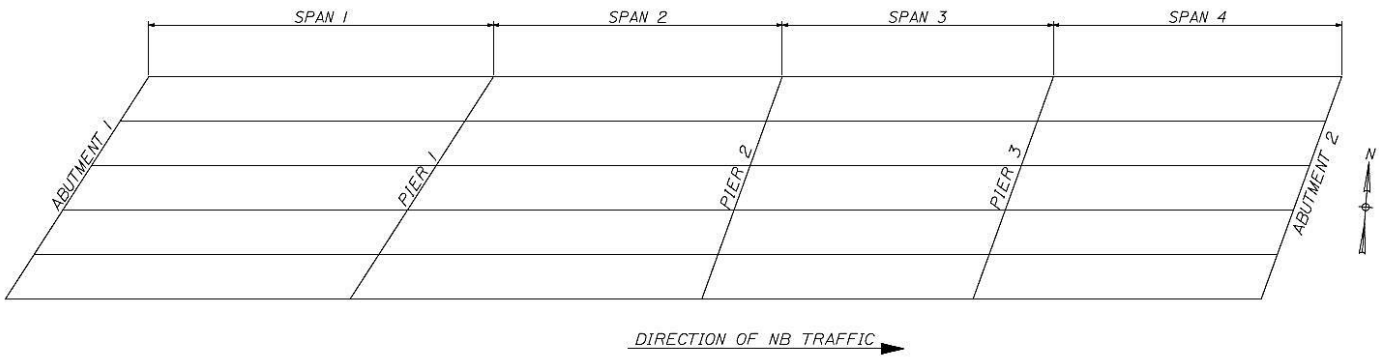


Figure 1: Bridge No. 5969 Terminology

ASSUMPTIONS, CRITERIA, AND REFERENCES

Assumptions and Criteria

The load rating is in accordance with MBE Section 6A.1.5. Where the MBE is silent, the analysis is per the AASHTO LRFD Manual.

Scope:

Rate both interior and exterior beams for all four spans; shear at the ends, moment at midspan, and at cover plate transitions. All four spans are considered to be unique. Since the beams in Span 2 are each a different length with different cover plates, evaluate all 6 members to determine the controlling members.

Structural Analysis:

- Deterioration: None. A reduced member capacity was not considered.
- Composite: Yes. Shear studs shown on the plans.
- Unbraced Length: Fully braced. Composite construction and the top flange is embedded in the concrete.
- Haunch / Blocking: The rolled members are over cambered as observed during the site visit. The haunch dimension used for analysis was the nominal minus the camber and the over camber tolerance. However, for calculating dead load, the nominal haunch was used.
- Stiffeners: None.
- Beam Section Properties: The rolled member properties from the year of construction, 1962, (ASD 6th Edition) are compared to the default rolled sections (LRFD3) in the Merlin Dash analysis. Since strong-axis values do not vary by more than 1%, the default LRFD3 member section properties are used in the analysis.
- Span 2: Since Span 2 has a different skew at each support and each beam has a different cover plate, the Merlin model was created using live load distribution factors based on an average span and average section properties of all six beams. Then each beam is evaluated to determine the controlling members.

Dead Load:

- Factors: γ_{DC} = 1.25 per MBE 6A.2.2.1
  γ_{DW} = 1.50 per MBE 6A.2.2.1
- Railing: Calculated 260 plf.
- Deck: 6 ¾ inches thick based on the plans.
- Blocking: Applied as distributed load. Assumed constant thickness, utilizing the maximum thickness at the bearings.
- Wearing Surface: 3 ½ inches concrete based on field measured curb reveal.
- Curb: Granite per plans and confirmed in field.
- Fence: None
- Cross-Frames: C-Channel, applied as point loads

Dead Load Distribution:

Loads applied to the non-composite section were distributed to each beam based on the tributary width. Loads applied to the composite section were evenly distributed to each beam (i.e. total load divided by number of beams).

Live Load: (Rating Guide Section 3.3 & MBE 6A.2.3)

- Factor: The load factors are summarized below:

	Strength	Service
HL-93	1.75	1.30

- Dynamic Allowance: IM = 1.33 for all AASHTO and Legal Loads (Rating Guide Section 3.2)
- Loads: HL-93, Maine Legal Loads – See loading figures on pages 6 through 8.
- Number of Lanes: Two based on current striping.

Live Load Distribution:

Per MBE 6A.3.2, the live load distribution factors for moment and shear were computed in accordance with AASHTO *LRFD* Section 4.6.2.2. The live load distribution factors for the interior beams were determined in accordance with AASHTO LRFD Table 4.6.2.2.2b-1 for concrete decks.

The live load distribution factors for the exterior beams were determined in accordance with AASHTO LRFD Section 4.6.2.2.2d. For the rigid body rotation and lever rule calculations, the position of the vehicles and the number of lanes are based on the existing striped lanes per Rating Guide 3.3.1.7. The first wheel is assumed to be on the white stripe.

The live load distribution factors corrected for skew are summarized below:

Element	Moment	Shear
Interior Stringers – Span 1	0.654	0.899
Exterior Stringers – Span 1	0.526	0.575
Interior Stringers – Span 2	0.698	0.878
Exterior Stringers – Span 2	0.562	0.561
Interior Stringers – Span 3	0.710	0.860
Exterior Stringers – Span 3	0.571	0.550
Interior Stringers – Span 4	0.705	0.860
Exterior Stringers – Span 4	0.567	0.550

Condition Factor: (*Rating Guide* Section 3.5.1)

A condition factor of $\Phi_c = 1.00$ was used, based on MBE Table 6A4.2.3-1 and the visual inspection and the condition assessments of good.

System Factor: (*Rating Guide* Section 3.5.2)

A system factor of $\Phi_s = 1.00$ was used, based on a "Redundant Stringer" system as indicated in MBE Table 6A4.2.4-1.

Material Properties:

Steel: The 1961 bridge construction plans specify the beams and cover plates as ASTM A7 (f_y = 33 ksi). Since the top flange of the beam is embedded in the concrete deck, it is considered to be continuously braced. Lateral torsional buckling will not control the beam capacity.

Concrete: Deck is assumed to be normal weight concrete with a compressive strength f'c = 3 ksi based on the year of construction and MBE Table 6A.5.2.1-1.

Serviceability and Fatigue: (*Rating Guide* Section 3.6.3)

Although the serviceability ratings were low, no serviceability related issues were noted. Fatigue live load distribution factors for one lane loaded without multiple presence were calculated in accordance with AASHTO LRFD 3.6.1.4.3b utilizing the previously calculated HL-93 live load distribution factors. Category E and E' weld details are present for the cover plates. The evaluation finite fatigue life analysis for HL-93 was performed assuming 4% growth rate in truck traffic per MBE 7.2.5. The interior beam of Span 2 (S18) controls the finite fatigue life.

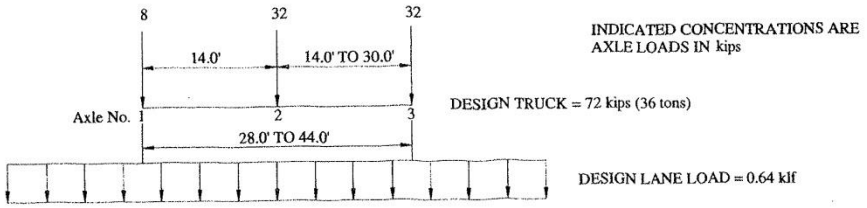
References

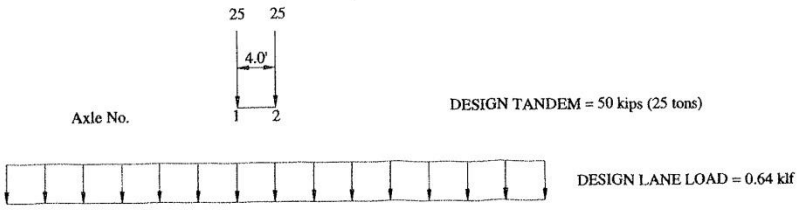
MBE	AASHTO Manual for Bridge Evaluation, 2nd Edition, with 2011 and 2013 Interims.
AASHTO LRFD	AASHTO *LRFD Bridge Design Specifications*, 5th Edition, 2010, with 2010 Interim Revisions.
BDG	MaineDOT *Bridge Design Guide*, 2003, with revisions through August 2008.
Rating Guide	MaineDOT *Draft Load Rating Guide*, November 2011.
SI&A	MaineDOT Structure Inventory and Appraisal Sheet: July 19, 2011.
AISC	AISC Manual of Steel Construction – AISC Shapes Database v14.0 Historic.xls, www.aisc.org
LRFD3	Manual of Steel Construction: Load and Resistance Factor Design, American Institute of Steel Construction, Inc., Third Edition, 2001.

LOADINGS USED FOR BRIDGE RATINGS

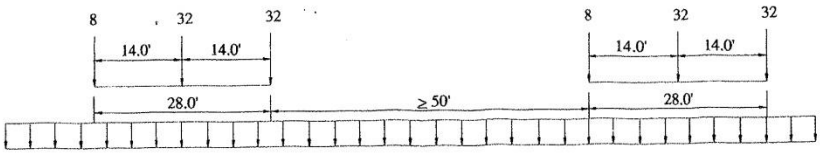
The following are the standard truck diagrams used in rating the structure:

APPENIDX C6A—LRFD DESIGN LIVE LOAD (HL-93) (LRFD DESIGN ARTICLE 3.6.1)





ADDITIONAL LOAD MODEL FOR NEGATIVE MOMENT AND INTERIOR REACTION
(REDUCE ALL LOADS TO 90%)

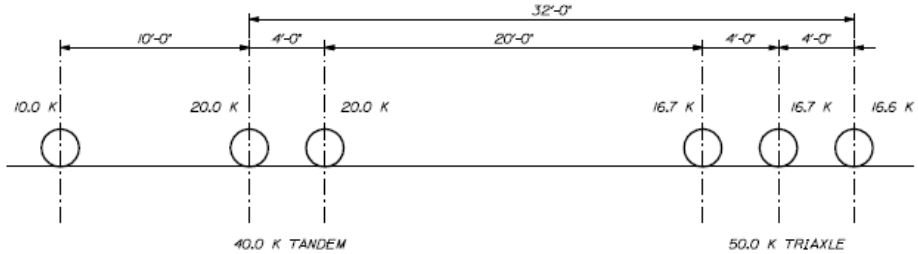


DESIGN LANE LOAD = 0.64 klf

Figure C6A-1—LRFD Design Live Load (HL-93)

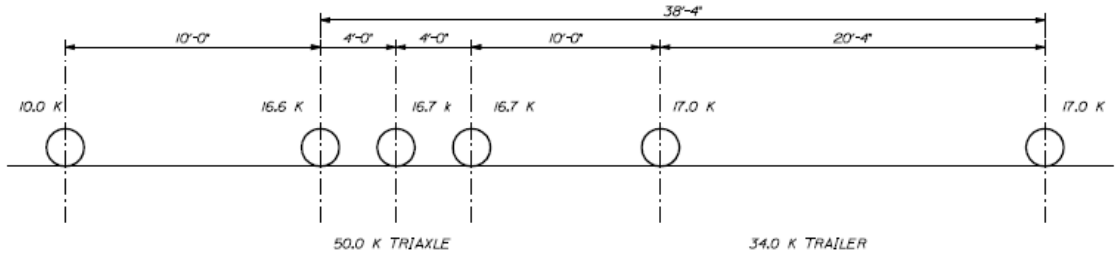
LOADINGS USED FOR BRIDGE RATINGS, CONT'D

Maine Legal Load Configurations:



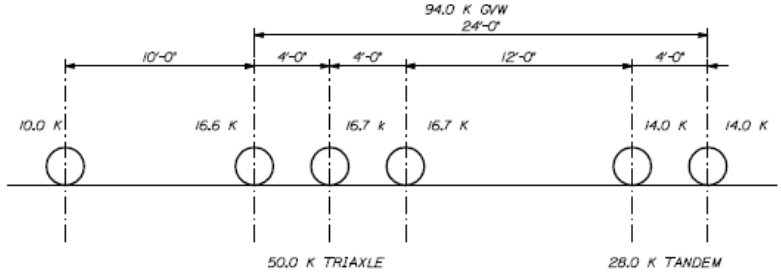
CONFIGURATON 1

SIX AXLE
3 AXLE TRACTOR
TRIAXLE SEMI-TRAILER
100.0 K GVW



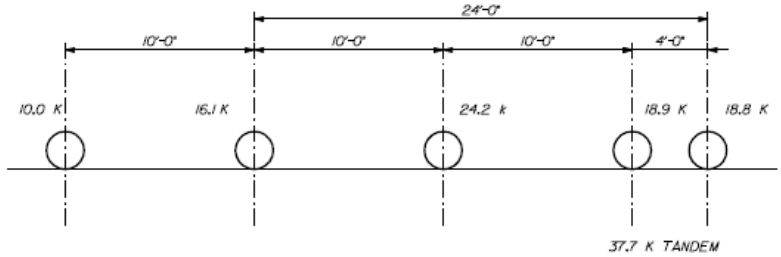
CONFIGURATON 2

SIX AXLE
TRIAXLE TRUCK
2 AXLE TRAILER
94.0 K GVW



CONFIGURATON 3

SIX AXLE
TRIAXLE TRUCK/TRACTOR
TANDEM SEMI-TRAILER
88.0 K GVW



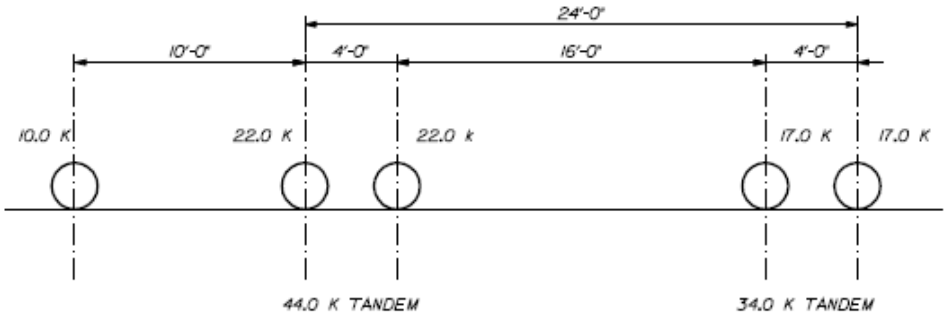
CONFIGURATON 4

FIVE AXLE
TWO AXLE TRUCK/TRACTOR
THREE AXLE TRAILER
88.0 K GVW

REVISION 03-07-11

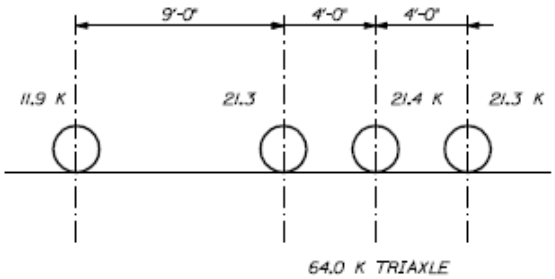
LOADINGS USED FOR BRIDGE RATINGS, CONT'D

Maine Legal Load Configurations:



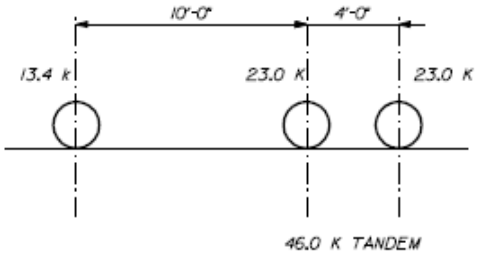
CONFIGURATON 5 FIVE AXLE TANDEM AXLE TRUCK OR TRACTOR SEMI-TRAILER

88.0 K GVW



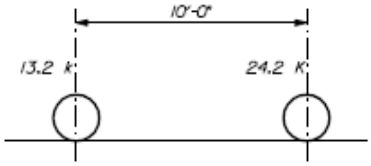
CONFIGURATON 6 FOUR AXLE TRUCK

75.9 K GVW



CONFIGURATON 7 THREE AXLE TRUCK

59.0 K GVW



CONFIGURATON 8 TWO AXLE TRUCK

37.4 K GVW

PHOTOGRAPHS

Photo No. 1

Location:
Span 1

Description:
South Side Elevation

Note: Paint failure on exterior beam.

Photo No. 2

Location:
Span 2

Description:
South Side Elevation

Note: Paint failure on exterior beam.

Photo No. 3

Location:
Span 3

Description:
South Side Elevation

Note: Paint failure on exterior beam.

Photo No. 4

Location:
Span 4

Description:
South Side Elevation

Note: Paint failure on exterior beam, particularly over Pier No. 3 at the joint.

Photo No. 5

Location:
Span 2

Description:
Pier No. 1 Elevation with Abutment No. 1 in Background

Photo No. 6

Location:
Span 2

Description:
Pier No. 2 Elevation with Abutment No. 2 and Pier No. 3 in Background

Note: Spalling and exposed reinforcing steel near the pier cap and column interface.

Photo No. 7

Location:
Span 3

Description:
Pier No. 3 Elevation with Abutment No. 2 in Background

Photo No. 8

Location:
Span 2

Description:
Typical Beam Bay

Photo No. 9

Location:
Underside of Bridge

Description:
Typical Interior Diaphragm

Photo No. 10

Location:
Exterior Beam

Description:
Typical Paint Failure on Exterior Beam

Photo No. 11

Location:
B&A Railroad

Description:
Looking South along Railroad at Bridge No. 1430

Photo No. 12

Location:
Emerson Mill Road

Description:
Looking South along Emerson Mill Road at Bridge No. 1430

Photo No. 13

Location:
Abutment No. 1

Description:
Bridge Rail with Concrete Parapet and Granite Curb

Photo No. 14

Location:
Abutment No. 1

Description:
Bridge Joint at Abutment No. 1

Photo No. 15

Location:
Abutment No. 1

Description:
Typical Expansion Bearing

Photo No. 16

Location:
Span 1

Description:
Typical Overhang

APPENDIX A

BACKGROUND INFORMATION

(See CD)

- Existing Inspection Report
- Material Tests – N/A
- Construction Plans
- Shop Drawings
- Site Visit

Structure Inventory and Appraisal Sheet

Bridge Key: 5969 Agency ID: 5969 SR: 94.1 SD/FO: ND

IDENTIFICATION

State 1: 23 Maine
Struc Num 8: 5969
Facility Carried 7: I-95 NORTHBOUND
Location 9: MILE 179.2/4.5 MILE E TL
Rte.(On/Under)5A: Route On Structure
Rte. Signing Prefix 5B: 3 State Hwy
Level of Service 5C: 1 Mainline
Rte. Number 5D: 00095
Directional Suffix 5E: 1 North
% Responsibility : 0
SHD District 2: 04 Eastern
County Code 3: 019 Penobscot
Place Code 4: 19280 Hampden
Kilometer Post 11: 286.6 km
Feature Intersected 6: B&ARR,EMERSON MILL RD
Latitude 16: 44d 46' 13"
Longitude 17: 068d 52' 09"
Border Bridge Code 98: Not Applicable (P)
Border Bridge Number 99: n/a

STRUCTURE TYPE AND MATERIALS

Number of Approach Spans 46: 0
Number of Spans Main Unit 45: 4
Main Span Material/Design 43A/B:
3 Steel
02 Stringer/Girder
Deck Type 107: 1 Concrete-Cast-in-Place
Wearing Surface 108A: 1 Monolithic Concrete
Membrane 108B: 0 None
Deck Protection 108C: None

AGE AND SERVICE

Year Built 27: 1962
Year Reconstructed 106: -4
Type of Service on 42A: 1 Highway
Type of Service under 42B: 4 Highway-railroad
Lanes on 28A: 2
Lanes Under 28B: 2
Detour Length 19: 6.6 km
ADT 29: 12,190
Truck ADT 109: 11 %
Year of ADT 30: 2011

GEOMETRIC DATA

Length Max Span 48: 15.85 m
Structure Length 49: 62.79 m
Curb/Sdwlk Width L 50A: 0.15 m
Curb/Sidewalk Width R 50B: 0.15 m
Width Curb to Curb 51: 12.19 m
Width Out to Out 52: 13.11 m
Approach Roadway Width 32:(w/ shoulders) 12.19 m
Median 33: 0 No median
Deck Area: 822.92 m²
Skew 34: 19.00 °
Structure Flared 35: 0 No flare
Vertical Clearance 10: 99.99 m
Horiz. Clearance 47: 12.19 m
Minimum Vertical Clearance Over Bridge 53: 99.90 m
Minimum Vertical Underclearance Reference 54A: H Hwy beneath struct
Minimum Vertical Underclearance 54B: 05.01 m
Minimum Lateral Underclearance Reference R 55A: H Hwy beneath struct
Minimum Lateral Underclearance R 55: 03.40 m
Minimum Lateral Underclearance L 56: 08.30 m

INSPECTION

Frequency 91: 24 months
Inspection Date 90: 7/19/2011
Next Inspection: 07/19/2013
FC Frequency 92A: NA
FC Inspection Date 93A: NA
Next FC Inspection: NA
UW Frequency 92B: NA
UW Inspection Date 93B: NA
Next UW Inspection: NA
SI Frequency 92C: NA
SI Date 93C: NA
Next SI: NA
Element Frequency: 24 months
Element Inspection Date: 07/19/2011
Next Elem. Insp. Due: 07/19/2013

CLASSIFICATION

Defense Highway 100: 1 On Interstate STRAHNET
Parallel Structure 101: Right of || bridge
Direction of Traffic 102: 1 1-way traffic
Temporary Structure 103: Not Applicable (P)
Highway System 104: 1 On the NHS
NBIS Length 112: Long Enough
Toll Facility 20: 3 On free road
Functional Class 26: 01 Rural Interstate
Defense Hwy 110:: 1 On Interstate STRAHNET
Historical Significance 37: 4 Hist sign not determin
Owner 22: **01** State Highway Agency
Custodian 21: **01** State Highway Agency

CONDITION

Deck 58: 6 Satisfactory
Super 59: 7 Good
Sub 60: 6 Satisfactory
Culvert 62: N N/A (NBI)
Channel/Channel Protection 61: N N/A (NBI)

LOAD RATING AND POSTING

Inventory Rating Method 65: 2 AS Allowable Stress
Operating Rating Method 63:2 AS Allowable Stress
Inventory Rating 66: MS17.7
Operating Rating 64: MS30.2
Design Load 31: 6 MS18(HS20)+mod
Posting 70: 5 At/Above Legal Loads
Posting status 41: A Open, no restriction

APPRAISAL

Bridge Rail 36A: 0 Substandard
Approach Rail 36C: 1 Meets Standards
Transition 36B: 1 Meets Standards
Approach Rail Ends 36D: 1 Meets Standards
Str. Evaluation 67: 6 Equal Min Criteria
Deck Geometry 68: 7 Above Min Criteria
Underclearance, Vertical and Horizontal 69: 7 Above Minimum
Waterway Adequacy 71: N Not applicable
Approach Alignment 72: 7 Above Min Criteria
Scour Critical 113: N Not Over Waterway

PROPOSED IMPROVEMENTS

Bridge Cost 94: NA
Type of Work 75: Unknown (P)
Roadway Cost 95: Unknown
Length of Improvement 76:
Total Cost 96: Unknown
Future ADT 114: 17,066
Year of Cost Estimate 97: Unknown
Year of Future ADT 115: 2031

NAVIGATION DATA

Navigation Control 38: N NA-no waterway
Vertical Clearance 39: 0.00 m
Horizontal Clearance 40: 0.00 m
Pier Protection 111: Not Applicable (P)
Lift Bridge Vertical Clearance 116: 0.00 m

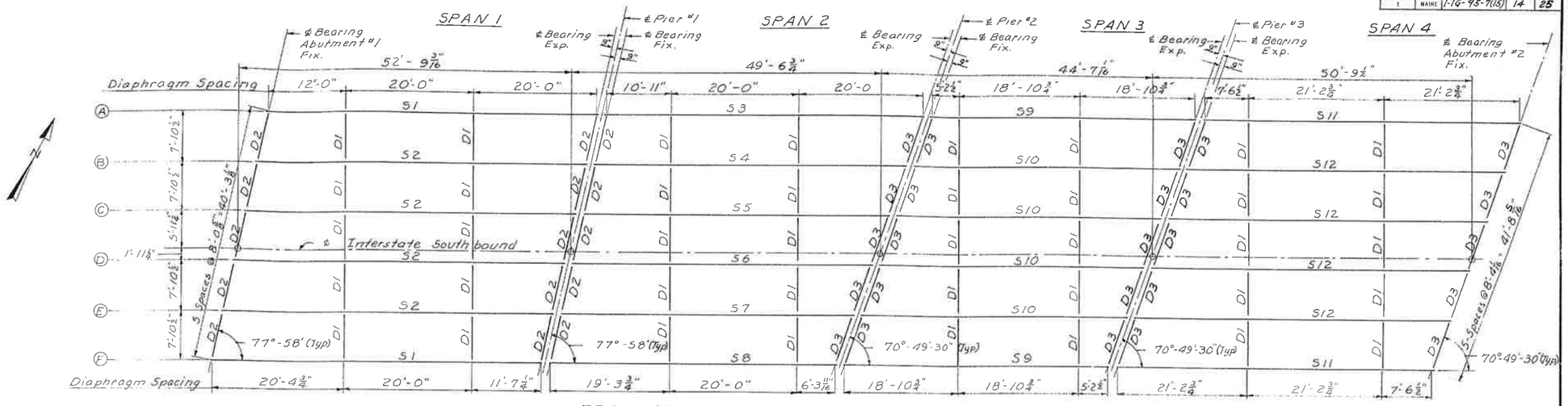
ELEMENT CONDITION STATE DATA

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	22/2	P Conc Deck/Rigid Ov	sq.m.	823	0 %	0	100 %	823	0 %	0	0 %	0	0 %	0
1	107/2	Paint Stl Opn Girder	m.	377	77 %	290	10 %	38	7 %	26	5 %	19	1 %	4
1	205/2	R/Conc Column	ea.	12	50 %	6	40 %	5	10 %	1	0 %	0	0 %	0
1	215/2	R/Conc Abutment	m.	26	30 %	8	55 %	14	15 %	4	0 %	0	0 %	0
1	218/2	Undefined Wall Elem.	m.	8	50 %	4	30 %	2	20 %	2	0 %	0	0 %	0

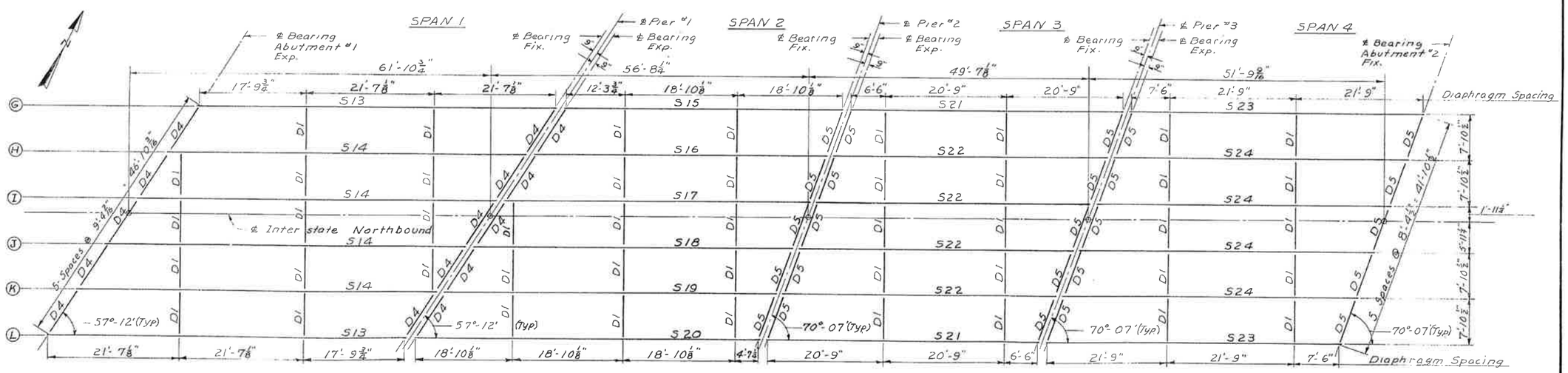
Structure Inventory and Appraisal Sheet

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	234/2	R/Conc Cap	m.	39	50 %	20	35 %	14	15 %	6	0 %	0	0 %	0
1	302/2	Compressn Joint Seal	m.	66	80 %	52	20 %	13	0 %	0	0 %	0	0 %	0
1	311/2	Moveable Bearing	ea.	24	40 %	10	50 %	12	10 %	2	0 %	0	0 %	0
1	313/2	Fixed Bearing	ea.	24	40 %	10	50 %	12	10 %	2	0 %	0	0 %	0
1	333/2	Other Bridge Railing	m.	126	50 %	63	50 %	63	0 %	0	0 %	0	0 %	0
1	385/2	Wear.Surf. - Rigid	sq.m.	766	100 %	766	0 %	0	0 %	0	0 %	0	0 %	0
1	388/2	Paint	sq.m.	1,767	80 %	1,414	10 %	177	8 %	141	2 %	35	0 %	0
1	389/2	Reinfor conc dk/slab	sq.m.	823	33 %	272	33 %	272	34 %	280	0 %	0	0 %	0

D. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-IG-95-7(15)	14	25



FRAMING PLAN SOUTHBOUND



FRAMING PLAN NORTHBOUND

DESIGN- C.D.H. DETAIL- C.A.M.
TRACE- G.E.A.
CHECK- A.L.B.
BRIDGE NO.
SURVEY-
PLOT-

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

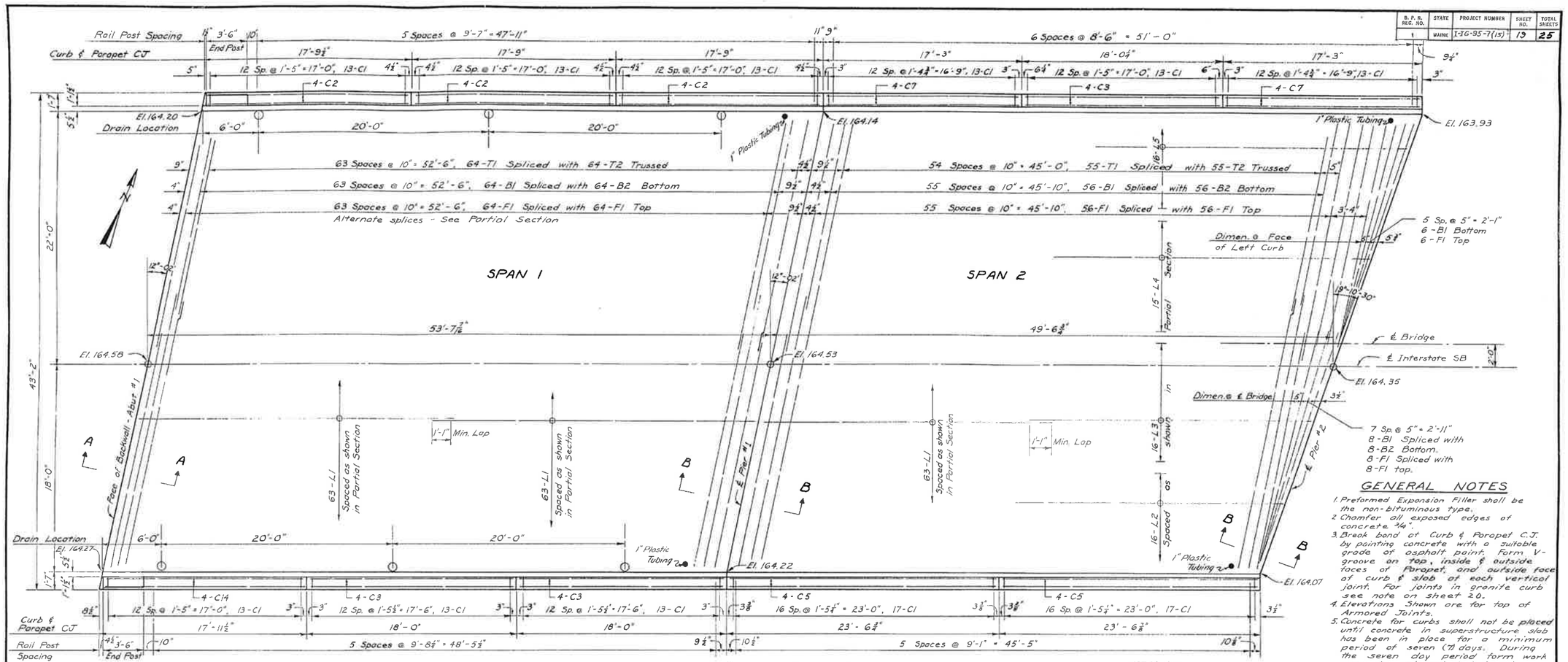
INTERSTATE 95
OVER
EMERSON MILL ROAD & BANGOR & AROOSTOOK RAILROAD
IN THE TOWN OF
HAMPDEN
PENOBSCOT COUNTY

STRUCTURAL STEEL FRAMING PLANS

SHEET 14 OF 25 AUGUSTA, MAINE SEPT., 1961

0 1 2 3 4 5 INCHES

B. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-IG-95-7(13)	19	25



GENERAL NOTES

1. Preformed Expansion Filler shall be the non-bituminous type.
2. Chamfer all exposed edges of concrete 3/4".
3. Break bond at Curb & Parapet C.J. by painting concrete with a suitable grade of asphalt paint. Form V-groove on top, inside & outside faces of Parapet, and outside face of curb & slab at each vertical joint. For joints in granite curb see note on sheet 20.
4. Elevations Shown are for top of Armored Joints.
5. Concrete for curbs shall not be placed until concrete in superstructure slab has been in place for a minimum period of seven (7) days. During the seven day period form work may be placed but hand equipment only shall be allowed on the slab.
6. At the low points in slabs, exact location to be determined by the Engineer, place 1" ø plastic tubing through slab for drainage. Do not cover with waterproofing. Payment to be considered as incidental to contract items.

DESIGN- C.D.H.	BRIDGE NO.
TRACE- J.W.M.	SURVEY-
CHECK- R.E.P.	PLOT-

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

INTERSTATE 95
OVER
EMERSON MILL ROAD & BANGOR & AROOSTOOK RAILROAD
IN THE TOWN OF
HAMPDEN
PENOBSCOT COUNTY

SUPERSTRUCTURE - SPANS 1 & 2 - SB

SHEET 19 OF 25 AUGUSTA, MAINE SEPT. 1961

APPENDIX B

LOAD RATING COMPUTATIONS

(See CD)

- Bridge Beam Dead Load Calculations
  - Span 1
  - Span 2
  - Span 3
  - Span 4

- Bridge Live Load Distribution Factor Calculation
  - Span 1
  - Span 2
  - Span 3
  - Span 4

- Bridge Moment of Inertia Calculation
  - Span 1
  - Span 2
  - Span 3
  - Span 4

- MathCad Calculations
  - Rigid Body Rotation Calculation
  - Span 2 Interior Beam Fatigue
  - Span 3 Interior Beam Fatigue

- Merlin-Dash Output:
  - HL-93 Rating Interior Beam – Span 1 Strength at Cover Plate
  - HL-93 Rating Interior Beam – Span 1 Fatigue & Mid M
  - HL-93 Rating Exterior Beam – Span 1 Strength
  - HL-93 Rating Exterior Beam – Span 1 Fatigue
  - HL-93 Rating Exterior Beam S15 – Span 2 Strength Mid M
  - HL-93 Rating Exterior Beam S15 – Span 2 Strength at CP
  - HL-93 Rating Exterior Beam S15 – Span 2 Fatigue
  - HL-93 Rating Interior Beam S16 – Span 2 Strength at CP
  - HL-93 Rating Interior Beam S16 – Span 2 Fatigue & Mid M
  - HL-93 Rating Interior Beam S17 – Span 2 Strength Mid M
  - HL-93 Rating Interior Beam S17 – Span 2 Fatigue and CP
  - HL-93 Rating Interior Beam S18 – Span 2 Strength
  - HL-93 Rating Interior Beam S18 – Span 2 Fatigue
  - HL-93 Rating Interior Beam S19 – Span 2
  - HL-93 Rating Exterior Beam S20 – Span 2 Fatigue & Mid M

- HL-93 Rating Exterior Beam S20 – Span 2 Strength at CP
- HL-93 Rating Interior Beam – Span 3 Strength
- HL-93 Rating Interior Beam – Span 3 Fatigue
- HL-93 Rating Exterior Beam – Span 3 Strength
- HL-93 Rating Exterior Beam – Span 3 Fatigue
- HL-93 Rating Interior Beam – Span 4 Strength
- HL-93 Rating Interior Beam – Span 4 Fatigue
- HL-93 Rating Exterior Beam – Span 4 Strength
- HL-93 Rating Exterior Beam – Span 4 Fatigue

- Hand Calculations

The Manual for Bridge Evaluation - Second Edition 2011 & 2013 Interims
AASHTO LRFD Bridge Design Specifications - Fifth Edition (2010) with Interims
MaineDOT Bridge Design Guide 2003 with Revisions through 2008
MaineDOT Draft Load Rating Guide, November 2011
Symmetrical Deck, Equal Girder Spacing

MERLIN Dead Loads - 5969 Span 1

References

- This sheet determines the applied loads for input into MERLIN girder runs. MERLIN internally calculates the dead load of the steel girders.

Geometry of Bridge:

length of bridge =	L_{span} =	61.00'	
curb-to-curb width of bridge =	b_{pvmt} =	40.00'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	43.17'	
number of girders =	N_g =	6	
girder spacing =	S_g =	7.875'	
maximum top flange width =	b_{f_max} =	11.51"	W33x130
maximum top flange thickness =	t_{f_max} =	0.86"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~6.75"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	6.75"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	7.50"	Based on Field Measurment
width of overhang =	$b_{overhang}$ =	1.896 ft	(assumes symmetrical)

Material Weights:

				LRFD
Unit weight of concrete =	w_c =	150 pcf	with reinforcing.	3.5.1 - Table 3.5.1-1
Unit weight of asphalt =	w_a =	140 pcf		3.5.1 - Table 3.5.1-2
Unit weight of granite =	w_g =	170 pcf		3.5.1 - Table 3.5.1-3
Unit weight of utility fluid =	w_{fluid} =	62.4 pcf		
Unit weight of gas =	w_{gas} =	8.0 pcf		

Girder Haunch:

Top flange within haunch? =		Yes	
thickness of haunch =	t_{haunch} =	3.61"	See hand calculation
width of haunch =	b_{haunch} =	19.50"	
weight of single girder haunch =	W_{haunch} =	**0.063 klf**	bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.875 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	5.833 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	**0.664 klf**	Average Thickness = 0.56'
deck weight for exterior girder =	W_{deck_ext} =	**0.492 klf**	Average Thickness = 0.56'

MERLIN Dead Loads - 5969 Span 1 (CONT'D)

References

Wearing Surface:

type of wearing surface =		Concrete	
thickness of wearing surface =	t_{pvmt} =	3.50"	Based on field measured curb reveal
thickness of waterproofing membrane =	t_{wm} =	0.000"	Inclusive to Wearing Surface Thickness
total weight of wearing surface =	W_{ws} =	1.750 klf	

Sidewalk:

Note: The dead load due to the sidewalk concrete is determined by measuring areas in CAD. The concrete area is determined by projecting the cross-slope of the roadway to the edge of deck and calculating the area above the theoretical line, less the volume of granite curb.

number of sidewalks =	N_{sw} =	0	(No sidewalk on this structure)
cross-sectional area of sidewalk =	A_{sw} =	0.00 ft²	(concrete portion)
cross-sectional area of granite curb =	A_g =	0.00 ft²	(granite curb only, if used)
weight of sidewalk =	W_{sw_conc} =	0.000 klf	

Curbs:

Note: The dead load due to curbs, is of the curbs that are not associated with sidewalks

number of curbs =	N_{curbs} =	2	
cross-sect area of curb and mortar bed =	A_{curbs} =	1.04 ft²	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	0.42 ft²	(granite curb only, if used)
weight of curbs =	W_{curbs} =	0.455 klf	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	0.00 Ft.	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	0.000 klf	

MERLIN Dead Loads - 5969 Span 1 (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$=	Other	0.260 klf	See hand calculation
$Rail_{Right}$=	Other	0.260 klf	See hand calculation
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.520 klf	

Snow / Pedestrian Fences:

left fence height =	NONE	Weight = 0 plf.	Verify Load
right fence height =	NONE	Weight = 0 plf.	Verify Load
total fence weight =	W_{fences} =	0.000 klf	

Diaphragms:

Note: This dead load calculation is for an individual diaphragm.

# of Attachment Plate 1 =	2	
Attachment Plate 1 Height =	29.00"	Field Measurement
Attachment Plate 1 Width =	6.00"	Field Measurement
Attachment Plate 1 Thickness =	0.375"	Field Measurement
Attachment Plate 1 Weight =	0.037 kip	
# of Diagonals =	0	
Diagonal Weight Per Foot =	0 plf	
Length of Diagonal member =	0.00'	
Diagonal Weight =	0.000 kip	
# of Horizontal Members =	1	
Horizontal Member Weight Per Foot =	33.9 plf	Per Plans
Length of Horizontal Member =	7.38'	
Horizontal Member Weight =	0.250 kip	
Interior Girder Diaphragm Weight =	0.29 kips	
Exterior Girder Diaphragm Weight =	0.14 kips	1/2 Interior Girder Diaphragm Weight

MERLIN Dead Loads - 5969 Span 1 (CONT'D)

References

Bridge Mounted Utilities:

Are utilities present on this bridge?	*No*
Are sewer main(s) present on this bridge?	*No*
Are water main(s) present on this bridge?	*No*
Are gas main(s) present on this bridge?	*No*
Are duct bank(s) present on this bridge?	*No*

Weight of sewer main line =		*0.0 lbs/ft*	
Inside diameter of sewer main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{sewer} =	**0.000 klf**	

Weight of water main line =		*0.0 lbs/ft*	
Inside diameter of water main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{water} =	**0.000 klf**	

Weight of gas line =		*0.0 lbs/ft*	
Inside diameter of gas line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{gas} * A_i)
Total line weight =	W_{gas} =	**0.000 klf**	

Weight of electrical duct bank =		*0.0 lbs/ft*
Hardware Allowance =		*0.0 lbs/ft*
Total duct bank weight =	$W_{electric}$ =	**0.000 klf**

Girder 1 N/C utility load =	0.000 klf
Girder 2 N/C utility load =	0.000 klf
Girder 3 N/C utility load =	0.000 klf
Girder 4 N/C utility load =	0.000 klf
Girder 5 N/C utility load =	0.000 klf
Girder 6 N/C utility load =	0.000 klf

MERLIN Dead Loads - 5969 Span 1 (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = **0.727 klf** = $W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.555 klf** = $W_{deck\_ext} + W_{haunch} + W_{soffit}$

MERLIN loads - Additional non-composite loads (DC2):

utility loads (interior girder) = **0.000 klf** = W_{util_int}

utility loads (exterior girder) = **0.000 klf** = W_{util_ext}

Diaphragm (interior girder) = **0.287 kip** per location

Diaphragm (exterior girder) = **0.144 kip** per location

MERLIN loads - Composite (DC2):

wearing surface = **0.292 klf** = $W_{ws} \div N_g$

sidewalk = **0.000 klf** = $W_{sw} \div N_g$

railing = **0.087 klf** = $W_{rails} \div N_g$

snow fence = **0.000 klf** = $W_{fence} \div N_g$

curb = **0.076 klf** = $W_{curbs} \div N_g$

Diaphragm Locations:

	Spacing		Distance from Left End		Check
Exterior	17.81		17.81		
	21.60		39.41		61.00
					21.59
Interior	11.51		11.51		
	21.59		33.10		61.00
	21.59		54.69		6.31

The Manual for Bridge Evaluation - Second Edition 2011 & 2013 Interims
AASHTO LRFD Bridge Design Specifications - Fifth Edition (2010) with Interims
MaineDOT Bridge Design Guide 2003 with Revisions through 2008
MaineDOT Draft Load Rating Guide, November 2011
Symmetrical Deck, Equal Girder Spacing

MERLIN Dead Loads - 5969 Span 2

References

- This sheet determines the applied loads for input into MERLIN girder runs.
  MERLIN internally calculates the dead load of the steel girders.

Geometry of Bridge:

length of bridge =	L_{span} =	55.57'	Average
curb-to-curb width of bridge =	b_{pvmt} =	40.00'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	43.17'	
number of girders =	N_g =	6	
girder spacing =	S_g =	7.875'	
maximum top flange width =	b_{f_max} =	11.51"	W33x130
maximum top flange thickness =	t_{f_max} =	0.86"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~6.75"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	6.75"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	7.50"	Based on Field Measurment
width of overhang =	$b_{overhang}$ =	1.896 ft	(assumes symmetrical)

Material Weights:

				LRFD
Unit weight of concrete =	w_c =	150 pcf	with reinforcing.	3.5.1 - Table 3.5.1-1
Unit weight of asphalt =	w_a =	140 pcf		3.5.1 - Table 3.5.1-2
Unit weight of granite =	w_g =	170 pcf		3.5.1 - Table 3.5.1-3
Unit weight of utility fluid =	w_{fluid} =	62.4 pcf		
Unit weight of gas =	w_{gas} =	8.0 pcf		

Girder Haunch:

Top flange within haunch? =		Yes	
thickness of haunch =	t_{haunch} =	3.61"	See hand calculation
width of haunch =	b_{haunch} =	19.50"	
weight of single girder haunch =	W_{haunch} =	0.063 klf	bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.875 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	5.833 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	0.664 klf	Average Thickness = 0.56'
deck weight for exterior girder =	W_{deck_ext} =	0.492 klf	Average Thickness = 0.56'

MERLIN Dead Loads - 5969 Span 2 (CONT'D)

References

Wearing Surface:

type of wearing surface =		Concrete	
thickness of wearing surface =	t_{pvmt} =	3.50"	Based on field measured curb reveal
thickness of waterproofing membrane =	t_{wm} =	0.000"	Inclusive to Wearing Surface Thickness
total weight of wearing surface =	W_{ws} =	1.750 klf	

Sidewalk:

Note: The dead load due to the sidewalk concrete is determined by measuring areas in CAD. The concrete area is determined by projecting the cross-slope of the roadway to the edge of deck and calculating the area above the theoretical line, less the volume of granite curb.

number of sidewalks =	N_{sw} =	0	(No sidewalk on this structure)
cross-sectional area of sidewalk =	A_{sw} =	0.00 ft²	(concrete portion)
cross-sectional area of granite curb =	A_g =	0.00 ft²	(granite curb only, if used)
weight of sidewalk =	W_{sw_conc} =	0.000 klf	

Curbs:

Note: The dead load due to curbs, is of the curbs that are not associated with sidewalks

number of curbs =	N_{curbs} =	2	
cross-sect area of curb and mortar bed =	A_{curbs} =	1.04 ft²	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	0.42 ft²	(granite curb only, if used)
weight of curbs =	W_{curbs} =	0.455 klf	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	0.00 Ft.	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	0.000 klf	

MERLIN Dead Loads - 5969 Span 2 (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$=	Other	0.260 klf	See hand calculation
$Rail_{Right}$=	Other	0.260 klf	See hand calculation
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.520 klf	

Snow / Pedestrian Fences:

left fence height =	NONE	Weight = 0 plf.	Verify Load
right fence height =	NONE	Weight = 0 plf.	Verify Load
total fence weight =	W_{fences} =	0.000 klf	

Diaphragms:

Note: This dead load calculation is for an individual diaphragm.

# of Attachment Plate 1 =	2	
Attachment Plate 1 Height =	29.00"	Field Measurement
Attachment Plate 1 Width =	6.00"	Field Measurement
Attachment Plate 1 Thickness =	0.375"	Field Measurement
Attachment Plate 1 Weight =	0.037 kip	
# of Diagonals =	0	
Diagonal Weight Per Foot =	0 plf	
Length of Diagonal member =	0.00'	
Diagonal Weight =	0.000 kip	
# of Horizontal Members =	1	
Horizontal Member Weight Per Foot =	33.9 plf	Per Plans
Length of Horizontal Member =	7.38'	
Horizontal Member Weight =	0.250 kip	
Interior Girder Diaphragm Weight =	0.29 kips	
Exterior Girder Diaphragm Weight =	0.14 kips	1/2 Interior Girder Diaphragm Weight

MERLIN Dead Loads - 5969 Span 2 (CONT'D)

References

Bridge Mounted Utilities:

Are utilities present on this bridge?	*No*
Are sewer main(s) present on this bridge?	*No*
Are water main(s) present on this bridge?	*No*
Are gas main(s) present on this bridge?	*No*
Are duct bank(s) present on this bridge?	*No*

Weight of sewer main line =		*0.0 lbs/ft*	
Inside diameter of sewer main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{sewer} =	**0.000 klf**	

Weight of water main line =		*0.0 lbs/ft*	
Inside diameter of water main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{water} =	**0.000 klf**	

Weight of gas line =		*0.0 lbs/ft*	
Inside diameter of gas line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{gas} * A_i)
Total line weight =	W_{gas} =	**0.000 klf**	

Weight of electrical duct bank =		*0.0 lbs/ft*
Hardware Allowance =		*0.0 lbs/ft*
Total duct bank weight =	$W_{electric}$ =	**0.000 klf**

Girder 1 N/C utility load =	0.000 klf
Girder 2 N/C utility load =	0.000 klf
Girder 3 N/C utility load =	0.000 klf
Girder 4 N/C utility load =	0.000 klf
Girder 5 N/C utility load =	0.000 klf
Girder 6 N/C utility load =	0.000 klf

MERLIN Dead Loads - 5969 Span 2 (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = **0.727 klf** = $W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.555 klf** = $W_{deck\_ext} + W_{haunch} + W_{soffit}$

MERLIN loads - Additional non-composite loads (DC2):

utility loads (interior girder) = **0.000 klf** = W_{util_int}

utility loads (exterior girder) = **0.000 klf** = W_{util_ext}

Diaphragm (interior girder) = **0.287 kip** per location

Diaphragm (exterior girder) = **0.144 kip** per location

MERLIN loads - Composite (DC2):

wearing surface = **0.292 klf** = $W_{ws} \div N_g$

sidewalk = **0.000 klf** = $W_{sw} \div N_g$

railing = **0.087 klf** = $W_{rails} \div N_g$

snow fence = **0.000 klf** = $W_{fence} \div N_g$

curb = **0.076 klf** = $W_{curbs} \div N_g$

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Symmetrical Deck, Equal Girder Spacing

MERLIN Dead Loads - 5969 Span 3

References

- This sheet determines the applied loads for input into MERLIN girder runs.
  MERLIN internally calculates the dead load of the steel girders.

Geometry of Bridge:

length of bridge =	L_{span} =	48.00'	
curb-to-curb width of bridge =	b_{pvmt} =	40.00'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	43.17'	
number of girders =	N_g =	6	
girder spacing =	S_g =	7.875'	
maximum top flange width =	b_{f_max} =	11.48"	W33x118
maximum top flange thickness =	t_{f_max} =	0.74"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~6.75"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	6.75"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	7.50"	Based on Field Measurment
width of overhang =	$b_{overhang}$ =	1.896 ft	(assumes symmetrical)

Material Weights:

				LRFD
Unit weight of concrete =	w_c =	150 pcf	with reinforcing.	3.5.1 - Table 3.5.1-1
Unit weight of asphalt =	w_a =	140 pcf		3.5.1 - Table 3.5.1-2
Unit weight of granite =	w_g =	170 pcf		3.5.1 - Table 3.5.1-3
Unit weight of utility fluid =	w_{fluid} =	62.4 pcf		
Unit weight of gas =	w_{gas} =	8.0 pcf		

Girder Haunch:

Top flange within haunch? =		Yes	
thickness of haunch =	t_{haunch} =	3.49"	See hand calculation
width of haunch =	b_{haunch} =	19.50"	
weight of single girder haunch =	W_{haunch} =	0.062 klf	bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.875 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	5.833 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	0.664 klf	Average Thickness = 0.56'
deck weight for exterior girder =	W_{deck_ext} =	0.492 klf	Average Thickness = 0.56'

MERLIN Dead Loads - 5969 Span 3 (CONT'D)

References

Wearing Surface:

type of wearing surface =		Concrete	
thickness of wearing surface =	t_{pvmt} =	3.50"	Based on field measured curb reveal
thickness of waterproofing membrane =	t_{wm} =	0.000"	Inclusive to Wearing Surface Thickness
total weight of wearing surface =	W_{ws} =	1.750 klf	

Sidewalk:

Note: The dead load due to the sidewalk concrete is determined by measuring areas in CAD. The concrete area is determined by projecting the cross-slope of the roadway to the edge of deck and calculating the area above the theoretical line, less the volume of granite curb.

number of sidewalks =	N_{sw} =	0	(No sidewalk on this structure)
cross-sectional area of sidewalk =	A_{sw} =	0.00 ft²	(concrete portion)
cross-sectional area of granite curb =	A_g =	0.00 ft²	(granite curb only, if used)
weight of sidewalk =	W_{sw_conc} =	0.000 klf	

Curbs:

Note: The dead load due to curbs, is of the curbs that are not associated with sidewalks

number of curbs =	N_{curbs} =	2	
cross-sect area of curb and mortar bed =	A_{curbs} =	1.04 ft²	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	0.42 ft²	(granite curb only, if used)
weight of curbs =	W_{curbs} =	0.455 klf	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	0.00 Ft.	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	0.000 klf	

MERLIN Dead Loads - 5969 Span 3 (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$=	Other	0.260 klf	See hand calculation
$Rail_{Right}$=	Other	0.260 klf	See hand calculation
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.520 klf	

Snow / Pedestrian Fences:

left fence height =	*NONE*	Weight = 0 plf.	Verify Load
right fence height =	*NONE*	Weight = 0 plf.	Verify Load
total fence weight =	W_{fences} =	0.000 klf	

Diaphragms:

Note: This dead load calculation is for an individual diaphragm.

# of Attachment Plate 1 =	*2*	
Attachment Plate 1 Height =	*29.00"*	Field Measurement
Attachment Plate 1 Width =	*6.00"*	Field Measurement
Attachment Plate 1 Thickness =	*0.375"*	Field Measurement
Attachment Plate 1 Weight =	0.037 kip	

# of Diagonals =	*0*	
Diagonal Weight Per Foot =	*0 plf*	
Length of Diagonal member =	*0.00'*	
Diagonal Weight =	0.000 kip	

# of Horizontal Members =	*1*	
Horizontal Member Weight Per Foot =	*33.9 plf*	Per Plans
Length of Horizontal Member =	*7.38'*	
Horizontal Member Weight =	0.250 kip	

Interior Girder Diaphragm Weight =	*0.29 kips*	
Exterior Girder Diaphragm Weight =	*0.14 kips*	1/2 Interior Girder Diaphragm Weight

MERLIN Dead Loads - 5969 Span 3 (CONT'D)

References

Bridge Mounted Utilities:

Are utilities present on this bridge?	*No*
Are sewer main(s) present on this bridge?	*No*
Are water main(s) present on this bridge?	*No*
Are gas main(s) present on this bridge?	*No*
Are duct bank(s) present on this bridge?	*No*

Weight of sewer main line =		*0.0 lbs/ft*	
Inside diameter of sewer main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{sewer} =	**0.000 klf**	

Weight of water main line =		*0.0 lbs/ft*	
Inside diameter of water main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{water} =	**0.000 klf**	

Weight of gas line =		*0.0 lbs/ft*	
Inside diameter of gas line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{gas} * A_i)
Total line weight =	W_{gas} =	**0.000 klf**	

Weight of electrical duct bank =		*0.0 lbs/ft*
Hardware Allowance =		*0.0 lbs/ft*
Total duct bank weight =	$W_{electric}$ =	**0.000 klf**

Girder 1 N/C utility load =	0.000 klf
Girder 2 N/C utility load =	0.000 klf
Girder 3 N/C utility load =	0.000 klf
Girder 4 N/C utility load =	0.000 klf
Girder 5 N/C utility load =	0.000 klf
Girder 6 N/C utility load =	0.000 klf

MERLIN Dead Loads - 5969 Span 3 (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = **0.726 klf** = $W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.554 klf** = $W_{deck\_ext} + W_{haunch} + W_{soffit}$

MERLIN loads - Additional non-composite loads (DC2):

utility loads (interior girder) = **0.000 klf** = W_{util_int}

utility loads (exterior girder) = **0.000 klf** = W_{util_ext}

Diaphragm (interior girder) = **0.287 kip** per location

Diaphragm (exterior girder) = **0.144 kip** per location

MERLIN loads - Composite (DC2):

wearing surface = **0.292 klf** = $W_{ws} \div N_g$

sidewalk = **0.000 klf** = $W_{sw} \div N_g$

railing = **0.087 klf** = $W_{rails} \div N_g$

snow fence = **0.000 klf** = $W_{fence} \div N_g$

curb = **0.076 klf** = $W_{curbs} \div N_g$

Diaphragm Locations:

	Spacing		Distance from Left End		Check
Exterior	6.50		6.50		
	20.75		27.25		48.00
					20.75
Interior	15.10		15.10		
	20.75		35.85		48.00
					12.15

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Symmetrical Deck, Equal Girder Spacing

References

MERLIN Dead Loads - 5969 Span 4

- This sheet determines the applied loads for input into MERLIN girder runs.
  MERLIN internally calculates the dead load of the steel girders.

Geometry of Bridge:

length of bridge =	L_{span} =	51.00'	
curb-to-curb width of bridge =	b_{pvmt} =	40.00'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	43.167'	
number of girders =	N_g =	6	
girder spacing =	S_g =	7.875'	
maximum top flange width =	b_{f_max} =	11.48"	W33x118
maximum top flange thickness =	t_{f_max} =	0.74"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~6.75"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	6.75"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	7.50"	Based on Field Measurment
width of overhang =	$b_{overhang}$ =	1.896 ft	(assumes symmetrical)

Material Weights:

				LRFD
Unit weight of concrete =	w_c =	150 pcf	with reinforcing.	3.5.1 - Table 3.5.1-1
Unit weight of asphalt =	w_a =	140 pcf		3.5.1 - Table 3.5.1-2
Unit weight of granite =	w_g =	170 pcf		3.5.1 - Table 3.5.1-3
Unit weight of utility fluid =	w_{fluid} =	62.4 pcf		
Unit weight of gas =	w_{gas} =	8.0 pcf		

Girder Haunch:

Top flange within haunch? =		Yes	
thickness of haunch =	t_{haunch} =	3.49"	See hand calculation
width of haunch =	b_{haunch} =	19.50"	
weight of single girder haunch =	W_{haunch} =	0.062 **klf**	bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.875 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	5.833 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	0.664 **klf**	Average Thickness = 0.56'
deck weight for exterior girder =	W_{deck_ext} =	0.492 **klf**	Average Thickness = 0.56'

MERLIN Dead Loads - 5969 Span 4 (CONT'D)

References

Wearing Surface:

type of wearing surface =		Concrete	
thickness of wearing surface =	t_{pvmt} =	3.50"	Based on field measured curb reveal
thickness of waterproofing membrane =	t_{wm} =	0.000"	Inclusive to Wearing Surface Thickness
total weight of wearing surface =	W_{ws} =	1.750 klf	

Sidewalk:

Note: The dead load due to the sidewalk concrete is determined by measuring areas in CAD. The concrete area is determined by projecting the cross-slope of the roadway to the edge of deck and calculating the area above the theoretical line, less the volume of granite curb.

number of sidewalks =	N_{sw} =	0	(No sidewalk on this structure)
cross-sectional area of sidewalk =	A_{sw} =	0.00 ft²	(concrete portion)
cross-sectional area of granite curb =	A_g =	0.00 ft²	(granite curb only, if used)
weight of sidewalk =	W_{sw_conc} =	0.000 klf	

Curbs:

Note: The dead load due to curbs, is of the curbs that are not associated with sidewalks

number of curbs =	N_{curbs} =	2	
cross-sect area of curb and mortar bed =	A_{curbs} =	1.04 ft²	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	0.42 ft²	(granite curb only, if used)
weight of curbs =	W_{curbs} =	0.455 klf	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	0.00 Ft.	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	0.000 **klf**	

MERLIN Dead Loads - 5969 Span 4 (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$=	Other	0.260 klf	See hand calculation
$Rail_{Right}$=	Other	0.260 klf	See hand calculation
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.520 klf	

Snow / Pedestrian Fences:

left fence height =	*NONE*	Weight = 0 plf.	Verify Load
right fence height =	*NONE*	Weight = 0 plf.	Verify Load
total fence weight =	W_{fences} =	0.000 klf	

Diaphragms:

Note: This dead load calculation is for an individual diaphragm.

# of Attachment Plate 1 =	*2*	
Attachment Plate 1 Height =	*29.00"*	Field Measurement
Attachment Plate 1 Width =	*6.00"*	Field Measurement
Attachment Plate 1 Thickness =	*0.375"*	Field Measurement
Attachment Plate 1 Weight =	0.037 kip	

# of Diagonals =	*0*	
Diagonal Weight Per Foot =	*0 plf*	
Length of Diagonal member =	*0.00'*	
Diagonal Weight =	0.000 kip	

# of Horizontal Members =	*1*	
Horizontal Member Weight Per Foot =	*33.9 plf*	Per Plans
Length of Horizontal Member =	*7.38'*	
Horizontal Member Weight =	0.250 kip	

Interior Girder Diaphragm Weight =	*0.29 kips*	
Exterior Girder Diaphragm Weight =	*0.14 kips*	1/2 Interior Girder Diaphragm Weight

MERLIN Dead Loads - 5969 Span 4 (CONT'D)

References

Bridge Mounted Utilities:

Are utilities present on this bridge?	*No*
Are sewer main(s) present on this bridge?	*No*
Are water main(s) present on this bridge?	*No*
Are gas main(s) present on this bridge?	*No*
Are duct bank(s) present on this bridge?	*No*

Weight of sewer main line =		*0.0 lbs/ft*	
Inside diameter of sewer main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{sewer} =	0.000 klf	

Weight of water main line =		*0.0 lbs/ft*	
Inside diameter of water main line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{fluid} * A_i)
Total line weight =	W_{water} =	0.000 klf	

Weight of gas line =		*0.0 lbs/ft*	
Inside diameter of gas line =		*0.00"*	
Insulation & Hardware Allowance =		*0.0 lbs/ft*	
liquid weight =	W_{liquid} =	*0.0 lbs/ft*	(w_{gas} * A_i)
Total line weight =	W_{gas} =	0.000 klf	

Weight of electrical duct bank =		*0.0 lbs/ft*
Hardware Allowance =		*0.0 lbs/ft*
Total duct bank weight =	$W_{electric}$ =	0.000 klf

Girder 1 N/C utility load =	0.000 klf
Girder 2 N/C utility load =	0.000 klf
Girder 3 N/C utility load =	0.000 klf
Girder 4 N/C utility load =	0.000 klf
Girder 5 N/C utility load =	0.000 klf
Girder 6 N/C utility load =	0.000 klf

MERLIN Dead Loads - 5969 Span 4 (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = 0.726 klf = $W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = 0.554 klf = $W_{deck\_ext} + W_{haunch} + W_{soffit}$

MERLIN loads - Additional non-composite loads (DC2):

utility loads (interior girder) = 0.000 klf = W_{util_int}

utility loads (exterior girder) = 0.000 klf = W_{util_ext}

Diaphragm (interior girder) = 0.287 kip per location

Diaphragm (exterior girder) = 0.144 kip per location

MERLIN loads - Composite (DC2):

wearing surface = 0.292 klf = $W_{ws} \div N_g$

sidewalk = 0.000 klf = $W_{sw} \div N_g$

railing = 0.087 klf = $W_{rails} \div N_g$

snow fence = 0.000 klf = $W_{fence} \div N_g$

curb = 0.076 klf = $W_{curbs} \div N_g$

Diaphragm Locations:

	Spacing		Distance from Left End		Check
Exterior	7.50		7.50		
	21.75		29.25		51.00
					21.75
Interior	16.10		16.10		
	21.75		37.85		51.00
					13.15

BRIDGE RATING

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Live Load Distribution: SPAN 1

The live load distribution factors for the HL-93 load case computed by the Merlin-Dash software are not inclusive of the lever rule or rigid body rotation. Therefore, this calculation in accordance with AASHTO LRFD 4.6.2.2 is input directly into Merlin-Dash. Distribution factors used in this analysis reflect the most recent AASHTO specifications and actual lane position. Since the Maine Legal Loads are unrestricted and may mix with normal traffic, the same live load distribution factors developed for the HL-93 case are used for the legal loads.

The following controlling live load distribution factors were used to determine bridge load ratings and have been corrected for skew:

	Strength		Fatigue	
Element	**Moment**	**Shear**	**Moment**	**Shear**
Interior Stringer	0.654	0.899	0.399	0.628
Exterior Stringer	0.526	0.575	0.347	0.408

Live Loads used in the analysis are listed as follows:

Description	**GVW (kips)**
HL-93	72
Maine Legal Loads	
1	100
2	94
3	88
4	88
5	88
6	75.9
7	59.4
8	37.4

Specific truck and lane load configurations are shown on the following pages.

ADT		12190	From SI&A
$\%_{trucks}$		11%	From SI&A
ADTT = $\%_{trucks}$*ADT		1341	Merlin Dash 06012
Fatigue Loading 0.85*ADTT		1140	
Condition Factor	ϕ_c =	1	
System Factor	ϕ_s =	1	
	ϕ_c*ϕ_s =	1	Merlin Dash 09012 input as ETA for Strength

LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES **LRFD AASHTO 4.6.2.2**

Application
- simple span
- constant deck width
- number of beams is not less than 4
- parallel beams with same approximate stiffness
- roadway overhang (d_e)is less than or equal to 3 ft.
- beam horizontal curvature is less than 12 degrees
- cross section is consistent with Table 4.6.2.2-1 (a)
- number of lanes for curb-curb widths between 18 ft. and 24 ft. equals 2

Variables

Skew Angle	θ	32.8	degrees	
Minimum Shoulder Width	$W_{shoulder}$	6	Ft.	Field Measurement
Girder/Stringer Spacing	S	7.875	Ft.	
Girder/Stringer Span Length	L	61	Ft.	
Deck Overhang	d_{ovhg}	1.895833	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.583333	Ft.	
Roadway Overhang	d_e	0.3125	Ft.	
Corrugated Steel Plank ? (Yes/No)		No		
Depth of steel grid or corrugated steel plank	t_g	0	in.	
Depth of concrete slab	t_s	6.75	in.	
Deck Concrete Strength	f'_c	3	ksi	**MBE Table 6A.5.2.1-1**
Unit Weight of Concrete	γ_{conc}	0.145	kcf	**LRFD Table 3.5.1-1**
Type of Beam		Rolled Beam		
No. of Beams	N_b	6		
Number of Design Lanes	N_L	3		**LRFD 3.6.1.1.1**
Average Depth of beam	d	33.77	in.	**See Hand Calculation**
Average Area of Beam	A	46.36	in^2	**See Hand Calculation**
Avg. Moment of Inertia for Beam	I	8485.58	in^4	**See Hand Calculation**
Top Flange Thickness	t_{tf}	0.86	in	**From AISC Steel Manual**
Overcamber Allowance		0.75	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	3156	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	9.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	24.04	in.	**See Hand Calculation**
Longitudinal Stiffness Parameter	K_g	317548		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.03		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.09		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.90		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.12		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		0.95		**LRFD Table 4.6.2.2.2e-1**

Interior Beams **REFERENCE**

Moment

Corrugated Steel Plank Deck LRFD Table 4.6.2.2.2c-1

Check Range of Applicability:

Spacing	N/A	
Deck	N/A	
g_1 =	N/A	One lane loaded
g_m =	N/A	Two or more lanes loaded
Control g_c =	**N/A**	

Reinforced Concrete Deck LRFD Table 4.6.2.2.2b-1

Check Range of Applicability:

Spacing	7.875	OK	
Slab	*6.75*	OK	
Length	61	OK	
No. Beams	6	OK	
K_g	3.18E+05	OK	
g_1 =	0.505		One lane loaded
g_m =	0.689		Two or more lanes loaded
g_{1_fat} =	0.399		One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.654**		Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

g_1 =	0.675	One lane loaded
g_m =	0.806	Two or more lanes loaded
g_{1_fat} =	0.628	One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.899**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:

Roadway Overhang	*0.31*	OK	

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.554		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.347		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.526**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.509		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.408		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.575**		Corrected for skew

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Live Load Distribution: SPAN 2 - Determine factors based on the average span geometry.

The live load distribution factors for the HL-93 load case computed by the Merlin-Dash software are not inclusive of the lever rule or rigid body rotation. Therefore, this calculation in accordance with AASHTO LRFD 4.6.2.2 is input directly into Merlin-Dash. Distribution factors used in this analysis reflect the most recent AASHTO specifications and actual lane position. Since the Maine Legal Loads are unrestricted and may mix with normal traffic, the same live load distribution factors developed for the HL-93 case are used for the legal loads.

The following controlling live load distribution factors were used to determine bridge load ratings and have been corrected for skew:

	Strength		Fatigue	
Element	**Moment**	**Shear**	**Moment**	**Shear**
Interior Stringer	0.698	0.878	0.430	0.613
Exterior Stringer	0.562	0.561	0.366	0.399

Live Loads used in the analysis are listed as follows:

Description	**GVW (kips)**
HL-93	72
Maine Legal Loads	
1	100
2	94
3	88
4	88
5	88
6	75.9
7	59.4
8	37.4

Specific truck and lane load configurations are shown on the following pages.

ADT		12190	From SI&A
$\%_{trucks}$		11%	From SI&A
ADTT = $\%_{trucks}$*ADT		1341	
Fatigue Loading 0.85*ADTT		1140	
Condition Factor	ϕ_c =	1	
System Factor	ϕ_s =	1	
	$\phi_c*\phi_s$ =	1	

Merlin Dash 06012

Merlin Dash 09012
input as ETA for Strength

LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES **LRFD AASHTO 4.6.2.2**

Application
- simple span
- constant deck width
- number of beams is not less than 4
- parallel beams with same approximate stiffness
- roadway overhang (d_e)is less than or equal to 3 ft.
- beam horizontal curvature is less than 12 degrees
- cross section is consistent with Table 4.6.2.2-1 (a)
- number of lanes for curb-curb widths between 18 ft. and 24 ft. equals 2

Variables

Skew Angle	θ	26.34	Average	
Minimum Shoulder Width	$W_{shoulder}$	6	Ft.	Field Measurement
Girder/Stringer Spacing	S	7.875	Ft.	
Girder/Stringer Span Length	L	55.56833	Ft.	
Deck Overhang	d_{ovhg}	1.895833	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.58	Ft.	
Roadway Overhang	d_e	0.3125	Ft.	
Corrugated Steel Plank ? (Yes/No)		No		
Depth of steel grid or corrugated steel plank	t_g	0	in.	
Depth of concrete slab	t_s	6.75	in.	
Deck Concrete Strength	f'_c	3	ksi	**MBE Table 6A.5.2.1-1**
Unit Weight of Concrete	γ_{conc}	0.145	kcf	**LRFD Table 3.5.1-1**
Type of Beam		Rolled Beam		
No. of Beams	N_b	6		
Number of Design Lanes	N_L	3		**LRFD 3.6.1.1.1**
Average Depth of beam	d	33.52	in.	**See Hand Calculation**
Average Area of Beam	A	42.24	in^2	**See Hand Calculation**
Avg. Moment of Inertia for Beam	I	7608.38	in^4	**See Hand Calculation**
Avg. Top Flange Thickness	t_{tf}	0.8	in	**From AISC Steel Manual**
Overcamber Allowance		0.625	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	3156	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	9.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	23.53	in.	**See Hand Calculation**
Longitudinal Stiffness Parameter	K_g	278997		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.03		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.08		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.91		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.09		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		1.00		**LRFD Table 4.6.2.2.2e-1**

Interior Beams **REFERENCE**

Moment

Corrugated Steel Plank Deck LRFD Table 4.6.2.2.2c-1

Check Range of Applicability:

Spacing	N/A	
Deck	N/A	
g_1 =	N/A	One lane loaded
g_m =	N/A	Two or more lanes loaded
Control g_c =	**N/A**	

Reinforced Concrete Deck LRFD Table 4.6.2.2.2b-1

Check Range of Applicability:

Spacing	7.875	OK	
Slab	*6.75*	OK	
Length	55.56833	OK	
No. Beams	6	OK	
K_g	2.79E+05	OK	
g_1 =	0.516		One lane loaded
g_m =	0.698		Two or more lanes loaded
g_{1_fat} =	0.430		One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.698**		Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

g_1 =	0.675	One lane loaded
g_m =	0.806	Two or more lanes loaded
g_{1_fat} =	0.613	One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.878**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:

Roadway Overhang	*0.31*	OK	

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.562		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.366		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.562**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.509		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.399		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.561**		Corrected for skew

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Live Load Distribution: SPAN 3

The live load distribution factors for the HL-93 load case computed by the Merlin-Dash software are not inclusive of the lever rule or rigid body rotation. Therefore, this calculation in accordance with AASHTO LRFD 4.6.2.2 is input directly into Merlin-Dash. Distribution factors used in this analysis reflect the most recent AASHTO specifications and actual lane position. Since the Maine Legal Loads are unrestricted and may mix with normal traffic, the same live load distribution factors developed for the HL-93 case are used for the legal loads.

The following controlling live load distribution factors were used to determine bridge load ratings and have been corrected for skew:

	Strength		Fatigue	
Element	**Moment**	**Shear**	**Moment**	**Shear**
Interior Stringer	0.710	0.860	0.443	0.601
Exterior Stringer	0.571	0.550	0.366	0.391

Live Loads used in the analysis are listed as follows:

Description	**GVW (kips)**
HL-93	72
Maine Legal Loads	
1	100
2	94
3	88
4	88
5	88
6	75.9
7	59.4
8	37.4

Specific truck and lane load configurations are shown on the following pages.

ADT		12190	From SI&A
$\%_{trucks}$		11%	From SI&A
ADTT = $\%_{trucks}$*ADT		1341	Merlin Dash 06012
Fatigue Loading 0.85*ADTT		1140	
Condition Factor	ϕ_c =	1	
System Factor	ϕ_s =	1	
	ϕ_c*ϕ_s =	1	Merlin Dash 09012 input as ETA for Strength

LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES **LRFD AASHTO 4.6.2.2**

Application
- simple span
- constant deck width
- number of beams is not less than 4
- parallel beams with same approximate stiffness
- roadway overhang (d_e)is less than or equal to 3 ft.
- beam horizontal curvature is less than 12 degrees
- cross section is consistent with Table 4.6.2.2-1 (a)
- number of lanes for curb-curb widths between 18 ft. and 24 ft. equals 2

Variables

Skew Angle	θ	19.88	degrees	
Minimum Shoulder Width	$W_{shoulder}$	6	Ft.	Field Measurement
Girder/Stringer Spacing	S	7.875	Ft.	
Girder/Stringer Span Length	L	48	Ft.	
Deck Overhang	d_{ovhg}	1.90	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.58	Ft.	
Roadway Overhang	d_e	0.31	Ft.	
Corrugated Steel Plank ? (Yes/No)		No		
Depth of steel grid or corrugated steel plank	t_g	0	in.	
Depth of concrete slab	t_s	6.75	in.	
Deck Concrete Strength	f'_c	3	ksi	**MBE Table 6A.5.2.1-1**
Unit Weight of Concrete	γ_{conc}	0.145	kcf	**LRFD Table 3.5.1-1**
Type of Beam		Rolled Beam		
No. of Beams	N_b	6		
Number of Design Lanes	N_L	3		**LRFD 3.6.1.1.1**
Average Depth of beam	d	33.06	in.	**See Moment of Inertia Tab**
Average Area of Beam	A	36.55	in^2	**See Moment of Inertia Tab**
Avg. Moment of Inertia for Beam	I	6344.75	in^4	**See Moment of Inertia Tab**
Top Flange Thickness	t_{tf}	0.74	in	**From AISC Steel Manual**
Overcamber Allowance		0.5	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	3156	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	9.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	22.01	in.	**See Hand Calculation**
Longitudinal Stiffness Parameter	K_g	216447		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.02		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.05		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.94		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.07		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		1.00		**LRFD Table 4.6.2.2.2e-1**

Interior Beams **REFERENCE**

Moment

Corrugated Steel Plank Deck LRFD Table 4.6.2.2.2c-1

Check Range of Applicability:

Spacing	N/A	
Deck	N/A	
g_1 =	N/A	One lane loaded
g_m =	N/A	Two or more lanes loaded
Control g_c =	**N/A**	

Reinforced Concrete Deck LRFD Table 4.6.2.2.2b-1

Check Range of Applicability:

Spacing	7.875	OK	
Slab	*6.75*	OK	
Length	48	OK	
No. Beams	6	OK	
K_g	2.16E+05	OK	
g_1 =	0.531		One lane loaded
g_m =	0.710		Two or more lanes loaded
g_{1_fat} =	0.443		One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.710**		Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

g_1 =	0.675	One lane loaded
g_m =	0.806	Two or more lanes loaded
g_{1_fat} =	0.601	One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.860**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:

Roadway Overhang	*0.31*	OK	

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.571		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.366		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.571**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.486	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.509		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	$g_{1\_}$ fat =	0.391		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ =	**0.550**		Corrected for skew

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Live Load Distribution: SPAN 4

The live load distribution factors for the HL-93 load case computed by the Merlin-Dash software are not inclusive of the lever rule or rigid body rotation. Therefore, this calculation in accordance with AASHTO LRFD 4.6.2.2 is input directly into Merlin-Dash. Distribution factors used in this analysis reflect the most recent AASHTO specifications and actual lane position. Since the Maine Legal Loads are unrestricted and may mix with normal traffic, the same live load distribution factors developed for the HL-93 case are used for the legal loads.

The following controlling live load distribution factors were used to determine bridge load ratings and have been corrected for skew:

	Strength		Fatigue	
Element	**Moment**	**Shear**	**Moment**	**Shear**
Interior Stringer	0.705	0.860	0.437	0.600
Exterior Stringer	0.567	0.550	0.366	0.390

Live Loads used in the analysis are listed as follows:

Description	**GVW (kips)**
HL-93	72
Maine Legal Loads	
1	100
2	94
3	88
4	88
5	88
6	75.9
7	59.4
8	37.4

Specific truck and lane load configurations are shown on the following pages.

ADT		12190	From SI&A
$\%_{trucks}$		11%	From SI&A
ADTT = $\%_{trucks}$*ADT		1341	
Fatigue Loading	0.85*ADTT	1140	
Condition Factor	ϕ_c =	1	
System Factor	ϕ_s =	1	
	ϕ_c*ϕ_s =	1	

Merlin Dash 06012

Merlin Dash 09012
input as ETA for Strength

LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES **LRFD AASHTO 4.6.2.2**

Application
- simple span
- constant deck width
- number of beams is not less than 4
- parallel beams with same approximate stiffness
- roadway overhang (d_e)is less than or equal to 3 ft.
- beam horizontal curvature is less than 12 degrees
- cross section is consistent with Table 4.6.2.2-1 (a)
- number of lanes for curb-curb widths between 18 ft. and 24 ft. equals 2

Variables

Skew Angle	θ	19.88	degrees	
Minimum Shoulder Width	$W_{shoulder}$	6	Ft.	Field Measurement
Girder/Stringer Spacing	S	7.875	Ft.	
Girder/Stringer Span Length	L	51	Ft.	
Deck Overhang	d_{ovhg}	1.896	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.58	Ft.	
Roadway Overhang	d_e	0.3125	Ft.	
Corrugated Steel Plank ? (Yes/No)		No		
Depth of steel grid or corrugated steel plank	t_g	0	in.	
Depth of concrete slab	t_s	6.75	in.	
Deck Concrete Strength	f'_c	3	ksi	**MBE Table 6A.5.2.1-1**
Unit Weight of Concrete	γ_{conc}	0.145	kcf	**LRFD Table 3.5.1-1**
Type of Beam		Rolled Beam		
No. of Beams	N_b	6		
Number of Design Lanes	N_L	3		**LRFD 3.6.1.1.1**
Average Depth of beam	d	33.20	in.	**See Moment of Inertia Tab**
Average Area of Beam	A	38.16	in^2	**See Moment of Inertia Tab**
Avg. Moment of Inertia for Beam	I	6711.14	in^4	**See Moment of Inertia Tab**
Top Flange Thickness	t_{tf}	0.74	in	**From AISC Steel Manual**
Overcamber Allowance		0.625	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	3156	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	9.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	22.75	in.	**See Hand Calculation**
Longitudinal Stiffness Parameter	K_g	238135		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.02		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.06		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.93		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.07		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		1.00		**LRFD Table 4.6.2.2.2e-1**

Interior Beams **REFERENCE**

Moment

Corrugated Steel Plank Deck LRFD Table 4.6.2.2.2c-1

Check Range of Applicability:

Spacing	N/A	
Deck	N/A	

	g_1 =	N/A	One lane loaded
	g_m =	N/A	Two or more lanes loaded
Control	g_c =	**N/A**	

Reinforced Concrete Deck LRFD Table 4.6.2.2.2b-1

Check Range of Applicability:

Spacing	7.875	OK
Slab	*6.75*	OK
Length	51	OK
No. Beams	6	OK
K_g	2.38E+05	OK

	g_1 =	0.524	One lane loaded
	g_m =	0.705	Two or more lanes loaded
	g_{1_fat} =	0.437	One lane loaded without multiple presence for fatigue analysis
Control	g_c =	**0.705**	Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

	g_1 =	0.675	One lane loaded
	g_m =	0.806	Two or more lanes loaded
	g_{1_fat} =	0.600	One lane loaded without multiple presence for fatigue analysis
Control	g_c =	**0.860**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:
Roadway Overhang *0.31* OK

	g_1 =	0.486	Lever Rule	One lane loaded (m =1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m =1.2) accounting for lane line.
	g_m =	0.567		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	g_{1_fat} =	0.366		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ **=**	**0.567**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.486	Lever Rule	One lane loaded (m =1.2) without accounting for lane line.
	g_{1l} =	0.167	Lever Rule	One lane loaded (m =1.2) accounting for lane line.
	g_m =	0.509		Two or more lanes loaded
	g_{RB1} =	0.439		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.515		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	g_{1_fat} =	0.390		One lane loaded without multiple presence for fatigue analysis
Control	$\boldsymbol{g_c}$ **=**	**0.550**		Corrected for skew

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Moment of Inertia Calculation for Rolled Shape - AISC Values									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	33.10	11.51	38.26	16.55	633.20	0.00	0.00	6699.00	6699.00
				$\bar{y}$ (in)=	16.55				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Exterior Midspan									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	33.10	11.51	38.26	17.55	671.46	3.53	477.55	6699.00	7176.55
Plate (bottom)	1.000	10.00	10.00	0.50	5.00	-13.52	1827.11	0.83	1827.94
Total	34.10		48.26		676.46				9004.49
				$\bar{y}$ (in)=	14.02				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Interior Midspan									
Section	**Depth (in)**		**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	33.10	11.51	38.26	17.55	671.46	4.32	715.36	6699.00	7414.36
Plate (bottom)	1.00	13.00	13.00	0.50	6.50	-12.73	2105.35	1.08	2106.44
Total	34.10		51.26		677.96				9520.79
				$\bar{y}$ (in)=	13.23				

Span Length	61	ft	
Length of Exterior Coverplate	39	ft	Does not Include Transition Length
Length of Interior Coverplate	42	ft	Does not Include Transition Length

Find Weighted Average Properties of Interior and Exterior Beams

Average Depth	33.77	in	See Hand Calculation
Average Area	46.36	in^2	See Hand Calculation
Average I_o	8485.58	in^4	See Hand Calculation
Average $\bar{y}$	14.48	in	See Hand Calculation

5969 - Span 2

Moment of Inertia Calculation for Rolled Shape - AISC Values (W33x118)

Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	32.86	11.48	34.71	16.43	570.29	0.00	0.00	5886.90	5886.90
				ȳ (in)=	16.43				

Moment of Inertia Calculation for Rolled Shape - AISC Values (W33x130)

Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	33.10	11.51	38.26	16.55	633.20	0.00	0.00	6699.00	6699.00
				ȳ (in)=	16.55				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S15 Midspan

Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	32.86	11.48	34.71	16.87	585.47	1.86	120.55	5886.90	6007.45
Plate (bottom)	0.44	10.00	4.38	0.22	0.96	-14.79	956.38	0.07	956.45
Total	33.30		39.09		586.43				6963.90
				ȳ (in)=	15.00				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S16 Midspan

Section	Depth (in)		A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	32.86	11.48	34.71	17.18	596.32	2.99	309.47	5886.90	6196.37
Plate (bottom)	0.75	10.00	7.50	0.38	2.81	-13.82	1432.24	0.35	1432.59
Total	33.61		42.21		599.13				7628.97
				ȳ (in)=	14.19				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S17 Midspan

Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	32.86	11.48	34.71	17.37	602.83	3.59	448.25	5886.90	6335.15
Plate (bottom)	0.94	10.00	9.38	0.47	4.39	-13.31	1659.62	0.69	1660.30
Total	33.80		44.09		607.22				7995.46
				ȳ (in)=	13.77				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S18 Midspan

Section	Depth (in)		A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	33.10	11.51	38.26	17.55	671.46	3.53	477.55	6699.00	7176.55
Plate (bottom)	1.00	10.00	10.00	0.50	5.00	-13.52	1827.11	0.83	1827.94
Total	34.10		48.26		676.46				9004.49
				ȳ (in)=	14.02				

(continued)

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S19 Midspan									
Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	33.10	11.51	38.26	17.43	666.68	3.89	579.87	6699.00	7278.87
Plate (bottom)	0.875	13.00	11.38	0.44	4.98	-13.09	1950.40	0.73	1951.13
Total	33.98		49.64		671.66				9230.00
				ȳ (in)=	13.53				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Girder S20 Midspan									
Section	Depth (in)		A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	33.10	11.51	38.26	17.55	671.46	3.53	477.55	6699.00	7176.55
Plate (bottom)	1.00	10.00	10.00	0.50	5.00	-13.52	1827.11	0.83	1827.94
Total	34.10		48.26		676.46				9004.49
				ȳ (in)=	14.02				

Span Length - Girder S15	50.00	ft
Span Length - Girder S16	52.23	ft
Span Length - Girder S17	54.45	ft
Span Length - Girder S18	56.68	ft
Span Length - Girder S19	58.91	ft
Span Length - Girder S20	61.14	ft

Average Span Length 55.57 ft

Length of Girder S15 Cover PL	23	ft	Does not Include Transition Length
Length of Girder S16 Cover PL	33	ft	Does not Include Transition Length
Length of Girder S17 Cover PL	35	ft	Does not Include Transition Length
Length of Girder S18 Cover PL	35	ft	Does not Include Transition Length
Length of Girder S19 Cover PL	41	ft	Does not Include Transition Length
Length of Girder S20 Cover PL	39	ft	Does not Include Transition Length

Find Average Depth of Interior and Exterior Beams

Average Depth	33.52	in
Average Area	42.24	in^2
Average I_o	7608.38	in^4
Average ȳ	14.96	in
Average t_f	0.8	in

5969 Span 3

Moment of Inertia Calculation for Rolled Shape - AISC Values									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	16.43	570.29	0.00	0.00	5886.90	5886.90
				$\bar{y}$ (in)=	16.43				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Exterior Midspan									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	16.74	581.13	1.05	38.20	5886.90	5925.10
Plate (bottom)	0.313	7.50	2.34	0.16	0.37	-15.54	565.79	0.02	565.81
Total	33.17		37.05		581.50				6490.91
				$\bar{y}$ (in)=	15.69				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Interior Midspan									
Section	**Depth (in)**		**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	16.87	585.47	1.86	120.55	5886.90	6007.45
Plate (bottom)	0.44	10.00	4.38	0.22	0.96	-14.79	956.38	0.07	956.45
Total	33.30		39.09		586.43				6963.90
				$\bar{y}$ (in)=	15.00				

Span Length	48	ft	
Length of Exterior Coverplate	20	ft	Does not Include Transition Length
Length of Interior Coverplate	25	ft	Does not Include Transition Length

Find Weighted Average Properties of Interior and Exterior Beams

Average Depth	33.06	in
Average Area	36.55	in^2
Average I_o	6344.75	in^4
Average $\bar{y}$	15.83	in

5969 Span 4

Moment of Inertia Calculation for Rolled Shape - AISC Values									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	16.43	570.29	0.00	0.00	5886.90	5886.90
				ȳ (in)=	16.43				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Exterior Midspan									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	16.93	587.64	2.10	153.10	5886.90	6040.00
Plate (bottom)	0.500	10.00	5.00	0.25	1.25	-14.58	1062.85	0.10	1062.95
Total	33.36		39.71		588.89				7102.96
				ȳ (in)=	14.83				

Moment of Inertia Calculation for Rolled Shape with Cover Plate - Interior Midspan									
Section	**Depth (in)**		**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	32.86	11.48	34.71	17.06	591.98	2.55	226.54	5886.90	6113.44
Plate (bottom)	0.625	10.00	6.25	0.31	1.95	-14.19	1258.08	0.20	1258.29
Total	33.49		40.96		593.93				7371.72
				ȳ (in)=	14.50				

Span Length	51	ft	
Length of Exterior Coverplate	28	ft	Does not Include Transition Length
Length of Interior Coverplate	31	ft	Does not Include Transition Length

Find Weighted Average Properties of Interior and Exterior Beams

Average Depth	33.20	in
Average Area	38.16	in^2
Average I_o	6711.14	in^4
Average ȳ	15.36	in

Reference - AASHTO LRFD Bridge Design Specifications - Fifth Edition (2010) with Interims

This calculation can consider the striped lane markings per Maine DOT Rating Guide 3.3.1.7. It is assumed that the wheel is either 2 feet from the face of curb or on the white stripe, whichever is further.

RIGID CROSS SECTION ANALYSIS

$B_{lane} := 12 \cdot ft$

Width of Travel Lane Striped - Defaults to 12' design lane. For roadways between 18 and 24, use half the roadway width for the lane width

$B_{shoulder} := 6 ft$

Width of Shoulder

$d_{shoulder} := B_{shoulder} - 2ft$

Default to 0' when not accounting for Lane Striping Lines, or shoulder of 2' or less. When shoulder is greater than 2', subtract 2' so the wheel is on the white stripe.

$N_L := 3$ $N_{L.Striped} := 2$

Number of Design/Striped Lanes

$N_b := 6$

Number of Beams

$S := 7 \cdot ft + 10.5 \cdot in$ $S = 7.875\ ft$

Girder Spacing

$d_{e.act} := 3.75 \cdot in$ $d_e := d_{e.act} - d_{shoulder}$ $d_e = -3.688\ ft$

Actual distance from the centerline of the exterior web to the "face of the curb" accounting for shoulder

$X_{ext} := \frac{(N_b - 1)}{2} \cdot S$ $X_{ext} = 19.688\ ft$

Horizontal Distance from the center of gravity of the pattern of girders to the exterior girder.

$e := \begin{pmatrix} X_{ext} + d_e - 5 \cdot ft \\ X_{ext} + d_e - 5 \cdot ft - B_{lane} \end{pmatrix}$ $e = \begin{pmatrix} 11 \\ -1 \end{pmatrix} ft$

Eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders

$m_p := \begin{pmatrix} 1.20 \\ 1.00 \end{pmatrix}$

Multiple presence factor for one, two or three lanes loaded AASHTO LRFD Table 3.6.1.1.2-1.

RIGID CROSS SECTION ANALYSIS (cont'd)

$$R_1 := \left[\frac{1}{N_b} + \frac{X_{ext} \cdot e_1}{2\cdot(2.5\cdot S)^2 + 2\cdot(1.5\cdot S)^2 + 2\cdot(0.5\cdot S)^2}\right]\cdot m_{p_1}$$

Distribution Factor for 1 lane loaded - Equation C4.6.2.2.2d-1

$$R_2 := \left[\frac{2}{N_b} + \frac{X_{ext} \cdot (e_1 + e_2)}{2\cdot(2.5\cdot S)^2 + 2\cdot(1.5\cdot S)^2 + 2\cdot(0.5\cdot S)^2}\right]\cdot m_{p_2}$$

Distribution Factor for 2 lanes loaded - Equation C4.6.2.2.2d-1

$$R = \begin{pmatrix} 0.439 \\ 0.515 \end{pmatrix}$$

Since there are 2 striped lanes on the bridge use a factor of 0.515.

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BY: DGB/TDM DATE: 8/20/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 8/23/13 SUBJECT: No. 5969 - I-95 NB Over B&A RR & Emerson Mill Rd

References

- The Manual for Bridge Evaluation - Second edition 2011 & 2013 Interims
- AASHTO LRFD Bridge Design Specifications - Fifth Edition (2010) with Interims

Fatigue Rating Interior Girder Span 2 S18 (Controls for all W33 x 130 beams)

$t_{bf} := \mathbf{0.86} \cdot in$ — Thickness of the bottom flange for W33 x 130 from AISC Historic Database ASD 6th edition.

The cover plate transition is Detail Category E' per AASHTO LRFD Table 6.6.1.2.3-1 description 3.5.

$A := \mathbf{3.9} \times 10^{8} \cdot ksi^{3}$ — Constant A AASHTO LRFD Table 6.6.1.2.3-1 description 3.5 Detail Category E'.

$\Delta f_{th} := \mathbf{2.6} \cdot ksi$ — Stress Threshold AASHTO LRFD Table 6.6.1.2.3-1 description 3.5 Detail Category E'.

$\Delta f_{LL\_IM} := \mathbf{2.9} \cdot ksi$ — Unfactored fatigue live load stress range at 7.56' for N = 15 from Merlin Dash Table 1.2.9.3A.

$R_{sa} := \mathbf{1.0}$ — Analysis partial load factor - MBE Table 7.2.2.1-1 for stress range by simplified analysis.

$R_{st} := \mathbf{1.0}$ — Truck-Weight partial load factor - MBE Table 7.2.2.1-1 for stress range by simplified analysis.

$R_s := R_{sa} \cdot R_{st}$ $\quad R_s = 1$ — Stress-Range estimate partial load factor - MBE Table 7.2.2.1-1 and Equation 7.2.2.1.1-1 for stress range by simplified analysis.

$\gamma_{LL} := \mathbf{0.75}$ — Load Factor for live load for the Fatigue II Load Combination AASHTO LRFD Table 3.4.1-1.

$\Delta f_{eff} := R_s \cdot \gamma_{LL} \cdot \Delta f_{LL\_IM}$ $\quad \Delta f_{eff} = 2.175 \cdot ksi$ — Effective stress range - MBE equation 7.2.2-1

$\Delta f_{LL\_IM.max} := 2.0 \cdot \Delta f_{eff}$ — Maximum stress range expected at the fatigue-prone detail per MBE 7.2.4.

$\Delta f_{LL\_IM.max} = 4.35 \cdot ksi$

$Infinite_{check} := if\left(\Delta f_{LL\_IM.max} \le \Delta f_{th}, \text{"Infinite Fatigue Life"}, \text{"Finite Fatigue Life"}\right)$

$Infinite_{check} = \text{"Finite Fatigue Life"}$ — Check that the structure has infinite fatigue life per MBE equation 7.2.4-1.

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BY: DGB/TDM DATE: 8/20/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 8/23/13 SUBJECT: No. 5969 - I-95 NB Over B&A RR & Emerson Mill Rd

Use the following only for cases with Finite Fatigue Life.

$Year_{built} := \mathbf{1962}$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $\quad Age = 51$ — Age of bridge in 2013.

$R_R := \mathbf{1.6}$ — Resistance Factor for Evaluation Life - MBE Table 7.2.5.2-1.

$n := \mathbf{1.0}$ — Cycles per truck passage, n, AASHTO LRFD Table 6.6.1.2.5-2.

$ADTT := \mathbf{1341}$ — Average daily truck traffic from excel spreadsheet.

$\rho := \mathbf{0.85}$ — Fraction of a truck in a single lane - AASHTO LRFD Table 3.6.1.4.2-1.

$g := \mathbf{1.00}$ — Growth rate from MBE Figure C7.2.5.1-1. Assumed growth rate of 4%.

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $\quad ADTT_{SL} = 1140$ — Average daily truck traffic in a single lane assuming 0 for the 51 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\,3}} \qquad Y = 146$$

Total finite fatigue life in the bridge - MBE equation 7.2.5.1-1.

$Y_{remaining} := Y - Age$ $\quad Y_{remaining} = 95$ — Remaining Fatigue Life

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BY: DGB/TDM DATE: 8/20/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 8/23/13 SUBJECT: No. 5969 - I-95 NB Over B&A RR & Emerson Mill Rd

References

- The Manual for Bridge Evaluation - Second edition 2011 & 2013 Interims
- AASHTO LRFD Bridge Design Specifications - Fifth Edition (2010) with Interims

Fatigue Rating Interior Girder Span 3 (Controls for all W33 x 118 beams)

$t_{bf} := \mathbf{0.74} \cdot in$ — Thickness of the bottom flange for W33 x 118 from AISC Historic Database ASD 6th edition.

The cover plate transition is Detail Category E per AASHTO LRFD Table 6.6.1.2.3-1 description 3.5.

$A := \mathbf{11} \times 10^{8} \cdot ksi^{3}$ — Constant A AASHTO LRFD Table 6.6.1.2.3-1 description 3.5 Detail Category E.

$\Delta f_{th} := \mathbf{4.5} \cdot ksi$ — Stress Threshold AASHTO LRFD Table 6.6.1.2.3-1 description 3.5 Detail Category E.

$\Delta f_{LL\_IM} := \mathbf{3.4} \cdot ksi$ — Unfactored fatigue live load stress range at 9.60' for N = 15 from Merlin Dash Table 1.2.9.3A.

$R_{sa} := \mathbf{1.0}$ — Analysis partial load factor - MBE Table 7.2.2.1-1 for stress range by simplified analysis.

$R_{st} := \mathbf{1.0}$ — Truck-Weight partial load factor - MBE Table 7.2.2.1-1 for stress range by simplified analysis.

$R_s := R_{sa} \cdot R_{st}$ $\quad R_s = 1$ — Stress-Range estimate partial load factor - MBE Table 7.2.2.1-1 and Equation 7.2.2.1.1-1 for stress range by simplified analysis.

$\gamma_{LL} := \mathbf{0.75}$ — Load Factor for live load for the Fatigue II Load Combination AASHTO LRFD Table 3.4.1-1.

$\Delta f_{eff} := R_s \cdot \gamma_{LL} \cdot \Delta f_{LL\_IM}$ $\quad \Delta f_{eff} = 2.55 \cdot ksi$ — Effective stress range - MBE equation 7.2.2-1

$\Delta f_{LL\_IM.max} := 2.0 \cdot \Delta f_{eff}$ — Maximum stress range expected at the fatigue-prone detail per MBE 7.2.4.

$\Delta f_{LL\_IM.max} = 5.1 \cdot ksi$

$Infinite_{check} := if\left(\Delta f_{LL\_IM.max} \le \Delta f_{th}, \text{"Infinite Fatigue Life"}, \text{"Finite Fatigue Life"}\right)$

$Infinite_{check} = \text{"Finite Fatigue Life"}$ — Check that the structure has infinite fatigue life per MBE equation 7.2.4-1.

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BY: DGB/TDM DATE: 8/20/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 8/23/13 SUBJECT: No. 5969 - I-95 NB Over B&A RR & Emerson Mill Rd

Use the following only for cases with Finite Fatigue Life.

$Year_{built} := \mathbf{1962}$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $\quad Age = 51$ — Age of bridge in 2013.

$R_R := \mathbf{1.3}$ — Resistance Factor for Evaluation Life - MBE Table 7.2.5.2-1.

$n := \mathbf{1.0}$ — Cycles per truck passage, n, AASHTO LRFD Table 6.6.1.2.5-2.

$ADTT := \mathbf{1341}$ — Average daily truck traffic from excel spreadsheet.

$\rho := \mathbf{0.85}$ — Fraction of a truck in a single lane - AASHTO LRFD Table 3.6.1.4.2-1.

$g := \mathbf{1.00}$ — Growth rate from MBE Figure C7.2.5.1-1. Assumed growth rate of 4%.

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $\quad ADTT_{SL} = 1140$ — Average daily truck traffic in a single lane assuming 0 for the 51 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\,3}} \qquad Y = 207$$

Total finite fatigue life in the bridge - MBE equation 7.2.5.1-1.

$Y_{remaining} := Y - Age$ $\quad Y_{remaining} = 156$ — Remaining Fatigue Life

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MERLIN V 10.4
COMPOSITE
LRF -- 2010
INPUT

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 1\5969\Rating\5969

TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span 1 Interior Strength CP	8/20/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5939	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	13	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

1

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.24	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT (lb/ft)	WEB DEPTH (in)	WEB THICK. (in)	STANDARD SECTN / PLATE GIRDER: TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION: AREA (in**2)	Ix (in**4)
1	W	33	130		0.00	0.0000	0.00	0.0000		
2	W	33	130		0.00	0.0000	13.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
			-->TYPE<--						
1	1	1	9.50				33.	33.	33.
2	2	2	42.00				33.	33.	33.
3	1	1	9.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C
1	33.00	0	0.00	0.40	0.00	15.00	1 1 1 1 1	0.65	2 2 2 2 2
1	0.00	0	0.00	0.63	0.00	0.00	0 0 0 0 0	0.90	3 3 3 3 3

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	61.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	11.51
2	2	Diaphragm	0.000	0.00	0.00	0.29	33.10
3	2	Diaphragm	0.000	0.00	0.00	0.29	54.69
4	1	Railing	0.087	0.00	61.00	0.00	0.00
5	1	Curb	0.076	0.00	61.00	0.00	0.00
6	0	Pavement	0.292	0.00	61.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	DIST. FROM TOP (in**2)	COMP. STRENG. AT 28 DAYS (in)	COMP. ALLOW -ABLE (ksi)	(ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	11.51	33.0	33.0	11.51
11.51	33.10	33.0	33.0	21.59
33.10	54.69	33.0	33.0	21.59
54.69	61.00	33.0	33.0	6.31

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.727	0.1608	0.8878

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	61.00
1	DC2	0.087	0.000	61.000
2	DC2	0.076	0.000	61.000

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
	V O L U M E		TOTAL WEIGHT	TOTAL WEIGHT	(kip)
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	(kip)	(kip)	
150.00	1481.2	54.9	222.2	58.8	281.01

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.654	0.654	0.702	0.702	FOR STRENGTH POSITIVE MOMENT
	0.899				FOR STRENGTH POSITIVE SHEAR
1	0.399				FOR FATIGUE POSITIVE MOMENT
	0.628				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	4.69	6710.0	33.1	16.55	405.6	405.6
1	2	9.38	6710.0	33.1	16.55	405.6	405.6
1	3	14.08	9539.7	33.1	13.22	721.4	457.2
1	4	18.77	9539.7	33.1	13.22	721.4	457.2
1	5	23.46	9539.7	33.1	13.22	721.4	457.2
1	6	28.15	9539.7	33.1	13.22	721.4	457.2
1	7	32.85	9539.7	33.1	13.22	721.4	457.2
1	8	37.54	9539.7	33.1	13.22	721.4	457.2

1	9	42.23	9539.7	33.1	13.22	721.4	457.2
1	10	46.92	9539.7	33.1	13.22	721.4	457.2
1	11	51.62	6710.0	33.1	16.55	405.6	405.6
1	12	56.31	6710.0	33.1	16.55	405.6	405.6
1	13	61.00	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	13532.1	0.023717	544.5	1642.8	826.3
1	1	4.69	13532.1	0.023717	544.5	1642.8	826.3
1	2	9.38	13532.1	0.023717	544.5	1642.8	826.3
1	3	14.08	20472.1	0.021012	951.0	1629.4	988.8
1	4	18.77	20472.1	0.021012	951.0	1629.4	988.8
1	5	23.46	20472.1	0.021012	951.0	1629.4	988.8
1	6	28.15	20472.1	0.021012	951.0	1629.4	988.8
1	7	32.85	20472.1	0.021012	951.0	1629.4	988.8
1	8	37.54	20472.1	0.021012	951.0	1629.4	988.8
1	9	42.23	20472.1	0.021012	951.0	1629.4	988.8
1	10	46.92	20472.1	0.021012	951.0	1629.4	988.8
1	11	51.62	13532.1	0.023717	544.5	1642.8	826.3
1	12	56.31	13532.1	0.023717	544.5	1642.8	826.3
1	13	61.00	13532.1	0.023717	544.5	1642.8	826.3

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	18306.4	0.029290	600.5	7031.5	1704.0
1	1	4.69	18306.4	0.029290	600.5	7031.5	1704.0
1	2	9.38	18306.4	0.029290	600.5	7031.5	1704.0
1	3	14.08	29510.1	0.026378	1043.2	5086.0	2116.6
1	4	18.77	29510.1	0.026378	1043.2	5086.0	2116.6
1	5	23.46	29510.1	0.026378	1043.2	5086.0	2116.6
1	6	28.15	29510.1	0.026378	1043.2	5086.0	2116.6
1	7	32.85	29510.1	0.026378	1043.2	5086.0	2116.6
1	8	37.54	29510.1	0.026378	1043.2	5086.0	2116.6
1	9	42.23	29510.1	0.026378	1043.2	5086.0	2116.6
1	10	46.92	29510.1	0.026378	1043.2	5086.0	2116.6
1	11	51.62	18306.4	0.029290	600.5	7031.5	1704.0
1	12	56.31	18306.4	0.029290	600.5	7031.5	1704.0
1	13	61.00	18306.4	0.029290	600.5	7031.5	1704.0

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 Please read NOTE on the following page
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     NOTE [1] If the section modulus for the top flange indicates
              overflows (***), the neutral axis may be very closed to
              the top of the top flange.

          [2] The section properties shown in this table are used
              for the calculation of stresses.

          [3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 ---
              Composite sections in simple spans and
              the positive moment regions of continuous
              spans should preferably be proportioned so that the
              neutral axis lies below the top surface of the steel
              beam. Concrete 'on the tension side of the neutral axis
              shall not be considered in calculating resulting moments.
              In the negative moment regions of continuous spans, only
              the slab reinforcement can be considered to act
              compositely with the steel beams in calculating resisting
              moments. Mechanical anchorages shall be provided in the
              composite regions to develop stresses on the plane
              joining the concrete and the steel. Concrete on the
              tension side of the neutral axis may be considered in
              computing moments of inertia for deflections and for
              determining stiffness factors used in calculating moments
              and shears

          [4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be
              using short-term composite section for positeve & negative flxure.
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 TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
               *********************************************

                    DEAD  LOAD                                TOTAL (k-ft)
 SP IN D FROM  ------------------- CONCENTRATED  UNIFORM  ---------------
 NO NO L SUPT     BEAM      SLAB        LOADS      LOADS    NONCOMPOSITE
        (ft)     (k-ft)    (k-ft)       (k-ft)     (k-ft)    DEAD  LOAD
 ---------------------------------------------------------------------

  1  0   0.00       0.0       0.0         0.0        0.0           0.0
  1  1   4.69      21.2      96.0         1.8        0.0         119.1
  1  2   9.38      38.9     176.1         3.7        0.0         218.7
  1  3  14.08      53.1     240.1         4.8        0.0         298.0
  1  4  18.77      63.7     288.1         5.3        0.0         357.1
  1  5  23.46      70.8     320.1         5.8        0.0         396.7
  1  6  28.15      74.3     336.1         6.3        0.0         416.8
  1  7  32.85      74.3     336.1         6.8        0.0         417.3
  1  8  37.54      70.8     320.1         6.0        0.0         397.0
  1  9  42.23      63.7     288.1         5.2        0.0         357.0
  1 10  46.92      53.1     240.1         4.3        0.0         297.5
  1 11  51.62      38.9     176.1         3.5        0.0         218.5
  1 12  56.31      21.2      96.0         2.2        0.0         119.5
  1 13  61.00       0.0       0.0         0.0        0.0           0.0


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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	4.69	38.6	21.5	0.0	0.0	0.0	60.1
1	2	9.38	70.7	39.5	0.0	0.0	0.0	110.2
1	3	14.08	96.4	53.8	0.0	0.0	0.0	150.3
1	4	18.77	115.7	64.6	0.0	0.0	0.0	180.3
1	5	23.46	128.6	71.8	0.0	0.0	0.0	200.4
1	6	28.15	135.0	75.4	0.0	0.0	0.0	210.4
1	7	32.85	135.0	75.4	0.0	0.0	0.0	210.4
1	8	37.54	128.6	71.8	0.0	0.0	0.0	200.4
1	9	42.23	115.7	64.6	0.0	0.0	0.0	180.3
1	10	46.92	96.4	53.8	0.0	0.0	0.0	150.3
1	11	51.62	70.7	39.5	0.0	0.0	0.0	110.2
1	12	56.31	38.6	21.5	0.0	0.0	0.0	60.1
1	13	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	4.69	0.0	0.0	281.6	HL-93	0.0	HL-93
1	2	9.38	0.0	0.0	508.8	HL-93	0.0	HL-93
1	3	14.08	0.0	0.0	679.4	HL-93	0.0	HL-93
1	4	18.77	0.0	0.0	801.0	HL-93	0.0	HL-93
1	5	23.46	0.0	0.0	879.5	HL-93	0.0	HL-93
1	6	28.15	0.0	0.0	903.7	HL-93	0.0	HL-93
1	7	32.85	0.0	0.0	903.7	HL-93	0.0	HL-93
1	8	37.54	0.0	0.0	879.5	HL-93	0.0	HL-93
1	9	42.23	0.0	0.0	801.0	HL-93	0.0	HL-93
1	10	46.92	0.0	0.0	679.4	HL-93	0.0	HL-93
1	11	51.62	0.0	0.0	508.8	HL-93	0.0	HL-93
1	12	56.31	0.0	0.0	281.6	HL-93	0.0	HL-93
1	13	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	4.69	97.	0.	97.
1	2	9.38	168.	0.	168.

1	3	14.08	222.	0.	222.
1	4	18.77	258.	0.	258.
1	5	23.46	276.	0.	276.
1	6	28.15	261.	0.	261.
1	7	32.85	261.	0.	261.
1	8	37.54	276.	0.	276.
1	9	42.23	258.	0.	258.
1	10	46.92	222.	0.	222.
1	11	51.62	168.	0.	168.
1	12	56.31	97.	0.	97.
1	13	61.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	4.69	119.1	60.1	281.6	HL-93	0.0	HL-93	460.8	179.2
1	2	9.38	218.7	110.2	508.8	HL-93	0.0	HL-93	837.7	328.9
1	3	14.08	298.0	150.3	679.4	HL-93	0.0	HL-93	1127.7	448.3
1	4	18.77	357.1	180.3	801.0	HL-93	0.0	HL-93	1338.4	537.5
1	5	23.46	396.7	200.4	879.5	HL-93	0.0	HL-93	1476.6	597.1
1	6	28.15	416.8	210.4	903.7	HL-93	0.0	HL-93	1530.9	627.2
1	7	32.85	417.3	210.4	903.7	HL-93	0.0	HL-93	1531.4	627.7
1	8	37.54	397.0	200.4	879.5	HL-93	0.0	HL-93	1476.9	597.3
1	9	42.23	357.0	180.3	801.0	HL-93	0.0	HL-93	1338.3	537.4
1	10	46.92	297.5	150.3	679.4	HL-93	0.0	HL-93	1127.2	447.8
1	11	51.62	218.5	110.2	508.8	HL-93	0.0	HL-93	837.5	328.7
1	12	56.31	119.5	60.1	281.6	HL-93	0.0	HL-93	461.2	179.6
1	13	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	4.69	460.8	545.3	726.5	613.8	268.9	72.6
1	2	9.38	837.7	990.3	1319.1	1115.6	493.4	126.1
1	3	14.08	1127.7	1331.5	1773.4	1501.7	672.4	166.3
1	4	18.77	1338.4	1578.7	2102.4	1782.1	806.2	193.6
1	5	23.46	1476.6	1740.5	2317.7	1965.9	895.6	206.9
1	6	28.15	1530.9	1802.0	2399.2	2037.7	940.7	195.4
1	7	32.85	1531.4	1802.5	2399.8	2038.3	941.5	195.4
1	8	37.54	1476.9	1740.7	2318.0	1966.2	896.0	206.9
1	9	42.23	1338.3	1578.6	2102.3	1781.9	806.0	193.6
1	10	46.92	1127.2	1331.1	1772.9	1501.1	671.7	166.3
1	11	51.62	837.5	990.1	1318.9	1115.4	493.1	126.1

1 12	56.31	461.2	545.6	726.9	614.3	269.4	72.6
1 13	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.9	22.2	0.4	0.0	27.5
1	1	4.69	4.1	18.8	0.4	0.0	23.3
1	2	9.38	3.4	15.4	0.4	0.0	19.1
1	3	14.08	2.6	11.9	0.1	0.0	14.7
1	4	18.77	1.9	8.5	0.1	0.0	10.5
1	5	23.46	1.1	5.1	0.1	0.0	6.4
1	6	28.15	0.4	1.7	0.1	0.0	2.2
1	7	32.85	-0.4	-1.7	0.1	0.0	-2.0
1	8	37.54	-1.1	-5.1	-0.2	0.0	-6.4
1	9	42.23	-1.9	-8.5	-0.2	0.0	-10.6
1	10	46.92	-2.6	-11.9	-0.2	0.0	-14.8
1	11	51.62	-3.4	-15.4	-0.2	0.0	-18.9
1	12	56.31	-4.1	-18.8	-0.5	0.0	-23.4
1	13	61.00	-4.9	-22.2	-0.5	0.0	-27.5

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	27.5	41.3
1	1	4.69	7.5	4.2	0.0	0.0	11.7	23.3	35.0
1	2	9.38	6.2	3.4	0.0	0.0	9.6	19.1	28.7
1	3	14.08	4.8	2.7	0.0	0.0	7.5	14.7	22.2
1	4	18.77	3.4	1.9	0.0	0.0	5.3	10.5	15.9
1	5	23.46	2.1	1.1	0.0	0.0	3.2	6.4	9.6
1	6	28.15	0.7	0.4	0.0	0.0	1.1	2.2	3.3
1	7	32.85	-0.7	-0.4	0.0	0.0	-1.1	-2.0	-3.0
1	8	37.54	-2.1	-1.1	0.0	0.0	-3.2	-6.4	-9.6
1	9	42.23	-3.4	-1.9	0.0	0.0	-5.3	-10.6	-15.9
1	10	46.92	-4.8	-2.7	0.0	0.0	-7.5	-14.8	-22.2
1	11	51.62	-6.2	-3.4	0.0	0.0	-9.6	-18.9	-28.5
1	12	56.31	-7.5	-4.2	0.0	0.0	-11.7	-23.4	-35.1
1	13	61.00	-8.9	-5.0	0.0	0.0	-13.9	-27.5	-41.4

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	90.4	HL-93	0.0	HL-93
1	1	4.69	0.0	0.0	81.2	HL-93	-3.0	HL-93
1	2	9.38	0.0	0.0	72.2	HL-93	-7.6	HL-93
1	3	14.08	0.0	0.0	63.4	HL-93	-12.8	HL-93
1	4	18.77	0.0	0.0	54.8	HL-93	-18.1	HL-93
1	5	23.46	0.0	0.0	46.4	HL-93	-23.6	HL-93
1	6	28.15	0.0	0.0	38.2	HL-93	-30.4	HL-93
1	7	32.85	0.0	0.0	30.4	HL-93	-38.2	HL-93
1	8	37.54	0.0	0.0	23.6	HL-93	-46.4	HL-93
1	9	42.23	0.0	0.0	18.1	HL-93	-54.8	HL-93
1	10	46.92	0.0	0.0	12.8	HL-93	-63.4	HL-93
1	11	51.62	0.0	0.0	7.6	HL-93	-72.2	HL-93
1	12	56.31	0.0	0.0	3.0	HL-93	-81.2	HL-93
1	13	61.00	0.0	0.0	0.0	HL-93	-90.4	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	36.	0.	36.
1	1	4.69	32.	-2.	34.
1	2	9.38	28.	-4.	32.
1	3	14.08	24.	-5.	30.
1	4	18.77	21.	-8.	28.
1	5	23.46	17.	-10.	27.
1	6	28.15	13.	-11.	24.
1	7	32.85	11.	-13.	24.
1	8	37.54	10.	-17.	27.
1	9	42.23	8.	-21.	28.
1	10	46.92	5.	-24.	30.
1	11	51.62	4.	-28.	32.
1	12	56.31	2.	-32.	34.
1	13	61.00	0.	-36.	36.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	36.5	.30737E-01	0.56	0.3125*	.14860E-01	0.27	0.3125*

1	1	4.69	34.2	.30737E-01	0.53	0.3125*	.14860E-01	0.25	0.3125*
1	2	9.38	31.9	.30737E-01	0.49	0.3125*	.14860E-01	0.24	0.3125*
1	3	14.08	29.9	.28303E-01	0.42	0.3125*	.81917E-02	0.12	0.3125*
1	4	18.77	28.2	.28303E-01	0.40	0.3125*	.81917E-02	0.12	0.3125*
1	5	23.46	26.9	.28303E-01	0.38	0.3125*	.81917E-02	0.11	0.3125*
1	6	28.15	23.9	.28303E-01	0.34	0.3125*	.81917E-02	0.10	0.3125*
1	7	32.85	23.9	.28303E-01	0.34	0.3125*	.81917E-02	0.10	0.3125*
1	8	37.54	26.9	.28303E-01	0.38	0.3125*	.81917E-02	0.11	0.3125*
1	9	42.23	28.2	.28303E-01	0.40	0.3125*	.81917E-02	0.12	0.3125*
1	10	46.92	29.9	.28303E-01	0.42	0.3125*	.81917E-02	0.12	0.3125*
1	11	51.62	31.9	.30737E-01	0.49	0.3125*	.14860E-01	0.24	0.3125*
1	12	56.31	34.2	.30737E-01	0.53	0.3125*	.14860E-01	0.25	0.3125*
1	13	61.00	36.5	.30737E-01	0.56	0.3125*	.14860E-01	0.27	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	41.3	90.4	HL-93	0.0	HL-93	131.8	41.3
1	1	4.69	35.0	81.2	HL-93	-3.0	HL-93	116.3	32.0
1	2	9.38	28.7	72.2	HL-93	-7.6	HL-93	101.0	21.1
1	3	14.08	22.2	63.4	HL-93	-12.8	HL-93	85.6	9.4
1	4	18.77	15.9	54.8	HL-93	-18.1	HL-93	70.7	-2.2
1	5	23.46	9.6	46.4	HL-93	-23.6	HL-93	56.0	-14.1
1	6	28.15	3.3	38.2	HL-93	-30.4	HL-93	41.4	-27.2
1	7	32.85	-3.0	30.4	HL-93	-38.2	HL-93	27.4	-41.2
1	8	37.54	-9.6	23.6	HL-93	-46.4	HL-93	14.0	-56.1
1	9	42.23	-15.9	18.1	HL-93	-54.8	HL-93	2.2	-70.8
1	10	46.92	-22.2	12.8	HL-93	-63.4	HL-93	-9.5	-85.7
1	11	51.62	-28.5	7.6	HL-93	-72.2	HL-93	-20.9	-100.8
1	12	56.31	-35.1	3.0	HL-93	-81.2	HL-93	-32.1	-116.4
1	13	61.00	-41.4	0.0	HL-93	-90.4	HL-93	-41.4	-131.9

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	131.8	158.9	212.2	176.0	62.0	27.3
1	1	4.69	116.3	140.6	187.8	155.4	52.6	25.7
1	2	9.38	101.0	122.6	163.9	135.0	43.1	24.0
1	3	14.08	85.6	104.6	139.9	114.5	33.2	22.4
1	4	18.77	70.7	87.1	116.6	94.7	23.8	21.2
1	5	23.46	56.0	69.9	93.7	75.1	14.3	20.2
1	6	28.15	41.4	52.9	71.1	55.8	4.9	17.9
1	7	32.85	-41.2	-52.7	-70.8	-55.5	-4.6	17.9
1	8	37.54	-56.1	-70.0	-93.8	-75.2	-14.4	20.2
1	9	42.23	-70.8	-87.2	-116.7	-94.8	-23.9	21.2
1	10	46.92	-85.7	-104.7	-140.0	-114.6	-33.3	22.4
1	11	51.62	-100.8	-122.4	-163.6	-134.7	-42.8	24.0
1	12	56.31	-116.4	-140.7	-187.9	-155.4	-52.7	25.7
1	13	61.00	-131.9	-159.0	-212.3	-176.1	-62.1	27.3

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	41.35	0.00	HL-93	90.45	HL-93	ST1	34.99	212.20
						ST2	34.99	176.02
						ST4	48.66	62.02
						SE1	41.35	131.80
						SE2	41.35	158.93
2	41.42	0.00	HL-93	90.45	HL-93	ST1	35.05	212.29
						ST2	35.05	176.11
						ST4	48.77	62.13
						SE1	41.42	131.87
						SE2	41.42	159.01

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)

1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	4.69	-0.0463	-0.2093	0.0000	-0.0617	-0.2597	-0.0617	-0.3213
1	2	9.38	-0.0885	-0.4002	0.0000	-0.1176	-0.4963	-0.1176	-0.6140
1	3	14.08	-0.1243	-0.5623	0.0000	-0.1649	-0.6974	-0.1649	-0.8624
1	4	18.77	-0.1529	-0.6917	0.0000	-0.2026	-0.8578	-0.2026	-1.0604
1	5	23.46	-0.1728	-0.7816	0.0000	-0.2289	-0.9692	-0.2289	-1.1981
1	6	28.15	-0.1830	-0.8276	0.0000	-0.2423	-1.0263	-0.2423	-1.2686
1	7	32.85	-0.1830	-0.8276	0.0000	-0.2423	-1.0263	-0.2423	-1.2686
1	8	37.54	-0.1728	-0.7816	0.0000	-0.2289	-0.9692	-0.2289	-1.1981
1	9	42.23	-0.1529	-0.6917	0.0000	-0.2026	-0.8577	-0.2026	-1.0604
1	10	46.92	-0.1243	-0.5623	0.0000	-0.1649	-0.6974	-0.1649	-0.8623
1	11	51.62	-0.0885	-0.4002	0.0000	-0.1176	-0.4963	-0.1176	-0.6139
1	12	56.31	-0.0463	-0.2093	0.0000	-0.0617	-0.2597	-0.0617	-0.3214
1	13	61.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	4.7	-0.046	0 1/16	-0.213	0 1/ 4	-0.062	0 1/16	-0.321	0 3/ 8
1	2	9.4	-0.088	0 1/ 8	-0.408	0 7/16	-0.118	0 1/ 8	-0.614	0 5/ 8
1	3	14.1	-0.124	0 1/ 8	-0.573	0 5/ 8	-0.165	0 3/16	-0.862	0 7/ 8
1	4	18.8	-0.153	0 3/16	-0.705	0 3/ 4	-0.203	0 1/ 4	-1.060	1 1/16
1	5	23.5	-0.173	0 3/16	-0.796	0 13/16	-0.229	0 1/ 4	-1.198	1 1/ 4
1	6	28.2	-0.183	0 3/16	-0.843	0 7/ 8	-0.242	0 1/ 4	-1.269	1 5/16
1	7	32.8	-0.183	0 3/16	-0.843	0 7/ 8	-0.242	0 1/ 4	-1.269	1 5/16
1	8	37.5	-0.173	0 3/16	-0.796	0 13/16	-0.229	0 1/ 4	-1.198	1 1/ 4
1	9	42.2	-0.153	0 3/16	-0.705	0 3/ 4	-0.203	0 1/ 4	-1.060	1 1/16
1	10	46.9	-0.124	0 1/ 8	-0.573	0 5/ 8	-0.165	0 3/16	-0.862	0 7/ 8
1	11	51.6	-0.088	0 1/ 8	-0.408	0 7/16	-0.118	0 1/ 8	-0.614	0 5/ 8
1	12	56.3	-0.046	0 1/16	-0.213	0 1/ 4	-0.062	0 1/16	-0.321	0 3/ 8
1	13	61.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.50	3 0.425	-0.348	MAX HL-93	0.92	0.00279
			0.028	MIN		

```
                                  5969 Span 1 Interior Strength CP.RES
                                   -0.102 MAX LANE
                                    0.028 MIN LANE

                    4    0.433     -0.355 MAX HL-93     0.92            0.00279
                                    0.029 MIN
                                   -0.104 MAX LANE
                                    0.029 MIN LANE

                    3    0.425     -0.255 MAX FA
                                    0.000 MIN FA

     ---------------------------------------------------------------------
     NOTE: [1] " - " indicates downward deflection

           [2] The distribution factor for LL+I deflection is defined as

               DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

               where NL= no. of traffic lanes
                     Ng= no. of girders
                     (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

           [3] This table is based upon the optional criteria specified
               in AASHTO LRFD Art. 3.6.1.3.2

           [4] The number of traffic lanes is determined according to
               AASHTO LRFD Art.3.6.1.1.1.
               The 1st line is for the most probable number of lanes
               and the 2nd line is for the next probable number of lanes.

           [5] Max rotations at left (1st line) & right (2nd line)
               supports of the span without averaging, factor and impact

           [6] If ADTT is between 100 and 1000, multi-presence
               factor of 0.95 is applied. If ADTT is below 100, factor
               is 0.9 (AASHTO LRFD C3.6.1.1.2).

           [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
               *********************************************

              STEEL DEAD LOAD    OTHER DEAD LOAD   TOTAL DEAD LOAD
 SP IN D FROM ----------------  ----------------  ----------------
 NO NO L SUPT    STEEL BEAM        STEEL BEAM        STEEL BEAM
                TOP      BOT      TOP      BOT      TOP      BOT
        (ft)       (ksi)             (ksi)             (ksi)
 ---------------------------------------------------------------------

  1  0   0.00    0.00    0.00      0.00    0.00      0.00    0.00
  1  1   4.69   -0.63    0.63     -2.90    2.90     -3.52    3.52
  1  2   9.38   -1.15    1.15     -5.32    5.32     -6.47    6.47
  1  3  14.08   -1.39    0.88     -6.43    4.07     -7.82    4.96
  1  4  18.77   -1.67    1.06     -7.70    4.88     -9.37    5.94
  1  5  23.46   -1.86    1.18     -8.55    5.42    -10.41    6.60
  1  6  28.15   -1.95    1.24     -8.99    5.70    -10.94    6.93
  1  7  32.85   -1.95    1.24     -9.00    5.71    -10.95    6.94
  1  8  37.54   -1.86    1.18     -8.56    5.43    -10.42    6.60
  1  9  42.23   -1.67    1.06     -7.70    4.88     -9.37    5.94
  1 10  46.92   -1.39    0.88     -6.42    4.07     -7.81    4.95
  1 11  51.62   -1.15    1.15     -5.31    5.31     -6.47    6.47
  1 12  56.31   -0.63    0.63     -2.91    2.91     -3.53    3.53
  1 13  61.00    0.00    0.00      0.00    0.00      0.00    0.00


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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	-0.03	-0.44	1.32	-0.03	-3.96	4.85
1	2	9.38	-0.06	-0.80	2.43	-0.06	-7.28	8.90
1	3	14.08	-0.07	-1.11	1.90	-0.07	-8.93	6.85
1	4	18.77	-0.08	-1.33	2.28	-0.08	-10.70	8.22
1	5	23.46	-0.09	-1.48	2.53	-0.09	-11.89	9.13
1	6	28.15	-0.09	-1.55	2.65	-0.09	-12.49	9.59
1	7	32.85	-0.09	-1.55	2.65	-0.09	-12.50	9.60
1	8	37.54	-0.09	-1.48	2.53	-0.09	-11.89	9.13
1	9	42.23	-0.08	-1.33	2.28	-0.08	-10.70	8.21
1	10	46.92	-0.07	-1.11	1.90	-0.07	-8.92	6.85
1	11	51.62	-0.06	-0.80	2.43	-0.06	-7.27	8.89
1	12	56.31	-0.03	-0.44	1.32	-0.03	-3.97	4.86
1	13	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	-0.22	-0.48	5.63	0.00	0.00	0.00
1	2	9.38	-0.40	-0.87	10.17	0.00	0.00	0.00
1	3	14.08	-0.43	-1.60	7.82	0.00	0.00	0.00
1	4	18.77	-0.50	-1.89	9.21	0.00	0.00	0.00
1	5	23.46	-0.55	-2.08	10.12	0.00	0.00	0.00
1	6	28.15	-0.57	-2.13	10.40	0.00	0.00	0.00
1	7	32.85	-0.57	-2.13	10.40	0.00	0.00	0.00
1	8	37.54	-0.55	-2.08	10.12	0.00	0.00	0.00
1	9	42.23	-0.50	-1.89	9.21	0.00	0.00	0.00
1	10	46.92	-0.43	-1.60	7.82	0.00	0.00	0.00
1	11	51.62	-0.40	-0.87	10.17	0.00	0.00	0.00
1	12	56.31	-0.22	-0.48	5.63	0.00	0.00	0.00
1	13	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE

1	0	0.00	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL
1	1	4.69	0.2	0.0	0.2 C	HL	1.9	0.0	1.9 T	HL
1	2	9.38	0.3	0.0	0.3 C	HL	3.4	0.0	3.4 T	HL
1	3	14.08	0.5	0.0	0.5 C	HL	2.6	0.0	2.6 T	HL
1	4	18.77	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1	5	23.46	0.7	0.0	0.7 C	HL	3.2	0.0	3.2 T	HL
1	6	28.15	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1	7	32.85	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1	8	37.54	0.7	0.0	0.7 C	HL	3.2	0.0	3.2 T	HL
1	9	42.23	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1	10	46.92	0.5	0.0	0.5 C	HL	2.6	0.0	2.6 T	HL
1	11	51.62	0.3	0.0	0.3 C	HL	3.4	0.0	3.4 T	HL
1	12	56.31	0.2	0.0	0.2 C	HL	1.9	0.0	1.9 T	HL
1	13	61.00	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.25	-4.44	10.48	-0.03	-3.96	4.85
1	2	9.38	33.	33.	-0.46	-8.14	19.07	-0.06	-7.28	8.90
1	3	14.08	33.	33.	-0.50	-10.53	14.67	-0.07	-8.93	6.85
1	4	18.77	33.	33.	-0.59	-12.59	17.43	-0.08	-10.70	8.22
1	5	23.46	33.	33.	-0.64	-13.96	19.25	-0.09	-11.89	9.13
1	6	28.15	33.	33.	-0.66	-14.62	19.98	-0.09	-12.49	9.59
1	7	32.85	33.	33.	-0.66	-14.63	19.99	-0.09	-12.50	9.60
1	8	37.54	33.	33.	-0.64	-13.97	19.25	-0.09	-11.89	9.13
1	9	42.23	33.	33.	-0.59	-12.59	17.43	-0.08	-10.70	8.21
1	10	46.92	33.	33.	-0.50	-10.52	14.66	-0.07	-8.92	6.85
1	11	51.62	33.	33.	-0.46	-8.14	19.06	-0.06	-7.27	8.89
1	12	56.31	33.	33.	-0.25	-4.45	10.49	-0.03	-3.97	4.86
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.32	-4.59	12.17	-0.03	-3.96	4.85
1	2	9.38	33.	33.	-0.58	-8.40	22.12	-0.06	-7.28	8.90
1	3	14.08	33.	33.	-0.62	-11.01	17.01	-0.07	-8.93	6.85

1	4	18.77	33.	33.	-0.74	-13.16	20.19	-0.08	-10.70	8.22
1	5	23.46	33.	33.	-0.81	-14.59	22.28	-0.09	-11.89	9.13
1	6	28.15	33.	33.	-0.83	-15.26	23.10	-0.09	-12.49	9.59
1	7	32.85	33.	33.	-0.83	-15.27	23.11	-0.09	-12.50	9.60
1	8	37.54	33.	33.	-0.81	-14.59	22.28	-0.09	-11.89	9.13
1	9	42.23	33.	33.	-0.74	-13.16	20.19	-0.08	-10.70	8.21
1	10	46.92	33.	33.	-0.62	-11.00	17.01	-0.07	-8.92	6.85
1	11	51.62	33.	33.	-0.58	-8.40	22.11	-0.06	-7.27	8.89
1	12	56.31	33.	33.	-0.32	-4.60	12.18	-0.03	-3.97	4.86
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.43	-5.87	16.12	-0.05	-5.03	6.27
1	2	9.38	33.	33.	-0.78	-10.74	29.31	-0.08	-9.22	11.51
1	3	14.08	33.	33.	-0.84	-14.14	22.55	-0.10	-11.34	8.87
1	4	18.77	33.	33.	-1.00	-16.90	26.76	-0.11	-13.59	10.64
1	5	23.46	33.	33.	-1.10	-18.73	29.52	-0.13	-15.10	11.82
1	6	28.15	33.	33.	-1.13	-19.59	30.60	-0.13	-15.86	12.41
1	7	32.85	33.	33.	-1.13	-19.61	30.61	-0.13	-15.88	12.42
1	8	37.54	33.	33.	-1.10	-18.74	29.53	-0.13	-15.10	11.82
1	9	42.23	33.	33.	-1.00	-16.89	26.76	-0.11	-13.59	10.63
1	10	46.92	33.	33.	-0.84	-14.13	22.54	-0.10	-11.32	8.86
1	11	51.62	33.	33.	-0.78	-10.74	29.30	-0.08	-9.22	11.51
1	12	56.31	33.	33.	-0.43	-5.88	16.13	-0.05	-5.04	6.29
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.41	-4.34	14.00	-0.02	-3.50	4.15
1	2	9.38	33.	33.	-0.74	-7.94	25.41	-0.04	-6.42	7.62
1	3	14.08	33.	33.	-0.80	-10.66	19.54	-0.05	-7.86	5.86
1	4	18.77	33.	33.	-0.94	-12.73	23.15	-0.06	-9.42	7.03
1	5	23.46	33.	33.	-1.04	-14.09	25.51	-0.07	-10.46	7.81
1	6	28.15	33.	33.	-1.07	-14.72	26.39	-0.07	-10.99	8.20
1	7	32.85	33.	33.	-1.07	-14.73	26.40	-0.07	-11.00	8.21
1	8	37.54	33.	33.	-1.04	-14.10	25.52	-0.07	-10.47	7.81
1	9	42.23	33.	33.	-0.94	-12.72	23.15	-0.06	-9.42	7.03
1	10	46.92	33.	33.	-0.80	-10.65	19.53	-0.05	-7.85	5.86
1	11	51.62	33.	33.	-0.74	-7.93	25.41	-0.04	-6.41	7.62
1	12	56.31	33.	33.	-0.41	-4.35	14.01	-0.02	-3.51	4.16
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.34	-5.67	13.87	-0.05	-5.03	6.27
1	2	9.38	33.	33.	-0.62	-10.40	25.24	-0.08	-9.22	11.51
1	3	14.08	33.	33.	-0.67	-13.50	19.42	-0.10	-11.34	8.87
1	4	18.77	33.	33.	-0.80	-16.14	23.07	-0.11	-13.59	10.64
1	5	23.46	33.	33.	-0.88	-17.90	25.47	-0.13	-15.10	11.82
1	6	28.15	33.	33.	-0.90	-18.74	26.44	-0.13	-15.86	12.41
1	7	32.85	33.	33.	-0.90	-18.75	26.45	-0.13	-15.88	12.42
1	8	37.54	33.	33.	-0.88	-17.91	25.48	-0.13	-15.10	11.82
1	9	42.23	33.	33.	-0.80	-16.14	23.07	-0.11	-13.59	10.63
1	10	46.92	33.	33.	-0.67	-13.49	19.41	-0.10	-11.32	8.86
1	11	51.62	33.	33.	-0.62	-10.39	25.23	-0.08	-9.22	11.51
1	12	56.31	33.	33.	-0.34	-5.69	13.88	-0.05	-5.04	6.29
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.69	33.	33.	-0.32	-4.15	11.75	-0.02	-3.50	4.15
1	2	9.38	33.	33.	-0.58	-7.59	21.35	-0.04	-6.42	7.62
1	3	14.08	33.	33.	-0.63	-10.02	16.41	-0.05	-7.86	5.86
1	4	18.77	33.	33.	-0.74	-11.97	19.47	-0.06	-9.42	7.03
1	5	23.46	33.	33.	-0.81	-13.26	21.47	-0.07	-10.46	7.81
1	6	28.15	33.	33.	-0.84	-13.87	22.24	-0.07	-10.99	8.20
1	7	32.85	33.	33.	-0.84	-13.88	22.24	-0.07	-11.00	8.21
1	8	37.54	33.	33.	-0.81	-13.27	21.47	-0.07	-10.47	7.81
1	9	42.23	33.	33.	-0.74	-11.97	19.47	-0.06	-9.42	7.03
1	10	46.92	33.	33.	-0.63	-10.01	16.41	-0.05	-7.85	5.86
1	11	51.62	33.	33.	-0.58	-7.59	21.34	-0.04	-6.41	7.62
1	12	56.31	33.	33.	-0.32	-4.15	11.76	-0.02	-3.51	4.16
1	13	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT	DL1, N=INF. MOM.	STRESS	DL2, N=27.0 MOM.	STRESS	POS. LL+I MOM.	STRESS	NEG. LL+I MOM.	STRESS	TOTAL STRESS --AND-- STRESS

		(ft)	(k-ft)	(ksi)	(k-ft)	(ksi)	(k-ft)	(ksi)	(k-ft)	(ksi)	RANGE
1	LT	9.50	221.	-6.5	111.	-0.8	513.	-0.9	0.	0.0	-8.22
											0.88
	LB			6.5		2.5		10.3		0.0	19.23
											10.25
	RT			-5.8		-0.8		-1.2		0.0	-7.82
											1.21
	RB			3.7		1.4		5.9		0.0	10.97
											5.90
1	LT	51.50	220.	-5.8	111.	-0.8	513.	-1.2	0.	0.0	-7.82
											1.21
	LB			3.7		1.4		5.9		0.0	10.97
											5.90
	RT			-6.5		-0.8		-0.9		0.0	-8.21
											0.88
	RB			6.5		2.5		10.3		0.0	19.22
											10.25

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	9.50	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	42.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	13.00	1.0000
3	9.50	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 61.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1]) [3]= GIRDER D. [3]/(12*[1])		<=>	LIMITATION 1/25 1/30	STATUS
1	9.50	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK
				[3]=	33.09	0.04520	>	0.03300	OK
2	42.00	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK
				[3]=	33.09	0.04520	>	0.03300	OK
3	9.50	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK
				[3]=	33.09	0.04520	>	0.03300	OK

```
 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                            Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                          [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  1   4.69  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  2   9.38  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  3  14.08  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  4  18.77  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  5  23.46  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  6  28.15  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  7  32.85  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  8  37.54  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  9  42.23  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1 10  46.92  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1 11  51.62  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1 12  56.31  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1 13  61.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0


-------------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE  1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
                  *********************************

 SP IN D FROM  Lo Co       Web                   Web
                                                 Page 25
```

NO	NO	L SUPT	ng [1]	mp	depth	thick	2Dc/tw [2]	[3]	Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	4.69	0	1	31.39	0.579	54.21	168.97	2
1	2	9.38	0	1	31.39	0.579	54.21	168.97	2
1	3	14.08	0	1	31.39	0.579	69.30	168.97	2
1	4	18.77	0	1	31.39	0.579	69.30	168.97	2
1	5	23.46	0	1	31.39	0.579	69.30	168.97	2
1	6	28.15	0	1	31.39	0.579	69.30	168.97	2
1	7	32.85	0	1	31.39	0.579	69.30	168.97	2
1	8	37.54	0	1	31.39	0.579	69.30	168.97	2
1	9	42.23	0	1	31.39	0.579	69.30	168.97	2
1	10	46.92	0	1	31.39	0.579	69.30	168.97	2
1	11	51.62	0	1	31.39	0.579	54.21	168.97	2
1	12	56.31	0	1	31.39	0.579	54.21	168.97	2
1	13	61.00	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	1	4.69	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	2	9.38	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	3	14.08	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	4	18.77	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	5	23.46	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	6	28.15	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	7	32.85	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	8	37.54	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	9	42.23	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	10	46.92	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	11	51.62	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	12	56.31	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	13	61.00	6.76	12.	11.5	5.2	0.850	0.637	
			6.76	12.	11.5	5.2	0.850	0.637	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. FROM LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	11.5	0.000	10.624	6.288	1.952	1.567
			0.000	0.000	0.000	0.000	1.000
2	11.5	33.1	10.624	16.385	15.235	14.085	1.069
			0.000	0.000	0.000	0.000	1.000
3	33.1	54.7	6.818	16.385	13.225	10.066	1.218
			0.000	0.000	0.000	0.000	1.000
4	54.7	61.0	0.000	6.818	3.565	0.313	1.702
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	FLB CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	1		4.69	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	2		9.38	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	3		14.08	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	4		18.77	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	5		23.46	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	6		28.15	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	7		32.85	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	8		37.54	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	9		42.23	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	10		46.92	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	11		51.62	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	12		56.31	1	6.76	11.26	19.84	0	6.31	7.17	26.91	0	0
1	13		61.00	1	6.76	11.26	19.84	0	6.31	7.17	26.91	0	0

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length

```
       Lp = limiting unbraced length to achieve the nominal flexural
            of RbRhFyc under uniform bending
          = 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
       Lr = limiting unbraced length to achieve the onset of
            nominal yielding in either flange under uniform
            bending with consideration of comp. flange
            residual stress effects
          = pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

       In negative moment region, the fist line is for non-composite sections
       and the second line is for composite sections

       Flange Category
        0 = compact section
        2 = non-compact section
        3 = slender section

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 TABLE  1.2.22.7B=FLB AND LTB RESISTANCE
                  **********************

 SP IN D FROM  Co                        FLB       LTB       GOV
 NO NO L SUPT  mp  Rh    Rb    Cb        Fnc       Fnc       Fnc
         (ft)                            (Ksi)     (Ksi)     (Ksi)
-----------------------------------------------------------------

  1  0   0.00  1 1.000 1.000  1.57     33.00     33.00     33.00
  1  1   4.69  1 1.000 1.000  1.57     33.00     33.00     33.00
  1  2   9.38  1 1.000 1.000  1.57     33.00     33.00     33.00
  1  3  14.08  1 1.000 1.000  1.07     33.00     27.17     27.17
  1  4  18.77  1 1.000 1.000  1.07     33.00     27.17     27.17
  1  5  23.46  1 1.000 1.000  1.07     33.00     27.17     27.17
  1  6  28.15  1 1.000 1.000  1.07     33.00     27.17     27.17
  1  7  32.85  1 1.000 1.000  1.07     33.00     27.17     27.17
  1  8  37.54  1 1.000 1.000  1.22     33.00     30.96     30.96
  1  9  42.23  1 1.000 1.000  1.22     33.00     30.96     30.96
  1 10  46.92  1 1.000 1.000  1.22     33.00     30.96     30.96
  1 11  51.62  1 1.000 1.000  1.22     33.00     31.39     31.39
  1 12  56.31  1 1.000 1.000  1.70     33.00     33.00     33.00
  1 13  61.00  1 1.000 1.000  1.70     33.00     33.00     33.00


-----------------------------------------------------------------

 Note: In the positive moment region, the result is for DL case
       In the negative moment region, the 1st line is for DL case
       and the 2nd line is for LL case
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 TABLE  1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
                  ******************************

 SP IN D FROM  Co
 NO NO L SUPT  mp     Dp       0.42Dt
         (ft)        (in)       (in)
-------------------------------------

  1  0   0.00  1       5.17      17.32
  1  1   4.69  1       5.17      17.32
  1  2   9.38  1       5.17      17.32
  1  3  14.08  1       8.20      17.74
  1  4  18.77  1       8.20      17.74
  1  5  23.46  1       8.20      17.74
  1  6  28.15  1       8.20      17.74
  1  7  32.85  1       8.20      17.74
  1  8  37.54  1       8.20      17.74
  1  9  42.23  1       8.20      17.74
  1 10  46.92  1       8.20      17.74
```

```
 1 11  51.62  1       5.17      17.32
 1 12  56.31  1       5.17      17.32
 1 13  61.00  1       5.17      17.32
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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	4.69	1	0	2	0		2	0
1	2	9.38	1	0	2	0		2	0
1	3	14.08	1	0	2	0		2	0
1	4	18.77	1	0	2	0		2	0
1	5	23.46	1	0	2	0		2	0
1	6	28.15	1	0	2	0		2	0
1	7	32.85	1	0	2	0		2	0
1	8	37.54	1	0	2	0		2	0
1	9	42.23	1	0	2	0		2	0
1	10	46.92	1	0	2	0		2	0
1	11	51.62	1	0	2	0		2	0
1	12	56.31	1	0	0	0		0	0
1	13	61.00	1	0	0	0		0	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

1

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	4.69	0.0	19.8	5.3	33.0	5.3	33.0	5.3	33.0	0
			0.0	-	5.3	-	5.3	33.0	-	-	0
1	2	9.38	0.0	19.8	9.7	33.0	9.7	33.0	9.7	33.0	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	3	14.08	0.0	19.8	11.7	33.0	11.7	33.0	11.7	27.2	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	4	18.77	0.0	19.8	14.1	33.0	14.1	33.0	14.1	27.2	0
			0.0	-	8.9	-	8.9	33.0	-	-	0
1	5	23.46	0.0	19.8	15.6	33.0	15.6	33.0	15.6	27.2	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	6	28.15	0.0	19.8	16.4	33.0	16.4	33.0	16.4	27.2	0
			0.0	-	10.4	-	10.4	33.0	-	-	0
1	7	32.85	0.0	19.8	16.4	33.0	16.4	33.0	16.4	27.2	0
			0.0	-	10.4	-	10.4	33.0	-	-	0
1	8	37.54	0.0	19.8	15.6	33.0	15.6	33.0	15.6	31.0	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	9	42.23	0.0	19.8	14.1	33.0	14.1	33.0	14.1	31.0	0
			0.0	-	8.9	-	8.9	33.0	-	-	0
1	10	46.92	0.0	19.8	11.7	33.0	11.7	33.0	11.7	31.0	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	11	51.62	0.0	19.8	9.7	33.0	9.7	33.0	9.7	31.4	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	12	56.31	0.0	19.8	5.3	33.0	5.3	33.0	5.3	33.0	0
			0.0	-	5.3	-	5.3	33.0	-	-	0
1	13	61.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	4.69	0.000	0.160	0.160	0.160	0.160	2
					0.160		0.160	3
1	2	9.38	0.000	0.294	0.294	0.294	0.294	2
					0.294		0.294	3
1	3	14.08	0.000	0.356	0.356	0.432	0.432	4

					0.225		0.225	3
1	4	18.77	0.000	0.426	0.426	0.517	0.517	4
					0.270		0.270	3
1	5	23.46	0.000	0.473	0.473	0.575	0.575	4
					0.300		0.300	3
1	6	28.15	0.000	0.497	0.497	0.604	0.604	4
					0.315		0.315	3
1	7	32.85	0.000	0.498	0.498	0.605	0.605	4
					0.316		0.316	3
1	8	37.54	0.000	0.474	0.474	0.505	0.505	4
					0.300		0.300	3
1	9	42.23	0.000	0.426	0.426	0.454	0.454	4
					0.270		0.270	3
1	10	46.92	0.000	0.355	0.355	0.378	0.378	4
					0.225		0.225	3
1	11	51.62	0.000	0.294	0.294	0.309	0.309	4
					0.294		0.294	3
1	12	56.31	0.000	0.161	0.161	0.161	0.161	2
					0.161		0.161	3
1	13	61.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2252.3	-	-	0.0	19.8	-	-	0--0
1	1		4.69	0	726.5	2252.3	-	-	0.0	19.8	-	-	0--0
1	2		9.38	0	1319.1	2252.3	-	-	0.0	19.8	-	-	0--0
1	3		14.08	0	1773.4	3331.5	-	-	0.0	19.8	-	-	0--0
1	4		18.77	0	2102.4	3331.5	-	-	0.0	19.8	-	-	0--0
1	5		23.46	0	2317.7	3331.5	-	-	0.0	19.8	-	-	0--0
1	6		28.15	0	2399.2	3331.5	-	-	0.0	19.8	-	-	0--0
1	7		32.85	0	2399.8	3331.5	-	-	0.0	19.8	-	-	0--0
1	8		37.54	0	2318.0	3331.5	-	-	0.0	19.8	-	-	0--0
1	9		42.23	0	2102.3	3331.5	-	-	0.0	19.8	-	-	0--0
1	10		46.92	0	1772.9	3331.5	-	-	0.0	19.8	-	-	0--0
1	11		51.62	0	1318.9	2252.3	-	-	0.0	19.8	-	-	0--0
1	12		56.31	0	726.9	2252.3	-	-	0.0	19.8	-	-	0--0
1	13		61.00	0	0.0	2252.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)

```
        [4] = (PHI)f * Fnt for non-compact tension flange
              of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
            = (PHI)f * Fnc for comp. flange of composite sections
              in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

        "-" is N.A.

        Under FLAG Column, 0 = OK; 1= NG
        M = Moment; C = Comp. Flange; T = Tension Flange
        D = Ductility

        For negative moment region or non-compact sections in
        positive moment region, the 1st line is for top
        flange and the 2nd line is for bottom flange

        The values of fbu, Mu and fl shall be determined based on
        factored loads, and shall be taken as positive in
        sign in all resistance equations (Art. 6.10.1.6)

        The value of fbu is the actual stress in this table,
        the users can use the maximum value within the
        unbraced length to do their own check
1
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   TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                   ***********************************************

   SP IN D FROM ID Mu+1/3flSxt/       fbu/        fl/ fbu+1/3fl/  MAX.
                           [1]        [2]        [3]        [4]  RAT. GOVN.
   ---------------------------------------------------------------------------

    1  0   0.00  0        0.000                                  0.000     1
    1  1   4.69  0        0.323                                  0.323     1
    1  2   9.38  0        0.586                                  0.586     1
    1  3  14.08  0        0.532                                  0.532     1
    1  4  18.77  0        0.631                                  0.631     1
    1  5  23.46  0        0.696                                  0.696     1
    1  6  28.15  0        0.720                                  0.720     1
    1  7  32.85  0        0.720                                  0.720     1
    1  8  37.54  0        0.696                                  0.696     1
    1  9  42.23  0        0.631                                  0.631     1
    1 10  46.92  0        0.532                                  0.532     1
    1 11  51.62  0        0.586                                  0.586     1
    1 12  56.31  0        0.323                                  0.323     1
    1 13  61.00  0        0.000                                  0.000     1


   ---------------------------------------------------------------------------

   NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
         [2] = (PHI)f * Fnc for comp. flange of composite
               sections in positive flexure (Eq. 6.10.7.2.1-1)
             = (PHI)f * Rh * Fyt for tension flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
         [3] = 0.6*Fyt for composite sections in positive flexure
               or tension flange for composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
             = 0.6*Fyc for comp. flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
         [4] = (PHI)f * Fnt for non-compact tension flange
               of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
             = (PHI)f * Fnc for comp. flange of composite sections
               in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

         The governing number is listed as below.
           1 = (Mu+1/3flSxt) / [1]
           2 = fbu / [2]
           3 = fl / [3]
           4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	4.69	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	9.38	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	14.08	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	18.77	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	23.46	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	28.15	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	32.85	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	37.54	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	42.23	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	46.92	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	51.62	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	56.31	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	61.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	212.2	<	347.9	0	0
1	1	4.69	0	187.8	<	347.9	0	0
1	2	9.38	0	163.9	<	347.9	0	0
1	3	14.08	0	139.9	<	347.9	0	0
1	4	18.77	0	116.6	<	347.9	0	0
1	5	23.46	0	93.7	<	347.9	0	0
1	6	28.15	0	71.1	<	347.9	0	0
1	7	32.85	0	70.8	<	347.9	0	0
1	8	37.54	0	93.8	<	347.9	0	0
1	9	42.23	0	116.7	<	347.9	0	0
1	10	46.92	0	140.0	<	347.9	0	0
1	11	51.62	0	163.6	<	347.9	0	0
1	12	56.31	0	187.9	<	347.9	0	0
1	13	61.00	0	212.3	<	347.9	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN D NO	L FROM SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	212.20	347.87	0	
1	1	4.69	33.	187.85	347.87	0	
1	2	9.38	33.	163.86	347.87	0	
1	3	14.08	33.	139.88	347.87	0	
1	4	18.77	33.	116.62	347.87	0	
1	5	23.46	33.	93.72	347.87	0	
1	6	28.15	33.	71.07	347.87	0	
1	7	32.85	33.	70.81	347.87	0	
1	8	37.54	33.	93.82	347.87	0	
1	9	42.23	33.	116.71	347.87	0	
1	10	46.92	33.	139.97	347.87	0	
1	11	51.62	33.	163.59	347.87	0	
1	12	56.31	33.	187.94	347.87	0	
1	13	61.00	33.	212.29	347.87	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D NO	L FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	BOT. FLANGE [1] ff+fl/2 (ksi)	[2] (ksi)	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	4.69	4.6	-	31.4	12.2	31.4	-	-	00-
1	2	9.38	8.4	-	31.4	22.1	31.4	-	-	00-
1	3	14.08	11.0	-	31.4	17.0	31.4	-	-	00-
1	4	18.77	13.2	-	31.4	20.2	31.4	-	-	00-
1	5	23.46	14.6	-	31.4	22.3	31.4	-	-	00-
1	6	28.15	15.3	-	31.4	23.1	31.4	-	-	00-
1	7	32.85	15.3	-	31.4	23.1	31.4	-	-	00-
1	8	37.54	14.6	-	31.4	22.3	31.4	-	-	00-
1	9	42.23	13.2	-	31.4	20.2	31.4	-	-	00-
1	10	46.92	11.0	-	31.4	17.0	31.4	-	-	00-
1	11	51.62	8.4	-	31.4	22.1	31.4	-	-	00-
1	12	56.31	4.6	-	31.4	12.2	31.4	-	-	00-
1	13	61.00	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient

= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)

where:

Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.69	TR	-0.2	A B B^C^C D E E^	TR	2.9	A B B^C^C D E
1	2	9.38	TR	-0.4	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1		9.50	SCG					
1	3	14.08	TR	-0.8	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	4	18.77	TR	-0.9	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	5	23.46	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	6	28.15	TR	-0.9	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	7	32.85	TR	-0.9	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	8	37.54	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9	42.23	TR	-0.9	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	10	46.92	TR	-0.8	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1		51.50	SCG					
1	11	51.62	TR	-0.4	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	12	56.31	TR	-0.2	A B B^C^C D E E^	TR	2.9	A B B^C^C D E
1	13	61.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.69	TR	-0.1	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	2	9.38	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1		9.50	SCG					
1	3	14.08	TR	-0.4	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	4	18.77	TR	-0.5	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	5	23.46	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	6	28.15	TR	-0.5	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	7	32.85	TR	-0.5	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	8	37.54	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	9	42.23	TR	-0.5	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	10	46.92	TR	-0.4	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1		51.50	SCG					
1	11	51.62	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	12	56.31	TR	-0.1	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	13	61.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	AASHTO E6.10.10.1.2 Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	54.7	.29290E-01	1.6	TR 3.09	3	5.80
1	1	4.69	51.3	.29290E-01	1.5	TR 3.09	3	6.17
1	2	9.38	47.9	.29290E-01	1.4	TR 3.09	3	6.61
1	3	14.08	44.8	.26378E-01	1.2	TR 3.09	3	7.85
1	4	18.77	42.3	.26378E-01	1.1	TR 3.09	3	8.31
1	5	23.46	40.4	.26378E-01	1.1	TR 3.09	3	8.71
1	6	28.15	35.9	.26378E-01	0.9	TR 3.09	3	9.80
1	7	32.85	35.9	.26378E-01	0.9	TR 3.09	3	9.80
1	8	37.54	40.4	.26378E-01	1.1	TR 3.09	3	8.71
1	9	42.23	42.3	.26378E-01	1.1	TR 3.09	3	8.31
1	10	46.92	44.8	.26378E-01	1.2	TR 3.09	3	7.85
1	11	51.62	47.9	.29290E-01	1.4	TR 3.09	3	6.61
1	12	56.31	51.3	.29290E-01	1.5	TR 3.09	3	6.17
1	13	61.00	54.7	.29290E-01	1.6	TR 3.09	3	5.80

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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
           *** If ADTT Single Lane is less than 960, use Fatigue II Table.
1
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                                                                    CODE CHECK
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                  *******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-2     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

  1  0   0.00  27.3 .29290E-01    0.8      TR    1.36        3           5.11
  1  1   4.69  25.7 .29290E-01    0.8      TR    1.36        3           5.44
  1  2   9.38  24.0 .29290E-01    0.7      TR    1.36        3           5.83
  1  3  14.08  22.4 .26378E-01    0.6      TR    1.36        3           6.92
  1  4  18.77  21.2 .26378E-01    0.6      TR    1.36        3           7.33
  1  5  23.46  20.2 .26378E-01    0.5      TR    1.36        3           7.68
  1  6  28.15  17.9 .26378E-01    0.5      TR    1.36        3           8.64
  1  7  32.85  17.9 .26378E-01    0.5      TR    1.36        3           8.64
  1  8  37.54  20.2 .26378E-01    0.5      TR    1.36        3           7.68
  1  9  42.23  21.2 .26378E-01    0.6      TR    1.36        3           7.33
                                                 Page 38
```

```
 1 10  46.92  22.4 .26378E-01    0.6      TR    1.36        3           6.92
 1 11  51.62  24.0 .29290E-01    0.7      TR    1.36        3           5.83
 1 12  56.31  25.7 .29290E-01    0.8      TR    1.36        3           5.44
 1 13  61.00  27.3 .29290E-01    0.8      TR    1.36        3           5.11


 ------------------------------------------------------------------------------
 Please read NOTE on the following page

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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

            * Default ALPHA value based on 7/8" diameter and input road type


        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
               where ADTT Single Lane= 1140 & No. of Cycles=   31207500
           *** If ADTT Single Lane greater than or equal to 960, refer
               to Fatigue I Table.
1
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------
```

```
1  7  32.85   1 1673.9  1626.6          237         >  85
1 13  61.00   1 1673.9  1626.6          237         >  85


-----------------------------------------------------------------------
Please read NOTE on the Following Page
```



```
    Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the
     section of maximum positive moment and each adjacent
     point of 0.0 moment, or between each adjacent point
     of 0.0 moment and centerline of an interior support
     shall be checked to ensure that adequate connectors
     are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed
     the number given by the formula :

           N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
     Where
       N1 = Number of connectors between points of maximum
            positive moment and each adjacent point of 0.0 moment
       N2 = number of connectors between each adjacent point of
            0.0 moment and the centerline of an interior support
       P = total nominal horizontal shear force
       Qr = factored shear resistance of one shear connector
          = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
       Qn = nominal shear resistance
       (Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of
     maximum positive moment and each adjacent point of 0.0 moment
     shall be the lesser either:

          P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
     or
       P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
     Where
       f'c = specified 28-day compressive strength of the concrete
       bs = effective width of the slab
       bfc = width of compression flange
       bft = width of tension flange
       ts = slab thickness
       Fyw = specified minimum yield strength of the web
       Fyt = specified minimum yield strength of the tension flange
       Fyc = specified minimum yield strength of the compression flange
       D = web depth
       tft = thickness of tension flange
       tfc = thickness of compression flange
       tw = web thickness


[4] For continuous-span composite sections, the total horizontal
     shear force between each adjacent point of 0.0 moment
     and the centerline of an interior support shall be taken as:

          P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
     Where
       bs = effective width of the concrete deck
       ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
    If ADTT Single Lane is less than 960, the pitch should use Fatigue II.
```

TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	32.846	5.000
1	32.846	61.000	5.000

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	61.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	NEG. MAX. STRESS	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2252.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	4.69	2252.3	119.1	60.1	281.6	0.0	5.31	HL-93	4.10	0
1	2	9.38	2252.3	218.7	110.2	508.8	0.0	2.65	HL-93	2.05	0
1	3	14.08	3331.5	298.0	150.3	679.4	0.0	2.99	HL-93	2.31	0
1	4	18.77	3331.5	357.1	180.3	801.0	0.0	2.43	HL-93	1.88	0
1	5	23.46	3331.5	396.7	200.4	879.5	0.0	2.15	HL-93	1.66	0
1	6	28.15	3331.5	416.8	210.4	903.7	0.0	2.06	HL-93	1.59	0
1	7	32.85	3331.5	417.3	210.4	903.7	0.0	2.06	HL-93	1.59	0
1	8	37.54	3331.5	397.0	200.4	879.5	0.0	2.15	HL-93	1.66	0
1	9	42.23	3331.5	357.0	180.3	801.0	0.0	2.43	HL-93	1.88	0
1	10	46.92	3331.5	297.5	150.3	679.4	0.0	3.00	HL-93	2.31	0
1	11	51.62	2252.3	218.5	110.2	508.8	0.0	2.66	HL-93	2.05	0
1	12	56.31	2252.3	119.5	60.1	281.6	0.0	5.31	HL-93	4.10	0
1	13	61.00	2252.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR

1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	4.69	31.4	-3.96	4.85	-0.48	5.63	0.00	0.00	4.71	HL-93	3.62
1	2	9.38	31.4	-7.28	8.90	-0.87	10.17	0.00	0.00	2.21	HL-93	1.70
1	3	14.08	31.4	-8.93	6.85	-1.60	7.82	0.00	0.00	3.13	HL-93	2.41
1	4	18.77	31.4	-10.70	8.22	-1.89	9.21	0.00	0.00	2.51	HL-93	1.93
1	5	23.46	31.4	-11.89	9.13	-2.08	10.12	0.00	0.00	2.20	HL-93	1.69
1	6	28.15	31.4	-12.49	9.59	-2.13	10.40	0.00	0.00	2.09	HL-93	1.61
1	7	32.85	31.4	-12.50	9.60	-2.13	10.40	0.00	0.00	2.09	HL-93	1.61
1	8	37.54	31.4	-11.89	9.13	-2.08	10.12	0.00	0.00	2.20	HL-93	1.69
1	9	42.23	31.4	-10.70	8.21	-1.89	9.21	0.00	0.00	2.51	HL-93	1.93
1	10	46.92	31.4	-8.92	6.85	-1.60	7.82	0.00	0.00	3.14	HL-93	2.41
1	11	51.62	31.4	-7.27	8.89	-0.87	10.17	0.00	0.00	2.21	HL-93	1.70
1	12	56.31	31.4	-3.97	4.86	-0.48	5.63	0.00	0.00	4.71	HL-93	3.62
1	13	61.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	32.85	STRENGTH L.S.	2.06	1.59	HL-93
1	61.00	32.85	SERVICEBILITY	2.09	1.61	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.060
Inventory= 1.589

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN CRITICAL LOCATION GOVERNING
RATING LEGAL LOAD MAXIMUM

5969 Span 1 Interior Strength CP.RES

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	LL MOMENT (k-ft)
1	61.00	37.54	STRENGTH L.S.	2.93	871.85
1	61.00	9.38	SERVICEBILITY	2.71	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.711

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	27.5	13.9	90.4	0.0	2.41	HL-93	1.86
1	1	4.69	347.9	23.3	11.7	81.2	-3.0	2.76	HL-93	2.13
1	2	9.38	347.9	19.1	9.6	72.2	-7.6	3.18	HL-93	2.46
1	3	14.08	347.9	14.7	7.5	63.4	-12.8	3.73	HL-93	2.87
1	4	18.77	347.9	10.5	5.3	54.8	-18.1	4.42	HL-93	3.41
1	5	23.46	347.9	6.4	3.2	46.4	-23.6	5.35	HL-93	4.13
1	6	28.15	347.9	2.2	1.1	38.2	-30.4	6.67	HL-93	5.14
1	7	32.85	347.9	-2.0	-1.1	30.4	-38.2	6.67	HL-93	5.15
1	8	37.54	347.9	-6.4	-3.2	23.6	-46.4	5.35	HL-93	4.13
1	9	42.23	347.9	-10.6	-5.3	18.1	-54.8	4.42	HL-93	3.41
1	10	46.92	347.9	-14.8	-7.5	12.8	-63.4	3.72	HL-93	2.87
1	11	51.62	347.9	-18.9	-9.6	7.6	-72.2	3.19	HL-93	2.46
1	12	56.31	347.9	-23.4	-11.7	3.0	-81.2	2.75	HL-93	2.13
1	13	61.00	347.9	-27.5	-13.9	0.0	-90.4	2.41	HL-93	1.86

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	61.00	STRENGTH L.S.	2.41	1.86	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.407
Inventory= 1.857

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH	CRITICAL LOCATION D FROM L SUPT	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR

```
       (ft)         (ft)                                    (kips)
 ----------------------------------------------------------------

   1    61.00        61.00       STRENGTH L.S.  2.92        100.61

 ---------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
                Legal Load Rating= 2.921
1
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                          UNIVERSITY OF MARYLAND                    LRF -- 2010
                                                                    RATING
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 TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
                ****************************

                  NUMBER OF LANE   LL + I.
     SPAN  D FROM -----AND------ DEFLECTION GOVERN. RATING FACTOR FOR LL DEFL.
      NO.  L SUPT  DIST. FACTOR  ----------  LOAD   --------------------------
            (ft)   FOR LL DEFL.     (inch)   TYPE      RF = L/(800 * LL DEFL.)
     ---------------------------------------------------------------------------

       1    30.50    3   0.425       -0.35    HL-93            2.63

                     4   0.433       -0.36                     2.58

     ---------------------------------------------------------------------------
     NOTE: [1] "-" indicates downward deflection

           [2] RF * (live load defl.) = span length/800, therefore

               RF = L/(800 * LL Defl.), if = 99, No rating
```

```
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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span 1 Interior Fat and Mid M	8/20/201

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5939	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	20	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

1

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.24	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO. ID.	STANDARD SECTN NOMINAL DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130		0.00	0.0000	0.00	0.0000		
2	W	33	130		0.00	0.000013	.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

1

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB TYPE 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	9.50				33.	33.	33.
2	2	2	42.00				33.	33.	33.
3	1	1	9.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.40	0.00	15.00	1 1 1 1 1	0.65	2 2 2 2 2
1	0.00	0	0.00	0.63	0.00	0.00	0 0 0 0 0	0.90	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

1

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	61.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	11.51
2	2	Diaphragm	0.000	0.00	0.00	0.29	33.10
3	2	Diaphragm	0.000	0.00	0.00	0.29	54.69
4	1	Railing	0.087	0.00	61.00	0.00	0.00
5	1	Curb	0.076	0.00	61.00	0.00	0.00
6	0	Pavement	0.292	0.00	61.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	DIST. FROM TOP (in**2)	(in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	11.51	33.0	33.0	11.51
11.51	33.10	33.0	33.0	21.59
33.10	54.69	33.0	33.0	21.59
54.69	61.00	33.0	33.0	6.31

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.727	0.1608	0.8878

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	61.00
1	DC2	0.087	0.000	61.000
2	DC2	0.076	0.000	61.000

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
	V O L U M E		TOTAL	TOTAL	
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	WEIGHT (kip)	WEIGHT (kip)	(kip)
150.00	1481.2	54.9	222.2	58.8	281.01

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.654	0.654	0.702	0.702	FOR STRENGTH POSITIVE MOMENT
	0.899				FOR STRENGTH POSITIVE SHEAR
1	0.399				FOR FATIGUE POSITIVE MOMENT
	0.628				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.05	6710.0	33.1	16.55	405.6	405.6
1	2	6.10	6710.0	33.1	16.55	405.6	405.6
1	3	9.15	6710.0	33.1	16.55	405.6	405.6
1	4	12.20	9539.7	33.1	13.22	721.4	457.2
1	5	15.25	9539.7	33.1	13.22	721.4	457.2
1	6	18.30	9539.7	33.1	13.22	721.4	457.2
1	7	21.35	9539.7	33.1	13.22	721.4	457.2
1	8	24.40	9539.7	33.1	13.22	721.4	457.2

1	9	27.45	9539.7	33.1	13.22	721.4	457.2
1	10	30.50	9539.7	33.1	13.22	721.4	457.2
1	11	33.55	9539.7	33.1	13.22	721.4	457.2
1	12	36.60	9539.7	33.1	13.22	721.4	457.2
1	13	39.65	9539.7	33.1	13.22	721.4	457.2
1	14	42.70	9539.7	33.1	13.22	721.4	457.2
1	15	45.75	9539.7	33.1	13.22	721.4	457.2
1	16	48.80	9539.7	33.1	13.22	721.4	457.2
1	17	51.85	6710.0	33.1	16.55	405.6	405.6
1	18	54.90	6710.0	33.1	16.55	405.6	405.6
1	19	57.95	6710.0	33.1	16.55	405.6	405.6
1	20	61.00	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

1

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L (ft)	SUPT	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00		13532.1	0.023717	544.5	1642.8	826.3
1	1	3.05		13532.1	0.023717	544.5	1642.8	826.3
1	2	6.10		13532.1	0.023717	544.5	1642.8	826.3
1	3	9.15		13532.1	0.023717	544.5	1642.8	826.3
1	4	12.20		20472.1	0.021012	951.0	1629.4	988.8
1	5	15.25		20472.1	0.021012	951.0	1629.4	988.8
1	6	18.30		20472.1	0.021012	951.0	1629.4	988.8
1	7	21.35		20472.1	0.021012	951.0	1629.4	988.8
1	8	24.40		20472.1	0.021012	951.0	1629.4	988.8
1	9	27.45		20472.1	0.021012	951.0	1629.4	988.8
1	10	30.50		20472.1	0.021012	951.0	1629.4	988.8
1	11	33.55		20472.1	0.021012	951.0	1629.4	988.8
1	12	36.60		20472.1	0.021012	951.0	1629.4	988.8
1	13	39.65		20472.1	0.021012	951.0	1629.4	988.8
1	14	42.70		20472.1	0.021012	951.0	1629.4	988.8
1	15	45.75		20472.1	0.021012	951.0	1629.4	988.8
1	16	48.80		20472.1	0.021012	951.0	1629.4	988.8
1	17	51.85		13532.1	0.023717	544.5	1642.8	826.3
1	18	54.90		13532.1	0.023717	544.5	1642.8	826.3
1	19	57.95		13532.1	0.023717	544.5	1642.8	826.3
1	20	61.00		13532.1	0.023717	544.5	1642.8	826.3

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO IN NO D FROM L SUPT (ft) | MOMENT OF INERTIA Ix (in**4) | Q/Ix Q=1ST. MOMENT OF INERTIA (1/in) | ELASTIC SECTION MODULUS,(in**3) STEEL BOT. | STEEL TOP. | CONCRETE(SLAB) TOP.

1	0	0.00	18306.4	0.029290	600.5	7031.5	1704.0
1	1	3.05	18306.4	0.029290	600.5	7031.5	1704.0
1	2	6.10	18306.4	0.029290	600.5	7031.5	1704.0
1	3	9.15	18306.4	0.029290	600.5	7031.5	1704.0
1	4	12.20	29510.1	0.026378	1043.2	5086.0	2116.6
1	5	15.25	29510.1	0.026378	1043.2	5086.0	2116.6
1	6	18.30	29510.1	0.026378	1043.2	5086.0	2116.6
1	7	21.35	29510.1	0.026378	1043.2	5086.0	2116.6
1	8	24.40	29510.1	0.026378	1043.2	5086.0	2116.6
1	9	27.45	29510.1	0.026378	1043.2	5086.0	2116.6
1	10	30.50	29510.1	0.026378	1043.2	5086.0	2116.6
1	11	33.55	29510.1	0.026378	1043.2	5086.0	2116.6
1	12	36.60	29510.1	0.026378	1043.2	5086.0	2116.6
1	13	39.65	29510.1	0.026378	1043.2	5086.0	2116.6
1	14	42.70	29510.1	0.026378	1043.2	5086.0	2116.6
1	15	45.75	29510.1	0.026378	1043.2	5086.0	2116.6
1	16	48.80	29510.1	0.026378	1043.2	5086.0	2116.6
1	17	51.85	18306.4	0.029290	600.5	7031.5	1704.0
1	18	54.90	18306.4	0.029290	600.5	7031.5	1704.0
1	19	57.95	18306.4	0.029290	600.5	7031.5	1704.0
1	20	61.00	18306.4	0.029290	600.5	7031.5	1704.0

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.05	14.2	64.2	1.2	0.0		79.7
1	2	6.10	26.9	121.7	2.4	0.0		151.1

1	3	9.15	38.1	172.5	3.6	0.0	214.2
1	4	12.20	47.9	216.4	4.6	0.0	268.9
1	5	15.25	56.1	253.6	4.9	0.0	314.6
1	6	18.30	62.8	284.0	5.3	0.0	352.1
1	7	21.35	68.0	307.7	5.6	0.0	381.3
1	8	24.40	71.8	324.6	5.9	0.0	402.3
1	9	27.45	74.0	334.8	6.2	0.0	415.0
1	10	30.50	74.8	338.1	6.6	0.0	419.5
1	11	33.55	74.0	334.8	6.8	0.0	415.5
1	12	36.60	71.8	324.6	6.2	0.0	402.6
1	13	39.65	68.0	307.7	5.7	0.0	381.4
1	14	42.70	62.8	284.0	5.1	0.0	352.0
1	15	45.75	56.1	253.6	4.6	0.0	314.2
1	16	48.80	47.9	216.4	4.0	0.0	268.3
1	17	51.85	38.1	172.5	3.5	0.0	214.0
1	18	54.90	26.9	121.7	2.8	0.0	151.5
1	19	57.95	14.2	64.2	1.4	0.0	79.9
1	20	61.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.05	25.8	14.4	0.0	0.0	0.0	40.2
1	2	6.10	48.9	27.3	0.0	0.0	0.0	76.2
1	3	9.15	69.3	38.7	0.0	0.0	0.0	107.9
1	4	12.20	86.9	48.5	0.0	0.0	0.0	135.4
1	5	15.25	101.9	56.9	0.0	0.0	0.0	158.7
1	6	18.30	114.1	63.7	0.0	0.0	0.0	177.8
1	7	21.35	123.6	69.0	0.0	0.0	0.0	192.6
1	8	24.40	130.4	72.8	0.0	0.0	0.0	203.2
1	9	27.45	134.5	75.1	0.0	0.0	0.0	209.5
1	10	30.50	135.8	75.8	0.0	0.0	0.0	211.6
1	11	33.55	134.5	75.1	0.0	0.0	0.0	209.5
1	12	36.60	130.4	72.8	0.0	0.0	0.0	203.2
1	13	39.65	123.6	69.0	0.0	0.0	0.0	192.6
1	14	42.70	114.1	63.7	0.0	0.0	0.0	177.8
1	15	45.75	101.9	56.9	0.0	0.0	0.0	158.7
1	16	48.80	86.9	48.5	0.0	0.0	0.0	135.4
1	17	51.85	69.3	38.7	0.0	0.0	0.0	107.9
1	18	54.90	48.9	27.3	0.0	0.0	0.0	76.2
1	19	57.95	25.8	14.4	0.0	0.0	0.0	40.2
1	20	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE

1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.05	0.0	0.0	189.2	HL-93	0.0	HL-93
1	2	6.10	0.0	0.0	355.5	HL-93	0.0	HL-93
1	3	9.15	0.0	0.0	498.7	HL-93	0.0	HL-93
1	4	12.20	0.0	0.0	618.9	HL-93	0.0	HL-93
1	5	15.25	0.0	0.0	713.7	HL-93	0.0	HL-93
1	6	18.30	0.0	0.0	790.4	HL-93	0.0	HL-93
1	7	21.35	0.0	0.0	850.2	HL-93	0.0	HL-93
1	8	24.40	0.0	0.0	889.4	HL-93	0.0	HL-93
1	9	27.45	0.0	0.0	902.0	HL-93	0.0	HL-93
1	10	30.50	0.0	0.0	906.2	HL-93	0.0	HL-93
1	11	33.55	0.0	0.0	902.0	HL-93	0.0	HL-93
1	12	36.60	0.0	0.0	889.4	HL-93	0.0	HL-93
1	13	39.65	0.0	0.0	850.2	HL-93	0.0	HL-93
1	14	42.70	0.0	0.0	790.4	HL-93	0.0	HL-93
1	15	45.75	0.0	0.0	713.7	HL-93	0.0	HL-93
1	16	48.80	0.0	0.0	618.9	HL-93	0.0	HL-93
1	17	51.85	0.0	0.0	498.7	HL-93	0.0	HL-93
1	18	54.90	0.0	0.0	355.5	HL-93	0.0	HL-93
1	19	57.95	0.0	0.0	189.2	HL-93	0.0	HL-93
1	20	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.05	66.	0.	66.
1	2	6.10	121.	0.	121.
1	3	9.15	165.	0.	165.
1	4	12.20	202.	0.	202.
1	5	15.25	232.	0.	232.
1	6	18.30	255.	0.	255.
1	7	21.35	271.	0.	271.
1	8	24.40	277.	0.	277.
1	9	27.45	262.	0.	262.
1	10	30.50	258.	0.	258.
1	11	33.55	262.	0.	262.
1	12	36.60	277.	0.	277.
1	13	39.65	271.	0.	271.
1	14	42.70	255.	0.	255.
1	15	45.75	232.	0.	232.
1	16	48.80	202.	0.	202.
1	17	51.85	165.	0.	165.
1	18	54.90	121.	0.	121.
1	19	57.95	66.	0.	66.
1	20	61.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT	DEAD LOAD NON COMP. N=Infin.	COMP. N=27.0	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE	GOVERN LOAD	MAXIMUM NEGATIVE	GOVERN LOAD	TOTAL MAXIMUM POSITIVE	NEGATIVE

		(ft)	(k-ft)	(k-ft)	(k-ft)	TYPE	(k-ft)	TYPE	(k-ft)	(k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.05	79.7	40.2	189.2	HL-93	0.0	HL-93	309.1	119.9
1	2	6.10	151.1	76.2	355.5	HL-93	0.0	HL-93	582.7	227.2
1	3	9.15	214.2	107.9	498.7	HL-93	0.0	HL-93	820.8	322.1
1	4	12.20	268.9	135.4	618.9	HL-93	0.0	HL-93	1023.2	404.3
1	5	15.25	314.6	158.7	713.7	HL-93	0.0	HL-93	1187.1	473.3
1	6	18.30	352.1	177.8	790.4	HL-93	0.0	HL-93	1320.3	529.9
1	7	21.35	381.3	192.6	850.2	HL-93	0.0	HL-93	1424.1	573.9
1	8	24.40	402.3	203.2	889.4	HL-93	0.0	HL-93	1494.9	605.5
1	9	27.45	415.0	209.5	902.0	HL-93	0.0	HL-93	1526.5	624.5
1	10	30.50	419.5	211.6	906.2	HL-93	0.0	HL-93	1537.3	631.1
1	11	33.55	415.5	209.5	902.0	HL-93	0.0	HL-93	1527.1	625.1
1	12	36.60	402.6	203.2	889.4	HL-93	0.0	HL-93	1495.2	605.8
1	13	39.65	381.4	192.6	850.2	HL-93	0.0	HL-93	1424.2	574.0
1	14	42.70	352.0	177.8	790.4	HL-93	0.0	HL-93	1320.2	529.7
1	15	45.75	314.2	158.7	713.7	HL-93	0.0	HL-93	1186.7	473.0
1	16	48.80	268.3	135.4	618.9	HL-93	0.0	HL-93	1022.7	403.7
1	17	51.85	214.0	107.9	498.7	HL-93	0.0	HL-93	820.7	322.0
1	18	54.90	151.5	76.2	355.5	HL-93	0.0	HL-93	583.1	227.7
1	19	57.95	79.9	40.2	189.2	HL-93	0.0	HL-93	309.3	120.1
1	20	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN D NO L	FROM SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.05	309.1	365.9	487.4	411.7	179.8	49.2
1	2	6.10	582.7	689.3	918.3	776.1	340.9	90.9
1	3	9.15	820.8	970.4	1292.7	1093.2	483.2	123.8
1	4	12.20	1023.2	1208.9	1610.3	1362.7	606.5	151.5
1	5	15.25	1187.1	1401.2	1866.2	1580.7	710.0	174.4
1	6	18.30	1320.3	1557.4	2074.1	1757.9	794.8	191.3
1	7	21.35	1424.1	1679.2	2236.2	1896.1	860.9	203.1
1	8	24.40	1494.9	1761.7	2345.9	1990.2	908.2	207.4
1	9	27.45	1526.5	1797.1	2392.8	2032.0	936.8	196.9
1	10	30.50	1537.3	1809.2	2408.7	2046.2	946.7	193.4
1	11	33.55	1527.1	1797.7	2393.4	2032.6	937.6	196.9
1	12	36.60	1495.2	1762.0	2346.3	1990.5	908.7	207.4
1	13	39.65	1424.2	1679.3	2236.2	1896.2	861.0	203.1
1	14	42.70	1320.2	1557.3	2073.9	1757.8	794.6	191.3
1	15	45.75	1186.7	1400.8	1865.7	1580.2	709.5	174.4
1	16	48.80	1022.7	1208.3	1609.5	1361.9	605.6	151.5
1	17	51.85	820.7	970.3	1292.5	1093.0	483.0	123.8
1	18	54.90	583.1	689.8	918.9	776.7	341.5	90.9
1	19	57.95	309.3	366.1	487.7	412.0	180.1	49.2
1	20	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

5969 Span 1 Interior Fatigue and Mid M.RES

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL DEAD LOAD (kip)	NONCOMPOSITE
1	0	0.00	4.9	22.2	0.4	0.0	27.5	
1	1	3.05	4.4	20.0	0.4	0.0	24.8	
1	2	6.10	3.9	17.7	0.4	0.0	22.1	
1	3	9.15	3.4	15.5	0.4	0.0	19.3	
1	4	12.20	2.9	13.3	0.1	0.0	16.4	
1	5	15.25	2.5	11.1	0.1	0.0	13.6	
1	6	18.30	2.0	8.9	0.1	0.0	10.9	
1	7	21.35	1.5	6.7	0.1	0.0	8.2	
1	8	24.40	1.0	4.4	0.1	0.0	5.5	
1	9	27.45	0.5	2.2	0.1	0.0	2.8	
1	10	30.50	0.0	0.0	0.1	0.0	0.1	
1	11	33.55	-0.5	-2.2	-0.2	0.0	-2.9	
1	12	36.60	-1.0	-4.4	-0.2	0.0	-5.6	
1	13	39.65	-1.5	-6.7	-0.2	0.0	-8.3	
1	14	42.70	-2.0	-8.9	-0.2	0.0	-11.0	
1	15	45.75	-2.5	-11.1	-0.2	0.0	-13.7	
1	16	48.80	-2.9	-13.3	-0.2	0.0	-16.4	
1	17	51.85	-3.4	-15.5	-0.2	0.0	-19.1	
1	18	54.90	-3.9	-17.7	-0.5	0.0	-22.1	
1	19	57.95	-4.4	-20.0	-0.5	0.0	-24.8	
1	20	61.00	-4.9	-22.2	-0.5	0.0	-27.5	

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	27.5	41.3
1	1	3.05	8.0	4.5	0.0	0.0	12.5	24.8	37.3
1	2	6.10	7.1	4.0	0.0	0.0	11.1	22.1	33.2
1	3	9.15	6.2	3.5	0.0	0.0	9.7	19.3	29.1
1	4	12.20	5.3	3.0	0.0	0.0	8.3	16.4	24.7
1	5	15.25	4.5	2.5	0.0	0.0	6.9	13.6	20.6
1	6	18.30	3.6	2.0	0.0	0.0	5.6	10.9	16.5
1	7	21.35	2.7	1.5	0.0	0.0	4.2	8.2	12.4
1	8	24.40	1.8	1.0	0.0	0.0	2.8	5.5	8.3
1	9	27.45	0.9	0.5	0.0	0.0	1.4	2.8	4.2
1	10	30.50	0.0	0.0	0.0	0.0	0.0	0.1	0.1
1	11	33.55	-0.9	-0.5	0.0	0.0	-1.4	-2.9	-4.3
1	12	36.60	-1.8	-1.0	0.0	0.0	-2.8	-5.6	-8.4
1	13	39.65	-2.7	-1.5	0.0	0.0	-4.2	-8.3	-12.5
1	14	42.70	-3.6	-2.0	0.0	0.0	-5.6	-11.0	-16.6
1	15	45.75	-4.5	-2.5	0.0	0.0	-6.9	-13.7	-20.7
1	16	48.80	-5.3	-3.0	0.0	0.0	-8.3	-16.4	-24.8
1	17	51.85	-6.2	-3.5	0.0	0.0	-9.7	-19.1	-28.8
1	18	54.90	-7.1	-4.0	0.0	0.0	-11.1	-22.1	-33.2
1	19	57.95	-8.0	-4.5	0.0	0.0	-12.5	-24.8	-37.3
1	20	61.00	-8.9	-5.0	0.0	0.0	-13.9	-27.5	-41.4

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	90.4	HL-93	0.0	HL-93
1	1	3.05	0.0	0.0	84.4	HL-93	-2.0	HL-93
1	2	6.10	0.0	0.0	78.5	HL-93	-4.2	HL-93
1	3	9.15	0.0	0.0	72.7	HL-93	-7.4	HL-93
1	4	12.20	0.0	0.0	66.9	HL-93	-10.7	HL-93
1	5	15.25	0.0	0.0	61.3	HL-93	-14.1	HL-93
1	6	18.30	0.0	0.0	55.7	HL-93	-17.5	HL-93
1	7	21.35	0.0	0.0	50.2	HL-93	-21.1	HL-93
1	8	24.40	0.0	0.0	44.8	HL-93	-24.8	HL-93
1	9	27.45	0.0	0.0	39.4	HL-93	-29.3	HL-93
1	10	30.50	0.0	0.0	34.2	HL-93	-34.2	HL-93
1	11	33.55	0.0	0.0	29.3	HL-93	-39.4	HL-93
1	12	36.60	0.0	0.0	24.8	HL-93	-44.8	HL-93
1	13	39.65	0.0	0.0	21.1	HL-93	-50.2	HL-93
1	14	42.70	0.0	0.0	17.5	HL-93	-55.7	HL-93
1	15	45.75	0.0	0.0	14.1	HL-93	-61.3	HL-93
1	16	48.80	0.0	0.0	10.7	HL-93	-66.9	HL-93
1	17	51.85	0.0	0.0	7.4	HL-93	-72.7	HL-93
1	18	54.90	0.0	0.0	4.2	HL-93	-78.5	HL-93
1	19	57.95	0.0	0.0	2.0	HL-93	-84.4	HL-93
1	20	61.00	0.0	0.0	0.0	HL-93	-90.4	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	36.	0.	36.
1	1	3.05	34.	-1.	35.
1	2	6.10	31.	-2.	34.
1	3	9.15	29.	-3.	32.
1	4	12.20	26.	-5.	31.
1	5	15.25	23.	-6.	29.
1	6	18.30	21.	-7.	28.
1	7	21.35	19.	-9.	28.
1	8	24.40	16.	-10.	27.
1	9	27.45	14.	-11.	24.
1	10	30.50	12.	-12.	23.
1	11	33.55	11.	-14.	24.
1	12	36.60	10.	-16.	27.
1	13	39.65	9.	-19.	28.
1	14	42.70	7.	-21.	28.
1	15	45.75	6.	-23.	29.
1	16	48.80	5.	-26.	31.
1	17	51.85	3.	-29.	32.
1	18	54.90	2.	-31.	34.
1	19	57.95	1.	-34.	35.
1	20	61.00	0.	-36.	36.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	36.5	.30737E-01	0.56	0.3125*	.14860E-01	0.27	0.3125*
1	1	3.05	35.0	.30737E-01	0.54	0.3125*	.14860E-01	0.26	0.3125*
1	2	6.10	33.6	.30737E-01	0.52	0.3125*	.14860E-01	0.25	0.3125*
1	3	9.15	32.1	.30737E-01	0.49	0.3125*	.14860E-01	0.24	0.3125*
1	4	12.20	30.7	.28303E-01	0.43	0.3125*	.81917E-02	0.13	0.3125*
1	5	15.25	29.4	.28303E-01	0.42	0.3125*	.81917E-02	0.12	0.3125*
1	6	18.30	28.3	.28303E-01	0.40	0.3125*	.81917E-02	0.12	0.3125*
1	7	21.35	27.6	.28303E-01	0.39	0.3125*	.81917E-02	0.11	0.3125*
1	8	24.40	26.6	.28303E-01	0.38	0.3125*	.81917E-02	0.11	0.3125*
1	9	27.45	24.3	.28303E-01	0.34	0.3125*	.81917E-02	0.10	0.3125*
1	10	30.50	23.5	.28303E-01	0.33	0.3125*	.81917E-02	0.10	0.3125*
1	11	33.55	24.3	.28303E-01	0.34	0.3125*	.81917E-02	0.10	0.3125*
1	12	36.60	26.6	.28303E-01	0.38	0.3125*	.81917E-02	0.11	0.3125*
1	13	39.65	27.6	.28303E-01	0.39	0.3125*	.81917E-02	0.11	0.3125*
1	14	42.70	28.3	.28303E-01	0.40	0.3125*	.81917E-02	0.12	0.3125*
1	15	45.75	29.4	.28303E-01	0.42	0.3125*	.81917E-02	0.12	0.3125*
1	16	48.80	30.7	.28303E-01	0.43	0.3125*	.81917E-02	0.13	0.3125*
1	17	51.85	32.1	.30737E-01	0.49	0.3125*	.14860E-01	0.24	0.3125*
1	18	54.90	33.6	.30737E-01	0.52	0.3125*	.14860E-01	0.25	0.3125*
1	19	57.95	35.0	.30737E-01	0.54	0.3125*	.14860E-01	0.26	0.3125*
1	20	61.00	36.5	.30737E-01	0.56	0.3125*	.14860E-01	0.27	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	41.3	90.4	HL-93	0.0	HL-93	131.8	41.3
1	1	3.05	37.3	84.4	HL-93	-2.0	HL-93	121.7	35.3

1	2	6.10	33.2	78.5	HL-93	-4.2	HL-93	111.7	29.0
1	3	9.15	29.1	72.7	HL-93	-7.4	HL-93	101.7	21.7
1	4	12.20	24.7	66.9	HL-93	-10.7	HL-93	91.6	14.0
1	5	15.25	20.6	61.3	HL-93	-14.1	HL-93	81.8	6.5
1	6	18.30	16.5	55.7	HL-93	-17.5	HL-93	72.2	-1.1
1	7	21.35	12.4	50.2	HL-93	-21.1	HL-93	62.6	-8.7
1	8	24.40	8.3	44.8	HL-93	-24.8	HL-93	53.1	-16.5
1	9	27.45	4.2	39.4	HL-93	-29.3	HL-93	43.6	-25.1
1	10	30.50	0.1	34.2	HL-93	-34.2	HL-93	34.4	-34.1
1	11	33.55	-4.3	29.3	HL-93	-39.4	HL-93	25.0	-43.7
1	12	36.60	-8.4	24.8	HL-93	-44.8	HL-93	16.4	-53.2
1	13	39.65	-12.5	21.1	HL-93	-50.2	HL-93	8.6	-62.7
1	14	42.70	-16.6	17.5	HL-93	-55.7	HL-93	1.0	-72.2
1	15	45.75	-20.7	14.1	HL-93	-61.3	HL-93	-6.6	-81.9
1	16	48.80	-24.8	10.7	HL-93	-66.9	HL-93	-14.1	-91.7
1	17	51.85	-28.8	7.4	HL-93	-72.7	HL-93	-21.5	-101.5
1	18	54.90	-33.2	4.2	HL-93	-78.5	HL-93	-29.0	-111.7
1	19	57.95	-37.3	2.0	HL-93	-84.4	HL-93	-35.4	-121.8
1	20	61.00	-41.4	0.0	HL-93	-90.4	HL-93	-41.4	-131.9

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	131.8	158.9	212.2	176.0	62.0	27.3
1	1	3.05	121.7	147.0	196.3	162.6	55.9	26.3
1	2	6.10	111.7	135.2	180.6	149.2	49.7	25.2
1	3	9.15	101.7	123.5	165.1	136.0	43.6	24.0
1	4	12.20	91.6	111.7	149.3	122.5	37.0	23.0
1	5	15.25	81.8	100.2	134.0	109.5	30.9	22.1
1	6	18.30	72.2	88.9	118.9	96.7	24.7	21.2
1	7	21.35	62.6	77.6	104.0	83.9	18.6	20.7
1	8	24.40	53.1	66.5	89.2	71.3	12.4	19.9
1	9	27.45	43.6	55.4	74.4	58.7	6.3	18.2
1	10	30.50	34.4	44.6	60.1	46.4	0.2	17.6
1	11	33.55	-43.7	-55.5	-74.5	-58.8	-6.4	18.2
1	12	36.60	-53.2	-66.6	-89.3	-71.4	-12.6	19.9
1	13	39.65	-62.7	-77.7	-104.1	-84.0	-18.7	20.7
1	14	42.70	-72.2	-88.9	-119.0	-96.8	-24.8	21.2
1	15	45.75	-81.9	-100.3	-134.1	-109.6	-31.0	22.1
1	16	48.80	-91.7	-111.7	-149.4	-122.6	-37.1	23.0
1	17	51.85	-101.5	-123.3	-164.8	-135.7	-43.3	24.0
1	18	54.90	-111.7	-135.3	-180.7	-149.3	-49.8	25.2
1	19	57.95	-121.8	-147.1	-196.4	-162.6	-56.0	26.3
1	20	61.00	-131.9	-159.0	-212.3	-176.1	-62.1	27.3

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM

1	41.35	0.00	HL-93	90.45	HL-93	ST1	34.99	212.20
						ST2	34.99	176.02
						ST4	48.66	62.02
						SE1	41.35	131.80
						SE2	41.35	158.93
2	41.42	0.00	HL-93	90.45	HL-93	ST1	35.05	212.29
						ST2	35.05	176.11
						ST4	48.77	62.13
						SE1	41.42	131.87
						SE2	41.42	159.01

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.05	-0.0304	-0.1373	0.0000	-0.0405	-0.1703	-0.0405	-0.2108
1	2	6.10	-0.0595	-0.2693	0.0000	-0.0793	-0.3340	-0.0793	-0.4133
1	3	9.15	-0.0865	-0.3913	0.0000	-0.1151	-0.4854	-0.1151	-0.6004
1	4	12.20	-0.1108	-0.5010	0.0000	-0.1470	-0.6213	-0.1470	-0.7684
1	5	15.25	-0.1322	-0.5980	0.0000	-0.1753	-0.7417	-0.1753	-0.9170
1	6	18.30	-0.1504	-0.6804	0.0000	-0.1994	-0.8438	-0.1994	-1.0432
1	7	21.35	-0.1650	-0.7463	0.0000	-0.2186	-0.9255	-0.2186	-1.1441
1	8	24.40	-0.1756	-0.7944	0.0000	-0.2326	-0.9851	-0.2326	-1.2177
1	9	27.45	-0.1821	-0.8236	0.0000	-0.2411	-1.0214	-0.2411	-1.2625
1	10	30.50	-0.1843	-0.8334	0.0000	-0.2440	-1.0335	-0.2440	-1.2775
1	11	33.55	-0.1821	-0.8236	0.0000	-0.2411	-1.0214	-0.2411	-1.2625
1	12	36.60	-0.1756	-0.7944	0.0000	-0.2326	-0.9851	-0.2326	-1.2177
1	13	39.65	-0.1650	-0.7463	0.0000	-0.2186	-0.9255	-0.2186	-1.1441
1	14	42.70	-0.1504	-0.6804	0.0000	-0.1994	-0.8438	-0.1994	-1.0431
1	15	45.75	-0.1322	-0.5980	0.0000	-0.1753	-0.7416	-0.1753	-0.9170
1	16	48.80	-0.1108	-0.5010	0.0000	-0.1470	-0.6213	-0.1470	-0.7683
1	17	51.85	-0.0865	-0.3913	0.0000	-0.1151	-0.4853	-0.1151	-0.6004
1	18	54.90	-0.0595	-0.2693	0.0000	-0.0793	-0.3340	-0.0793	-0.4133
1	19	57.95	-0.0304	-0.1373	0.0000	-0.0405	-0.1703	-0.0405	-0.2108
1	20	61.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT	NONCOMPOSITE DEAD LOADS STEEL (in)	CAMBER (in)	SLAB (in)	CAMBER (in)	COMPOSITE DL DEFL. (in)	CAMBER (in)	T O T A L DEFL. (in)	CAMBER (in)

	(ft)	DEFL.	SIZE	DEFL.	SIZE		SIZE		SIZE		SIZE
1 0	0.0	0.000	--	0.000	--	0.000	--	0.000	--		
1 1	3.0	-0.030	0	1/16	-0.140	0	3/16	-0.040	0	1/16	-0.211	0	1/ 4
1 2	6.1	-0.060	0	1/16	-0.274	0	5/16	-0.079	0	1/ 8	-0.413	0	7/16
1 3	9.1	-0.087	0	1/ 8	-0.399	0	7/16	-0.115	0	1/ 8	-0.600	0	5/ 8
1 4	12.2	-0.111	0	1/ 8	-0.511	0	9/16	-0.147	0	3/16	-0.768	0	13/16
1 5	15.2	-0.132	0	3/16	-0.609	0	5/ 8	-0.175	0	3/16	-0.917	0	15/16
1 6	18.3	-0.150	0	3/16	-0.693	0	3/ 4	-0.199	0	1/ 4	-1.043	1	1/16
1 7	21.3	-0.165	0	3/16	-0.761	0	13/16	-0.219	0	1/ 4	-1.144	1	3/16
1 8	24.4	-0.176	0	3/16	-0.809	0	13/16	-0.233	0	1/ 4	-1.218	1	1/ 4
1 9	27.4	-0.182	0	3/16	-0.839	0	7/ 8	-0.241	0	1/ 4	-1.262	1	5/16
1 10	30.5	-0.184	0	3/16	-0.849	0	7/ 8	-0.244	0	1/ 4	-1.278	1	5/16
1 11	33.5	-0.182	0	3/16	-0.839	0	7/ 8	-0.241	0	1/ 4	-1.263	1	5/16
1 12	36.6	-0.176	0	3/16	-0.809	0	13/16	-0.233	0	1/ 4	-1.218	1	1/ 4
1 13	39.6	-0.165	0	3/16	-0.760	0	13/16	-0.219	0	1/ 4	-1.144	1	3/16
1 14	42.7	-0.150	0	3/16	-0.693	0	3/ 4	-0.199	0	1/ 4	-1.043	1	1/16
1 15	45.7	-0.132	0	3/16	-0.609	0	5/ 8	-0.175	0	3/16	-0.917	0	15/16
1 16	48.8	-0.111	0	1/ 8	-0.511	0	9/16	-0.147	0	3/16	-0.768	0	13/16
1 17	51.8	-0.087	0	1/ 8	-0.399	0	7/16	-0.115	0	1/ 8	-0.600	0	5/ 8
1 18	54.9	-0.060	0	1/16	-0.274	0	5/16	-0.079	0	1/ 8	-0.413	0	7/16
1 19	57.9	-0.030	0	1/16	-0.140	0	3/16	-0.040	0	1/16	-0.211	0	1/ 4
1 20	61.0	0.000	--	0.000	--	0.000	--	0.000	--		

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.50	3	0.425	-0.348 MAX HL-93	0.92	0.00279
				0.028 MIN		
				-0.102 MAX LANE		
				0.028 MIN LANE		
		4	0.433	-0.355 MAX HL-93	0.92	0.00279
				0.029 MIN		
				-0.104 MAX LANE		
				0.029 MIN LANE		
		3	0.425	-0.255 MAX FA		
				0.000 MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	-0.42	0.42	-1.94	1.94	-2.36	2.36
1	2	6.10	-0.80	0.80	-3.67	3.67	-4.47	4.47
1	3	9.15	-1.13	1.13	-5.21	5.21	-6.34	6.34
1	4	12.20	-1.26	0.80	-5.80	3.68	-7.06	4.47
1	5	15.25	-1.47	0.93	-6.79	4.30	-8.26	5.23
1	6	18.30	-1.65	1.04	-7.59	4.81	-9.24	5.86
1	7	21.35	-1.79	1.13	-8.22	5.21	-10.01	6.34
1	8	24.40	-1.88	1.19	-8.68	5.50	-10.56	6.69
1	9	27.45	-1.94	1.23	-8.95	5.67	-10.89	6.90
1	10	30.50	-1.96	1.24	-9.05	5.73	-11.01	6.98
1	11	33.55	-1.94	1.23	-8.96	5.68	-10.91	6.91
1	12	36.60	-1.88	1.19	-8.68	5.50	-10.57	6.70
1	13	39.65	-1.79	1.13	-8.22	5.21	-10.01	6.34
1	14	42.70	-1.65	1.04	-7.59	4.81	-9.24	5.85
1	15	45.75	-1.47	0.93	-6.78	4.29	-8.25	5.23
1	16	48.80	-1.26	0.80	-5.79	3.67	-7.04	4.46
1	17	51.85	-1.13	1.13	-5.21	5.21	-6.33	6.33
1	18	54.90	-0.80	0.80	-3.69	3.69	-4.48	4.48
1	19	57.95	-0.42	0.42	-1.94	1.94	-2.36	2.36
1	20	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	-0.02	-0.29	0.89	-0.02	-2.65	3.24
1	2	6.10	-0.04	-0.56	1.68	-0.04	-5.03	6.15
1	3	9.15	-0.06	-0.79	2.38	-0.06	-7.13	8.72
1	4	12.20	-0.06	-1.00	1.71	-0.06	-8.05	6.18
1	5	15.25	-0.07	-1.17	2.00	-0.07	-9.43	7.24
1	6	18.30	-0.08	-1.31	2.24	-0.08	-10.55	8.10
1	7	21.35	-0.09	-1.42	2.43	-0.09	-11.43	8.77
1	8	24.40	-0.09	-1.50	2.56	-0.09	-12.06	9.26
1	9	27.45	-0.09	-1.54	2.64	-0.09	-12.44	9.55
1	10	30.50	-0.10	-1.56	2.67	-0.10	-12.57	9.65
1	11	33.55	-0.09	-1.54	2.64	-0.09	-12.45	9.56

1 12	36.60	-0.09	-1.50	2.56	-0.09	-12.06	9.26
1 13	39.65	-0.09	-1.42	2.43	-0.09	-11.43	8.77
1 14	42.70	-0.08	-1.31	2.24	-0.08	-10.55	8.10
1 15	45.75	-0.07	-1.17	2.00	-0.07	-9.42	7.23
1 16	48.80	-0.06	-1.00	1.71	-0.06	-8.04	6.17
1 17	51.85	-0.06	-0.79	2.38	-0.06	-7.12	8.71
1 18	54.90	-0.04	-0.56	1.68	-0.04	-5.04	6.16
1 19	57.95	-0.02	-0.29	0.89	-0.02	-2.66	3.25
1 20	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	-0.15	-0.32	3.78	0.00	0.00	0.00
1	2	6.10	-0.28	-0.61	7.10	0.00	0.00	0.00
1	3	9.15	-0.39	-0.85	9.97	0.00	0.00	0.00
1	4	12.20	-0.39	-1.46	7.12	0.00	0.00	0.00
1	5	15.25	-0.45	-1.68	8.21	0.00	0.00	0.00
1	6	18.30	-0.50	-1.86	9.09	0.00	0.00	0.00
1	7	21.35	-0.54	-2.01	9.78	0.00	0.00	0.00
1	8	24.40	-0.56	-2.10	10.23	0.00	0.00	0.00
1	9	27.45	-0.57	-2.13	10.38	0.00	0.00	0.00
1	10	30.50	-0.57	-2.14	10.42	0.00	0.00	0.00
1	11	33.55	-0.57	-2.13	10.38	0.00	0.00	0.00
1	12	36.60	-0.56	-2.10	10.23	0.00	0.00	0.00
1	13	39.65	-0.54	-2.01	9.78	0.00	0.00	0.00
1	14	42.70	-0.50	-1.86	9.09	0.00	0.00	0.00
1	15	45.75	-0.45	-1.68	8.21	0.00	0.00	0.00
1	16	48.80	-0.39	-1.46	7.12	0.00	0.00	0.00
1	17	51.85	-0.39	-0.85	9.97	0.00	0.00	0.00
1	18	54.90	-0.28	-0.61	7.10	0.00	0.00	0.00
1	19	57.95	-0.15	-0.32	3.78	0.00	0.00	0.00
1	20	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	3.05	0.1	0.0	0.1	C HL	1.3	0.0	1.3	T HL
1	2	6.10	0.2	0.0	0.2	C HL	2.4	0.0	2.4	T HL
1	3	9.15	0.3	0.0	0.3	C HL	3.3	0.0	3.3	T HL
1	4	12.20	0.5	0.0	0.5	C HL	2.3	0.0	2.3	T HL
1	5	15.25	0.5	0.0	0.5	C HL	2.7	0.0	2.7	T HL
1	6	18.30	0.6	0.0	0.6	C HL	2.9	0.0	2.9	T HL
1	7	21.35	0.6	0.0	0.6	C HL	3.1	0.0	3.1	T HL
1	8	24.40	0.7	0.0	0.7	C HL	3.2	0.0	3.2	T HL

1 9	27.45	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1 10	30.50	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1 11	33.55	0.6	0.0	0.6 C	HL	3.0	0.0	3.0 T	HL
1 12	36.60	0.7	0.0	0.7 C	HL	3.2	0.0	3.2 T	HL
1 13	39.65	0.6	0.0	0.6 C	HL	3.1	0.0	3.1 T	HL
1 14	42.70	0.6	0.0	0.6 C	HL	2.9	0.0	2.9 T	HL
1 15	45.75	0.5	0.0	0.5 C	HL	2.7	0.0	2.7 T	HL
1 16	48.80	0.5	0.0	0.5 C	HL	2.3	0.0	2.3 T	HL
1 17	51.85	0.3	0.0	0.3 C	HL	3.3	0.0	3.3 T	HL
1 18	54.90	0.2	0.0	0.2 C	HL	2.4	0.0	2.4 T	HL
1 19	57.95	0.1	0.0	0.1 C	HL	1.3	0.0	1.3 T	HL
1 20	61.00	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN D NO	FROM L (ft)	SUPT Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	33.	33.	-0.17	-2.97	7.02	-0.02	-2.65	3.24
1	2	6.10	33.	33.	-0.32	-5.63	13.25	-0.04	-5.03	6.15
1	3	9.15	33.	33.	-0.45	-7.98	18.68	-0.06	-7.13	8.72
1	4	12.20	33.	33.	-0.45	-9.51	13.30	-0.06	-8.05	6.18
1	5	15.25	33.	33.	-0.52	-11.11	15.45	-0.07	-9.43	7.24
1	6	18.30	33.	33.	-0.58	-12.42	17.19	-0.08	-10.55	8.10
1	7	21.35	33.	33.	-0.62	-13.43	18.55	-0.09	-11.43	8.77
1	8	24.40	33.	33.	-0.65	-14.15	19.49	-0.09	-12.06	9.26
1	9	27.45	33.	33.	-0.66	-14.56	19.92	-0.09	-12.44	9.55
1	10	30.50	33.	33.	-0.67	-14.71	20.07	-0.10	-12.57	9.65
1	11	33.55	33.	33.	-0.66	-14.58	19.93	-0.09	-12.45	9.56
1	12	36.60	33.	33.	-0.65	-14.16	19.49	-0.09	-12.06	9.26
1	13	39.65	33.	33.	-0.62	-13.43	18.55	-0.09	-11.43	8.77
1	14	42.70	33.	33.	-0.58	-12.41	17.19	-0.08	-10.55	8.10
1	15	45.75	33.	33.	-0.52	-11.10	15.44	-0.07	-9.42	7.23
1	16	48.80	33.	33.	-0.45	-9.50	13.29	-0.06	-8.04	6.17
1	17	51.85	33.	33.	-0.45	-7.97	18.68	-0.06	-7.12	8.71
1	18	54.90	33.	33.	-0.32	-5.65	13.27	-0.04	-5.04	6.16
1	19	57.95	33.	33.	-0.17	-2.98	7.03	-0.02	-2.66	3.25
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L (ft)	SUPT Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

1	1	3.05	33.	33.	-0.21	-3.07	8.16	-0.02	-2.65	3.24
1	2	6.10	33.	33.	-0.40	-5.81	15.38	-0.04	-5.03	6.15
1	3	9.15	33.	33.	-0.57	-8.23	21.67	-0.06	-7.13	8.72
1	4	12.20	33.	33.	-0.57	-9.95	15.44	-0.06	-8.05	6.18
1	5	15.25	33.	33.	-0.66	-11.62	17.91	-0.07	-9.43	7.24
1	6	18.30	33.	33.	-0.73	-12.98	19.92	-0.08	-10.55	8.10
1	7	21.35	33.	33.	-0.78	-14.03	21.49	-0.09	-11.43	8.77
1	8	24.40	33.	33.	-0.82	-14.78	22.56	-0.09	-12.06	9.26
1	9	27.45	33.	33.	-0.83	-15.20	23.04	-0.09	-12.44	9.55
1	10	30.50	33.	33.	-0.84	-15.35	23.20	-0.10	-12.57	9.65
1	11	33.55	33.	33.	-0.83	-15.22	23.04	-0.09	-12.45	9.56
1	12	36.60	33.	33.	-0.82	-14.79	22.56	-0.09	-12.06	9.26
1	13	39.65	33.	33.	-0.78	-14.04	21.49	-0.09	-11.43	8.77
1	14	42.70	33.	33.	-0.73	-12.97	19.92	-0.08	-10.55	8.10
1	15	45.75	33.	33.	-0.66	-11.61	17.90	-0.07	-9.42	7.23
1	16	48.80	33.	33.	-0.57	-9.94	15.43	-0.06	-8.04	6.17
1	17	51.85	33.	33.	-0.57	-8.23	21.67	-0.06	-7.12	8.71
1	18	54.90	33.	33.	-0.40	-5.83	15.40	-0.04	-5.04	6.16
1	19	57.95	33.	33.	-0.21	-3.08	8.17	-0.02	-2.66	3.25
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	33.	33.	-0.29	-3.93	10.81	-0.03	-3.36	4.20
1	2	6.10	33.	33.	-0.54	-7.43	20.39	-0.06	-6.37	7.96
1	3	9.15	33.	33.	-0.76	-10.52	28.72	-0.08	-9.03	11.28
1	4	12.20	33.	33.	-0.77	-12.78	20.46	-0.09	-10.23	8.00
1	5	15.25	33.	33.	-0.89	-14.92	23.73	-0.10	-11.97	9.37
1	6	18.30	33.	33.	-0.98	-16.66	26.40	-0.11	-13.40	10.49
1	7	21.35	33.	33.	-1.06	-18.02	28.47	-0.12	-14.51	11.36
1	8	24.40	33.	33.	-1.11	-18.98	29.89	-0.13	-15.31	11.98
1	9	27.45	33.	33.	-1.13	-19.52	30.52	-0.13	-15.79	12.36
1	10	30.50	33.	33.	-1.13	-19.70	30.73	-0.13	-15.96	12.49
1	11	33.55	33.	33.	-1.13	-19.53	30.53	-0.13	-15.81	12.37
1	12	36.60	33.	33.	-1.11	-18.99	29.89	-0.13	-15.32	11.99
1	13	39.65	33.	33.	-1.06	-18.02	28.47	-0.12	-14.51	11.36
1	14	42.70	33.	33.	-0.98	-16.66	26.39	-0.11	-13.39	10.48
1	15	45.75	33.	33.	-0.89	-14.91	23.73	-0.10	-11.96	9.36
1	16	48.80	33.	33.	-0.77	-12.76	20.45	-0.09	-10.21	7.99
1	17	51.85	33.	33.	-0.76	-10.52	28.71	-0.08	-9.03	11.27
1	18	54.90	33.	33.	-0.54	-7.45	20.40	-0.06	-6.39	7.97
1	19	57.95	33.	33.	-0.29	-3.93	10.82	-0.03	-3.37	4.20
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	33.	33.	-0.28	-2.90	9.39	-0.02	-2.34	2.78
1	2	6.10	33.	33.	-0.52	-5.50	17.70	-0.03	-4.43	5.26
1	3	9.15	33.	33.	-0.73	-7.78	24.90	-0.04	-6.29	7.46
1	4	12.20	33.	33.	-0.73	-9.64	17.75	-0.05	-7.09	5.29
1	5	15.25	33.	33.	-0.84	-11.24	20.56	-0.05	-8.30	6.19
1	6	18.30	33.	33.	-0.93	-12.55	22.84	-0.06	-9.29	6.93
1	7	21.35	33.	33.	-1.00	-13.57	24.62	-0.06	-10.06	7.51
1	8	24.40	33.	33.	-1.05	-14.28	25.82	-0.07	-10.61	7.92
1	9	27.45	33.	33.	-1.06	-14.67	26.33	-0.07	-10.94	8.17
1	10	30.50	33.	33.	-1.07	-14.80	26.50	-0.07	-11.06	8.26
1	11	33.55	33.	33.	-1.06	-14.68	26.33	-0.07	-10.96	8.18
1	12	36.60	33.	33.	-1.05	-14.29	25.83	-0.07	-10.62	7.92
1	13	39.65	33.	33.	-1.00	-13.57	24.62	-0.06	-10.06	7.51
1	14	42.70	33.	33.	-0.93	-12.55	22.84	-0.06	-9.28	6.93
1	15	45.75	33.	33.	-0.84	-11.23	20.55	-0.05	-8.29	6.19
1	16	48.80	33.	33.	-0.73	-9.63	17.74	-0.05	-7.07	5.28
1	17	51.85	33.	33.	-0.73	-7.77	24.90	-0.04	-6.28	7.46
1	18	54.90	33.	33.	-0.52	-5.51	17.71	-0.03	-4.45	5.28
1	19	57.95	33.	33.	-0.28	-2.91	9.40	-0.02	-2.34	2.78
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	33.	33.	-0.23	-3.80	9.30	-0.03	-3.36	4.20
1	2	6.10	33.	33.	-0.43	-7.19	17.54	-0.06	-6.37	7.96
1	3	9.15	33.	33.	-0.61	-10.18	24.73	-0.08	-9.03	11.28
1	4	12.20	33.	33.	-0.61	-12.20	17.61	-0.09	-10.23	8.00
1	5	15.25	33.	33.	-0.71	-14.24	20.45	-0.10	-11.97	9.37
1	6	18.30	33.	33.	-0.78	-15.92	22.76	-0.11	-13.40	10.49
1	7	21.35	33.	33.	-0.85	-17.22	24.56	-0.12	-14.51	11.36
1	8	24.40	33.	33.	-0.89	-18.14	25.79	-0.13	-15.31	11.98
1	9	27.45	33.	33.	-0.90	-18.67	26.37	-0.13	-15.79	12.36
1	10	30.50	33.	33.	-0.90	-18.85	26.56	-0.13	-15.96	12.49
1	11	33.55	33.	33.	-0.90	-18.68	26.38	-0.13	-15.81	12.37
1	12	36.60	33.	33.	-0.89	-18.15	25.80	-0.13	-15.32	11.99
1	13	39.65	33.	33.	-0.85	-17.22	24.56	-0.12	-14.51	11.36
1	14	42.70	33.	33.	-0.78	-15.91	22.76	-0.11	-13.39	10.48
1	15	45.75	33.	33.	-0.71	-14.23	20.44	-0.10	-11.96	9.36
1	16	48.80	33.	33.	-0.61	-12.18	17.60	-0.09	-10.21	7.99
1	17	51.85	33.	33.	-0.61	-10.18	24.73	-0.08	-9.03	11.27
1	18	54.90	33.	33.	-0.43	-7.21	17.56	-0.06	-6.39	7.97
1	19	57.95	33.	33.	-0.23	-3.80	9.31	-0.03	-3.37	4.20
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.05	33.	33.	-0.22	-2.77	7.88	-0.02	-2.34	2.78
1	2	6.10	33.	33.	-0.41	-5.25	14.85	-0.03	-4.43	5.26
1	3	9.15	33.	33.	-0.57	-7.44	20.92	-0.04	-6.29	7.46
1	4	12.20	33.	33.	-0.57	-9.06	14.90	-0.05	-7.09	5.29
1	5	15.25	33.	33.	-0.66	-10.57	17.28	-0.05	-8.30	6.19
1	6	18.30	33.	33.	-0.73	-11.80	19.21	-0.06	-9.29	6.93
1	7	21.35	33.	33.	-0.79	-12.76	20.71	-0.06	-10.06	7.51
1	8	24.40	33.	33.	-0.82	-13.44	21.73	-0.07	-10.61	7.92
1	9	27.45	33.	33.	-0.84	-13.82	22.18	-0.07	-10.94	8.17
1	10	30.50	33.	33.	-0.84	-13.95	22.33	-0.07	-11.06	8.26
1	11	33.55	33.	33.	-0.84	-13.83	22.18	-0.07	-10.96	8.18
1	12	36.60	33.	33.	-0.82	-13.45	21.74	-0.07	-10.62	7.92
1	13	39.65	33.	33.	-0.79	-12.77	20.71	-0.06	-10.06	7.51
1	14	42.70	33.	33.	-0.73	-11.80	19.20	-0.06	-9.28	6.93
1	15	45.75	33.	33.	-0.66	-10.56	17.27	-0.05	-8.29	6.19
1	16	48.80	33.	33.	-0.57	-9.05	14.89	-0.05	-7.07	5.28
1	17	51.85	33.	33.	-0.57	-7.43	20.91	-0.04	-6.28	7.46
1	18	54.90	33.	33.	-0.41	-5.26	14.87	-0.03	-4.45	5.28
1	19	57.95	33.	33.	-0.22	-2.78	7.89	-0.02	-2.34	2.78
1	20	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	9.50	220.	-6.5	111.	-0.8	512.	-0.9	0.	0.0	-8.21
											0.87
	LB			6.5		2.4		10.2		0.0	19.21
											10.24
	RT			-5.8		-0.8		-1.2		0.0	-7.81
											1.21
	RB			3.7		1.4		5.9		0.0	10.96
											5.90
1	LT	51.50	220.	-5.8	111.	-0.8	512.	-1.2	0.	0.0	-7.81
											1.21
	LB			3.7		1.4		5.9		0.0	10.96
											5.90
	RT			-6.5		-0.8		-0.9		0.0	-8.20
											0.87
	RB			6.5		2.4		10.2		0.0	19.21
											10.24

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK

```
  1   9.50    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

  2  42.00    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 13.00 1.0000

  3   9.50    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

 ------------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   61.0
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------

  1   9.50  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

  2  42.00  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

  3   9.50  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

 ---------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                            Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  1   3.05  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  2   6.10  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  3   9.15  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  4  12.20  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  5  15.25  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
                                                 Page 27
```

1	6	18.30	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	7	21.35	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	8	24.40	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	9	27.45	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	10	30.50	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	11	33.55	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	12	36.60	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	13	39.65	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	14	42.70	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	15	45.75	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	16	48.80	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	17	51.85	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	18	54.90	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	19	57.95	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	20	61.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0		0.00	0	1	31.39	0.579	54.21	168.97	2
1	1		3.05	0	1	31.39	0.579	54.21	168.97	2
1	2		6.10	0	1	31.39	0.579	54.21	168.97	2
1	3		9.15	0	1	31.39	0.579	54.21	168.97	2
1	4		12.20	0	1	31.39	0.579	69.30	168.97	2
1	5		15.25	0	1	31.39	0.579	69.30	168.97	2
1	6		18.30	0	1	31.39	0.579	69.30	168.97	2
1	7		21.35	0	1	31.39	0.579	69.30	168.97	2
1	8		24.40	0	1	31.39	0.579	69.30	168.97	2
1	9		27.45	0	1	31.39	0.579	69.30	168.97	2
1	10		30.50	0	1	31.39	0.579	69.30	168.97	2
1	11		33.55	0	1	31.39	0.579	69.30	168.97	2
1	12		36.60	0	1	31.39	0.579	69.30	168.97	2
1	13		39.65	0	1	31.39	0.579	69.30	168.97	2
1	14		42.70	0	1	31.39	0.579	69.30	168.97	2
1	15		45.75	0	1	31.39	0.579	69.30	168.97	2
1	16		48.80	0	1	31.39	0.579	69.30	168.97	2
1	17		51.85	0	1	31.39	0.579	54.21	168.97	2
1	18		54.90	0	1	31.39	0.579	54.21	168.97	2
1	19		57.95	0	1	31.39	0.579	54.21	168.97	2
1	20		61.00	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO L (ft)	SUPT	bf/2tf	[1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	1	3.05	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	2	6.10	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	3	9.15	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	4	12.20	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	5	15.25	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	6	18.30	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	7	21.35	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	8	24.40	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	9	27.45	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	10	30.50	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	11	33.55	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	12	36.60	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	13	39.65	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	14	42.70	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	15	45.75	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	16	48.80	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	17	51.85	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	18	54.90	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	19	57.95	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	20	61.00	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. FROM LEFT SUPT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	11.5	0.000	10.341	6.346	2.350	1.527
			0.000	0.000	0.000	0.000	1.000
2	11.5	33.1	10.341	16.383	15.272	14.161	1.067
			0.000	0.000	0.000	0.000	1.000
3	33.1	54.7	6.915	16.383	13.275	10.167	1.214
			0.000	0.000	0.000	0.000	1.000
4	54.7	61.0	0.000	6.915	3.655	0.394	1.691
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	1	3.05	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	2	6.10	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	3	9.15	1	6.76	11.26	19.84	0	11.51	7.17	26.91	2	2
1	4	12.20	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	5	15.25	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	6	18.30	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	7	21.35	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	8	24.40	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	9	27.45	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	10	30.50	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	11	33.55	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	12	36.60	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	13	39.65	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	14	42.70	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	15	45.75	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	16	48.80	1	6.76	11.26	19.84	0	21.59	6.94	26.06	2	2
1	17	51.85	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	18	54.90	1	6.76	11.26	19.84	0	6.31	7.17	26.91	0	0
1	19	57.95	1	6.76	11.26	19.84	0	6.31	7.17	26.91	0	0
1	20	61.00	1	6.76	11.26	19.84	0	6.31	7.17	26.91	0	0

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange

residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.53	33.00	33.00	33.00
1	1	3.05	1	1.000	1.000	1.53	33.00	33.00	33.00
1	2	6.10	1	1.000	1.000	1.53	33.00	33.00	33.00
1	3	9.15	1	1.000	1.000	1.53	33.00	33.00	33.00
1	4	12.20	1	1.000	1.000	1.07	33.00	27.11	27.11
1	5	15.25	1	1.000	1.000	1.07	33.00	27.11	27.11
1	6	18.30	1	1.000	1.000	1.07	33.00	27.11	27.11
1	7	21.35	1	1.000	1.000	1.07	33.00	27.11	27.11
1	8	24.40	1	1.000	1.000	1.07	33.00	27.11	27.11
1	9	27.45	1	1.000	1.000	1.07	33.00	27.11	27.11
1	10	30.50	1	1.000	1.000	1.07	33.00	27.11	27.11
1	11	33.55	1	1.000	1.000	1.21	33.00	30.85	30.85
1	12	36.60	1	1.000	1.000	1.21	33.00	30.85	30.85
1	13	39.65	1	1.000	1.000	1.21	33.00	30.85	30.85
1	14	42.70	1	1.000	1.000	1.21	33.00	30.85	30.85
1	15	45.75	1	1.000	1.000	1.21	33.00	30.85	30.85
1	16	48.80	1	1.000	1.000	1.21	33.00	30.85	30.85
1	17	51.85	1	1.000	1.000	1.21	33.00	31.28	31.28
1	18	54.90	1	1.000	1.000	1.69	33.00	33.00	33.00
1	19	57.95	1	1.000	1.000	1.69	33.00	33.00	33.00
1	20	61.00	1	1.000	1.000	1.69	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0	0.00	1	5.17	17.32
1	1	3.05	1	5.17	17.32
1	2	6.10	1	5.17	17.32
1	3	9.15	1	5.17	17.32
1	4	12.20	1	8.20	17.74
1	5	15.25	1	8.20	17.74
1	6	18.30	1	8.20	17.74
1	7	21.35	1	8.20	17.74
1	8	24.40	1	8.20	17.74
1	9	27.45	1	8.20	17.74

1 10	30.50	1	8.20	17.74
1 11	33.55	1	8.20	17.74
1 12	36.60	1	8.20	17.74
1 13	39.65	1	8.20	17.74
1 14	42.70	1	8.20	17.74
1 15	45.75	1	8.20	17.74
1 16	48.80	1	8.20	17.74
1 17	51.85	1	5.17	17.32
1 18	54.90	1	5.17	17.32
1 19	57.95	1	5.17	17.32
1 20	61.00	1	5.17	17.32

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L (ft)	FROM SUPT	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Category Non-Comp.	Category Comp.
1	0	0.00	1		0	2	0		2	0
1	1	3.05	1		0	2	0		2	0
1	2	6.10	1		0	2	0		2	0
1	3	9.15	1		0	2	0		2	0
1	4	12.20	1		0	2	0		2	0
1	5	15.25	1		0	2	0		2	0
1	6	18.30	1		0	2	0		2	0
1	7	21.35	1		0	2	0		2	0
1	8	24.40	1		0	2	0		2	0
1	9	27.45	1		0	2	0		2	0
1	10	30.50	1		0	2	0		2	0
1	11	33.55	1		0	2	0		2	0
1	12	36.60	1		0	2	0		2	0
1	13	39.65	1		0	2	0		2	0
1	14	42.70	1		0	2	0		2	0
1	15	45.75	1		0	2	0		2	0
1	16	48.80	1		0	2	0		2	0
1	17	51.85	1		0	2	0		2	0
1	18	54.90	1		0	0	0		0	0
1	19	57.95	1		0	0	0		0	0
1	20	61.00	1		0	0	0		0	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.05	0.0	19.8	3.5	33.0	3.5	33.0	3.5	33.0	0
				0.0	-	3.5	-	3.5	33.0	-	-	0
1	2		6.10	0.0	19.8	6.7	33.0	6.7	33.0	6.7	33.0	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	3		9.15	0.0	19.8	9.5	33.0	9.5	33.0	9.5	33.0	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	4		12.20	0.0	19.8	10.6	33.0	10.6	33.0	10.6	27.1	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	5		15.25	0.0	19.8	12.4	33.0	12.4	33.0	12.4	27.1	0
				0.0	-	7.9	-	7.9	33.0	-	-	0
1	6		18.30	0.0	19.8	13.9	33.0	13.9	33.0	13.9	27.1	0
				0.0	-	8.8	-	8.8	33.0	-	-	0
1	7		21.35	0.0	19.8	15.0	33.0	15.0	33.0	15.0	27.1	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	8		24.40	0.0	19.8	15.8	33.0	15.8	33.0	15.8	27.1	0
				0.0	-	10.0	-	10.0	33.0	-	-	0
1	9		27.45	0.0	19.8	16.3	33.0	16.3	33.0	16.3	27.1	0
				0.0	-	10.4	-	10.4	33.0	-	-	0
1	10		30.50	0.0	19.8	16.5	33.0	16.5	33.0	16.5	27.1	0
				0.0	-	10.5	-	10.5	33.0	-	-	0
1	11		33.55	0.0	19.8	16.4	33.0	16.4	33.0	16.4	30.9	0
				0.0	-	10.4	-	10.4	33.0	-	-	0
1	12		36.60	0.0	19.8	15.9	33.0	15.9	33.0	15.9	30.9	0
				0.0	-	10.0	-	10.0	33.0	-	-	0
1	13		39.65	0.0	19.8	15.0	33.0	15.0	33.0	15.0	30.9	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	14		42.70	0.0	19.8	13.9	33.0	13.9	33.0	13.9	30.9	0
				0.0	-	8.8	-	8.8	33.0	-	-	0
1	15		45.75	0.0	19.8	12.4	33.0	12.4	33.0	12.4	30.9	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	16		48.80	0.0	19.8	10.6	33.0	10.6	33.0	10.6	30.9	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	17		51.85	0.0	19.8	9.5	33.0	9.5	33.0	9.5	31.3	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	18		54.90	0.0	19.8	6.7	33.0	6.7	33.0	6.7	33.0	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	19		57.95	0.0	19.8	3.5	33.0	3.5	33.0	3.5	33.0	0
				0.0	-	3.5	-	3.5	33.0	-	-	0
1	20		61.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.05	0.000	0.107	0.107	0.107	0.107	2
					0.107		0.107	3
1	2	6.10	0.000	0.203	0.203	0.203	0.203	2
					0.203		0.203	3
1	3	9.15	0.000	0.288	0.288	0.288	0.288	2
					0.288		0.288	3
1	4	12.20	0.000	0.321	0.321	0.391	0.391	4
					0.203		0.203	3
1	5	15.25	0.000	0.375	0.375	0.457	0.457	4
					0.238		0.238	3
1	6	18.30	0.000	0.420	0.420	0.511	0.511	4
					0.266		0.266	3
1	7	21.35	0.000	0.455	0.455	0.554	0.554	4
					0.288		0.288	3
1	8	24.40	0.000	0.480	0.480	0.584	0.584	4
					0.304		0.304	3
1	9	27.45	0.000	0.495	0.495	0.603	0.603	4
					0.314		0.314	3
1	10	30.50	0.000	0.500	0.500	0.609	0.609	4
					0.317		0.317	3
1	11	33.55	0.000	0.496	0.496	0.530	0.530	4
					0.314		0.314	3
1	12	36.60	0.000	0.480	0.480	0.514	0.514	4
					0.304		0.304	3
1	13	39.65	0.000	0.455	0.455	0.487	0.487	4
					0.288		0.288	3
1	14	42.70	0.000	0.420	0.420	0.449	0.449	4
					0.266		0.266	3
1	15	45.75	0.000	0.375	0.375	0.401	0.401	4
					0.238		0.238	3
1	16	48.80	0.000	0.320	0.320	0.342	0.342	4
					0.203		0.203	3
1	17	51.85	0.000	0.288	0.288	0.304	0.304	4
					0.288		0.288	3
1	18	54.90	0.000	0.204	0.204	0.204	0.204	2
					0.204		0.204	3
1	19	57.95	0.000	0.107	0.107	0.107	0.107	2
					0.107		0.107	3
1	20	61.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2252.3	-	-	0.0	19.8	-	-	0--0
1	1		3.05	0	487.4	2252.3	-	-	0.0	19.8	-	-	0--0
1	2		6.10	0	918.3	2252.3	-	-	0.0	19.8	-	-	0--0
1	3		9.15	0	1292.7	2252.3	-	-	0.0	19.8	-	-	0--0
1	4		12.20	0	1610.3	3331.5	-	-	0.0	19.8	-	-	0--0
1	5		15.25	0	1866.2	3331.5	-	-	0.0	19.8	-	-	0--0
1	6		18.30	0	2074.1	3331.5	-	-	0.0	19.8	-	-	0--0
1	7		21.35	0	2236.2	3331.5	-	-	0.0	19.8	-	-	0--0
1	8		24.40	0	2345.9	3331.5	-	-	0.0	19.8	-	-	0--0
1	9		27.45	0	2392.8	3331.5	-	-	0.0	19.8	-	-	0--0
1	10		30.50	0	2408.7	3331.5	-	-	0.0	19.8	-	-	0--0
1	11		33.55	0	2393.4	3331.5	-	-	0.0	19.8	-	-	0--0
1	12		36.60	0	2346.3	3331.5	-	-	0.0	19.8	-	-	0--0
1	13		39.65	0	2236.2	3331.5	-	-	0.0	19.8	-	-	0--0
1	14		42.70	0	2073.9	3331.5	-	-	0.0	19.8	-	-	0--0
1	15		45.75	0	1865.7	3331.5	-	-	0.0	19.8	-	-	0--0
1	16		48.80	0	1609.5	3331.5	-	-	0.0	19.8	-	-	0--0
1	17		51.85	0	1292.5	2252.3	-	-	0.0	19.8	-	-	0--0
1	18		54.90	0	918.9	2252.3	-	-	0.0	19.8	-	-	0--0
1	19		57.95	0	487.7	2252.3	-	-	0.0	19.8	-	-	0--0
1	20		61.00	0	0.0	2252.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	3.05	0	0.216				0.216	1
1	2	6.10	0	0.408				0.408	1
1	3	9.15	0	0.574				0.574	1
1	4	12.20	0	0.483				0.483	1
1	5	15.25	0	0.560				0.560	1
1	6	18.30	0	0.623				0.623	1
1	7	21.35	0	0.671				0.671	1
1	8	24.40	0	0.704				0.704	1
1	9	27.45	0	0.718				0.718	1
1	10	30.50	0	0.723				0.723	1
1	11	33.55	0	0.718				0.718	1
1	12	36.60	0	0.704				0.704	1
1	13	39.65	0	0.671				0.671	1
1	14	42.70	0	0.623				0.623	1
1	15	45.75	0	0.560				0.560	1
1	16	48.80	0	0.483				0.483	1
1	17	51.85	0	0.574				0.574	1
1	18	54.90	0	0.408				0.408	1
1	19	57.95	0	0.217				0.217	1
1	20	61.00	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.05	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	6.10	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	9.15	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	12.20	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	15.25	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	18.30	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	21.35	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	24.40	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	27.45	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	30.50	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

1 11	33.55	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 12	36.60	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 13	39.65	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 14	42.70	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 15	45.75	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 16	48.80	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 17	51.85	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 18	54.90	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 19	57.95	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 20	61.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	212.2	<	347.9	0	0
1	1	3.05	0	196.3	<	347.9	0	0
1	2	6.10	0	180.6	<	347.9	0	0
1	3	9.15	0	165.1	<	347.9	0	0
1	4	12.20	0	149.3	<	347.9	0	0

1	5	15.25	0	134.0	<	347.9	0	0
1	6	18.30	0	118.9	<	347.9	0	0
1	7	21.35	0	104.0	<	347.9	0	0
1	8	24.40	0	89.2	<	347.9	0	0
1	9	27.45	0	74.4	<	347.9	0	0
1	10	30.50	0	60.1	<	347.9	0	0
1	11	33.55	0	74.5	<	347.9	0	0
1	12	36.60	0	89.3	<	347.9	0	0
1	13	39.65	0	104.1	<	347.9	0	0
1	14	42.70	0	119.0	<	347.9	0	0
1	15	45.75	0	134.1	<	347.9	0	0
1	16	48.80	0	149.4	<	347.9	0	0
1	17	51.85	0	164.8	<	347.9	0	0
1	18	54.90	0	180.7	<	347.9	0	0
1	19	57.95	0	196.4	<	347.9	0	0
1	20	61.00	0	212.3	<	347.9	0	0

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      NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

                For compact section  (composite sections in positive flexure)

                  w/o long. stiff.

                  D/tw <= 150 (Eq. 6.10.2.1.1-1)

                  w / long. stiff.

                  D/tw <= 300 (Eq. 6.10.2.1.2-1)

                  2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

                For compact or non-compact sections specified in Art. 6.10.8

                  (composite sections in negative flexure and
                   non-composite sections)

                  2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


            [2]: SHEAR CAPACITY ... web crippling control

                Vn = CVp .......... AASHTO Eq.6.10.9.2-1

                Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

     Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

                Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

     Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

                Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

              * For the detailed description of shear capacity,
                please refer to AASHTO Art.6.10.9.2 and .3

              * STRENGTH CATEGORY; 0 = compact section
                                   2 = braced non-compact section
                                   3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************
```

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	212.20	347.87	0	
1	1	3.05	33.	196.33	347.87	0	
1	2	6.10	33.	180.61	347.87	0	
1	3	9.15	33.	165.05	347.87	0	
1	4	12.20	33.	149.29	347.87	0	
1	5	15.25	33.	134.03	347.87	0	
1	6	18.30	33.	118.93	347.87	0	
1	7	21.35	33.	103.98	347.87	0	
1	8	24.40	33.	89.19	347.87	0	
1	9	27.45	33.	74.42	347.87	0	
1	10	30.50	33.	60.06	347.87	0	
1	11	33.55	33.	74.52	347.87	0	
1	12	36.60	33.	89.28	347.87	0	
1	13	39.65	33.	104.07	347.87	0	
1	14	42.70	33.	119.02	347.87	0	
1	15	45.75	33.	134.12	347.87	0	
1	16	48.80	33.	149.38	347.87	0	
1	17	51.85	33.	164.79	347.87	0	
1	18	54.90	33.	180.71	347.87	0	
1	19	57.95	33.	196.42	347.87	0	
1	20	61.00	33.	212.29	347.87	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D NO L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw (ksi)	[3]	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-		00-
1	1	3.05	3.1	-	31.4	8.2	31.4	-	-		00-
1	2	6.10	5.8	-	31.4	15.4	31.4	-	-		00-
1	3	9.15	8.2	-	31.4	21.7	31.4	-	-		00-
1	4	12.20	10.0	-	31.4	15.4	31.4	-	-		00-
1	5	15.25	11.6	-	31.4	17.9	31.4	-	-		00-
1	6	18.30	13.0	-	31.4	19.9	31.4	-	-		00-
1	7	21.35	14.0	-	31.4	21.5	31.4	-	-		00-
1	8	24.40	14.8	-	31.4	22.6	31.4	-	-		00-
1	9	27.45	15.2	-	31.4	23.0	31.4	-	-		00-
1	10	30.50	15.3	-	31.4	23.2	31.4	-	-		00-
1	11	33.55	15.2	-	31.4	23.0	31.4	-	-		00-
1	12	36.60	14.8	-	31.4	22.6	31.4	-	-		00-
1	13	39.65	14.0	-	31.4	21.5	31.4	-	-		00-
1	14	42.70	13.0	-	31.4	19.9	31.4	-	-		00-
1	15	45.75	11.6	-	31.4	17.9	31.4	-	-		00-
1	16	48.80	9.9	-	31.4	15.4	31.4	-	-		00-
1	17	51.85	8.2	-	31.4	21.7	31.4	-	-		00-
1	18	54.90	5.8	-	31.4	15.4	31.4	-	-		00-
1	19	57.95	3.1	-	31.4	8.2	31.4	-	-		00-
1	20	61.00	0.0	-	31.4	0.0	31.4	-	-		00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)

```
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
     [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
     [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
           but not to exceed the smaller of RhFyc and Fyw/0.7
           k = bending-buckling coefficient
             = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
           where:
           Dc = depth of the web in compression in the elastic
                range. For composite sections, Dc shall be
                determined as specified in Article D6.3.1

     "-" is N.A.

     Flag check - 0 = OK; 1 = NG
     T = Top Flange; B = Bottom Flange; C = Comp. Flange

     The values of fl shall be determined based on
     factored loads, and shall be taken as positive in
     sign in all resistance equations (Art. 6.10.1.6)
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
               ----------------------------- -----------------------------
SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
-----------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   3.05    TR     -0.2 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1  2   6.10    TR     -0.3 A B B^C^C D E E^  TR      3.6 A B B^C^C D E
 1  3   9.15    TR     -0.4 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1      9.50  SCG
 1  4  12.20    TR     -0.7 A B B^C^C D E E^  TR      3.5 A B B^C^C D E
 1  5  15.25    TR     -0.8 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
 1  6  18.30    TR     -0.9 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1  7  21.35    TR     -1.0 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1  8  24.40    TR     -1.0 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1  9  27.45    TR     -0.9 A B B^C^C D E E^  TR      4.5 A B B^C^C D
 1 10  30.50    TR     -0.9 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1 11  33.55    TR     -0.9 A B B^C^C D E E^  TR      4.5 A B B^C^C D
 1 12  36.60    TR     -1.0 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1 13  39.65    TR     -1.0 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1 14  42.70    TR     -0.9 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1 15  45.75    TR     -0.8 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
 1 16  48.80    TR     -0.7 A B B^C^C D E E^  TR      3.5 A B B^C^C D E
 1     51.50  SCG
 1 17  51.85    TR     -0.4 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 18  54.90    TR     -0.3 A B B^C^C D E E^  TR      3.6 A B B^C^C D E
 1 19  57.95    TR     -0.2 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 20  61.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


-------------------------------------------------------------------------------
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.
-------------------------------------------------------------------------------
NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
             POC = Dead load point of contraflexure
```

```
                I:P = Point where INT coincides with POC
                I:C = Point where INT coincides with SCG
                S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                   ***************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                       TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.05    TR     -0.1 A B B^C^C D E E^  TR      1.0 A B B^C^C D E E^
  1  2   6.10    TR     -0.2 A B B^C^C D E E^  TR      1.8 A B B^C^C D E E^
  1  3   9.15    TR     -0.2 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1      9.50  SCG
  1  4  12.20    TR     -0.4 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
  1  5  15.25    TR     -0.4 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1  6  18.30    TR     -0.5 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
  1  7  21.35    TR     -0.5 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1  8  24.40    TR     -0.5 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1  9  27.45    TR     -0.5 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1 10  30.50    TR     -0.5 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
  1 11  33.55    TR     -0.5 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1 12  36.60    TR     -0.5 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1 13  39.65    TR     -0.5 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1 14  42.70    TR     -0.5 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
  1 15  45.75    TR     -0.4 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1 16  48.80    TR     -0.4 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
  1     51.50  SCG
  1 17  51.85    TR     -0.2 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1 18  54.90    TR     -0.2 A B B^C^C D E E^  TR      1.8 A B B^C^C D E E^
  1 19  57.95    TR     -0.1 A B B^C^C D E E^  TR      1.0 A B B^C^C D E E^
  1 20  61.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   *******************************************
```

5969 Span 1 Interior Fatigue and Mid M.RES

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	54.7	.29290E-01	1.6	TR 3.09	3	5.80
1	1	3.05	52.5	.29290E-01	1.5	TR 3.09	3	6.03
1	2	6.10	50.3	.29290E-01	1.5	TR 3.09	3	6.29
1	3	9.15	48.1	.29290E-01	1.4	TR 3.09	3	6.59
1	4	12.20	46.0	.26378E-01	1.2	TR 3.09	3	7.65
1	5	15.25	44.1	.26378E-01	1.2	TR 3.09	3	7.97
1	6	18.30	42.5	.26378E-01	1.1	TR 3.09	3	8.28
1	7	21.35	41.4	.26378E-01	1.1	TR 3.09	3	8.49
1	8	24.40	39.9	.26378E-01	1.1	TR 3.09	3	8.82
1	9	27.45	36.4	.26378E-01	1.0	TR 3.09	3	9.67
1	10	30.50	35.2	.26378E-01	0.9	TR 3.09	3	9.99
1	11	33.55	36.4	.26378E-01	1.0	TR 3.09	3	9.67
1	12	36.60	39.9	.26378E-01	1.1	TR 3.09	3	8.82
1	13	39.65	41.4	.26378E-01	1.1	TR 3.09	3	8.49
1	14	42.70	42.5	.26378E-01	1.1	TR 3.09	3	8.28
1	15	45.75	44.1	.26378E-01	1.2	TR 3.09	3	7.97
1	16	48.80	46.0	.26378E-01	1.2	TR 3.09	3	7.65
1	17	51.85	48.1	.29290E-01	1.4	TR 3.09	3	6.59
1	18	54.90	50.3	.29290E-01	1.5	TR 3.09	3	6.29
1	19	57.95	52.5	.29290E-01	1.5	TR 3.09	3	6.03
1	20	61.00	54.7	.29290E-01	1.6	TR 3.09	3	5.80

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	27.3	.29290E-01	0.8	TR	1.36	3	5.11
1	1	3.05	26.3	.29290E-01	0.8	TR	1.36	3	5.32
1	2	6.10	25.2	.29290E-01	0.7	TR	1.36	3	5.55
1	3	9.15	24.0	.29290E-01	0.7	TR	1.36	3	5.81
1	4	12.20	23.0	.26378E-01	0.6	TR	1.36	3	6.74
1	5	15.25	22.1	.26378E-01	0.6	TR	1.36	3	7.03
1	6	18.30	21.2	.26378E-01	0.6	TR	1.36	3	7.30
1	7	21.35	20.7	.26378E-01	0.5	TR	1.36	3	7.49
1	8	24.40	19.9	.26378E-01	0.5	TR	1.36	3	7.78
1	9	27.45	18.2	.26378E-01	0.5	TR	1.36	3	8.53
1	10	30.50	17.6	.26378E-01	0.5	TR	1.36	3	8.81
1	11	33.55	18.2	.26378E-01	0.5	TR	1.36	3	8.53
1	12	36.60	19.9	.26378E-01	0.5	TR	1.36	3	7.78
1	13	39.65	20.7	.26378E-01	0.5	TR	1.36	3	7.49
1	14	42.70	21.2	.26378E-01	0.6	TR	1.36	3	7.30
1	15	45.75	22.1	.26378E-01	0.6	TR	1.36	3	7.03
1	16	48.80	23.0	.26378E-01	0.6	TR	1.36	3	6.74
1	17	51.85	24.0	.29290E-01	0.7	TR	1.36	3	5.81
1	18	54.90	25.2	.29290E-01	0.7	TR	1.36	3	5.55
1	19	57.95	26.3	.29290E-01	0.8	TR	1.36	3	5.32
1	20	61.00	27.3	.29290E-01	0.8	TR	1.36	3	5.11

Please read NOTE on the following page

NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D FROM L (ft)	SUPT 1=POS. 0=NEG.	MOMENT REGION	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) FATIG. CRI.	<=>	SEE NOTE N1	N2	STATUS BLANK=OK CHECK=**
1	10	30.50	1	1673.9	1626.6			222	>	85		
1	20	61.00	1	1673.9	1626.6			222	>	85		

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or
P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	30.500	5.000
1	30.500	61.000	5.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	61.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	POS. MAX. STRESS	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2252.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.05	2252.3	79.7	40.2	189.2	0.0	8.21	HL-93	6.33	0
1	2	6.10	2252.3	151.1	76.2	355.5	0.0	4.08	HL-93	3.14	0
1	3	9.15	2252.3	214.2	107.9	498.7	0.0	2.72	HL-93	2.10	0
1	4	12.20	3331.5	268.9	135.4	618.9	0.0	3.36	HL-93	2.59	0
1	5	15.25	3331.5	314.6	158.7	713.7	0.0	2.82	HL-93	2.17	0
1	6	18.30	3331.5	352.1	177.8	790.4	0.0	2.47	HL-93	1.91	0
1	7	21.35	3331.5	381.3	192.6	850.2	0.0	2.25	HL-93	1.74	0
1	8	24.40	3331.5	402.3	203.2	889.4	0.0	2.12	HL-93	1.63	0
1	9	27.45	3331.5	415.0	209.5	902.0	0.0	2.07	HL-93	1.59	0
1	10	30.50	3331.5	419.5	211.6	906.2	0.0	2.05	HL-93	1.58	0
1	11	33.55	3331.5	415.5	209.5	902.0	0.0	2.07	HL-93	1.59	0
1	12	36.60	3331.5	402.6	203.2	889.4	0.0	2.12	HL-93	1.63	0
1	13	39.65	3331.5	381.4	192.6	850.2	0.0	2.25	HL-93	1.74	0
1	14	42.70	3331.5	352.0	177.8	790.4	0.0	2.47	HL-93	1.91	0
1	15	45.75	3331.5	314.2	158.7	713.7	0.0	2.82	HL-93	2.17	0
1	16	48.80	3331.5	268.3	135.4	618.9	0.0	3.36	HL-93	2.59	0
1	17	51.85	2252.3	214.0	107.9	498.7	0.0	2.72	HL-93	2.10	0
1	18	54.90	2252.3	151.5	76.2	355.5	0.0	4.08	HL-93	3.14	0
1	19	57.95	2252.3	79.9	40.2	189.2	0.0	8.20	HL-93	6.33	0
1	20	61.00	2252.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

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RATING

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	SUPT	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. NEG. STEEL TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. RATING FACTOR	LIVE LOAD TYPE
1	0	0.00	31.4	0.00	0.00	0.00	0.00	9.99	7.68	HL-93
1	1	3.05	31.4	-2.65	3.24	-0.32	3.78	0.00	0.00	7.43	HL-93	5.72
1	2	6.10	31.4	-5.03	6.15	-0.61	7.10	0.00	0.00	3.55	HL-93	2.73
1	3	9.15	31.4	-7.13	8.72	-0.85	9.97	0.00	0.00	2.27	HL-93	1.75
1	4	12.20	31.4	-8.05	6.18	-1.46	7.12	0.00	0.00	3.54	HL-93	2.72
1	5	15.25	31.4	-9.43	7.24	-1.68	8.21	0.00	0.00	2.94	HL-93	2.26
1	6	18.30	31.4	-10.55	8.10	-1.86	9.09	0.00	0.00	2.56	HL-93	1.97
1	7	21.35	31.4	-11.43	8.77	-2.01	9.78	0.00	0.00	2.31	HL-93	1.78
1	8	24.40	31.4	-12.06	9.26	-2.10	10.23	0.00	0.00	2.16	HL-93	1.66
1	9	27.45	31.4	-12.44	9.55	-2.13	10.38	0.00	0.00	2.10	HL-93	1.62
1	10	30.50	31.4	-12.57	9.65	-2.14	10.42	0.00	0.00	2.08	HL-93	1.60
1	11	33.55	31.4	-12.45	9.56	-2.13	10.38	0.00	0.00	2.10	HL-93	1.62
1	12	36.60	31.4	-12.06	9.26	-2.10	10.23	0.00	0.00	2.16	HL-93	1.66
1	13	39.65	31.4	-11.43	8.77	-2.01	9.78	0.00	0.00	2.31	HL-93	1.78
1	14	42.70	31.4	-10.55	8.10	-1.86	9.09	0.00	0.00	2.56	HL-93	1.97
1	15	45.75	31.4	-9.42	7.23	-1.68	8.21	0.00	0.00	2.94	HL-93	2.26
1	16	48.80	31.4	-8.04	6.17	-1.46	7.12	0.00	0.00	3.54	HL-93	2.72
1	17	51.85	31.4	-7.12	8.71	-0.85	9.97	0.00	0.00	2.27	HL-93	1.75
1	18	54.90	31.4	-5.04	6.16	-0.61	7.10	0.00	0.00	3.55	HL-93	2.73
1	19	57.95	31.4	-2.66	3.25	-0.32	3.78	0.00	0.00	7.43	HL-93	5.72
1	20	61.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	30.50	STRENGTH L.S.	2.05	1.58	HL-93
1	61.00	30.50	SERVICEBILITY	2.08	1.60	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.051
Inventory= 1.582

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	61.00	36.60	STRENGTH L.S.	2.91	874.03
1	61.00	9.15	SERVICEBILITY	2.78	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.784

1

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RATING

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	27.5	13.9	90.4	0.0	2.41	HL-93	1.86
1	1	3.05	347.9	24.8	12.5	84.4	-2.0	2.63	HL-93	2.03
1	2	6.10	347.9	22.1	11.1	78.5	-4.2	2.87	HL-93	2.22

1	3	9.15	347.9	19.3	9.7	72.7	-7.4	3.16	HL-93	2.44
1	4	12.20	347.9	16.4	8.3	66.9	-10.7	3.49	HL-93	2.70
1	5	15.25	347.9	13.6	6.9	61.3	-14.1	3.88	HL-93	2.99
1	6	18.30	347.9	10.9	5.6	55.7	-17.5	4.34	HL-93	3.35
1	7	21.35	347.9	8.2	4.2	50.2	-21.1	4.90	HL-93	3.78
1	8	24.40	347.9	5.5	2.8	44.8	-24.8	5.57	HL-93	4.30
1	9	27.45	347.9	2.8	1.4	39.4	-29.3	6.44	HL-93	4.97
1	10	30.50	347.9	0.1	0.0	34.2	-34.2	7.52	HL-93	5.80
1	11	33.55	347.9	-2.9	-1.4	29.3	-39.4	6.44	HL-93	4.96
1	12	36.60	347.9	-5.6	-2.8	24.8	-44.8	5.57	HL-93	4.30
1	13	39.65	347.9	-8.3	-4.2	21.1	-50.2	4.89	HL-93	3.78
1	14	42.70	347.9	-11.0	-5.6	17.5	-55.7	4.34	HL-93	3.35
1	15	45.75	347.9	-13.7	-6.9	14.1	-61.3	3.88	HL-93	2.99
1	16	48.80	347.9	-16.4	-8.3	10.7	-66.9	3.49	HL-93	2.70
1	17	51.85	347.9	-19.1	-9.7	7.4	-72.7	3.16	HL-93	2.44
1	18	54.90	347.9	-22.1	-11.1	4.2	-78.5	2.87	HL-93	2.22
1	19	57.95	347.9	-24.8	-12.5	2.0	-84.4	2.62	HL-93	2.02
1	20	61.00	347.9	-27.5	-13.9	0.0	-90.4	2.41	HL-93	1.86

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	61.00	STRENGTH L.S.	2.41	1.86	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.407
Inventory= 1.857

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	61.00	61.00	STRENGTH L.S.	2.92	100.61

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 2.921

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

1	30.50	3	0.425	-0.35	HL-93	2.63
		4	0.433	-0.36		2.58

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

1

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```
TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 1 Exterior Strength   8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       12           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                    1 = detailed output

 * span interval   : maximum = 20
                    default = 10

 * structural type : 1 = composite (default)
                    2 = noncomposite
                    3 = reinforced concrete
                    4 = prestressed concrete

 * type of unit    : 0 = English (default)
                    1 = Metric
                    2 = Metric input English output
                    3 = English input Metric output

 * design option   : 0 = WSD (default)
                    1 = LFD
                    2 = LRFD


 * program flow    : 0 = DL analysis only
                    1 = DL + LL analysis
                    2 = code check
                    3 = rating
                    4 = design
                    5 = design + code check
                    6 = design + recycle + code check
                    7 = DL stage only
                    8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
1
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              *****************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.24	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	PLATE GIRDER WEB THICK. DEPTH (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130		0.00	0.0000	0.00	0.0000		
2	W	33	130		0.00	0.00001	0.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

1

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.00				33.	33.	33.
2	2	2	39.00				33.	33.	33.
3	1	1	11.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.35	0.00	15.00	1 1 1 1 1	0.53	2 2 2 2 2
1	0.00	0	0.00	0.41	0.00	0.00	0 0 0 0 0	0.57	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

1

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	61.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	17.81
2	2	Diaphragm	0.000	0.00	0.00	0.14	39.41
3	1	Railing	0.087	0.00	61.00	0.00	0.00
4	1	Curb	0.076	0.00	61.00	0.00	0.00
5	0	Pavement	0.292	0.00	61.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	17.81	33.0	33.0	17.81
17.81	39.41	33.0	33.0	21.60
39.41	61.00	33.0	33.0	21.59

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.555       0.1520     0.7070


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      61.00

     1   DC2     0.087       0.000     61.000

     2   DC2     0.076       0.000     61.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1481.2	54.9	222.2	55.6	277.82

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.526	0.526	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.575				FOR STRENGTH POSITIVE SHEAR
1	0.347				FOR FATIGUE POSITIVE MOMENT
	0.408				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	5.08	6710.0	33.1	16.55	405.6	405.6
1	2	10.17	6710.0	33.1	16.55	405.6	405.6
1	3	15.25	9020.9	33.1	14.02	643.6	449.4
1	4	20.33	9020.9	33.1	14.02	643.6	449.4
1	5	25.42	9020.9	33.1	14.02	643.6	449.4
1	6	30.50	9020.9	33.1	14.02	643.6	449.4
1	7	35.58	9020.9	33.1	14.02	643.6	449.4
1	8	40.67	9020.9	33.1	14.02	643.6	449.4
1	9	45.75	9020.9	33.1	14.02	643.6	449.4
1	10	50.83	6710.0	33.1	16.55	405.6	405.6
1	11	55.92	6710.0	33.1	16.55	405.6	405.6
1	12	61.00	6710.0	33.1	16.55	405.6	405.6

```
 ----------------------------------------------------------------------
  NOTE: For rolled section, the 5th column is the depth  d (inch)
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 TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
               ****************************************


              MOMENT OF    Q/Ix       ELASTIC SECTION MODULUS,(in**3)
 SP IN D FROM  INERTIA -------------- --------------------------------
 NO NO L SUPT --------- Q=1ST. MOMENT      STEEL       CONCRETE(SLAB)
        (ft)    Ix         OF INERTIA ---------------- ---------------
              (in**4)     (1/in)        BOT.    TOP.         TOP.
 ----------------------------------------------------------------------

  1  0   0.00   12362.3   0.021678     527.0     1283.3      695.6
  1  1   5.08   12362.3   0.021678     527.0     1283.3      695.6
  1  2  10.17   12362.3   0.021678     527.0     1283.3      695.6
  1  3  15.25   17252.9   0.019493     826.1     1306.4      808.2
  1  4  20.33   17252.9   0.019493     826.1     1306.4      808.2
  1  5  25.42   17252.9   0.019493     826.1     1306.4      808.2
  1  6  30.50   17252.9   0.019493     826.1     1306.4      808.2
  1  7  35.58   17252.9   0.019493     826.1     1306.4      808.2
  1  8  40.67   17252.9   0.019493     826.1     1306.4      808.2
  1  9  45.75   17252.9   0.019493     826.1     1306.4      808.2
  1 10  50.83   12362.3   0.021678     527.0     1283.3      695.6
  1 11  55.92   12362.3   0.021678     527.0     1283.3      695.6
  1 12  61.00   12362.3   0.021678     527.0     1283.3      695.6


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 TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N =  9.00
               ****************************************


              MOMENT OF    Q/Ix       ELASTIC SECTION MODULUS,(in**3)
 SP IN D FROM  INERTIA -------------- --------------------------------
 NO NO L SUPT --------- Q=1ST. MOMENT      STEEL       CONCRETE(SLAB)
        (ft)    Ix         OF INERTIA ---------------- ---------------
              (in**4)     (1/in)        BOT.    TOP.         TOP.
 ----------------------------------------------------------------------

  1  0   0.00   17045.3   0.028332     586.7     4224.1     1400.0
  1  1   5.08   17045.3   0.028332     586.7     4224.1     1400.0
  1  2  10.17   17045.3   0.028332     586.7     4224.1     1400.0
  1  3  15.25   24951.9   0.025851     916.4     3636.3     1663.3
  1  4  20.33   24951.9   0.025851     916.4     3636.3     1663.3
  1  5  25.42   24951.9   0.025851     916.4     3636.3     1663.3
  1  6  30.50   24951.9   0.025851     916.4     3636.3     1663.3
  1  7  35.58   24951.9   0.025851     916.4     3636.3     1663.3
  1  8  40.67   24951.9   0.025851     916.4     3636.3     1663.3
  1  9  45.75   24951.9   0.025851     916.4     3636.3     1663.3
  1 10  50.83   17045.3   0.028332     586.7     4224.1     1400.0
  1 11  55.92   17045.3   0.028332     586.7     4224.1     1400.0
  1 12  61.00   17045.3   0.028332     586.7     4224.1     1400.0


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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	5.08	21.6	78.9	0.8	0.0	101.3
1	2	10.17	39.3	143.4	1.6	0.0	184.3
1	3	15.25	53.0	193.6	2.3	0.0	249.0
1	4	20.33	62.9	229.5	2.7	0.0	295.1
1	5	25.42	68.8	251.0	2.8	0.0	322.5
1	6	30.50	70.7	258.1	2.8	0.0	331.7
1	7	35.58	68.8	251.0	2.9	0.0	322.6
1	8	40.67	62.9	229.5	2.7	0.0	295.1
1	9	45.75	53.0	193.6	2.1	0.0	248.7
1	10	50.83	39.3	143.4	1.4	0.0	184.1
1	11	55.92	21.6	78.9	0.7	0.0	101.2
1	12	61.00	0.0	0.0	0.0	0.0	0.0

TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft)	CONCENTRATED LOADS (k-ft)	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD

			DW	DC2	DW	DC2		
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	5.08	41.5	23.2	0.0	0.0	0.0	64.7
1	2	10.17	75.5	42.1	0.0	0.0	0.0	117.6
1	3	15.25	101.9	56.9	0.0	0.0	0.0	158.7
1	4	20.33	120.7	67.4	0.0	0.0	0.0	188.1
1	5	25.42	132.0	73.7	0.0	0.0	0.0	205.8
1	6	30.50	135.8	75.8	0.0	0.0	0.0	211.6
1	7	35.58	132.0	73.7	0.0	0.0	0.0	205.8
1	8	40.67	120.7	67.4	0.0	0.0	0.0	188.1
1	9	45.75	101.9	56.9	0.0	0.0	0.0	158.7
1	10	50.83	75.5	42.1	0.0	0.0	0.0	117.6
1	11	55.92	41.5	23.2	0.0	0.0	0.0	64.7
1	12	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	SIDEWALK (MAXIMUM) NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	5.08	0.0	0.0	243.4	HL-93	0.0	HL-93
1	2	10.17	0.0	0.0	435.4	HL-93	0.0	HL-93
1	3	15.25	0.0	0.0	574.0	HL-93	0.0	HL-93
1	4	20.33	0.0	0.0	669.6	HL-93	0.0	HL-93
1	5	25.42	0.0	0.0	719.5	HL-93	0.0	HL-93
1	6	30.50	0.0	0.0	728.8	HL-93	0.0	HL-93
1	7	35.58	0.0	0.0	719.5	HL-93	0.0	HL-93
1	8	40.67	0.0	0.0	669.6	HL-93	0.0	HL-93
1	9	45.75	0.0	0.0	574.0	HL-93	0.0	HL-93
1	10	50.83	0.0	0.0	435.4	HL-93	0.0	HL-93
1	11	55.92	0.0	0.0	243.4	HL-93	0.0	HL-93
1	12	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	5.08	90.	0.	90.
1	2	10.17	155.	0.	155.
1	3	15.25	202.	0.	202.
1	4	20.33	232.	0.	232.
1	5	25.42	236.	0.	236.
1	6	30.50	224.	0.	224.
1	7	35.58	236.	0.	236.
1	8	40.67	232.	0.	232.
1	9	45.75	202.	0.	202.
1	10	50.83	155.	0.	155.
1	11	55.92	90.	0.	90.
1	12	61.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	5.08	101.3	64.7	243.4	HL-93	0.0	HL-93	409.3	165.9
1	2	10.17	184.3	117.6	435.4	HL-93	0.0	HL-93	737.2	301.8
1	3	15.25	249.0	158.7	574.0	HL-93	0.0	HL-93	981.8	407.7
1	4	20.33	295.1	188.1	669.6	HL-93	0.0	HL-93	1152.8	483.2
1	5	25.42	322.5	205.8	719.5	HL-93	0.0	HL-93	1247.7	528.3
1	6	30.50	331.7	211.6	728.8	HL-93	0.0	HL-93	1272.2	543.3
1	7	35.58	322.6	205.8	719.5	HL-93	0.0	HL-93	1247.8	528.4
1	8	40.67	295.1	188.1	669.6	HL-93	0.0	HL-93	1152.8	483.2
1	9	45.75	248.7	158.7	574.0	HL-93	0.0	HL-93	981.5	407.4
1	10	50.83	184.1	117.6	435.4	HL-93	0.0	HL-93	737.0	301.6
1	11	55.92	101.2	64.7	243.4	HL-93	0.0	HL-93	409.2	165.8
1	12	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	5.08	409.3	482.3	643.7	546.3	248.9	67.7
1	2	10.17	737.2	867.8	1158.1	983.9	452.7	116.2
1	3	15.25	981.8	1154.0	1539.7	1310.1	611.6	151.6
1	4	20.33	1152.8	1353.7	1806.0	1538.1	724.8	174.0
1	5	25.42	1247.7	1463.6	1952.4	1664.6	792.4	176.7
1	6	30.50	1272.2	1490.8	1988.6	1697.0	815.0	168.2
1	7	35.58	1247.8	1463.7	1952.5	1664.8	792.5	176.7
1	8	40.67	1152.8	1353.7	1806.0	1538.1	724.8	174.0
1	9	45.75	981.5	1153.7	1539.3	1309.7	611.1	151.6
1	10	50.83	737.0	867.6	1157.8	983.7	452.5	116.2
1	11	55.92	409.2	482.2	643.6	546.2	248.8	67.7
1	12	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.6	16.9	0.2	0.0	21.7
1	1	5.08	3.9	14.1	0.2	0.0	18.1
1	2	10.17	3.1	11.3	0.2	0.0	14.5
1	3	15.25	2.3	8.5	0.2	0.0	10.9
1	4	20.33	1.5	5.6	0.0	0.0	7.2
1	5	25.42	0.8	2.8	0.0	0.0	3.6
1	6	30.50	0.0	0.0	0.0	0.0	0.0
1	7	35.58	-0.8	-2.8	0.0	0.0	-3.6
1	8	40.67	-1.5	-5.6	-0.1	0.0	-7.3
1	9	45.75	-2.3	-8.5	-0.1	0.0	-10.9
1	10	50.83	-3.1	-11.3	-0.1	0.0	-14.5
1	11	55.92	-3.9	-14.1	-0.1	0.0	-18.1
1	12	61.00	-4.6	-16.9	-0.1	0.0	-21.7

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	21.7	35.6
1	1	5.08	7.4	4.1	0.0	0.0	11.6	18.1	29.7
1	2	10.17	5.9	3.3	0.0	0.0	9.3	14.5	23.8
1	3	15.25	4.5	2.5	0.0	0.0	6.9	10.9	17.9
1	4	20.33	3.0	1.7	0.0	0.0	4.6	7.2	11.8
1	5	25.42	1.5	0.8	0.0	0.0	2.3	3.6	5.9
1	6	30.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	7	35.58	-1.5	-0.8	0.0	0.0	-2.3	-3.6	-5.9
1	8	40.67	-3.0	-1.7	0.0	0.0	-4.6	-7.3	-11.9
1	9	45.75	-4.5	-2.5	0.0	0.0	-6.9	-10.9	-17.9
1	10	50.83	-5.9	-3.3	0.0	0.0	-9.3	-14.5	-23.8
1	11	55.92	-7.4	-4.1	0.0	0.0	-11.6	-18.1	-29.7
1	12	61.00	-8.9	-5.0	0.0	0.0	-13.9	-21.7	-35.6

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	57.9	HL-93	0.0	HL-93

1	1	5.08	0.0	0.0	51.5	HL-93	-2.1	HL-93
1	2	10.17	0.0	0.0	45.2	HL-93	-5.4	HL-93
1	3	15.25	0.0	0.0	39.2	HL-93	-9.0	HL-93
1	4	20.33	0.0	0.0	33.3	HL-93	-12.7	HL-93
1	5	25.42	0.0	0.0	27.5	HL-93	-16.7	HL-93
1	6	30.50	0.0	0.0	21.9	HL-93	-21.9	HL-93
1	7	35.58	0.0	0.0	16.7	HL-93	-27.5	HL-93
1	8	40.67	0.0	0.0	12.7	HL-93	-33.3	HL-93
1	9	45.75	0.0	0.0	9.0	HL-93	-39.2	HL-93
1	10	50.83	0.0	0.0	5.4	HL-93	-45.2	HL-93
1	11	55.92	0.0	0.0	2.1	HL-93	-51.5	HL-93
1	12	61.00	0.0	0.0	0.0	HL-93	-57.9	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	24.	0.	24.
1	1	5.08	21.	-1.	22.
1	2	10.17	18.	-2.	21.
1	3	15.25	15.	-4.	19.
1	4	20.33	13.	-5.	18.
1	5	25.42	10.	-7.	17.
1	6	30.50	8.	-8.	15.
1	7	35.58	7.	-10.	17.
1	8	40.67	5.	-13.	18.
1	9	45.75	4.	-15.	19.
1	10	50.83	2.	-18.	21.
1	11	55.92	1.	-21.	22.
1	12	61.00	0.	-24.	24.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	23.7	.30655E-01	0.36	0.3125*	.15191E-01	0.18	0.3125*
1	1	5.08	22.1	.30655E-01	0.34	0.3125*	.15191E-01	0.17	0.3125*
1	2	10.17	20.5	.30655E-01	0.31	0.3125*	.15191E-01	0.16	0.3125*
1	3	15.25	19.1	.28520E-01	0.27	0.3125*	.92950E-02	0.09	0.3125*
1	4	20.33	18.1	.28520E-01	0.26	0.3125*	.92950E-02	0.08	0.3125*
1	5	25.42	16.7	.28520E-01	0.24	0.3125*	.92950E-02	0.08	0.3125*
1	6	30.50	15.3	.28520E-01	0.22	0.3125*	.92950E-02	0.07	0.3125*
1	7	35.58	16.7	.28520E-01	0.24	0.3125*	.92950E-02	0.08	0.3125*
1	8	40.67	18.1	.28520E-01	0.26	0.3125*	.92950E-02	0.08	0.3125*
1	9	45.75	19.1	.28520E-01	0.27	0.3125*	.92950E-02	0.09	0.3125*
1	10	50.83	20.5	.30655E-01	0.31	0.3125*	.15191E-01	0.16	0.3125*
1	11	55.92	22.1	.30655E-01	0.34	0.3125*	.15191E-01	0.17	0.3125*
1	12	61.00	23.7	.30655E-01	0.36	0.3125*	.15191E-01	0.18	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	35.6	57.9	HL-93	0.0	HL-93	93.4	35.6
1	1	5.08	29.7	51.5	HL-93	-2.1	HL-93	81.2	27.6
1	2	10.17	23.8	45.2	HL-93	-5.4	HL-93	69.0	18.4
1	3	15.25	17.9	39.2	HL-93	-9.0	HL-93	57.0	8.9
1	4	20.33	11.8	33.3	HL-93	-12.7	HL-93	45.1	-0.9
1	5	25.42	5.9	27.5	HL-93	-16.7	HL-93	33.4	-10.8
1	6	30.50	0.0	21.9	HL-93	-21.9	HL-93	21.9	-21.9
1	7	35.58	-5.9	16.7	HL-93	-27.5	HL-93	10.8	-33.4
1	8	40.67	-11.9	12.7	HL-93	-33.3	HL-93	0.8	-45.2
1	9	45.75	-17.9	9.0	HL-93	-39.2	HL-93	-8.9	-57.0
1	10	50.83	-23.8	5.4	HL-93	-45.2	HL-93	-18.3	-69.0
1	11	55.92	-29.7	2.1	HL-93	-51.5	HL-93	-27.5	-81.1
1	12	61.00	-35.6	0.0	HL-93	-57.9	HL-93	-35.6	-93.4

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	93.4	110.8	148.0	124.8	53.4	17.8
1	1	5.08	81.2	96.6	129.0	108.5	44.5	16.6
1	2	10.17	69.0	82.6	110.4	92.3	35.7	15.4
1	3	15.25	57.0	68.8	92.0	76.3	26.8	14.3
1	4	20.33	45.1	55.1	73.7	60.4	17.7	13.6
1	5	25.42	33.4	41.6	55.9	44.9	8.9	12.5

1	6	30.50	21.9	28.5	38.3	29.6	0.0	11.4
1	7	35.58	-33.4	-41.6	-55.8	-44.8	-8.8	12.5
1	8	40.67	-45.2	-55.2	-73.9	-60.6	-17.9	13.6
1	9	45.75	-57.0	-68.8	-92.0	-76.3	-26.8	14.3
1	10	50.83	-69.0	-82.6	-110.4	-92.3	-35.6	15.4
1	11	55.92	-81.1	-96.6	-129.0	-108.4	-44.5	16.6
1	12	61.00	-93.4	-110.8	-147.9	-124.8	-53.4	17.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	35.60	0.00	HL-93	57.85	HL-93	ST1	29.81	147.96
						ST2	29.81	124.82
						ST4	40.03	53.39
						SE1	35.60	93.45
						SE2	35.60	110.80
2	35.58	0.00	HL-93	57.85	HL-93	ST1	29.79	147.94
						ST2	29.79	124.80
						ST4	40.01	53.37
						SE1	35.58	93.43
						SE2	35.58	110.78

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	5.08	-0.0503	-0.1835	0.0000	-0.0793	-0.2358	-0.0793	-0.3151
1	2	10.17	-0.0957	-0.3492	0.0000	-0.1506	-0.4488	-0.1506	-0.5994
1	3	15.25	-0.1331	-0.4857	0.0000	-0.2091	-0.6242	-0.2091	-0.8334
1	4	20.33	-0.1614	-0.5892	0.0000	-0.2535	-0.7573	-0.2535	-1.0108
1	5	25.42	-0.1791	-0.6538	0.0000	-0.2812	-0.8403	-0.2812	-1.1214
1	6	30.50	-0.1851	-0.6757	0.0000	-0.2906	-0.8684	-0.2906	-1.1590
1	7	35.58	-0.1791	-0.6538	0.0000	-0.2812	-0.8402	-0.2812	-1.1214
1	8	40.67	-0.1614	-0.5892	0.0000	-0.2535	-0.7572	-0.2535	-1.0107
1	9	45.75	-0.1331	-0.4857	0.0000	-0.2091	-0.6241	-0.2091	-0.8333
1	10	50.83	-0.0957	-0.3492	0.0000	-0.1506	-0.4487	-0.1506	-0.5993
1	11	55.92	-0.0503	-0.1835	0.0000	-0.0793	-0.2358	-0.0793	-0.3151
1	12	61.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections

due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	5.1	-0.050	0 1/16	-0.186	0 3/16	-0.079	0 1/ 8	-0.315	0 3/ 8
1	2	10.2	-0.096	0 1/ 8	-0.353	0 3/ 8	-0.151	0 3/16	-0.599	0 5/ 8
1	3	15.2	-0.133	0 3/16	-0.491	0 1/ 2	-0.209	0 1/ 4	-0.833	0 7/ 8
1	4	20.3	-0.161	0 3/16	-0.596	0 5/ 8	-0.254	0 5/16	-1.011	1 1/16
1	5	25.4	-0.179	0 3/16	-0.661	0 11/16	-0.281	0 5/16	-1.121	1 1/ 8
1	6	30.5	-0.185	0 3/16	-0.683	0 11/16	-0.291	0 5/16	-1.159	1 3/16
1	7	35.6	-0.179	0 3/16	-0.661	0 11/16	-0.281	0 5/16	-1.121	1 1/ 8
1	8	40.7	-0.161	0 3/16	-0.596	0 5/ 8	-0.254	0 5/16	-1.011	1 1/16
1	9	45.7	-0.133	0 3/16	-0.491	0 1/ 2	-0.209	0 1/ 4	-0.833	0 7/ 8
1	10	50.8	-0.096	0 1/ 8	-0.353	0 3/ 8	-0.151	0 3/16	-0.599	0 5/ 8
1	11	55.9	-0.050	0 1/16	-0.186	0 3/16	-0.079	0 1/ 8	-0.315	0 3/ 8
1	12	61.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	FACTOR	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.50	3	0.425	-0.413	MAX HL-93	0.92	0.00330
				0.033	MIN		
				-0.121	MAX LANE		
				0.033	MIN LANE		
		4	0.433	-0.421	MAX HL-93	0.92	0.00330
				0.034	MIN		
				-0.123	MAX LANE		
				0.034	MIN LANE		
		3	0.425	-0.302	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	-0.64	0.64	-2.36	2.36	-3.00	3.00
1	2	10.17	-1.16	1.16	-4.29	4.29	-5.45	5.45
1	3	15.25	-1.42	0.99	-5.23	3.65	-6.65	4.64
1	4	20.33	-1.68	1.17	-6.20	4.33	-7.88	5.50
1	5	25.42	-1.84	1.28	-6.78	4.73	-8.61	6.01
1	6	30.50	-1.89	1.32	-6.97	4.87	-8.86	6.18
1	7	35.58	-1.84	1.28	-6.78	4.73	-8.62	6.01
1	8	40.67	-1.68	1.17	-6.20	4.33	-7.88	5.50
1	9	45.75	-1.42	0.99	-5.23	3.65	-6.64	4.64
1	10	50.83	-1.16	1.16	-4.28	4.28	-5.45	5.45
1	11	55.92	-0.64	0.64	-2.35	2.35	-2.99	2.99
1	12	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	-0.04	-0.60	1.47	-0.04	-3.60	4.47
1	2	10.17	-0.08	-1.10	2.68	-0.08	-6.55	8.13
1	3	15.25	-0.09	-1.46	2.31	-0.09	-8.11	6.95
1	4	20.33	-0.10	-1.73	2.73	-0.10	-9.61	8.23
1	5	25.42	-0.11	-1.89	2.99	-0.11	-10.50	9.00
1	6	30.50	-0.12	-1.94	3.07	-0.12	-10.80	9.26
1	7	35.58	-0.11	-1.89	2.99	-0.11	-10.51	9.00
1	8	40.67	-0.10	-1.73	2.73	-0.10	-9.61	8.23
1	9	45.75	-0.09	-1.46	2.31	-0.09	-8.10	6.94

1 10	50.83	-0.08	-1.10	2.68	-0.08	-6.55	8.12
1 11	55.92	-0.04	-0.60	1.47	-0.04	-3.60	4.47
1 12	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	-0.23	-0.69	4.98	0.00	0.00	0.00
1	2	10.17	-0.41	-1.24	8.91	0.00	0.00	0.00
1	3	15.25	-0.46	-1.89	7.52	0.00	0.00	0.00
1	4	20.33	-0.54	-2.21	8.77	0.00	0.00	0.00
1	5	25.42	-0.58	-2.37	9.42	0.00	0.00	0.00
1	6	30.50	-0.58	-2.41	9.54	0.00	0.00	0.00
1	7	35.58	-0.58	-2.37	9.42	0.00	0.00	0.00
1	8	40.67	-0.54	-2.21	8.77	0.00	0.00	0.00
1	9	45.75	-0.46	-1.89	7.52	0.00	0.00	0.00
1	10	50.83	-0.41	-1.24	8.91	0.00	0.00	0.00
1	11	55.92	-0.23	-0.69	4.98	0.00	0.00	0.00
1	12	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	5.08	0.3	0.0	0.3	C	HL	1.8	0.0	1.8	T	HL
1	2	10.17	0.4	0.0	0.4	C	HL	3.2	0.0	3.2	T	HL
1	3	15.25	0.7	0.0	0.7	C	HL	2.6	0.0	2.6	T	HL
1	4	20.33	0.8	0.0	0.8	C	HL	3.0	0.0	3.0	T	HL
1	5	25.42	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	6	30.50	0.7	0.0	0.7	C	HL	2.9	0.0	2.9	T	HL
1	7	35.58	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	8	40.67	0.8	0.0	0.8	C	HL	3.0	0.0	3.0	T	HL
1	9	45.75	0.7	0.0	0.7	C	HL	2.6	0.0	2.6	T	HL
1	10	50.83	0.4	0.0	0.4	C	HL	3.2	0.0	3.2	T	HL
1	11	55.92	0.3	0.0	0.3	C	HL	1.8	0.0	1.8	T	HL
1	12	61.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.27	-4.29	9.45	-0.04	-3.60	4.47
1	2	10.17	33.	33.	-0.49	-7.79	17.03	-0.08	-6.55	8.13
1	3	15.25	33.	33.	-0.55	-10.00	14.46	-0.09	-8.11	6.95
1	4	20.33	33.	33.	-0.64	-11.82	17.00	-0.10	-9.61	8.23
1	5	25.42	33.	33.	-0.69	-12.88	18.42	-0.11	-10.50	9.00
1	6	30.50	33.	33.	-0.70	-13.21	18.80	-0.12	-10.80	9.26
1	7	35.58	33.	33.	-0.69	-12.88	18.42	-0.11	-10.51	9.00
1	8	40.67	33.	33.	-0.64	-11.82	17.00	-0.10	-9.61	8.23
1	9	45.75	33.	33.	-0.55	-9.99	14.46	-0.09	-8.10	6.94
1	10	50.83	33.	33.	-0.49	-7.78	17.03	-0.08	-6.55	8.12
1	11	55.92	33.	33.	-0.27	-4.29	9.44	-0.04	-3.60	4.47
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.34	-4.50	10.94	-0.04	-3.60	4.47
1	2	10.17	33.	33.	-0.61	-8.16	19.71	-0.08	-6.55	8.13
1	3	15.25	33.	33.	-0.69	-10.57	16.72	-0.09	-8.11	6.95
1	4	20.33	33.	33.	-0.80	-12.48	19.63	-0.10	-9.61	8.23
1	5	25.42	33.	33.	-0.86	-13.59	21.25	-0.11	-10.50	9.00
1	6	30.50	33.	33.	-0.88	-13.93	21.67	-0.12	-10.80	9.26
1	7	35.58	33.	33.	-0.86	-13.59	21.25	-0.11	-10.51	9.00
1	8	40.67	33.	33.	-0.80	-12.48	19.63	-0.10	-9.61	8.23
1	9	45.75	33.	33.	-0.69	-10.56	16.71	-0.09	-8.10	6.94
1	10	50.83	33.	33.	-0.61	-8.15	19.70	-0.08	-6.55	8.12
1	11	55.92	33.	33.	-0.34	-4.50	10.94	-0.04	-3.60	4.47
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.46	-5.81	14.53	-0.06	-4.60	5.82
1	2	10.17	33.	33.	-0.83	-10.53	26.18	-0.11	-8.37	10.59
1	3	15.25	33.	33.	-0.93	-13.68	22.21	-0.12	-10.37	9.05
1	4	20.33	33.	33.	-1.09	-16.15	26.08	-0.15	-12.29	10.73
1	5	25.42	33.	33.	-1.17	-17.59	28.22	-0.16	-13.43	11.73
1	6	30.50	33.	33.	-1.19	-18.02	28.77	-0.16	-13.81	12.07
1	7	35.58	33.	33.	-1.17	-17.59	28.22	-0.16	-13.43	11.73
1	8	40.67	33.	33.	-1.09	-16.15	26.08	-0.15	-12.29	10.73
1	9	45.75	33.	33.	-0.93	-13.67	22.20	-0.12	-10.36	9.05
1	10	50.83	33.	33.	-0.83	-10.52	26.17	-0.11	-8.36	10.58
1	11	55.92	33.	33.	-0.46	-5.80	14.53	-0.06	-4.59	5.82
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.44	-4.35	12.50	-0.03	-3.14	3.79
1	2	10.17	33.	33.	-0.78	-7.88	22.47	-0.06	-5.72	6.89
1	3	15.25	33.	33.	-0.87	-10.38	19.04	-0.06	-7.06	5.88
1	4	20.33	33.	33.	-1.02	-12.24	22.32	-0.08	-8.37	6.97
1	5	25.42	33.	33.	-1.09	-13.30	24.11	-0.08	-9.15	7.62
1	6	30.50	33.	33.	-1.11	-13.62	24.54	-0.09	-9.41	7.84
1	7	35.58	33.	33.	-1.09	-13.31	24.11	-0.08	-9.15	7.62
1	8	40.67	33.	33.	-1.02	-12.24	22.32	-0.08	-8.37	6.97
1	9	45.75	33.	33.	-0.87	-10.37	19.03	-0.06	-7.06	5.88
1	10	50.83	33.	33.	-0.78	-7.88	22.47	-0.06	-5.71	6.88
1	11	55.92	33.	33.	-0.44	-4.35	12.50	-0.03	-3.14	3.78
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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COMPOSITE
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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.37	-5.53	12.54	-0.06	-4.60	5.82
1	2	10.17	33.	33.	-0.67	-10.04	22.61	-0.11	-8.37	10.59
1	3	15.25	33.	33.	-0.74	-12.92	19.20	-0.12	-10.37	9.05
1	4	20.33	33.	33.	-0.87	-15.27	22.57	-0.15	-12.29	10.73
1	5	25.42	33.	33.	-0.94	-16.64	24.45	-0.16	-13.43	11.73
1	6	30.50	33.	33.	-0.95	-17.06	24.95	-0.16	-13.81	12.07
1	7	35.58	33.	33.	-0.94	-16.64	24.45	-0.16	-13.43	11.73
1	8	40.67	33.	33.	-0.87	-15.27	22.57	-0.15	-12.29	10.73
1	9	45.75	33.	33.	-0.74	-12.92	19.20	-0.12	-10.36	9.05
1	10	50.83	33.	33.	-0.67	-10.03	22.61	-0.11	-8.36	10.58
1	11	55.92	33.	33.	-0.37	-5.53	12.54	-0.06	-4.59	5.82
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.08	33.	33.	-0.34	-4.08	10.51	-0.03	-3.14	3.79
1	2	10.17	33.	33.	-0.62	-7.39	18.91	-0.06	-5.72	6.89
1	3	15.25	33.	33.	-0.69	-9.62	16.03	-0.06	-7.06	5.88
1	4	20.33	33.	33.	-0.80	-11.35	18.81	-0.08	-8.37	6.97
1	5	25.42	33.	33.	-0.86	-12.35	20.34	-0.08	-9.15	7.62
1	6	30.50	33.	33.	-0.87	-12.66	20.72	-0.09	-9.41	7.84
1	7	35.58	33.	33.	-0.86	-12.36	20.34	-0.08	-9.15	7.62
1	8	40.67	33.	33.	-0.80	-11.35	18.81	-0.08	-8.37	6.97
1	9	45.75	33.	33.	-0.69	-9.61	16.03	-0.06	-7.06	5.88
1	10	50.83	33.	33.	-0.62	-7.38	18.90	-0.06	-5.71	6.88
1	11	55.92	33.	33.	-0.34	-4.07	10.50	-0.03	-3.14	3.78
1	12	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. STRESS (k-ft)	(ksi)	DL2, N=27.0 MOM. STRESS (k-ft)	(ksi)	POS. LL+I MOM. STRESS (k-ft)	(ksi)	NEG. LL+I MOM. STRESS (k-ft)	(ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.00	195.	-5.8	124.	-1.2	458.	-1.3	0.	0.0	-8.23
											1.30
	LB			5.8		2.8		9.4		0.0	17.97
											9.37
	RT			-5.2		-1.1		-1.5		0.0	-7.86
											1.51
	RB			3.6		1.8		6.0		0.0	11.44
											6.00
1	LT	50.00	195.	-5.2	124.	-1.1	458.	-1.5	0.	0.0	-7.85
											1.51
	LB			3.6		1.8		6.0		0.0	11.43
											6.00
	RT			-5.8		-1.2		-1.3		0.0	-8.22
											1.30
	RB			5.8		2.8		9.4		0.0	17.96
											9.37

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

```
                 **********************************

                                                          COVER PLATE -- (in)
 MEMBER          TOP FLANGE     WEB       BOT FLANGE  -------------------------
--------- STEEL ----------- ------------ ------------     TOP          BOT
NO LENGTH  TYPE WIDTH THICK DEPTH THICK  WIDTH THICK  -----------  ------------
    (ft)           (in)        (in)          (in)     WIDTH THICK  WIDTH THICK
-------------------------------------------------------------------------------

 1  11.00    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

 2  39.00    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 10.00 1.0000

 3  11.00    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

-------------------------------------------------------------------------------
NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
               ************

 ** [1] = Span length (ft) =   61.0
                                                          LIMITATION
  MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
--------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
    (ft)   (in)   (in)       (in)
-------------------------------------------------------------------------

 1  11.00  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

 2  39.00  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

 3  11.00  6.75   33.09 [2]=   41.23     0.05633      >   0.04000    OK

                        [3]=   33.09     0.04520      >   0.03300    OK

-------------------------------------------------------------------------

 NOTE : [1] -- Span length or Average of two adjacent span lengths

        [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

            * For composite girders, the minimum overall depth
              of girder (concrete slab plus haunch & girder) preferably should not
              be less than 0.040*Length of span (or 0.032*L if continuous spans).
              and the depth of steel girder alone preferably should not be
              less than 0.033*Length of span (or 0.027*L if continuous)

        [3] -- same as NOTE [2]

         ** -- cover plates not taken into account

         ** -- these criteria are related to the structural stability
               during construction and the limitation for the live load
               deflection.
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TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                 ******************************

SP IN D FROM  Lo Co       Web                                          Web
NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
```

			[1]					[2]	[3]	[4]	[5]	[6]
1	0	0.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	1	5.08	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	2	10.17	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	3	15.25	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	4	20.33	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	5	25.42	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	6	30.50	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	7	35.58	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	8	40.67	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	9	45.75	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	10	50.83	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	11	55.92	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	12	61.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	5.08	0	1	31.39	0.579	54.21	168.97	2
1	2	10.17	0	1	31.39	0.579	54.21	168.97	2
1	3	15.25	0	1	31.39	0.579	66.55	168.97	2
1	4	20.33	0	1	31.39	0.579	66.55	168.97	2
1	5	25.42	0	1	31.39	0.579	66.55	168.97	2
1	6	30.50	0	1	31.39	0.579	66.55	168.97	2
1	7	35.58	0	1	31.39	0.579	66.55	168.97	2
1	8	40.67	0	1	31.39	0.579	66.55	168.97	2
1	9	45.75	0	1	31.39	0.579	66.55	168.97	2
1	10	50.83	0	1	31.39	0.579	54.21	168.97	2
1	11	55.92	0	1	31.39	0.579	54.21	168.97	2
1	12	61.00	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 1	5.08	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 2	10.17	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 3	15.25	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 4	20.33	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 5	25.42	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 6	30.50	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 7	35.58	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 8	40.67	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 9	45.75	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 10	50.83	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 11	55.92	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 12	61.00	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	17.8	0.000	10.903	7.264	3.624	1.000
			0.000	0.000	0.000	0.000	1.000
2	17.8	39.4	10.903	12.092	13.150	14.208	1.000
			0.000	0.000	0.000	0.000	1.000
3	39.4	61.0	0.000	12.092	8.391	4.691	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange

is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	6.76	11.26	19.84	0	17.81	7.17	26.91	2	2
1	1	5.08	1	6.76	11.26	19.84	0	17.81	7.17	26.91	2	2
1	2	10.17	1	6.76	11.26	19.84	0	17.81	7.17	26.91	2	2
1	3	15.25	1	6.76	11.26	19.84	0	17.81	6.98	26.21	2	2
1	4	20.33	1	6.76	11.26	19.84	0	21.60	6.98	26.21	2	2
1	5	25.42	1	6.76	11.26	19.84	0	21.60	6.98	26.21	2	2
1	6	30.50	1	6.76	11.26	19.84	0	21.60	6.98	26.21	2	2
1	7	35.58	1	6.76	11.26	19.84	0	21.60	6.98	26.21	2	2
1	8	40.67	1	6.76	11.26	19.84	0	21.59	6.98	26.21	2	2
1	9	45.75	1	6.76	11.26	19.84	0	21.59	6.98	26.21	2	2
1	10	50.83	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	11	55.92	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	12	61.00	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.00	33.00	27.66	27.66
1	1	5.08	1	1.000	1.000	1.00	33.00	27.66	27.66
1	2	10.17	1	1.000	1.000	1.00	33.00	27.66	27.66
1	3	15.25	1	1.000	1.000	1.00	33.00	27.42	27.42
1	4	20.33	1	1.000	1.000	1.00	33.00	25.47	25.47
1	5	25.42	1	1.000	1.000	1.00	33.00	25.47	25.47
1	6	30.50	1	1.000	1.000	1.00	33.00	25.47	25.47

1	7	35.58	1	1.000	1.000	1.00	33.00	25.47	25.47
1	8	40.67	1	1.000	1.000	1.00	33.00	25.48	25.48
1	9	45.75	1	1.000	1.000	1.00	33.00	25.48	25.48
1	10	50.83	1	1.000	1.000	1.00	33.00	25.77	25.77
1	11	55.92	1	1.000	1.000	1.00	33.00	25.77	25.77
1	12	61.00	1	1.000	1.000	1.00	33.00	25.77	25.77

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0	0.00	1	8.19	17.32
1	1	5.08	1	8.19	17.32
1	2	10.17	1	8.19	17.32
1	3	15.25	1	8.63	17.74
1	4	20.33	1	8.63	17.74
1	5	25.42	1	8.63	17.74
1	6	30.50	1	8.63	17.74
1	7	35.58	1	8.63	17.74
1	8	40.67	1	8.63	17.74
1	9	45.75	1	8.63	17.74
1	10	50.83	1	8.19	17.32
1	11	55.92	1	8.19	17.32
1	12	61.00	1	8.19	17.32

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	5.08	1	0	2	0		2	0
1	2	10.17	1	0	2	0		2	0
1	3	15.25	1	0	2	0		2	0
1	4	20.33	1	0	2	0		2	0
1	5	25.42	1	0	2	0		2	0
1	6	30.50	1	0	2	0		2	0
1	7	35.58	1	0	2	0		2	0
1	8	40.67	1	0	2	0		2	0
1	9	45.75	1	0	2	0		2	0
1	10	50.83	1	0	2	0		2	0
1	11	55.92	1	0	2	0		2	0
1	12	61.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.7	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		5.08	0.0	19.8	4.5	33.0	4.5	33.0	4.5	27.7	0
				0.0	-	4.5	-	4.5	33.0	-	-	0
1	2		10.17	0.0	19.8	8.2	33.0	8.2	33.0	8.2	27.7	0
				0.0	-	8.2	-	8.2	33.0	-	-	0
1	3		15.25	0.0	19.8	10.0	33.0	10.0	33.0	10.0	27.4	0
				0.0	-	7.0	-	7.0	33.0	-	-	0
1	4		20.33	0.0	19.8	11.8	33.0	11.8	33.0	11.8	25.5	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	5		25.42	0.0	19.8	12.9	33.0	12.9	33.0	12.9	25.5	0
				0.0	-	9.0	-	9.0	33.0	-	-	0
1	6		30.50	0.0	19.8	13.3	33.0	13.3	33.0	13.3	25.5	0
				0.0	-	9.3	-	9.3	33.0	-	-	0
1	7		35.58	0.0	19.8	12.9	33.0	12.9	33.0	12.9	25.5	0
				0.0	-	9.0	-	9.0	33.0	-	-	0
1	8		40.67	0.0	19.8	11.8	33.0	11.8	33.0	11.8	25.5	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	9		45.75	0.0	19.8	10.0	33.0	10.0	33.0	10.0	25.5	0
				0.0	-	7.0	-	7.0	33.0	-	-	0
1	10		50.83	0.0	19.8	8.2	33.0	8.2	33.0	8.2	25.8	0
				0.0	-	8.2	-	8.2	33.0	-	-	0
1	11		55.92	0.0	19.8	4.5	33.0	4.5	33.0	4.5	25.8	0
				0.0	-	4.5	-	4.5	33.0	-	-	0
1	12		61.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	25.8	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	5.08	0.000	0.136	0.136	0.162	0.162	4
					0.136		0.136	3
1	2	10.17	0.000	0.248	0.248	0.296	0.296	4
					0.248		0.248	3
1	3	15.25	0.000	0.302	0.302	0.364	0.364	4
					0.211		0.211	3
1	4	20.33	0.000	0.358	0.358	0.464	0.464	4
					0.250		0.250	3
1	5	25.42	0.000	0.391	0.391	0.507	0.507	4
					0.273		0.273	3
1	6	30.50	0.000	0.403	0.403	0.522	0.522	4
					0.281		0.281	3
1	7	35.58	0.000	0.392	0.392	0.507	0.507	4
					0.273		0.273	3
1	8	40.67	0.000	0.358	0.358	0.464	0.464	4
					0.250		0.250	3
1	9	45.75	0.000	0.302	0.302	0.391	0.391	4
					0.211		0.211	3
1	10	50.83	0.000	0.248	0.248	0.317	0.317	4
					0.248		0.248	3
1	11	55.92	0.000	0.136	0.136	0.174	0.174	4
					0.136		0.136	3
1	12	61.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2046.3	-	-	0.0	19.8	-	-	0--0
1	1		5.08	0	643.7	2046.3	-	-	0.0	19.8	-	-	0--0
1	2		10.17	0	1158.1	2046.3	-	-	0.0	19.8	-	-	0--0
1	3		15.25	0	1539.7	2890.0	-	-	0.0	19.8	-	-	0--0
1	4		20.33	0	1806.0	2890.0	-	-	0.0	19.8	-	-	0--0
1	5		25.42	0	1952.4	2890.0	-	-	0.0	19.8	-	-	0--0
1	6		30.50	0	1988.6	2890.0	-	-	0.0	19.8	-	-	0--0
1	7		35.58	0	1952.5	2890.0	-	-	0.0	19.8	-	-	0--0
1	8		40.67	0	1806.0	2890.0	-	-	0.0	19.8	-	-	0--0
1	9		45.75	0	1539.3	2890.0	-	-	0.0	19.8	-	-	0--0
1	10		50.83	0	1157.8	2046.3	-	-	0.0	19.8	-	-	0--0
1	11		55.92	0	643.6	2046.3	-	-	0.0	19.8	-	-	0--0
1	12		61.00	0	0.0	2046.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		5.08	0	0.315				0.315	1
1	2		10.17	0	0.566				0.566	1
1	3		15.25	0	0.533				0.533	1
1	4		20.33	0	0.625				0.625	1
1	5		25.42	0	0.676				0.676	1
1	6		30.50	0	0.688				0.688	1
1	7		35.58	0	0.676				0.676	1
1	8		40.67	0	0.625				0.625	1
1	9		45.75	0	0.533				0.533	1
1	10		50.83	0	0.566				0.566	1

```
 1 11  55.92  0        0.315                                  0.315     1
 1 12  61.00  0        0.000                                  0.000     1


 ----------------------------------------------------------------------

 NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
       [2] = (PHI)f * Fnc for comp. flange of composite
             sections in positive flexure (Eq. 6.10.7.2.1-1)
           = (PHI)f * Rh * Fyt for tension flange of composite sections in
             negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
       [3] = 0.6*Fyt for composite sections in positive flexure
             or tension flange for composite sections in
             negative flexure and non-composite sections (Eq. 6.10.1.6-1)
           = 0.6*Fyc for comp. flange of composite sections in
             negative flexure and non-composite sections (Eq. 6.10.1.6-1)
       [4] = (PHI)f * Fnt for non-compact tension flange
             of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
           = (PHI)f * Fnc for comp. flange of composite sections
             in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

       The governing number is listed as below.
         1 = (Mu+1/3flSxt) / [1]
         2 = fbu / [2]
         3 = fl / [3]
         4 = (fbu + 1/3fl) / [4]

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 TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
                 *********************************

 SP IN D FROM  Fy     k     Art.6.10.9.2         C     Vp        SHEAR CAPACITY
 NO NO L SUPT ----- ----- -------------- D/tw  ----- ----- ----- -Eq.6.10.9.2--
        (ft)  (ksi)  [1]    [2]    [3]          [4]  (ksi)        Vn = C x Vp
 ---------------------------------------------------------------------------

  1  0   0.00  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  1   5.08  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  2  10.17  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  3  15.25  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  4  20.33  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  5  25.42  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  6  30.50  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  7  35.58  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  8  40.67  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1  9  45.75  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1 10  50.83  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1 11  55.92  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9
  1 12  61.00  33.0  5.0    74.2   92.8  54.2  1.000   347.9     347.9


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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0
```

```
For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                 ..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                 ..... AASHTO LRFD Eq.6.10.9.3.2-7


** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
   stiffened
   if (d0/D) < 3 without long. stiffener, or
   if (d0/D) > 3 with both transv. and long. stiffeners.
   For handling requirement, (d0/D) > (260/(D/tw))**2

        Vu = CVp

   where D = clear, unsupported distance between flange components

        d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot
   count on post-buckling shear resistance from tension-
   field action for an unstiffened girder.
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    TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                   ********************************

                         S T R E N G T H  C A T E G O R Y; SEE NOTE
                  ------------------------------------------------------------
    SP IN D FROM WEB STABILITY    LFD/LRFD      UNSTIFFENED            OVERALL
    NO NO L SUPT UNSTIFFENED,[1] MAX. SHEAR <=> SHEAR CAPA- CATEGORY     WEB
           (ft)  ---- D/tw -----    (kip)       CITY,   [2]
    ---------------------------------------------------------------------------

     1  0   0.00       0            148.0   <      347.9       0          0
     1  1   5.08       0            129.0   <      347.9       0          0
     1  2  10.17       0            110.4   <      347.9       0          0
     1  3  15.25       0             92.0   <      347.9       0          0
     1  4  20.33       0             73.7   <      347.9       0          0
     1  5  25.42       0             55.9   <      347.9       0          0
     1  6  30.50       0             38.3   <      347.9       0          0
     1  7  35.58       0             55.8   <      347.9       0          0
     1  8  40.67       0             73.9   <      347.9       0          0
     1  9  45.75       0             92.0   <      347.9       0          0
     1 10  50.83       0            110.4   <      347.9       0          0
     1 11  55.92       0            129.0   <      347.9       0          0
     1 12  61.00       0            147.9   <      347.9       0          0


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     NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

               For compact section  (composite sections in positive flexure)

                 w/o long. stiff.

                 D/tw <= 150 (Eq. 6.10.2.1.1-1)

                 w / long. stiff.

                 D/tw <= 300 (Eq. 6.10.2.1.2-1)

                 2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1
```

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	147.96	347.87	0	
1	1	5.08	33.	129.04	347.87	0	
1	2	10.17	33.	110.39	347.87	0	
1	3	15.25	33.	92.01	347.87	0	
1	4	20.33	33.	73.73	347.87	0	
1	5	25.42	33.	55.85	347.87	0	
1	6	30.50	33.	38.34	347.87	0	
1	7	35.58	33.	55.83	347.87	0	
1	8	40.67	33.	73.89	347.87	0	
1	9	45.75	33.	91.99	347.87	0	
1	10	50.83	33.	110.37	347.87	0	
1	11	55.92	33.	129.02	347.87	0	
1	12	61.00	33.	147.94	347.87	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	TOP FLANGE [1] (ksi)	BOT. FLANGE ff+fl/2 (ksi)	BOT. FLANGE [2] (ksi)	COMP. FLG fc (ksi)	FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	5.08	4.5	-	31.4	10.9	31.4	-	-	00-
1	2	10.17	8.2	-	31.4	19.7	31.4	-	-	00-

```
      1  3  15.25  10.6    -     31.4    16.7  31.4   -     -   00-
      1  4  20.33  12.5    -     31.4    19.6  31.4   -     -   00-
      1  5  25.42  13.6    -     31.4    21.2  31.4   -     -   00-
      1  6  30.50  13.9    -     31.4    21.7  31.4   -     -   00-
      1  7  35.58  13.6    -     31.4    21.3  31.4   -     -   00-
      1  8  40.67  12.5    -     31.4    19.6  31.4   -     -   00-
      1  9  45.75  10.6    -     31.4    16.7  31.4   -     -   00-
      1 10  50.83   8.2    -     31.4    19.7  31.4   -     -   00-
      1 11  55.92   4.5    -     31.4    10.9  31.4   -     -   00-
      1 12  61.00   0.0    -     31.4     0.0  31.4   -     -   00-


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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ***************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   5.08    TR     -0.4 A B B^C^C D E E^  TR      2.8 A B B^C^C D E
  1  2  10.17    TR     -0.7 A B B^C^C D E E^  TR      4.8 A B B^C^C D
  1     11.00  SCG
  1  3  15.25    TR     -1.0 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
  1  4  20.33    TR     -1.1 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  5  25.42    TR     -1.2 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  6  30.50    TR     -1.1 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
  1  7  35.58    TR     -1.2 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  8  40.67    TR     -1.1 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  9  45.75    TR     -1.0 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
  1     50.00  SCG
  1 10  50.83    TR     -0.7 A B B^C^C D E E^  TR      4.8 A B B^C^C D
  1 11  55.92    TR     -0.4 A B B^C^C D E E^  TR      2.8 A B B^C^C D E
  1 12  61.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
```

```
 ---------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ---------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   5.08    TR     -0.2 A B B^C^C D E E^  TR      1.4 A B B^C^C D E E^
  1  2  10.17    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1     11.00  SCG
  1  3  15.25    TR     -0.5 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1  4  20.33    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  5  25.42    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  6  30.50    TR     -0.6 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
  1  7  35.58    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  8  40.67    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  9  45.75    TR     -0.5 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1     50.00  SCG
  1 10  50.83    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1 11  55.92    TR     -0.2 A B B^C^C D E E^  TR      1.4 A B B^C^C D E E^
  1 12  61.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ---------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ---------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	35.5	.28332E-01	1.0	TR	3.09	3	9.22
1	1	5.08	33.2	.28332E-01	0.9	TR	3.09	3	9.87
1	2	10.17	30.8	.28332E-01	0.9	TR	3.09	3	10.65
1	3	15.25	28.7	.25851E-01	0.7	TR	3.09	3	12.52
1	4	20.33	27.2	.25851E-01	0.7	TR	3.09	3	13.21
1	5	25.42	25.0	.25851E-01	0.6	TR	3.09	3	14.37
1	6	30.50	22.9	.25851E-01	0.6	TR	3.09	3	15.69
1	7	35.58	25.0	.25851E-01	0.6	TR	3.09	3	14.37
1	8	40.67	27.2	.25851E-01	0.7	TR	3.09	3	13.21
1	9	45.75	28.7	.25851E-01	0.7	TR	3.09	3	12.52
1	10	50.83	30.8	.28332E-01	0.9	TR	3.09	3	10.65
1	11	55.92	33.2	.28332E-01	0.9	TR	3.09	3	9.87
1	12	61.00	35.5	.28332E-01	1.0	TR	3.09	3	9.22

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	<--Input [5], input	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	17.8	.28332E-01	0.5	TR	1.36	3	8.13
1	1	5.08	16.6	.28332E-01	0.5	TR	1.36	3	8.71
1	2	10.17	15.4	.28332E-01	0.4	TR	1.36	3	9.39
1	3	15.25	14.3	.25851E-01	0.4	TR	1.36	3	11.04
1	4	20.33	13.6	.25851E-01	0.4	TR	1.36	3	11.65
1	5	25.42	12.5	.25851E-01	0.3	TR	1.36	3	12.67
1	6	30.50	11.4	.25851E-01	0.3	TR	1.36	3	13.84
1	7	35.58	12.5	.25851E-01	0.3	TR	1.36	3	12.67
1	8	40.67	13.6	.25851E-01	0.4	TR	1.36	3	11.65
1	9	45.75	14.3	.25851E-01	0.4	TR	1.36	3	11.04
1	10	50.83	15.4	.28332E-01	0.4	TR	1.36	3	9.39
1	11	55.92	16.6	.28332E-01	0.5	TR	1.36	3	8.71
1	12	61.00	17.8	.28332E-01	0.5	TR	1.36	3	8.13

Please read NOTE on the following page

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

```
** The connector spacing shown here is calculated based upon
   AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
   where ADTT Single Lane= 1140 & No. of Cycles=   31207500
*** If ADTT Single Lane greater than or equal to 960, refer
   to Fatigue I Table.
```

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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                 ***************************************

                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  6  30.50   1 1574.9  1204.9          123         >  63
      1 12  61.00   1 1574.9  1204.9          123         >  63


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
             Qr = factored shear resistance of one shear connector
                = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
             Qn = nominal shear resistance
             (Phi)sc = resistance factor for shear connectors

      [3] The total horizontal shear force between the point of
           maximum positive moment and each adjacent point of 0.0 moment
           shall be the lesser either:

                P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
           or
             P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
           Where
             f'c = specified 28-day compressive strength of the concrete
             bs = effective width of the slab
             bfc = width of compression flange
             bft = width of tension flange
             ts = slab thickness
             Fyw = specified minimum yield strength of the web
             Fyt = specified minimum yield strength of the tension flange
             Fyc = specified minimum yield strength of the compression flange
```

D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	30.500	9.000
1	30.500	61.000	9.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	61.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK

OK 1.2.22.14=STRENGTH LIMIT STATE CHECK

OK 1.2.22.17=TRANSVERSE STIFFENER SPACING

OK 1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2046.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	5.08	2046.3	101.3	64.7	243.4	0.0	5.57	HL-93	4.29	0
1	2	10.17	2046.3	184.3	117.6	435.4	0.0	2.81	HL-93	2.17	0
1	3	15.25	2890.0	249.0	158.7	574.0	0.0	3.04	HL-93	2.34	0
1	4	20.33	2890.0	295.1	188.1	669.6	0.0	2.50	HL-93	1.93	0
1	5	25.42	2890.0	322.5	205.8	719.5	0.0	2.26	HL-93	1.74	0
1	6	30.50	2890.0	331.7	211.6	728.8	0.0	2.21	HL-93	1.71	0
1	7	35.58	2890.0	322.6	205.8	719.5	0.0	2.26	HL-93	1.74	0
1	8	40.67	2890.0	295.1	188.1	669.6	0.0	2.50	HL-93	1.93	0
1	9	45.75	2890.0	248.7	158.7	574.0	0.0	3.04	HL-93	2.34	0
1	10	50.83	2046.3	184.1	117.6	435.4	0.0	2.81	HL-93	2.17	0
1	11	55.92	2046.3	101.2	64.7	243.4	0.0	5.57	HL-93	4.29	0
1	12	61.00	2046.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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1

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD RATING TYPE FACTOR
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68
1	1	5.08	31.4	-3.60 4.47	-0.69 4.98	0.00 0.00	5.40	HL-93 4.15
1	2	10.17	31.4	-6.55 8.13	-1.24 8.91	0.00 0.00	2.61	HL-93 2.01
1	3	15.25	31.4	-8.11 6.95	-1.89 7.52	0.00 0.00	3.25	HL-93 2.50
1	4	20.33	31.4	-9.61 8.23	-2.21 8.77	0.00 0.00	2.64	HL-93 2.03
1	5	25.42	31.4	-10.50 9.00	-2.37 9.42	0.00 0.00	2.37	HL-93 1.82
1	6	30.50	31.4	-10.80 9.26	-2.41 9.54	0.00 0.00	2.31	HL-93 1.78
1	7	35.58	31.4	-10.51 9.00	-2.37 9.42	0.00 0.00	2.37	HL-93 1.82
1	8	40.67	31.4	-9.61 8.23	-2.21 8.77	0.00 0.00	2.64	HL-93 2.03
1	9	45.75	31.4	-8.10 6.94	-1.89 7.52	0.00 0.00	3.25	HL-93 2.50
1	10	50.83	31.4	-6.55 8.12	-1.24 8.91	0.00 0.00	2.61	HL-93 2.01
1	11	55.92	31.4	-3.60 4.47	-0.69 4.98	0.00 0.00	5.40	HL-93 4.15
1	12	61.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN CRITICAL LOCATION GOVERNING GOVERNING

NO.	LENGTH (ft)	D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	LIVE LOAD TYPE
1	61.00	30.50	STRENGTH L.S.	2.21	1.71	HL-93
1	61.00	30.50	SERVICEBILITY	2.31	1.78	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.212
Inventory= 1.707

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	61.00	35.58	STRENGTH L.S.	3.19	688.38
1	61.00	35.58	SERVICEBILITY	3.18	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 3.179

1

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I D2	MAX. SHEAR POS.	MAX. SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	21.7	13.9	57.9	0.0	3.86	HL-93	2.97
1	1	5.08	347.9	18.1	11.6	51.5	-2.1	4.45	HL-93	3.43
1	2	10.17	347.9	14.5	9.3	45.2	-5.4	5.18	HL-93	4.00
1	3	15.25	347.9	10.9	6.9	39.2	-9.0	6.13	HL-93	4.73
1	4	20.33	347.9	7.2	4.6	33.3	-12.7	7.40	HL-93	5.71
1	5	25.42	347.9	3.6	2.3	27.5	-16.7	9.17	HL-93	7.07
1	6	30.50	347.9	0.0	0.0	21.9	-21.9	9.99	HL-93	9.08
1	7	35.58	347.9	-3.6	-2.3	16.7	-27.5	9.17	HL-93	7.07
1	8	40.67	347.9	-7.3	-4.6	12.7	-33.3	7.40	HL-93	5.71
1	9	45.75	347.9	-10.9	-6.9	9.0	-39.2	6.13	HL-93	4.73
1	10	50.83	347.9	-14.5	-9.3	5.4	-45.2	5.18	HL-93	4.00
1	11	55.92	347.9	-18.1	-11.6	2.1	-51.5	4.45	HL-93	3.43
1	12	61.00	347.9	-21.7	-13.9	0.0	-57.9	3.86	HL-93	2.97

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	RATING	TYPE
1	61.00	0.00	STRENGTH L.S.	3.86	2.97	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.856
Inventory= 2.975

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING CRITERION	RATING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	61.00	0.00	STRENGTH L.S.	4.68		64.35

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 4.680

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	30.50	3 0.425	-0.41	HL-93	2.21
		4 0.433	-0.42		2.17

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 1 Exterior Fatigue   8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       16           1       AASHTO  2010     0     2            3
 ----------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
1
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                                                                INPUT
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.24	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130			0.00	0.0000	0.00	0.0000		
2	W	33	130			0.00	0.00001	0.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.00				33.	33.	33.
2	2	2	39.00				33.	33.	33.
3	1	1	11.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING	TANDEM LIVE LOAD	AASHTO ROAD TYPE			SIDEWALK
		1,2,3 OR 4			LIVE LOAD
HL - 93	1=YES : 0=NO	ADTT	ADTTSL		(k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M-ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.35	0.00	15.00	1 1 1 1 1	0.53	2 2 2 2 2
1	0.00	0	0.00	0.41	0.00	0.00	0 0 0 0 0	0.57	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied
to the indicated loading type.

1 = The special distribution factor defined is applied to
the indicated loading type of calculation of all
moment, shear and deflection.

2 = The special distribution factor defined is only applied
to the loading type for calculating moment.

3 = The special distribution factor defined is only applied
to the loading type for calculating shear.

4 = The special distribution factor defined is only applied
to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	61.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those
of normal weight concrete (W=145 pcf) of various design strength shall
be as follows:

fc' = unit ultimate compressive strength of concrete as determined
by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	17.81
2	2	Diaphragm	0.000	0.00	0.00	0.14	39.41
3	1	Railing	0.087	0.00	61.00	0.00	0.00
4	1	Curb	0.076	0.00	61.00	0.00	0.00
5	0	Pavement	0.292	0.00	61.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi) (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	17.81	33.0	33.0	17.81
17.81	39.41	33.0	33.0	21.60
39.41	61.00	33.0	33.0	21.59

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.555       0.1520     0.7070


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      61.00

     1   DC2     0.087       0.000     61.000

     2   DC2     0.076       0.000     61.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1481.2	54.9	222.2	55.6	277.82

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.526	0.526	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.575				FOR STRENGTH POSITIVE SHEAR
1	0.347				FOR FATIGUE POSITIVE MOMENT
	0.408				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.81	6710.0	33.1	16.55	405.6	405.6
1	2	7.62	6710.0	33.1	16.55	405.6	405.6
1	3	11.44	9020.9	33.1	14.02	643.6	449.4
1	4	15.25	9020.9	33.1	14.02	643.6	449.4
1	5	19.06	9020.9	33.1	14.02	643.6	449.4
1	6	22.87	9020.9	33.1	14.02	643.6	449.4
1	7	26.69	9020.9	33.1	14.02	643.6	449.4
1	8	30.50	9020.9	33.1	14.02	643.6	449.4
1	9	34.31	9020.9	33.1	14.02	643.6	449.4
1	10	38.12	9020.9	33.1	14.02	643.6	449.4
1	11	41.94	9020.9	33.1	14.02	643.6	449.4
1	12	45.75	9020.9	33.1	14.02	643.6	449.4

1 13	49.56	9020.9	33.1	14.02	643.6	449.4
1 14	53.37	6710.0	33.1	16.55	405.6	405.6
1 15	57.19	6710.0	33.1	16.55	405.6	405.6
1 16	61.00	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12362.3	0.021678	527.0	1283.3	695.6
1	1	3.81	12362.3	0.021678	527.0	1283.3	695.6
1	2	7.62	12362.3	0.021678	527.0	1283.3	695.6
1	3	11.44	17252.9	0.019493	826.1	1306.4	808.2
1	4	15.25	17252.9	0.019493	826.1	1306.4	808.2
1	5	19.06	17252.9	0.019493	826.1	1306.4	808.2
1	6	22.87	17252.9	0.019493	826.1	1306.4	808.2
1	7	26.69	17252.9	0.019493	826.1	1306.4	808.2
1	8	30.50	17252.9	0.019493	826.1	1306.4	808.2
1	9	34.31	17252.9	0.019493	826.1	1306.4	808.2
1	10	38.12	17252.9	0.019493	826.1	1306.4	808.2
1	11	41.94	17252.9	0.019493	826.1	1306.4	808.2
1	12	45.75	17252.9	0.019493	826.1	1306.4	808.2
1	13	49.56	17252.9	0.019493	826.1	1306.4	808.2
1	14	53.37	12362.3	0.021678	527.0	1283.3	695.6
1	15	57.19	12362.3	0.021678	527.0	1283.3	695.6
1	16	61.00	12362.3	0.021678	527.0	1283.3	695.6

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	17045.3	0.028332	586.7	4224.1	1400.0
1	1	3.81	17045.3	0.028332	586.7	4224.1	1400.0
1	2	7.62	17045.3	0.028332	586.7	4224.1	1400.0
1	3	11.44	24951.9	0.025851	916.4	3636.3	1663.3
1	4	15.25	24951.9	0.025851	916.4	3636.3	1663.3
1	5	19.06	24951.9	0.025851	916.4	3636.3	1663.3
1	6	22.87	24951.9	0.025851	916.4	3636.3	1663.3
1	7	26.69	24951.9	0.025851	916.4	3636.3	1663.3
1	8	30.50	24951.9	0.025851	916.4	3636.3	1663.3
1	9	34.31	24951.9	0.025851	916.4	3636.3	1663.3
1	10	38.12	24951.9	0.025851	916.4	3636.3	1663.3
1	11	41.94	24951.9	0.025851	916.4	3636.3	1663.3

1 12	45.75	24951.9	0.025851	916.4	3636.3	1663.3
1 13	49.56	24951.9	0.025851	916.4	3636.3	1663.3
1 14	53.37	17045.3	0.028332	586.7	4224.1	1400.0
1 15	57.19	17045.3	0.028332	586.7	4224.1	1400.0
1 16	61.00	17045.3	0.028332	586.7	4224.1	1400.0

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	3.81	16.6	60.5	0.6	0.0	77.7
1	2	7.62	30.9	112.9	1.2	0.0	145.0
1	3	11.44	43.1	157.3	1.7	0.0	202.2
1	4	15.25	53.0	193.6	2.3	0.0	249.0
1	5	19.06	60.8	221.8	2.7	0.0	285.4
1	6	22.87	66.3	242.0	2.8	0.0	311.1
1	7	26.69	69.6	254.1	2.8	0.0	326.5
1	8	30.50	70.7	258.1	2.8	0.0	331.7
1	9	34.31	69.6	254.1	2.9	0.0	326.6
1	10	38.12	66.3	242.0	2.9	0.0	311.2
1	11	41.94	60.8	221.8	2.6	0.0	285.2
1	12	45.75	53.0	193.6	2.1	0.0	248.7
1	13	49.56	43.1	157.3	1.5	0.0	201.9
1	14	53.37	30.9	112.9	1.0	0.0	144.9
1	15	57.19	16.6	60.5	0.5	0.0	77.6
1	16	61.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.81	31.8	17.8	0.0	0.0	0.0	49.6
1	2	7.62	59.4	33.2	0.0	0.0	0.0	92.6
1	3	11.44	82.8	46.2	0.0	0.0	0.0	129.0
1	4	15.25	101.9	56.9	0.0	0.0	0.0	158.7
1	5	19.06	116.7	65.2	0.0	0.0	0.0	181.9
1	6	22.87	127.3	71.1	0.0	0.0	0.0	198.4
1	7	26.69	133.7	74.6	0.0	0.0	0.0	208.3
1	8	30.50	135.8	75.8	0.0	0.0	0.0	211.6
1	9	34.31	133.7	74.6	0.0	0.0	0.0	208.3
1	10	38.12	127.3	71.1	0.0	0.0	0.0	198.4
1	11	41.94	116.7	65.2	0.0	0.0	0.0	181.9
1	12	45.75	101.9	56.9	0.0	0.0	0.0	158.7
1	13	49.56	82.8	46.2	0.0	0.0	0.0	129.0
1	14	53.37	59.4	33.2	0.0	0.0	0.0	92.6
1	15	57.19	31.8	17.8	0.0	0.0	0.0	49.6
1	16	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.81	0.0	0.0	187.3	HL-93	0.0	HL-93
1	2	7.62	0.0	0.0	345.8	HL-93	0.0	HL-93
1	3	11.44	0.0	0.0	475.4	HL-93	0.0	HL-93
1	4	15.25	0.0	0.0	574.0	HL-93	0.0	HL-93
1	5	19.06	0.0	0.0	649.3	HL-93	0.0	HL-93
1	6	22.87	0.0	0.0	701.6	HL-93	0.0	HL-93
1	7	26.69	0.0	0.0	723.6	HL-93	0.0	HL-93
1	8	30.50	0.0	0.0	728.8	HL-93	0.0	HL-93
1	9	34.31	0.0	0.0	723.6	HL-93	0.0	HL-93
1	10	38.12	0.0	0.0	701.6	HL-93	0.0	HL-93
1	11	41.94	0.0	0.0	649.3	HL-93	0.0	HL-93
1	12	45.75	0.0	0.0	574.0	HL-93	0.0	HL-93
1	13	49.56	0.0	0.0	475.4	HL-93	0.0	HL-93
1	14	53.37	0.0	0.0	345.8	HL-93	0.0	HL-93
1	15	57.19	0.0	0.0	187.3	HL-93	0.0	HL-93
1	16	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.81	70.	0.	70.
1	2	7.62	125.	0.	125.
1	3	11.44	168.	0.	168.
1	4	15.25	202.	0.	202.
1	5	19.06	226.	0.	226.
1	6	22.87	239.	0.	239.
1	7	26.69	231.	0.	231.
1	8	30.50	224.	0.	224.
1	9	34.31	231.	0.	231.
1	10	38.12	239.	0.	239.
1	11	41.94	226.	0.	226.
1	12	45.75	202.	0.	202.
1	13	49.56	168.	0.	168.
1	14	53.37	125.	0.	125.
1	15	57.19	70.	0.	70.
1	16	61.00	0.	0.	0.

TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.81	77.7	49.6	187.3	HL-93	0.0	HL-93	314.6	127.3
1	2	7.62	145.0	92.6	345.8	HL-93	0.0	HL-93	583.4	237.6
1	3	11.44	202.2	129.0	475.4	HL-93	0.0	HL-93	806.5	331.1
1	4	15.25	249.0	158.7	574.0	HL-93	0.0	HL-93	981.8	407.7
1	5	19.06	285.4	181.9	649.3	HL-93	0.0	HL-93	1116.5	467.2
1	6	22.87	311.1	198.4	701.6	HL-93	0.0	HL-93	1211.1	509.5
1	7	26.69	326.5	208.3	723.6	HL-93	0.0	HL-93	1258.4	534.9
1	8	30.50	331.7	211.6	728.8	HL-93	0.0	HL-93	1272.2	543.3
1	9	34.31	326.6	208.3	723.6	HL-93	0.0	HL-93	1258.5	534.9
1	10	38.12	311.2	198.4	701.6	HL-93	0.0	HL-93	1211.3	509.6
1	11	41.94	285.2	181.9	649.3	HL-93	0.0	HL-93	1116.4	467.1
1	12	45.75	248.7	158.7	574.0	HL-93	0.0	HL-93	981.5	407.4
1	13	49.56	201.9	129.0	475.4	HL-93	0.0	HL-93	806.3	330.9
1	14	53.37	144.9	92.6	345.8	HL-93	0.0	HL-93	583.3	237.5
1	15	57.19	77.6	49.6	187.3	HL-93	0.0	HL-93	314.5	127.2
1	16	61.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.81	314.6	370.8	494.9	420.0	190.9	52.5
1	2	7.62	583.4	687.2	917.0	778.7	356.5	93.9
1	3	11.44	806.5	949.1	1266.4	1076.3	496.7	126.2
1	4	15.25	981.8	1154.0	1539.7	1310.1	611.6	151.6
1	5	19.06	1116.5	1311.3	1749.5	1489.8	700.8	169.6
1	6	22.87	1211.1	1421.6	1896.6	1615.9	764.2	179.4
1	7	26.69	1258.4	1475.5	1968.2	1678.8	802.3	172.9
1	8	30.50	1272.2	1490.8	1988.6	1697.0	815.0	168.2
1	9	34.31	1258.5	1475.6	1968.3	1678.9	802.4	172.9
1	10	38.12	1211.3	1421.8	1896.7	1616.1	764.4	179.4
1	11	41.94	1116.4	1311.1	1749.3	1489.6	700.6	169.6
1	12	45.75	981.5	1153.7	1539.3	1309.7	611.1	151.6
1	13	49.56	806.3	948.9	1266.2	1076.1	496.4	126.2
1	14	53.37	583.3	687.0	916.9	778.6	356.2	93.9
1	15	57.19	314.5	370.7	494.8	419.9	190.8	52.5
1	16	61.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.6	16.9	0.2	0.0	21.7
1	1	3.81	4.1	14.8	0.2	0.0	19.0
1	2	7.62	3.5	12.7	0.2	0.0	16.3
1	3	11.44	2.9	10.6	0.2	0.0	13.6
1	4	15.25	2.3	8.5	0.2	0.0	10.9
1	5	19.06	1.7	6.3	0.0	0.0	8.1
1	6	22.87	1.2	4.2	0.0	0.0	5.4
1	7	26.69	0.6	2.1	0.0	0.0	2.7
1	8	30.50	0.0	0.0	0.0	0.0	0.0
1	9	34.31	-0.6	-2.1	0.0	0.0	-2.7
1	10	38.12	-1.2	-4.2	0.0	0.0	-5.4
1	11	41.94	-1.7	-6.3	-0.1	0.0	-8.2
1	12	45.75	-2.3	-8.5	-0.1	0.0	-10.9
1	13	49.56	-2.9	-10.6	-0.1	0.0	-13.6
1	14	53.37	-3.5	-12.7	-0.1	0.0	-16.3
1	15	57.19	-4.1	-14.8	-0.1	0.0	-19.0
1	16	61.00	-4.6	-16.9	-0.1	0.0	-21.7

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

COMPOSITE (k) TOTAL TOTAL

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	COMPOSITE (k)	NONCOMPOSITE (k)	DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	21.7	35.6
1	1	3.81	7.8	4.4	0.0	0.0	12.1	19.0	31.2
1	2	7.62	6.7	3.7	0.0	0.0	10.4	16.3	26.7
1	3	11.44	5.6	3.1	0.0	0.0	8.7	13.6	22.3
1	4	15.25	4.5	2.5	0.0	0.0	6.9	10.9	17.9
1	5	19.06	3.3	1.9	0.0	0.0	5.2	8.1	13.3
1	6	22.87	2.2	1.2	0.0	0.0	3.5	5.4	8.9
1	7	26.69	1.1	0.6	0.0	0.0	1.7	2.7	4.4
1	8	30.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	9	34.31	-1.1	-0.6	0.0	0.0	-1.7	-2.7	-4.4
1	10	38.12	-2.2	-1.2	0.0	0.0	-3.5	-5.4	-8.9
1	11	41.94	-3.3	-1.9	0.0	0.0	-5.2	-8.2	-13.4
1	12	45.75	-4.5	-2.5	0.0	0.0	-6.9	-10.9	-17.9
1	13	49.56	-5.6	-3.1	0.0	0.0	-8.7	-13.6	-22.3
1	14	53.37	-6.7	-3.7	0.0	0.0	-10.4	-16.3	-26.7
1	15	57.19	-7.8	-4.4	0.0	0.0	-12.1	-19.0	-31.1
1	16	61.00	-8.9	-5.0	0.0	0.0	-13.9	-21.7	-35.6

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	57.9	HL-93	0.0	HL-93
1	1	3.81	0.0	0.0	53.1	HL-93	-1.6	HL-93
1	2	7.62	0.0	0.0	48.3	HL-93	-3.7	HL-93
1	3	11.44	0.0	0.0	43.7	HL-93	-6.3	HL-93
1	4	15.25	0.0	0.0	39.2	HL-93	-9.0	HL-93
1	5	19.06	0.0	0.0	34.7	HL-93	-11.8	HL-93
1	6	22.87	0.0	0.0	30.4	HL-93	-14.7	HL-93
1	7	26.69	0.0	0.0	26.0	HL-93	-18.0	HL-93
1	8	30.50	0.0	0.0	21.9	HL-93	-21.9	HL-93
1	9	34.31	0.0	0.0	18.0	HL-93	-26.0	HL-93
1	10	38.12	0.0	0.0	14.7	HL-93	-30.4	HL-93
1	11	41.94	0.0	0.0	11.8	HL-93	-34.7	HL-93
1	12	45.75	0.0	0.0	9.0	HL-93	-39.2	HL-93
1	13	49.56	0.0	0.0	6.3	HL-93	-43.7	HL-93
1	14	53.37	0.0	0.0	3.7	HL-93	-48.3	HL-93
1	15	57.19	0.0	0.0	1.6	HL-93	-53.1	HL-93
1	16	61.00	0.0	0.0	0.0	HL-93	-57.9	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	TRUCK ONLY NEG	TRUCK ONLY RANGE
1	0	0.00	24.	0.	24.
1	1	3.81	22.	-1.	23.

1	2	7.62	19.	-2.	21.
1	3	11.44	17.	-3.	20.
1	4	15.25	15.	-4.	19.
1	5	19.06	13.	-5.	18.
1	6	22.87	11.	-6.	18.
1	7	26.69	9.	-7.	16.
1	8	30.50	8.	-8.	15.
1	9	34.31	7.	-9.	16.
1	10	38.12	6.	-11.	18.
1	11	41.94	5.	-13.	18.
1	12	45.75	4.	-15.	19.
1	13	49.56	3.	-17.	20.
1	14	53.37	2.	-19.	21.
1	15	57.19	1.	-22.	23.
1	16	61.00	0.	-24.	24.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	23.7	.30655E-01	0.36	0.3125*	.15191E-01	0.18	0.3125*
1	1		3.81	22.5	.30655E-01	0.34	0.3125*	.15191E-01	0.17	0.3125*
1	2		7.62	21.3	.30655E-01	0.33	0.3125*	.15191E-01	0.16	0.3125*
1	3		11.44	20.1	.28520E-01	0.29	0.3125*	.92950E-02	0.09	0.3125*
1	4		15.25	19.1	.28520E-01	0.27	0.3125*	.92950E-02	0.09	0.3125*
1	5		19.06	18.3	.28520E-01	0.26	0.3125*	.92950E-02	0.09	0.3125*
1	6		22.87	17.6	.28520E-01	0.25	0.3125*	.92950E-02	0.08	0.3125*
1	7		26.69	16.0	.28520E-01	0.23	0.3125*	.92950E-02	0.07	0.3125*
1	8		30.50	15.3	.28520E-01	0.22	0.3125*	.92950E-02	0.07	0.3125*
1	9		34.31	16.0	.28520E-01	0.23	0.3125*	.92950E-02	0.07	0.3125*
1	10		38.12	17.6	.28520E-01	0.25	0.3125*	.92950E-02	0.08	0.3125*
1	11		41.94	18.3	.28520E-01	0.26	0.3125*	.92950E-02	0.09	0.3125*
1	12		45.75	19.1	.28520E-01	0.27	0.3125*	.92950E-02	0.09	0.3125*
1	13		49.56	20.1	.28520E-01	0.29	0.3125*	.92950E-02	0.09	0.3125*
1	14		53.37	21.3	.30655E-01	0.33	0.3125*	.15191E-01	0.16	0.3125*
1	15		57.19	22.5	.30655E-01	0.34	0.3125*	.15191E-01	0.17	0.3125*
1	16		61.00	23.7	.30655E-01	0.36	0.3125*	.15191E-01	0.18	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	35.6	57.9	HL-93	0.0	HL-93	93.4	35.6
1	1	3.81	31.2	53.1	HL-93	-1.6	HL-93	84.2	29.6
1	2	7.62	26.7	48.3	HL-93	-3.7	HL-93	75.1	23.0
1	3	11.44	22.3	43.7	HL-93	-6.3	HL-93	66.0	16.0
1	4	15.25	17.9	39.2	HL-93	-9.0	HL-93	57.0	8.9
1	5	19.06	13.3	34.7	HL-93	-11.8	HL-93	48.0	1.5
1	6	22.87	8.9	30.4	HL-93	-14.7	HL-93	39.2	-5.8
1	7	26.69	4.4	26.0	HL-93	-18.0	HL-93	30.5	-13.5
1	8	30.50	0.0	21.9	HL-93	-21.9	HL-93	21.9	-21.9
1	9	34.31	-4.4	18.0	HL-93	-26.0	HL-93	13.6	-30.5
1	10	38.12	-8.9	14.7	HL-93	-30.4	HL-93	5.8	-39.2
1	11	41.94	-13.4	11.8	HL-93	-34.7	HL-93	-1.6	-48.2
1	12	45.75	-17.9	9.0	HL-93	-39.2	HL-93	-8.9	-57.0
1	13	49.56	-22.3	6.3	HL-93	-43.7	HL-93	-16.0	-66.0
1	14	53.37	-26.7	3.7	HL-93	-48.3	HL-93	-23.0	-75.1
1	15	57.19	-31.1	1.6	HL-93	-53.1	HL-93	-29.6	-84.2
1	16	61.00	-35.6	0.0	HL-93	-57.9	HL-93	-35.6	-93.4

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	93.4	110.8	148.0	124.8	53.4	17.8
1	1	3.81	84.2	100.1	133.7	112.5	46.7	16.9
1	2	7.62	75.1	89.6	119.7	100.3	40.1	16.0
1	3	11.44	66.0	79.1	105.8	88.3	33.5	15.1
1	4	15.25	57.0	68.8	92.0	76.3	26.8	14.3
1	5	19.06	48.0	58.4	78.2	64.3	19.9	13.7
1	6	22.87	39.2	48.3	64.8	52.6	13.3	13.2
1	7	26.69	30.5	38.3	51.4	41.0	6.7	12.0
1	8	30.50	21.9	28.5	38.3	29.6	0.0	11.4
1	9	34.31	-30.5	-38.3	-51.4	-41.0	-6.6	12.0
1	10	38.12	-39.2	-48.3	-64.8	-52.6	-13.3	13.2
1	11	41.94	-48.2	-58.6	-78.4	-64.5	-20.1	13.7
1	12	45.75	-57.0	-68.8	-92.0	-76.3	-26.8	14.3
1	13	49.56	-66.0	-79.1	-105.7	-88.3	-33.4	15.1
1	14	53.37	-75.1	-89.6	-119.7	-100.3	-40.1	16.0
1	15	57.19	-84.2	-100.1	-133.7	-112.5	-46.7	16.9
1	16	61.00	-93.4	-110.8	-147.9	-124.8	-53.4	17.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	35.60	0.00	HL-93	57.85	HL-93	ST1	29.81	147.96
						ST2	29.81	124.82
						ST4	40.03	53.39
						SE1	35.60	93.45
						SE2	35.60	110.80
2	35.58	0.00	HL-93	57.85	HL-93	ST1	29.79	147.94
						ST2	29.79	124.80
						ST4	40.01	53.37
						SE1	35.58	93.43
						SE2	35.58	110.78

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D FROM NO L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1 0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 1	3.81	-0.0380	-0.1387	0.0000	-0.0599	-0.1782	-0.0599	-0.2381
1 2	7.62	-0.0739	-0.2696	0.0000	-0.1164	-0.3465	-0.1164	-0.4629
1 3	11.44	-0.1058	-0.3861	0.0000	-0.1664	-0.4962	-0.1664	-0.6626
1 4	15.25	-0.1331	-0.4857	0.0000	-0.2091	-0.6242	-0.2091	-0.8334
1 5	19.06	-0.1553	-0.5668	0.0000	-0.2439	-0.7285	-0.2439	-0.9724
1 6	22.87	-0.1717	-0.6266	0.0000	-0.2696	-0.8054	-0.2696	-1.0750
1 7	26.69	-0.1817	-0.6633	0.0000	-0.2853	-0.8525	-0.2853	-1.1378
1 8	30.50	-0.1851	-0.6757	0.0000	-0.2906	-0.8684	-0.2906	-1.1590
1 9	34.31	-0.1817	-0.6633	0.0000	-0.2853	-0.8525	-0.2853	-1.1378
1 10	38.12	-0.1717	-0.6266	0.0000	-0.2696	-0.8054	-0.2696	-1.0749
1 11	41.94	-0.1553	-0.5668	0.0000	-0.2439	-0.7284	-0.2439	-0.9723
1 12	45.75	-0.1331	-0.4857	0.0000	-0.2091	-0.6241	-0.2091	-0.8333
1 13	49.56	-0.1058	-0.3861	0.0000	-0.1664	-0.4961	-0.1664	-0.6625
1 14	53.37	-0.0739	-0.2696	0.0000	-0.1164	-0.3464	-0.1164	-0.4628
1 15	57.19	-0.0380	-0.1387	0.0000	-0.0599	-0.1781	-0.0599	-0.2381
1 16	61.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	L FROM SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.8	-0.038	0 1/16	-0.140	0 3/16	-0.060	0 1/16	-0.238	0 1/ 4
1	2	7.6	-0.074	0 1/ 8	-0.273	0 5/16	-0.116	0 1/ 8	-0.463	0 1/ 2
1	3	11.4	-0.106	0 1/ 8	-0.390	0 7/16	-0.166	0 3/16	-0.663	0 11/16
1	4	15.2	-0.133	0 3/16	-0.491	0 1/ 2	-0.209	0 1/ 4	-0.833	0 7/ 8
1	5	19.1	-0.155	0 3/16	-0.573	0 5/ 8	-0.244	0 1/ 4	-0.972	1 0/16
1	6	22.9	-0.172	0 3/16	-0.634	0 11/16	-0.270	0 5/16	-1.075	1 1/ 8
1	7	26.7	-0.182	0 3/16	-0.671	0 11/16	-0.285	0 5/16	-1.138	1 3/16
1	8	30.5	-0.185	0 3/16	-0.683	0 11/16	-0.291	0 5/16	-1.159	1 3/16
1	9	34.3	-0.182	0 3/16	-0.671	0 11/16	-0.285	0 5/16	-1.138	1 3/16
1	10	38.1	-0.172	0 3/16	-0.634	0 11/16	-0.270	0 5/16	-1.075	1 1/ 8
1	11	41.9	-0.155	0 3/16	-0.573	0 5/ 8	-0.244	0 1/ 4	-0.972	1 0/16
1	12	45.7	-0.133	0 3/16	-0.491	0 1/ 2	-0.209	0 1/ 4	-0.833	0 7/ 8
1	13	49.6	-0.106	0 1/ 8	-0.390	0 7/16	-0.166	0 3/16	-0.663	0 11/16
1	14	53.4	-0.074	0 1/ 8	-0.273	0 5/16	-0.116	0 1/ 8	-0.463	0 1/ 2
1	15	57.2	-0.038	0 1/16	-0.140	0 3/16	-0.060	0 1/16	-0.238	0 1/ 4
1	16	61.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.50	3 0.425	-0.413	MAX HL-93	0.92	0.00330
			0.033	MIN		
			-0.121	MAX LANE		
			0.033	MIN LANE		
		4 0.433	-0.421	MAX HL-93	0.92	0.00330
			0.034	MIN		
			-0.123	MAX LANE		
			0.034	MIN LANE		
		3 0.425	-0.302	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	-0.49	0.49	-1.81	1.81	-2.30	2.30
1	2	7.62	-0.92	0.92	-3.38	3.38	-4.29	4.29
1	3	11.44	-1.15	0.80	-4.25	2.97	-5.40	3.77
1	4	15.25	-1.42	0.99	-5.23	3.65	-6.65	4.64
1	5	19.06	-1.62	1.13	-6.00	4.19	-7.62	5.32
1	6	22.87	-1.77	1.24	-6.54	4.56	-8.31	5.80
1	7	26.69	-1.86	1.30	-6.86	4.79	-8.72	6.09
1	8	30.50	-1.89	1.32	-6.97	4.87	-8.86	6.18
1	9	34.31	-1.86	1.30	-6.86	4.79	-8.72	6.09
1	10	38.12	-1.77	1.24	-6.54	4.57	-8.31	5.80
1	11	41.94	-1.62	1.13	-5.99	4.18	-7.62	5.32
1	12	45.75	-1.42	0.99	-5.23	3.65	-6.64	4.64
1	13	49.56	-1.15	0.80	-4.24	2.96	-5.39	3.77
1	14	53.37	-0.92	0.92	-3.37	3.37	-4.29	4.29
1	15	57.19	-0.49	0.49	-1.81	1.81	-2.30	2.30
1	16	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	-0.03	-0.46	1.13	-0.03	-2.76	3.43
1	2	7.62	-0.06	-0.87	2.11	-0.06	-5.16	6.40
1	3	11.44	-0.07	-1.18	1.87	-0.07	-6.58	5.64
1	4	15.25	-0.09	-1.46	2.31	-0.09	-8.11	6.95
1	5	19.06	-0.10	-1.67	2.64	-0.10	-9.29	7.96
1	6	22.87	-0.11	-1.82	2.88	-0.11	-10.13	8.68
1	7	26.69	-0.11	-1.91	3.03	-0.11	-10.63	9.11
1	8	30.50	-0.12	-1.94	3.07	-0.12	-10.80	9.26
1	9	34.31	-0.11	-1.91	3.03	-0.11	-10.64	9.12
1	10	38.12	-0.11	-1.82	2.88	-0.11	-10.13	8.68
1	11	41.94	-0.10	-1.67	2.64	-0.10	-9.29	7.96
1	12	45.75	-0.09	-1.46	2.31	-0.09	-8.10	6.94

1 13	49.56	-0.07	-1.18	1.87	-0.07	-6.58	5.64
1 14	53.37	-0.06	-0.87	2.11	-0.06	-5.15	6.40
1 15	57.19	-0.03	-0.46	1.13	-0.03	-2.76	3.43
1 16	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	-0.18	-0.53	3.83	0.00	0.00	0.00
1	2	7.62	-0.33	-0.98	7.07	0.00	0.00	0.00
1	3	11.44	-0.38	-1.57	6.22	0.00	0.00	0.00
1	4	15.25	-0.46	-1.89	7.52	0.00	0.00	0.00
1	5	19.06	-0.52	-2.14	8.50	0.00	0.00	0.00
1	6	22.87	-0.56	-2.32	9.19	0.00	0.00	0.00
1	7	26.69	-0.58	-2.39	9.47	0.00	0.00	0.00
1	8	30.50	-0.58	-2.41	9.54	0.00	0.00	0.00
1	9	34.31	-0.58	-2.39	9.47	0.00	0.00	0.00
1	10	38.12	-0.56	-2.32	9.19	0.00	0.00	0.00
1	11	41.94	-0.52	-2.14	8.50	0.00	0.00	0.00
1	12	45.75	-0.46	-1.89	7.52	0.00	0.00	0.00
1	13	49.56	-0.38	-1.57	6.22	0.00	0.00	0.00
1	14	53.37	-0.33	-0.98	7.07	0.00	0.00	0.00
1	15	57.19	-0.18	-0.53	3.83	0.00	0.00	0.00
1	16	61.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.81	0.2	0.0	0.2	C	HL	1.4	0.0	1.4	T	HL
1	2	7.62	0.4	0.0	0.4	C	HL	2.6	0.0	2.6	T	HL
1	3	11.44	0.6	0.0	0.6	C	HL	2.2	0.0	2.2	T	HL
1	4	15.25	0.7	0.0	0.7	C	HL	2.6	0.0	2.6	T	HL
1	5	19.06	0.7	0.0	0.7	C	HL	3.0	0.0	3.0	T	HL
1	6	22.87	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	7	26.69	0.8	0.0	0.8	C	HL	3.0	0.0	3.0	T	HL
1	8	30.50	0.7	0.0	0.7	C	HL	2.9	0.0	2.9	T	HL
1	9	34.31	0.8	0.0	0.8	C	HL	3.0	0.0	3.0	T	HL
1	10	38.12	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	11	41.94	0.7	0.0	0.7	C	HL	3.0	0.0	3.0	T	HL
1	12	45.75	0.7	0.0	0.7	C	HL	2.6	0.0	2.6	T	HL
1	13	49.56	0.6	0.0	0.6	C	HL	2.2	0.0	2.2	T	HL
1	14	53.37	0.4	0.0	0.4	C	HL	2.6	0.0	2.6	T	HL
1	15	57.19	0.2	0.0	0.2	C	HL	1.4	0.0	1.4	T	HL
1	16	61.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.21	-3.29	7.26	-0.03	-2.76	3.43
1	2	7.62	33.	33.	-0.39	-6.14	13.47	-0.06	-5.16	6.40
1	3	11.44	33.	33.	-0.45	-8.15	11.87	-0.07	-6.58	5.64
1	4	15.25	33.	33.	-0.55	-10.00	14.46	-0.09	-8.11	6.95
1	5	19.06	33.	33.	-0.62	-11.43	16.46	-0.10	-9.29	7.96
1	6	22.87	33.	33.	-0.67	-12.45	17.87	-0.11	-10.13	8.68
1	7	26.69	33.	33.	-0.69	-13.02	18.59	-0.11	-10.63	9.11
1	8	30.50	33.	33.	-0.70	-13.21	18.80	-0.12	-10.80	9.26
1	9	34.31	33.	33.	-0.69	-13.02	18.59	-0.11	-10.64	9.12
1	10	38.12	33.	33.	-0.67	-12.45	17.87	-0.11	-10.13	8.68
1	11	41.94	33.	33.	-0.62	-11.43	16.46	-0.10	-9.29	7.96
1	12	45.75	33.	33.	-0.55	-9.99	14.46	-0.09	-8.10	6.94
1	13	49.56	33.	33.	-0.45	-8.15	11.86	-0.07	-6.58	5.64
1	14	53.37	33.	33.	-0.39	-6.14	13.47	-0.06	-5.15	6.40
1	15	57.19	33.	33.	-0.21	-3.29	7.26	-0.03	-2.76	3.43
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.26	-3.45	8.41	-0.03	-2.76	3.43
1	2	7.62	33.	33.	-0.49	-6.43	15.60	-0.06	-5.16	6.40
1	3	11.44	33.	33.	-0.57	-8.62	13.73	-0.07	-6.58	5.64
1	4	15.25	33.	33.	-0.69	-10.57	16.72	-0.09	-8.11	6.95
1	5	19.06	33.	33.	-0.78	-12.08	19.01	-0.10	-9.29	7.96
1	6	22.87	33.	33.	-0.84	-13.14	20.63	-0.11	-10.13	8.68
1	7	26.69	33.	33.	-0.87	-13.74	21.43	-0.11	-10.63	9.11
1	8	30.50	33.	33.	-0.88	-13.93	21.67	-0.12	-10.80	9.26
1	9	34.31	33.	33.	-0.87	-13.74	21.43	-0.11	-10.64	9.12
1	10	38.12	33.	33.	-0.84	-13.14	20.63	-0.11	-10.13	8.68
1	11	41.94	33.	33.	-0.78	-12.07	19.01	-0.10	-9.29	7.96
1	12	45.75	33.	33.	-0.69	-10.56	16.71	-0.09	-8.10	6.94
1	13	49.56	33.	33.	-0.57	-8.62	13.73	-0.07	-6.58	5.64
1	14	53.37	33.	33.	-0.49	-6.43	15.59	-0.06	-5.15	6.40
1	15	57.19	33.	33.	-0.26	-3.45	8.41	-0.03	-2.76	3.43
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.36	-4.46	11.17	-0.04	-3.53	4.47
1	2	7.62	33.	33.	-0.66	-8.30	20.72	-0.08	-6.59	8.34
1	3	11.44	33.	33.	-0.77	-11.16	18.25	-0.10	-8.42	7.35
1	4	15.25	33.	33.	-0.93	-13.68	22.21	-0.12	-10.37	9.05
1	5	19.06	33.	33.	-1.05	-15.63	25.26	-0.14	-11.88	10.38
1	6	22.87	33.	33.	-1.14	-17.01	27.39	-0.15	-12.95	11.31
1	7	26.69	33.	33.	-1.18	-17.78	28.46	-0.16	-13.60	11.88
1	8	30.50	33.	33.	-1.19	-18.02	28.77	-0.16	-13.81	12.07
1	9	34.31	33.	33.	-1.18	-17.78	28.46	-0.16	-13.60	11.88
1	10	38.12	33.	33.	-1.14	-17.01	27.40	-0.15	-12.96	11.32
1	11	41.94	33.	33.	-1.05	-15.63	25.25	-0.14	-11.88	10.37
1	12	45.75	33.	33.	-0.93	-13.67	22.20	-0.12	-10.36	9.05
1	13	49.56	33.	33.	-0.77	-11.16	18.24	-0.10	-8.41	7.35
1	14	53.37	33.	33.	-0.66	-8.30	20.71	-0.08	-6.58	8.33
1	15	57.19	33.	33.	-0.36	-4.46	11.17	-0.04	-3.52	4.46
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.34	-3.34	9.61	-0.02	-2.41	2.90
1	2	7.62	33.	33.	-0.62	-6.22	17.80	-0.04	-4.50	5.42
1	3	11.44	33.	33.	-0.72	-8.48	15.67	-0.05	-5.73	4.78
1	4	15.25	33.	33.	-0.87	-10.38	19.04	-0.06	-7.06	5.88
1	5	19.06	33.	33.	-0.98	-11.84	21.62	-0.07	-8.09	6.74
1	6	22.87	33.	33.	-1.07	-12.88	23.43	-0.08	-8.82	7.35
1	7	26.69	33.	33.	-1.10	-13.44	24.30	-0.08	-9.26	7.72
1	8	30.50	33.	33.	-1.11	-13.62	24.54	-0.09	-9.41	7.84
1	9	34.31	33.	33.	-1.10	-13.44	24.30	-0.08	-9.26	7.72
1	10	38.12	33.	33.	-1.07	-12.88	23.43	-0.08	-8.83	7.35
1	11	41.94	33.	33.	-0.98	-11.84	21.62	-0.07	-8.09	6.74
1	12	45.75	33.	33.	-0.87	-10.37	19.03	-0.06	-7.06	5.88
1	13	49.56	33.	33.	-0.72	-8.47	15.67	-0.05	-5.73	4.77
1	14	53.37	33.	33.	-0.62	-6.22	17.80	-0.04	-4.50	5.42
1	15	57.19	33.	33.	-0.34	-3.34	9.61	-0.02	-2.41	2.90
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.29	-4.25	9.64	-0.04	-3.53	4.47
1	2	7.62	33.	33.	-0.53	-7.91	17.89	-0.08	-6.59	8.34
1	3	11.44	33.	33.	-0.61	-10.54	15.76	-0.10	-8.42	7.35
1	4	15.25	33.	33.	-0.74	-12.92	19.20	-0.12	-10.37	9.05
1	5	19.06	33.	33.	-0.84	-14.77	21.85	-0.14	-11.88	10.38
1	6	22.87	33.	33.	-0.91	-16.08	23.72	-0.15	-12.95	11.31
1	7	26.69	33.	33.	-0.94	-16.82	24.67	-0.16	-13.60	11.88
1	8	30.50	33.	33.	-0.95	-17.06	24.95	-0.16	-13.81	12.07
1	9	34.31	33.	33.	-0.94	-16.82	24.67	-0.16	-13.60	11.88
1	10	38.12	33.	33.	-0.91	-16.08	23.72	-0.15	-12.96	11.32
1	11	41.94	33.	33.	-0.84	-14.77	21.85	-0.14	-11.88	10.37
1	12	45.75	33.	33.	-0.74	-12.92	19.20	-0.12	-10.36	9.05
1	13	49.56	33.	33.	-0.61	-10.53	15.75	-0.10	-8.41	7.35
1	14	53.37	33.	33.	-0.53	-7.91	17.88	-0.08	-6.58	8.33
1	15	57.19	33.	33.	-0.29	-4.24	9.64	-0.04	-3.52	4.46
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.81	33.	33.	-0.26	-3.13	8.08	-0.02	-2.41	2.90
1	2	7.62	33.	33.	-0.49	-5.83	14.97	-0.04	-4.50	5.42
1	3	11.44	33.	33.	-0.57	-7.85	13.18	-0.05	-5.73	4.78
1	4	15.25	33.	33.	-0.69	-9.62	16.03	-0.06	-7.06	5.88
1	5	19.06	33.	33.	-0.78	-10.99	18.22	-0.07	-8.09	6.74
1	6	22.87	33.	33.	-0.84	-11.95	19.75	-0.08	-8.82	7.35
1	7	26.69	33.	33.	-0.87	-12.49	20.51	-0.08	-9.26	7.72
1	8	30.50	33.	33.	-0.87	-12.66	20.72	-0.09	-9.41	7.84
1	9	34.31	33.	33.	-0.87	-12.49	20.51	-0.08	-9.26	7.72
1	10	38.12	33.	33.	-0.84	-11.95	19.76	-0.08	-8.83	7.35
1	11	41.94	33.	33.	-0.78	-10.98	18.22	-0.07	-8.09	6.74
1	12	45.75	33.	33.	-0.69	-9.61	16.03	-0.06	-7.06	5.88
1	13	49.56	33.	33.	-0.57	-7.85	13.18	-0.05	-5.73	4.77
1	14	53.37	33.	33.	-0.49	-5.83	14.97	-0.04	-4.50	5.42
1	15	57.19	33.	33.	-0.26	-3.13	8.07	-0.02	-2.41	2.90
1	16	61.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP	JOINT D FROM	DL1, N=INF.	DL2, N=27.0	POS. LL+I	NEG. LL+I	TOTAL STRESS --AND--

NO	TYPE	L SUPT (ft)	MOM. (k-ft)	STRESS (ksi)	MOM. (k-ft)	STRESS (ksi)	MOM. (k-ft)	STRESS (ksi)	MOM. (k-ft)	STRESS (ksi)	STRESS RANGE
1	LT	11.00	196.	-5.8	125.	-1.2	460.	-1.3	0.	0.0	-8.26
											1.31
	LB			5.8		2.8		9.4		0.0	18.05
											9.42
	RT			-5.2		-1.1		-1.5		0.0	-7.89
											1.52
	RB			3.6		1.8		6.0		0.0	11.49
											6.03
1	LT	50.00	195.	-5.2	125.	-1.1	460.	-1.5	0.	0.0	-7.88
											1.52
	LB			3.6		1.8		6.0		0.0	11.49
											6.03
	RT			-5.8		-1.2		-1.3		0.0	-8.26
											1.31
	RB			5.8		2.8		9.4		0.0	18.04
											9.42

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	39.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	10.00	1.0000
3	11.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 61.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1])		<=>	LIMITATION 1/25	STATUS
				[3]= GIRDER D. [3]/(12*[1])				1/30	
1	11.00	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK
				[3]=	33.09	0.04520	>	0.03300	OK
2	39.00	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK
				[3]=	33.09	0.04520	>	0.03300	OK
3	11.00	6.75	33.09	[2]=	41.23	0.05633	>	0.04000	OK

[3]= 33.09 0.04520 > 0.03300 OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN D NO L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	1	3.81	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	2	7.62	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	3	11.44	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	4	15.25	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	5	19.06	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	6	22.87	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	7	26.69	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	8	30.50	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	9	34.31	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	10	38.12	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	11	41.94	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	12	45.75	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	13	49.56	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	14	53.37	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	15	57.19	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	16	61.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0

NOTE:

[1] 0 - No long. stiffeners
1 - long. stiffeners

[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)

[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0

[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)

[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw

[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	3.81	0	1	31.39	0.579	54.21	168.97	2
1	2	7.62	0	1	31.39	0.579	54.21	168.97	2
1	3	11.44	0	1	31.39	0.579	66.55	168.97	2
1	4	15.25	0	1	31.39	0.579	66.55	168.97	2
1	5	19.06	0	1	31.39	0.579	66.55	168.97	2
1	6	22.87	0	1	31.39	0.579	66.55	168.97	2
1	7	26.69	0	1	31.39	0.579	66.55	168.97	2
1	8	30.50	0	1	31.39	0.579	66.55	168.97	2
1	9	34.31	0	1	31.39	0.579	66.55	168.97	2
1	10	38.12	0	1	31.39	0.579	66.55	168.97	2
1	11	41.94	0	1	31.39	0.579	66.55	168.97	2
1	12	45.75	0	1	31.39	0.579	66.55	168.97	2
1	13	49.56	0	1	31.39	0.579	66.55	168.97	2
1	14	53.37	0	1	31.39	0.579	54.21	168.97	2
1	15	57.19	0	1	31.39	0.579	54.21	168.97	2
1	16	61.00	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0 0.00	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	1 3.81	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	2 7.62	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	3 11.44	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	4 15.25	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	5 19.06	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	6 22.87	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	7 26.69	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	8 30.50	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	9 34.31	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	10 38.12	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	11 41.94	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1	12 45.75	6.76	12.	11.5	5.2	0.850	0.637	

```
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
 1 13  49.56    6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
 1 14  53.37    6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
 1 15  57.19    6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
 1 16  61.00    6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
1
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0  17.8   0.000  10.952   6.995   3.038   1.000
                       0.000   0.000   0.000   0.000   1.000
    2     17.8  39.4  10.952  12.115  13.184  14.253   1.000
                       0.000   0.000   0.000   0.000   1.000
    3     39.4  61.0   0.000  12.115   7.810   3.505   1.000
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
1
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
         (ft)                         CAT  (ft)     (ft)     (ft)    CAT CAT
-------------------------------------------------------------------------------

  1  0   0.00  1  6.76  11.26  19.84   0    17.81     7.17    26.91   2   2
  1  1   3.81  1  6.76  11.26  19.84   0    17.81     7.17    26.91   2   2
  1  2   7.62  1  6.76  11.26  19.84   0    17.81     7.17    26.91   2   2
  1  3  11.44  1  6.76  11.26  19.84   0    17.81     6.98    26.21   2   2
  1  4  15.25  1  6.76  11.26  19.84   0    17.81     6.98    26.21   2   2
  1  5  19.06  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1  6  22.87  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1  7  26.69  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1  8  30.50  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1  9  34.31  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1 10  38.12  1  6.76  11.26  19.84   0    21.60     6.98    26.21   2   2
  1 11  41.94  1  6.76  11.26  19.84   0    21.59     6.98    26.21   2   2
  1 12  45.75  1  6.76  11.26  19.84   0    21.59     6.98    26.21   2   2
  1 13  49.56  1  6.76  11.26  19.84   0    21.59     6.98    26.21   2   2
```

1	14	53.37	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	15	57.19	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2
1	16	61.00	1	6.76	11.26	19.84	0	21.59	7.17	26.91	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.00	33.00	27.66	27.66
1	1	3.81	1	1.000	1.000	1.00	33.00	27.66	27.66
1	2	7.62	1	1.000	1.000	1.00	33.00	27.66	27.66
1	3	11.44	1	1.000	1.000	1.00	33.00	27.42	27.42
1	4	15.25	1	1.000	1.000	1.00	33.00	27.42	27.42
1	5	19.06	1	1.000	1.000	1.00	33.00	25.47	25.47
1	6	22.87	1	1.000	1.000	1.00	33.00	25.47	25.47
1	7	26.69	1	1.000	1.000	1.00	33.00	25.47	25.47
1	8	30.50	1	1.000	1.000	1.00	33.00	25.47	25.47
1	9	34.31	1	1.000	1.000	1.00	33.00	25.47	25.47
1	10	38.12	1	1.000	1.000	1.00	33.00	25.47	25.47
1	11	41.94	1	1.000	1.000	1.00	33.00	25.48	25.48
1	12	45.75	1	1.000	1.000	1.00	33.00	25.48	25.48
1	13	49.56	1	1.000	1.000	1.00	33.00	25.48	25.48
1	14	53.37	1	1.000	1.000	1.00	33.00	25.77	25.77
1	15	57.19	1	1.000	1.000	1.00	33.00	25.77	25.77
1	16	61.00	1	1.000	1.000	1.00	33.00	25.77	25.77

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

1

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	8.19	17.32
1	1		3.81	1	8.19	17.32
1	2		7.62	1	8.19	17.32
1	3		11.44	1	8.63	17.74
1	4		15.25	1	8.63	17.74
1	5		19.06	1	8.63	17.74
1	6		22.87	1	8.63	17.74
1	7		26.69	1	8.63	17.74
1	8		30.50	1	8.63	17.74
1	9		34.31	1	8.63	17.74
1	10		38.12	1	8.63	17.74
1	11		41.94	1	8.63	17.74
1	12		45.75	1	8.63	17.74
1	13		49.56	1	8.63	17.74
1	14		53.37	1	8.19	17.32
1	15		57.19	1	8.19	17.32
1	16		61.00	1	8.19	17.32

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0		0.00	1	0	2	0		2	0
1	1		3.81	1	0	2	0		2	0
1	2		7.62	1	0	2	0		2	0
1	3		11.44	1	0	2	0		2	0
1	4		15.25	1	0	2	0		2	0
1	5		19.06	1	0	2	0		2	0
1	6		22.87	1	0	2	0		2	0
1	7		26.69	1	0	2	0		2	0
1	8		30.50	1	0	2	0		2	0
1	9		34.31	1	0	2	0		2	0
1	10		38.12	1	0	2	0		2	0
1	11		41.94	1	0	2	0		2	0
1	12		45.75	1	0	2	0		2	0
1	13		49.56	1	0	2	0		2	0
1	14		53.37	1	0	2	0		2	0
1	15		57.19	1	0	2	0		2	0
1	16		61.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute

section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.7	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	3.81	0.0	19.8	3.4	33.0	3.4	33.0	3.4	27.7	0
			0.0	-	3.4	-	3.4	33.0	-	-	0
1	2	7.62	0.0	19.8	6.4	33.0	6.4	33.0	6.4	27.7	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	3	11.44	0.0	19.8	8.1	33.0	8.1	33.0	8.1	27.4	0
			0.0	-	5.7	-	5.7	33.0	-	-	0
1	4	15.25	0.0	19.8	10.0	33.0	10.0	33.0	10.0	27.4	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	5	19.06	0.0	19.8	11.4	33.0	11.4	33.0	11.4	25.5	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	6	22.87	0.0	19.8	12.5	33.0	12.5	33.0	12.5	25.5	0
			0.0	-	8.7	-	8.7	33.0	-	-	0
1	7	26.69	0.0	19.8	13.1	33.0	13.1	33.0	13.1	25.5	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	8	30.50	0.0	19.8	13.3	33.0	13.3	33.0	13.3	25.5	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	9	34.31	0.0	19.8	13.1	33.0	13.1	33.0	13.1	25.5	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	10	38.12	0.0	19.8	12.5	33.0	12.5	33.0	12.5	25.5	0
			0.0	-	8.7	-	8.7	33.0	-	-	0
1	11	41.94	0.0	19.8	11.4	33.0	11.4	33.0	11.4	25.5	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	12	45.75	0.0	19.8	10.0	33.0	10.0	33.0	10.0	25.5	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	13	49.56	0.0	19.8	8.1	33.0	8.1	33.0	8.1	25.5	0
			0.0	-	5.6	-	5.6	33.0	-	-	0
1	14	53.37	0.0	19.8	6.4	33.0	6.4	33.0	6.4	25.8	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	15	57.19	0.0	19.8	3.4	33.0	3.4	33.0	3.4	25.8	0
			0.0	-	3.4	-	3.4	33.0	-	-	0
1	16	61.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	25.8	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.81	0.000	0.104	0.104	0.125	0.125	4
					0.104		0.104	3
1	2	7.62	0.000	0.195	0.195	0.233	0.233	4
					0.195		0.195	3
1	3	11.44	0.000	0.245	0.245	0.295	0.295	4
					0.171		0.171	3
1	4	15.25	0.000	0.302	0.302	0.364	0.364	4
					0.211		0.211	3
1	5	19.06	0.000	0.346	0.346	0.449	0.449	4
					0.242		0.242	3
1	6	22.87	0.000	0.378	0.378	0.489	0.489	4
					0.264		0.264	3
1	7	26.69	0.000	0.396	0.396	0.513	0.513	4
					0.277		0.277	3
1	8	30.50	0.000	0.403	0.403	0.522	0.522	4
					0.281		0.281	3
1	9	34.31	0.000	0.396	0.396	0.514	0.514	4
					0.277		0.277	3
1	10	38.12	0.000	0.378	0.378	0.489	0.489	4
					0.264		0.264	3
1	11	41.94	0.000	0.346	0.346	0.448	0.448	4
					0.242		0.242	3
1	12	45.75	0.000	0.302	0.302	0.391	0.391	4
					0.211		0.211	3
1	13	49.56	0.000	0.245	0.245	0.317	0.317	4
					0.171		0.171	3
1	14	53.37	0.000	0.195	0.195	0.250	0.250	4
					0.195		0.195	3
1	15	57.19	0.000	0.104	0.104	0.134	0.134	4
					0.104		0.104	3
1	16	61.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK

5969 Span 1 Exterior Fatigue.RES
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2046.3	-	-	0.0	19.8	-	-	0--0
1	1		3.81	0	494.9	2046.3	-	-	0.0	19.8	-	-	0--0
1	2		7.62	0	917.0	2046.3	-	-	0.0	19.8	-	-	0--0
1	3		11.44	0	1266.4	2890.0	-	-	0.0	19.8	-	-	0--0
1	4		15.25	0	1539.7	2890.0	-	-	0.0	19.8	-	-	0--0
1	5		19.06	0	1749.5	2890.0	-	-	0.0	19.8	-	-	0--0
1	6		22.87	0	1896.6	2890.0	-	-	0.0	19.8	-	-	0--0
1	7		26.69	0	1968.2	2890.0	-	-	0.0	19.8	-	-	0--0
1	8		30.50	0	1988.6	2890.0	-	-	0.0	19.8	-	-	0--0
1	9		34.31	0	1968.3	2890.0	-	-	0.0	19.8	-	-	0--0
1	10		38.12	0	1896.7	2890.0	-	-	0.0	19.8	-	-	0--0
1	11		41.94	0	1749.3	2890.0	-	-	0.0	19.8	-	-	0--0
1	12		45.75	0	1539.3	2890.0	-	-	0.0	19.8	-	-	0--0
1	13		49.56	0	1266.2	2890.0	-	-	0.0	19.8	-	-	0--0
1	14		53.37	0	916.9	2046.3	-	-	0.0	19.8	-	-	0--0
1	15		57.19	0	494.8	2046.3	-	-	0.0	19.8	-	-	0--0
1	16		61.00	0	0.0	2046.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		3.81	0	0.242				0.242	1
1	2		7.62	0	0.448				0.448	1
1	3		11.44	0	0.438				0.438	1
1	4		15.25	0	0.533				0.533	1

1	5	19.06	0	0.605	0.605	1
1	6	22.87	0	0.656	0.656	1
1	7	26.69	0	0.681	0.681	1
1	8	30.50	0	0.688	0.688	1
1	9	34.31	0	0.681	0.681	1
1	10	38.12	0	0.656	0.656	1
1	11	41.94	0	0.605	0.605	1
1	12	45.75	0	0.533	0.533	1
1	13	49.56	0	0.438	0.438	1
1	14	53.37	0	0.448	0.448	1
1	15	57.19	0	0.242	0.242	1
1	16	61.00	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN D NO L	FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.81	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	7.62	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	11.44	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	15.25	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	19.06	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	22.87	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	26.69	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	30.50	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	34.31	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	38.12	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	41.94	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	45.75	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	49.56	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	14	53.37	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	15	57.19	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	16	61.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
           = 5 for unstiffened beams and girders
           = 5 for stiffened girders when (d0/D) > 3, or
                                          (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                     ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                     ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	148.0	<	347.9	0	0
1	1	3.81	0	133.7	<	347.9	0	0
1	2	7.62	0	119.7	<	347.9	0	0
1	3	11.44	0	105.8	<	347.9	0	0
1	4	15.25	0	92.0	<	347.9	0	0
1	5	19.06	0	78.2	<	347.9	0	0
1	6	22.87	0	64.8	<	347.9	0	0
1	7	26.69	0	51.4	<	347.9	0	0
1	8	30.50	0	38.3	<	347.9	0	0
1	9	34.31	0	51.4	<	347.9	0	0
1	10	38.12	0	64.8	<	347.9	0	0
1	11	41.94	0	78.4	<	347.9	0	0
1	12	45.75	0	92.0	<	347.9	0	0
1	13	49.56	0	105.7	<	347.9	0	0
1	14	53.37	0	119.7	<	347.9	0	0
1	15	57.19	0	133.7	<	347.9	0	0
1	16	61.00	0	147.9	<	347.9	0	0

Please read NOTE on the following page

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	147.96	347.87	0	
1	1	3.81	33.	133.74	347.87	0	
1	2	7.62	33.	119.68	347.87	0	
1	3	11.44	33.	105.77	347.87	0	
1	4	15.25	33.	92.01	347.87	0	
1	5	19.06	33.	78.23	347.87	0	
1	6	22.87	33.	64.78	347.87	0	
1	7	26.69	33.	51.41	347.87	0	
1	8	30.50	33.	38.34	347.87	0	
1	9	34.31	33.	51.39	347.87	0	
1	10	38.12	33.	64.76	347.87	0	
1	11	41.94	33.	78.39	347.87	0	
1	12	45.75	33.	91.99	347.87	0	
1	13	49.56	33.	105.75	347.87	0	
1	14	53.37	33.	119.66	347.87	0	
1	15	57.19	33.	133.72	347.87	0	
1	16	61.00	33.	147.94	347.87	0	

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     TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
                     *************************

     SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                  ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
     NO NO L SUPT                                          [3]  TBC
             (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi) (ksi)
     ---------------------------------------------------------------

      1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
      1  1   3.81   3.5    -     31.4     8.4  31.4   -     -   00-
      1  2   7.62   6.4    -     31.4    15.6  31.4   -     -   00-
      1  3  11.44   8.6    -     31.4    13.7  31.4   -     -   00-
      1  4  15.25  10.6    -     31.4    16.7  31.4   -     -   00-
      1  5  19.06  12.1    -     31.4    19.0  31.4   -     -   00-
      1  6  22.87  13.1    -     31.4    20.6  31.4   -     -   00-
      1  7  26.69  13.7    -     31.4    21.4  31.4   -     -   00-
      1  8  30.50  13.9    -     31.4    21.7  31.4   -     -   00-
      1  9  34.31  13.7    -     31.4    21.4  31.4   -     -   00-
      1 10  38.12  13.1    -     31.4    20.6  31.4   -     -   00-
      1 11  41.94  12.1    -     31.4    19.0  31.4   -     -   00-
      1 12  45.75  10.6    -     31.4    16.7  31.4   -     -   00-
      1 13  49.56   8.6    -     31.4    13.7  31.4   -     -   00-
      1 14  53.37   6.4    -     31.4    15.6  31.4   -     -   00-
      1 15  57.19   3.5    -     31.4     8.4  31.4   -     -   00-
      1 16  61.00   0.0    -     31.4     0.0  31.4   -     -   00-


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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                  = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                  = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members
```

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.81	TR	-0.3	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	2	7.62	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1		11.00	SCG					
1	3	11.44	TR	-0.8	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1	4	15.25	TR	-1.0	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	5	19.06	TR	-1.1	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	6	22.87	TR	-1.2	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	7	26.69	TR	-1.1	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	8	30.50	TR	-1.1	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	9	34.31	TR	-1.1	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	10	38.12	TR	-1.2	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	11	41.94	TR	-1.1	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	12	45.75	TR	-1.0	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	13	49.56	TR	-0.8	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1		50.00	SCG					
1	14	53.37	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	15	57.19	TR	-0.3	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	16	61.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.81	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2	7.62	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1		11.00	SCG					
1	3	11.44	TR	-0.4	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	4	15.25	TR	-0.5	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	5	19.06	TR	-0.6	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	6	22.87	TR	-0.6	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	7	26.69	TR	-0.6	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^

```
 1  8  30.50    TR     -0.6 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1  9  34.31    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1 10  38.12    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1 11  41.94    TR     -0.6 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1 12  45.75    TR     -0.5 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 13  49.56    TR     -0.4 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1     50.00  SCG
 1 14  53.37    TR     -0.3 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
 1 15  57.19    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
 1 16  61.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   ******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-1     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

  1  0   0.00  35.5 .28332E-01    1.0      TR    3.09        3           9.22
  1  1   3.81  33.8 .28332E-01    1.0      TR    3.09        3           9.70
  1  2   7.62  32.0 .28332E-01    0.9      TR    3.09        3          10.25
  1  3  11.44  30.2 .25851E-01    0.8      TR    3.09        3          11.88
  1  4  15.25  28.7 .25851E-01    0.7      TR    3.09        3          12.52
  1  5  19.06  27.5 .25851E-01    0.7      TR    3.09        3          13.08
  1  6  22.87  26.5 .25851E-01    0.7      TR    3.09        3          13.57
  1  7  26.69  24.1 .25851E-01    0.6      TR    3.09        3          14.92
  1  8  30.50  22.9 .25851E-01    0.6      TR    3.09        3          15.69
  1  9  34.31  24.1 .25851E-01    0.6      TR    3.09        3          14.92
  1 10  38.12  26.5 .25851E-01    0.7      TR    3.09        3          13.57
  1 11  41.94  27.5 .25851E-01    0.7      TR    3.09        3          13.08
  1 12  45.75  28.7 .25851E-01    0.7      TR    3.09        3          12.52
  1 13  49.56  30.2 .25851E-01    0.8      TR    3.09        3          11.88
  1 14  53.37  32.0 .28332E-01    0.9      TR    3.09        3          10.25
  1 15  57.19  33.8 .28332E-01    1.0      TR    3.09        3           9.70
  1 16  61.00  35.5 .28332E-01    1.0      TR    3.09        3           9.22


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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2
```

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	17.8	.28332E-01	0.5	TR 1.36	3	8.13
1	1	3.81	16.9	.28332E-01	0.5	TR 1.36	3	8.55
1	2	7.62	16.0	.28332E-01	0.5	TR 1.36	3	9.04
1	3	11.44	15.1	.25851E-01	0.4	TR 1.36	3	10.48
1	4	15.25	14.3	.25851E-01	0.4	TR 1.36	3	11.04
1	5	19.06	13.7	.25851E-01	0.4	TR 1.36	3	11.53
1	6	22.87	13.2	.25851E-01	0.3	TR 1.36	3	11.97
1	7	26.69	12.0	.25851E-01	0.3	TR 1.36	3	13.16
1	8	30.50	11.4	.25851E-01	0.3	TR 1.36	3	13.84
1	9	34.31	12.0	.25851E-01	0.3	TR 1.36	3	13.16
1	10	38.12	13.2	.25851E-01	0.3	TR 1.36	3	11.97
1	11	41.94	13.7	.25851E-01	0.4	TR 1.36	3	11.53
1	12	45.75	14.3	.25851E-01	0.4	TR 1.36	3	11.04
1	13	49.56	15.1	.25851E-01	0.4	TR 1.36	3	10.48
1	14	53.37	16.0	.28332E-01	0.5	TR 1.36	3	9.04
1	15	57.19	16.9	.28332E-01	0.5	TR 1.36	3	8.55
1	16	61.00	17.8	.28332E-01	0.5	TR 1.36	3	8.13

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```
 NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                section, the range of shear shall be taken as the difference
                in the minimum and maximum shear envelopes (excluding dead
                loads); AASHTO LRFD Eqn.6.10.10.1.2-2

       [2] Q/I : Q= statical moment about the neutral axis of the composite
                   section of the transformed compressive concrete area or
                   the area of reinforcement embedded in the concrete for
                   negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                 I= moment of inertia of the transformed composite girder in
                   positive moment regions or the moment of inertia provided
                   by the steel beam including or excluding the area of
                   reinforcement embedded in the concrete in negative moment
                   regions, in inches to the fourth power.

       [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                 of the slab and girder at the point in the span under
                 consideration.
               = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
           Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
           Ffat = radial fatigue shear range per unit length
                = Ffat2 = Frc/w for skews exceeding 20 degrees
           Frc = 25.0 kips for both exterior and interior girders
           w = the effective length of the deck
             = 48 inches except at end supports where w is taken
               as 24 inches

       [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

           * Default ALPHA value based on 7/8" diameter and input road type


       [5] No. of shear connectors per transverse section. This value is from
           input data type 12032.

       [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

           ** The connector spacing shown here is calculated based upon
              AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
              where ADTT Single Lane= 1140 & No. of Cycles=   31207500
          *** If ADTT Single Lane greater than or equal to 960, refer
              to Fatigue I Table.
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```
 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                 ************************************

                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  30.50   1 1574.9  1204.9          123         >  63
      1 16  61.00   1 1574.9  1204.9          123         >  63


     ---------------------------------------------------------------------------
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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	30.500	9.000
1	30.500	61.000	9.000

```
                ------------------------------------
1
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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                 *********************************

                 SPAN     CURRENT SPAN    T
                  NO.     FROM      TO    C
                          (ft)     (ft)   R
                ---------------------------

                    1       0.0      61.0 C

                ---------------------------

                NOTE: T: TENSION AREA, C: COMPRESSION AREA,
                R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
                STRESSES
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     TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
                    ************************

 STATUS                 TABLE NO.
-------------------------------------------------------------------------------

  OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

  OK     1.2.22.4=DEPTH RATIOS

  OK     1.2.22.10=CONSTRUCBILITY CHECK

  OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

  OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

  OK     1.2.22.21=SERVICE LIMIT STATE CHECK


         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
                ***********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT       MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
 NO NO L SUPT  CAP.                     --------------- FACTOR  LOAD  FACTOR  T
        (ft)  (k-ft)       D1; k-ft  D2     POS.     NEG.  [3]   TYPE
            or STRESS          ksi        MAX. STRESS
 ------------------------------------------------------------------------------

  1  0   0.00   2046.3      0.0      0.0      0.0      0.0  9.99  HL-93  9.99  0
                                          Page 43
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1	1	3.81	2046.3	77.7	49.6	187.3	0.0	7.43	HL-93	5.73	0
1	2	7.62	2046.3	145.0	92.6	345.8	0.0	3.72	HL-93	2.87	0
1	3	11.44	2890.0	202.2	129.0	475.4	0.0	3.83	HL-93	2.95	0
1	4	15.25	2890.0	249.0	158.7	574.0	0.0	3.04	HL-93	2.34	0
1	5	19.06	2890.0	285.4	181.9	649.3	0.0	2.60	HL-93	2.00	0
1	6	22.87	2890.0	311.1	198.4	701.6	0.0	2.35	HL-93	1.81	0
1	7	26.69	2890.0	326.5	208.3	723.6	0.0	2.24	HL-93	1.73	0
1	8	30.50	2890.0	331.7	211.6	728.8	0.0	2.21	HL-93	1.71	0
1	9	34.31	2890.0	326.6	208.3	723.6	0.0	2.24	HL-93	1.73	0
1	10	38.12	2890.0	311.2	198.4	701.6	0.0	2.34	HL-93	1.81	0
1	11	41.94	2890.0	285.2	181.9	649.3	0.0	2.60	HL-93	2.00	0
1	12	45.75	2890.0	248.7	158.7	574.0	0.0	3.04	HL-93	2.34	0
1	13	49.56	2890.0	201.9	129.0	475.4	0.0	3.83	HL-93	2.95	0
1	14	53.37	2046.3	144.9	92.6	345.8	0.0	3.72	HL-93	2.87	0
1	15	57.19	2046.3	77.6	49.6	187.3	0.0	7.43	HL-93	5.73	0
1	16	61.00	2046.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	3.81	31.4	-2.76 3.43	-0.53 3.83	0.00 0.00	7.29	HL-93	5.60
1	2	7.62	31.4	-5.16 6.40	-0.98 7.07	0.00 0.00	3.53	HL-93	2.71
1	3	11.44	31.4	-6.58 5.64	-1.57 6.22	0.00 0.00	4.13	HL-93	3.18
1	4	15.25	31.4	-8.11 6.95	-1.89 7.52	0.00 0.00	3.25	HL-93	2.50
1	5	19.06	31.4	-9.29 7.96	-2.14 8.50	0.00 0.00	2.75	HL-93	2.12
1	6	22.87	31.4	-10.13 8.68	-2.32 9.19	0.00 0.00	2.47	HL-93	1.90
1	7	26.69	31.4	-10.63 9.11	-2.39 9.47	0.00 0.00	2.35	HL-93	1.81
1	8	30.50	31.4	-10.80 9.26	-2.41 9.54	0.00 0.00	2.31	HL-93	1.78
1	9	34.31	31.4	-10.64 9.12	-2.39 9.47	0.00 0.00	2.35	HL-93	1.81

1	10	38.12	31.4	-10.13	8.68	-2.32	9.19	0.00	0.00	2.47	HL-93	1.90
1	11	41.94	31.4	-9.29	7.96	-2.14	8.50	0.00	0.00	2.75	HL-93	2.12
1	12	45.75	31.4	-8.10	6.94	-1.89	7.52	0.00	0.00	3.25	HL-93	2.50
1	13	49.56	31.4	-6.58	5.64	-1.57	6.22	0.00	0.00	4.13	HL-93	3.18
1	14	53.37	31.4	-5.15	6.40	-0.98	7.07	0.00	0.00	3.53	HL-93	2.71
1	15	57.19	31.4	-2.76	3.43	-0.53	3.83	0.00	0.00	7.29	HL-93	5.61
1	16	61.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	30.50	STRENGTH L.S.	2.21	1.71	HL-93
1	61.00	30.50	SERVICEBILITY	2.31	1.78	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.212
Inventory= 1.707

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	61.00	38.12	STRENGTH L.S.	3.18	698.89
1	61.00	38.12	SERVICEBILITY	3.18	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 3.176

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN D NO	FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	21.7	13.9	57.9	0.0	3.86	HL-93	2.97
1	1	3.81	347.9	19.0	12.1	53.1	-1.6	4.29	HL-93	3.31
1	2	7.62	347.9	16.3	10.4	48.3	-3.7	4.79	HL-93	3.70
1	3	11.44	347.9	13.6	8.7	43.7	-6.3	5.40	HL-93	4.16
1	4	15.25	347.9	10.9	6.9	39.2	-9.0	6.13	HL-93	4.73
1	5	19.06	347.9	8.1	5.2	34.7	-11.8	7.05	HL-93	5.44
1	6	22.87	347.9	5.4	3.5	30.4	-14.7	8.20	HL-93	6.33
1	7	26.69	347.9	2.7	1.7	26.0	-18.0	9.73	HL-93	7.50
1	8	30.50	347.9	0.0	0.0	21.9	-21.9	9.99	HL-93	9.08
1	9	34.31	347.9	-2.7	-1.7	18.0	-26.0	9.73	HL-93	7.50
1	10	38.12	347.9	-5.4	-3.5	14.7	-30.4	8.20	HL-93	6.33
1	11	41.94	347.9	-8.2	-5.2	11.8	-34.7	7.04	HL-93	5.43
1	12	45.75	347.9	-10.9	-6.9	9.0	-39.2	6.13	HL-93	4.73
1	13	49.56	347.9	-13.6	-8.7	6.3	-43.7	5.40	HL-93	4.17
1	14	53.37	347.9	-16.3	-10.4	3.7	-48.3	4.79	HL-93	3.70
1	15	57.19	347.9	-19.0	-12.1	1.6	-53.1	4.29	HL-93	3.31
1	16	61.00	347.9	-21.7	-13.9	0.0	-57.9	3.86	HL-93	2.97

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.00	0.00	STRENGTH L.S.	3.86	2.97	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.856
Inventory= 2.975

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	61.00	0.00	STRENGTH L.S.	4.68	64.35

```
-------------------------------------------------------------
    NOTE: control Rating Factor for Current Bridge:
          STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
               Legal Load Rating= 4.680
1
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 TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
               ****************************

                 NUMBER OF LANE   LL + I.
    SPAN  D FROM -----AND------ DEFLECTION GOVERN. RATING FACTOR FOR LL DEFL.
     NO.  L SUPT  DIST. FACTOR  ----------  LOAD   --------------------------
           (ft)   FOR LL DEFL.     (inch)   TYPE      RF = L/(800 * LL DEFL.)
    ---------------------------------------------------------------------------

      1    30.50    3   0.425       -0.41    HL-93            2.21

                    4   0.433       -0.42                     2.17

    ---------------------------------------------------------------------------
    NOTE: [1] "-" indicates downward deflection

          [2] RF * (live load defl.) = span length/800, therefore

              RF = L/(800 * LL Defl.), if = 99, No rating
```

```
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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span 2 S15 Exterior Strength M	8/19/13

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Exterior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	20	1	AASHTO	2010	0	2	3

```
* output level    : 0 = basic output
                    1 = detailed output

* span interval   : maximum = 20
                    default = 10

* structural type : 1 = composite (default)
                    2 = noncomposite
                    3 = reinforced concrete
                    4 = prestressed concrete

* type of unit    : 0 = English (default)
                    1 = Metric
                    2 = Metric input English output
                    3 = English input Metric output

* design option   : 0 = WSD (default)
                    1 = LFD
                    2 = LRFD


* program flow    : 0 = DL analysis only
                    1 = DL + LL analysis
                    2 = code check
                    3 = rating
                    4 = design
                    5 = design + code check
                    6 = design + recycle + code check
                    7 = DL stage only
                    8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION =    0
   0 - DEFAULT (2008)
   1 - "PRIOR TO 2007" WIDTH IS USED
```


TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
50.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	STANDARD SECTN DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.0000	10.00	0.4375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	13.50				33.	33.	33.
2	2	2	23.00				33.	33.	33.
3	1	1	13.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

5969 Span 2-S15 Exterior Strength Mid M.RES

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.56	2 2 2 2 2
1	0.00	0	0.00	0.40	0.00	0.00	0 0 0 0 0	0.56	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	50.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

5969 Span 2-S15 Exterior Strength Mid M.RES

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	12.31
2	2	Diaphragm	0.000	0.00	0.00	0.14	31.15
3	1	Railing	0.087	0.00	50.00	0.00	0.00
4	1	Curb	0.076	0.00	50.00	0.00	0.00
5	0	Pavement	0.292	0.00	50.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN.	DIA. SEC (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA PER FOOT OF SLAB (in**2)	Fy (ksi)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	12.31	33.0	33.0	12.31
12.31	31.15	33.0	33.0	18.84
31.15	50.00	33.0	33.0	18.85

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.555       0.1249     0.6799


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      50.00

     1   DC2     0.087       0.000     50.000

     2   DC2     0.076       0.000     50.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1214.1	45.0	182.1	37.5	219.59

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.562	0.562	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.561				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.399				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.50	5900.0	32.9	16.45	358.7	358.7
1	2	5.00	5900.0	32.9	16.45	358.7	358.7
1	3	7.50	5900.0	32.9	16.45	358.7	358.7
1	4	10.00	5900.0	32.9	16.45	358.7	358.7
1	5	12.50	5900.0	32.9	16.45	358.7	358.7
1	6	15.00	6981.8	32.9	15.02	464.8	381.2
1	7	17.50	6981.8	32.9	15.02	464.8	381.2
1	8	20.00	6981.8	32.9	15.02	464.8	381.2
1	9	22.50	6981.8	32.9	15.02	464.8	381.2
1	10	25.00	6981.8	32.9	15.02	464.8	381.2
1	11	27.50	6981.8	32.9	15.02	464.8	381.2
1	12	30.00	6981.8	32.9	15.02	464.8	381.2

1 13	32.50	6981.8	32.9	15.02	464.8	381.2
1 14	35.00	6981.8	32.9	15.02	464.8	381.2
1 15	37.50	5900.0	32.9	16.45	358.7	358.7
1 16	40.00	5900.0	32.9	16.45	358.7	358.7
1 17	42.50	5900.0	32.9	16.45	358.7	358.7
1 18	45.00	5900.0	32.9	16.45	358.7	358.7
1 19	47.50	5900.0	32.9	16.45	358.7	358.7
1 20	50.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11472.1	0.022815	479.7	1276.7	659.5
1	1	2.50	11472.1	0.022815	479.7	1276.7	659.5
1	2	5.00	11472.1	0.022815	479.7	1276.7	659.5
1	3	7.50	11472.1	0.022815	479.7	1276.7	659.5
1	4	10.00	11472.1	0.022815	479.7	1276.7	659.5
1	5	12.50	11472.1	0.022815	479.7	1276.7	659.5
1	6	15.00	13830.3	0.021457	613.8	1279.9	719.7
1	7	17.50	13830.3	0.021457	613.8	1279.9	719.7
1	8	20.00	13830.3	0.021457	613.8	1279.9	719.7
1	9	22.50	13830.3	0.021457	613.8	1279.9	719.7
1	10	25.00	13830.3	0.021457	613.8	1279.9	719.7
1	11	27.50	13830.3	0.021457	613.8	1279.9	719.7
1	12	30.00	13830.3	0.021457	613.8	1279.9	719.7
1	13	32.50	13830.3	0.021457	613.8	1279.9	719.7
1	14	35.00	13830.3	0.021457	613.8	1279.9	719.7
1	15	37.50	11472.1	0.022815	479.7	1276.7	659.5
1	16	40.00	11472.1	0.022815	479.7	1276.7	659.5
1	17	42.50	11472.1	0.022815	479.7	1276.7	659.5
1	18	45.00	11472.1	0.022815	479.7	1276.7	659.5
1	19	47.50	11472.1	0.022815	479.7	1276.7	659.5
1	20	50.00	11472.1	0.022815	479.7	1276.7	659.5

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15815.0	0.028983	534.7	4762.1	1348.1
1	1	2.50	15815.0	0.028983	534.7	4762.1	1348.1
1	2	5.00	15815.0	0.028983	534.7	4762.1	1348.1
1	3	7.50	15815.0	0.028983	534.7	4762.1	1348.1

1	4	10.00	15815.0	0.028983	534.7	4762.1	1348.1
1	5	12.50	15815.0	0.028983	534.7	4762.1	1348.1
1	6	15.00	19524.5	0.027497	681.3	4170.7	1491.4
1	7	17.50	19524.5	0.027497	681.3	4170.7	1491.4
1	8	20.00	19524.5	0.027497	681.3	4170.7	1491.4
1	9	22.50	19524.5	0.027497	681.3	4170.7	1491.4
1	10	25.00	19524.5	0.027497	681.3	4170.7	1491.4
1	11	27.50	19524.5	0.027497	681.3	4170.7	1491.4
1	12	30.00	19524.5	0.027497	681.3	4170.7	1491.4
1	13	32.50	19524.5	0.027497	681.3	4170.7	1491.4
1	14	35.00	19524.5	0.027497	681.3	4170.7	1491.4
1	15	37.50	15815.0	0.028983	534.7	4762.1	1348.1
1	16	40.00	15815.0	0.028983	534.7	4762.1	1348.1
1	17	42.50	15815.0	0.028983	534.7	4762.1	1348.1
1	18	45.00	15815.0	0.028983	534.7	4762.1	1348.1
1	19	47.50	15815.0	0.028983	534.7	4762.1	1348.1
1	20	50.00	15815.0	0.028983	534.7	4762.1	1348.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	2.50	7.4	33.0	0.4	0.0	40.8
1	2	5.00	14.1	62.4	0.8	0.0	77.3
1	3	7.50	19.9	88.5	1.2	0.0	109.6
1	4	10.00	25.0	111.0	1.6	0.0	137.6
1	5	12.50	29.3	130.1	2.0	0.0	161.4
1	6	15.00	32.8	145.7	2.1	0.0	180.5

1	7	17.50	35.5	157.8	2.1	0.0	195.5
1	8	20.00	37.5	166.5	2.1	0.0	206.1
1	9	22.50	38.6	171.7	2.2	0.0	212.5
1	10	25.00	39.0	173.4	2.2	0.0	214.7
1	11	27.50	38.6	171.7	2.3	0.0	212.6
1	12	30.00	37.5	166.5	2.3	0.0	206.3
1	13	32.50	35.5	157.8	2.2	0.0	195.5
1	14	35.00	32.8	145.7	1.9	0.0	180.4
1	15	37.50	29.3	130.1	1.6	0.0	160.9
1	16	40.00	25.0	111.0	1.3	0.0	137.2
1	17	42.50	19.9	88.5	0.9	0.0	109.3
1	18	45.00	14.1	62.4	0.6	0.0	77.1
1	19	47.50	7.4	33.0	0.3	0.0	40.7
1	20	50.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.50	17.3	9.7	0.0	0.0	0.0	27.0
1	2	5.00	32.8	18.3	0.0	0.0	0.0	51.2
1	3	7.50	46.5	26.0	0.0	0.0	0.0	72.5
1	4	10.00	58.4	32.6	0.0	0.0	0.0	91.0
1	5	12.50	68.4	38.2	0.0	0.0	0.0	106.6
1	6	15.00	76.6	42.8	0.0	0.0	0.0	119.4
1	7	17.50	83.0	46.4	0.0	0.0	0.0	129.4
1	8	20.00	87.6	48.9	0.0	0.0	0.0	136.5
1	9	22.50	90.3	50.4	0.0	0.0	0.0	140.8
1	10	25.00	91.2	50.9	0.0	0.0	0.0	142.2
1	11	27.50	90.3	50.4	0.0	0.0	0.0	140.8
1	12	30.00	87.6	48.9	0.0	0.0	0.0	136.5
1	13	32.50	83.0	46.4	0.0	0.0	0.0	129.4
1	14	35.00	76.6	42.8	0.0	0.0	0.0	119.4
1	15	37.50	68.4	38.2	0.0	0.0	0.0	106.6
1	16	40.00	58.4	32.6	0.0	0.0	0.0	91.0
1	17	42.50	46.5	26.0	0.0	0.0	0.0	72.5
1	18	45.00	32.8	18.3	0.0	0.0	0.0	51.2
1	19	47.50	17.3	9.7	0.0	0.0	0.0	27.0
1	20	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.50	0.0	0.0	124.1	HL-93	0.0	HL-93
1	2	5.00	0.0	0.0	232.4	HL-93	0.0	HL-93

1	3	7.50	0.0	0.0	325.1	HL-93	0.0	HL-93
1	4	10.00	0.0	0.0	402.0	HL-93	0.0	HL-93
1	5	12.50	0.0	0.0	461.2	HL-93	0.0	HL-93
1	6	15.00	0.0	0.0	508.8	HL-93	0.0	HL-93
1	7	17.50	0.0	0.0	546.0	HL-93	0.0	HL-93
1	8	20.00	0.0	0.0	569.5	HL-93	0.0	HL-93
1	9	22.50	0.0	0.0	574.2	HL-93	0.0	HL-93
1	10	25.00	0.0	0.0	575.8	HL-93	0.0	HL-93
1	11	27.50	0.0	0.0	574.2	HL-93	0.0	HL-93
1	12	30.00	0.0	0.0	569.5	HL-93	0.0	HL-93
1	13	32.50	0.0	0.0	546.0	HL-93	0.0	HL-93
1	14	35.00	0.0	0.0	508.8	HL-93	0.0	HL-93
1	15	37.50	0.0	0.0	461.2	HL-93	0.0	HL-93
1	16	40.00	0.0	0.0	402.0	HL-93	0.0	HL-93
1	17	42.50	0.0	0.0	325.1	HL-93	0.0	HL-93
1	18	45.00	0.0	0.0	232.4	HL-93	0.0	HL-93
1	19	47.50	0.0	0.0	124.1	HL-93	0.0	HL-93
1	20	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.50	44.	0.	44.
1	2	5.00	81.	0.	81.
1	3	7.50	111.	0.	111.
1	4	10.00	135.	0.	135.
1	5	12.50	151.	0.	151.
1	6	15.00	164.	0.	164.
1	7	17.50	175.	0.	175.
1	8	20.00	183.	0.	183.
1	9	22.50	186.	0.	186.
1	10	25.00	187.	0.	187.
1	11	27.50	186.	0.	186.
1	12	30.00	183.	0.	183.
1	13	32.50	175.	0.	175.
1	14	35.00	164.	0.	164.
1	15	37.50	151.	0.	151.
1	16	40.00	135.	0.	135.
1	17	42.50	111.	0.	111.
1	18	45.00	81.	0.	81.
1	19	47.50	44.	0.	44.
1	20	50.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE N=27.0 (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

1	1	2.50	40.8	27.0	124.1	HL-93	0.0	HL-93	191.8	67.8
1	2	5.00	77.3	51.2	232.4	HL-93	0.0	HL-93	360.9	128.5
1	3	7.50	109.6	72.5	325.1	HL-93	0.0	HL-93	507.2	182.1
1	4	10.00	137.6	91.0	402.0	HL-93	0.0	HL-93	630.6	228.6
1	5	12.50	161.4	106.6	461.2	HL-93	0.0	HL-93	729.2	268.0
1	6	15.00	180.5	119.4	508.8	HL-93	0.0	HL-93	808.8	300.0
1	7	17.50	195.5	129.4	546.0	HL-93	0.0	HL-93	870.8	324.8
1	8	20.00	206.1	136.5	569.5	HL-93	0.0	HL-93	912.2	342.6
1	9	22.50	212.5	140.8	574.2	HL-93	0.0	HL-93	927.6	353.3
1	10	25.00	214.7	142.2	575.8	HL-93	0.0	HL-93	932.7	356.9
1	11	27.50	212.6	140.8	574.2	HL-93	0.0	HL-93	927.7	353.4
1	12	30.00	206.3	136.5	569.5	HL-93	0.0	HL-93	912.3	342.8
1	13	32.50	195.5	129.4	546.0	HL-93	0.0	HL-93	870.9	324.9
1	14	35.00	180.4	119.4	508.8	HL-93	0.0	HL-93	808.6	299.8
1	15	37.50	160.9	106.6	461.2	HL-93	0.0	HL-93	728.7	267.6
1	16	40.00	137.2	91.0	402.0	HL-93	0.0	HL-93	630.2	228.2
1	17	42.50	109.3	72.5	325.1	HL-93	0.0	HL-93	506.9	181.8
1	18	45.00	77.1	51.2	232.4	HL-93	0.0	HL-93	360.7	128.3
1	19	47.50	40.7	27.0	124.1	HL-93	0.0	HL-93	191.8	67.7
1	20	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.50	191.8	229.1	306.2	256.6	101.7	33.0
1	2	5.00	360.9	430.6	575.5	482.6	192.7	60.9
1	3	7.50	507.2	604.7	808.1	678.1	273.1	83.2
1	4	10.00	630.6	751.2	1003.9	843.1	342.9	101.0
1	5	12.50	729.2	867.5	1159.2	974.7	402.0	112.9
1	6	15.00	808.8	961.4	1284.5	1081.0	450.0	123.0
1	7	17.50	870.8	1034.6	1382.3	1163.9	487.3	131.6
1	8	20.00	912.2	1083.0	1446.9	1219.0	513.9	137.3
1	9	22.50	927.6	1099.8	1469.2	1239.5	530.0	139.4
1	10	25.00	932.7	1105.5	1476.6	1246.3	535.4	140.1
1	11	27.50	927.7	1099.9	1469.3	1239.6	530.1	139.4
1	12	30.00	912.3	1083.2	1447.1	1219.3	514.2	137.3
1	13	32.50	870.9	1034.7	1382.4	1164.0	487.4	131.6
1	14	35.00	808.6	961.2	1284.3	1080.8	449.7	123.0
1	15	37.50	728.7	867.1	1158.6	974.1	401.3	112.9
1	16	40.00	630.2	750.9	1003.4	842.6	342.4	101.0
1	17	42.50	506.9	604.4	807.8	677.7	272.7	83.2
1	18	45.00	360.7	430.4	575.3	482.3	192.5	60.9
1	19	47.50	191.8	229.0	306.1	256.4	101.5	33.0
1	20	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT	STEEL BEAM	CONC. SLAB	CONCENT. LOAD	OTHER UNIFORM LOAD	TOTAL DEAD	NONCOMPOSITE LOAD

5969 Span 2-S15 Exterior Strength Mid M.RES

		(ft)	(kip)	(kip)	(kip)	(kip)	(kip)
1	0	0.00	3.1	13.9	0.2	0.0	17.2
1	1	2.50	2.8	12.5	0.2	0.0	15.5
1	2	5.00	2.5	11.1	0.2	0.0	13.8
1	3	7.50	2.2	9.7	0.2	0.0	12.1
1	4	10.00	1.9	8.3	0.2	0.0	10.4
1	5	12.50	1.6	6.9	0.0	0.0	8.5
1	6	15.00	1.2	5.5	0.0	0.0	6.8
1	7	17.50	0.9	4.2	0.0	0.0	5.1
1	8	20.00	0.6	2.8	0.0	0.0	3.4
1	9	22.50	0.3	1.4	0.0	0.0	1.7
1	10	25.00	0.0	0.0	0.0	0.0	0.0
1	11	27.50	-0.3	-1.4	0.0	0.0	-1.7
1	12	30.00	-0.6	-2.8	0.0	0.0	-3.4
1	13	32.50	-0.9	-4.2	-0.1	0.0	-5.2
1	14	35.00	-1.2	-5.6	-0.1	0.0	-6.9
1	15	37.50	-1.6	-6.9	-0.1	0.0	-8.6
1	16	40.00	-1.9	-8.3	-0.1	0.0	-10.3
1	17	42.50	-2.2	-9.7	-0.1	0.0	-12.0
1	18	45.00	-2.5	-11.1	-0.1	0.0	-13.7
1	19	47.50	-2.8	-12.5	-0.1	0.0	-15.4
1	20	50.00	-3.1	-13.9	-0.1	0.0	-17.1

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.3	4.1	0.0	0.0	11.4	17.2	28.5
1	1	2.50	6.6	3.7	0.0	0.0	10.2	15.5	25.7
1	2	5.00	5.8	3.3	0.0	0.0	9.1	13.8	22.9
1	3	7.50	5.1	2.9	0.0	0.0	8.0	12.1	20.0
1	4	10.00	4.4	2.4	0.0	0.0	6.8	10.4	17.2
1	5	12.50	3.6	2.0	0.0	0.0	5.7	8.5	14.2
1	6	15.00	2.9	1.6	0.0	0.0	4.5	6.8	11.4
1	7	17.50	2.2	1.2	0.0	0.0	3.4	5.1	8.5
1	8	20.00	1.5	0.8	0.0	0.0	2.3	3.4	5.7
1	9	22.50	0.7	0.4	0.0	0.0	1.1	1.7	2.9
1	10	25.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	11	27.50	-0.7	-0.4	0.0	0.0	-1.1	-1.7	-2.8
1	12	30.00	-1.5	-0.8	0.0	0.0	-2.3	-3.4	-5.7
1	13	32.50	-2.2	-1.2	0.0	0.0	-3.4	-5.2	-8.6
1	14	35.00	-2.9	-1.6	0.0	0.0	-4.6	-6.9	-11.5
1	15	37.50	-3.7	-2.0	0.0	0.0	-5.7	-8.6	-14.3
1	16	40.00	-4.4	-2.4	0.0	0.0	-6.8	-10.3	-17.1
1	17	42.50	-5.1	-2.9	0.0	0.0	-8.0	-12.0	-20.0
1	18	45.00	-5.8	-3.3	0.0	0.0	-9.1	-13.7	-22.8
1	19	47.50	-6.6	-3.7	0.0	0.0	-10.2	-15.4	-25.7
1	20	50.00	-7.3	-4.1	0.0	0.0	-11.4	-17.1	-28.5

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)

5969 Span 2-S15 Exterior Strength Mid M.RES
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	52.7	HL-93	0.0	HL-93
1	1	2.50	0.0	0.0	49.1	HL-93	-1.2	HL-93
1	2	5.00	0.0	0.0	45.6	HL-93	-2.5	HL-93
1	3	7.50	0.0	0.0	42.1	HL-93	-4.3	HL-93
1	4	10.00	0.0	0.0	38.7	HL-93	-6.3	HL-93
1	5	12.50	0.0	0.0	35.3	HL-93	-8.4	HL-93
1	6	15.00	0.0	0.0	32.0	HL-93	-10.5	HL-93
1	7	17.50	0.0	0.0	28.6	HL-93	-12.7	HL-93
1	8	20.00	0.0	0.0	25.4	HL-93	-14.9	HL-93
1	9	22.50	0.0	0.0	22.4	HL-93	-17.1	HL-93
1	10	25.00	0.0	0.0	19.4	HL-93	-19.4	HL-93
1	11	27.50	0.0	0.0	17.1	HL-93	-22.4	HL-93
1	12	30.00	0.0	0.0	14.9	HL-93	-25.4	HL-93
1	13	32.50	0.0	0.0	12.7	HL-93	-28.6	HL-93
1	14	35.00	0.0	0.0	10.5	HL-93	-32.0	HL-93
1	15	37.50	0.0	0.0	8.4	HL-93	-35.3	HL-93
1	16	40.00	0.0	0.0	6.3	HL-93	-38.7	HL-93
1	17	42.50	0.0	0.0	4.3	HL-93	-42.1	HL-93
1	18	45.00	0.0	0.0	2.5	HL-93	-45.6	HL-93
1	19	47.50	0.0	0.0	1.2	HL-93	-49.1	HL-93
1	20	50.00	0.0	0.0	0.0	HL-93	-52.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	21.	0.	21.
1	1	2.50	19.	-1.	20.
1	2	5.00	18.	-1.	19.
1	3	7.50	16.	-2.	18.
1	4	10.00	15.	-3.	18.
1	5	12.50	13.	-4.	17.
1	6	15.00	12.	-4.	16.
1	7	17.50	11.	-5.	16.
1	8	20.00	10.	-6.	16.
1	9	22.50	9.	-7.	16.
1	10	25.00	8.	-8.	16.
1	11	27.50	7.	-9.	16.
1	12	30.00	6.	-10.	16.
1	13	32.50	5.	-11.	16.
1	14	35.00	4.	-12.	16.
1	15	37.50	4.	-13.	17.
1	16	40.00	3.	-15.	18.
1	17	42.50	2.	-16.	18.
1	18	45.00	1.	-18.	19.
1	19	47.50	1.	-19.	20.
1	20	50.00	0.	-21.	21.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

5969 Span 2-S15 Exterior Strength Mid M.RES

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	21.0	.30967E-01	0.32	0.2500*	.14525E-01	0.15	0.2500*
1	1		2.50	20.0	.30967E-01	0.31	0.2500*	.14525E-01	0.15	0.2500*
1	2		5.00	19.2	.30967E-01	0.30	0.2500*	.14525E-01	0.14	0.2500*
1	3		7.50	18.4	.30967E-01	0.28	0.2500*	.14525E-01	0.13	0.2500*
1	4		10.00	17.6	.30967E-01	0.27	0.2500*	.14525E-01	0.13	0.2500*
1	5		12.50	16.7	.30967E-01	0.26	0.2500*	.14525E-01	0.12	0.2500*
1	6		15.00	16.2	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	7		17.50	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	8		20.00	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	9		22.50	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	10		25.00	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	11		27.50	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	12		30.00	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	13		32.50	16.3	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	14		35.00	16.2	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*
1	15		37.50	16.7	.30967E-01	0.26	0.2500*	.14525E-01	0.12	0.2500*
1	16		40.00	17.6	.30967E-01	0.27	0.2500*	.14525E-01	0.13	0.2500*
1	17		42.50	18.4	.30967E-01	0.28	0.2500*	.14525E-01	0.13	0.2500*
1	18		45.00	19.2	.30967E-01	0.30	0.2500*	.14525E-01	0.14	0.2500*
1	19		47.50	20.0	.30967E-01	0.31	0.2500*	.14525E-01	0.15	0.2500*
1	20		50.00	21.0	.30967E-01	0.32	0.2500*	.14525E-01	0.15	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0		0.00	28.5	52.7	HL-93	0.0	HL-93	81.2	28.5
1	1		2.50	25.7	49.1	HL-93	-1.2	HL-93	74.8	24.5
1	2		5.00	22.9	45.6	HL-93	-2.5	HL-93	68.4	20.4
1	3		7.50	20.0	42.1	HL-93	-4.3	HL-93	62.1	15.7
1	4		10.00	17.2	38.7	HL-93	-6.3	HL-93	55.9	10.9
1	5		12.50	14.2	35.3	HL-93	-8.4	HL-93	49.5	5.8

1	6	15.00	11.4	32.0	HL-93	-10.5	HL-93	43.3	0.9
1	7	17.50	8.5	28.6	HL-93	-12.7	HL-93	37.2	-4.1
1	8	20.00	5.7	25.4	HL-93	-14.9	HL-93	31.1	-9.2
1	9	22.50	2.9	22.4	HL-93	-17.1	HL-93	25.2	-14.2
1	10	25.00	0.0	19.4	HL-93	-19.4	HL-93	19.4	-19.4
1	11	27.50	-2.8	17.1	HL-93	-22.4	HL-93	14.3	-25.2
1	12	30.00	-5.7	14.9	HL-93	-25.4	HL-93	9.2	-31.1
1	13	32.50	-8.6	12.7	HL-93	-28.6	HL-93	4.0	-37.3
1	14	35.00	-11.5	10.5	HL-93	-32.0	HL-93	-1.0	-43.4
1	15	37.50	-14.3	8.4	HL-93	-35.3	HL-93	-5.9	-49.6
1	16	40.00	-17.1	6.3	HL-93	-38.7	HL-93	-10.8	-55.8
1	17	42.50	-20.0	4.3	HL-93	-42.1	HL-93	-15.7	-62.1
1	18	45.00	-22.8	2.5	HL-93	-45.6	HL-93	-20.4	-68.4
1	19	47.50	-25.7	1.2	HL-93	-49.1	HL-93	-24.4	-74.8
1	20	50.00	-28.5	0.0	HL-93	-52.7	HL-93	-28.5	-81.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	81.2	97.0	129.6	108.6	42.8	15.7
1	1	2.50	74.8	89.5	119.7	100.0	38.5	15.0
1	2	5.00	68.4	82.1	109.8	91.6	34.3	14.4
1	3	7.50	62.1	74.8	100.0	83.2	30.0	13.8
1	4	10.00	55.9	67.5	90.3	74.8	25.8	13.2
1	5	12.50	49.5	60.1	80.4	66.3	21.3	12.5
1	6	15.00	43.3	52.9	70.9	58.1	17.1	12.2
1	7	17.50	37.2	45.7	61.3	49.9	12.8	12.2
1	8	20.00	31.1	38.7	52.0	41.8	8.5	12.2
1	9	22.50	25.2	31.9	42.9	34.0	4.3	12.2
1	10	25.00	19.4	25.3	34.0	26.2	0.0	12.2
1	11	27.50	-25.2	-31.9	-42.9	-33.9	-4.2	12.2
1	12	30.00	-31.1	-38.7	-51.9	-41.8	-8.5	12.2
1	13	32.50	-37.3	-45.9	-61.4	-50.0	-13.0	12.2
1	14	35.00	-43.4	-53.0	-71.0	-58.2	-17.2	12.2
1	15	37.50	-49.6	-60.2	-80.6	-66.5	-21.5	12.5
1	16	40.00	-55.8	-67.4	-90.2	-74.8	-25.7	13.2
1	17	42.50	-62.1	-74.7	-100.0	-83.1	-30.0	13.8
1	18	45.00	-68.4	-82.1	-109.8	-91.5	-34.2	14.4
1	19	47.50	-74.8	-89.5	-119.6	-100.0	-38.5	15.0
1	20	50.00	-81.2	-97.0	-129.6	-108.5	-42.7	15.7

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	28.54	0.00	HL-93	52.66	HL-93	ST1	23.86	129.65
						ST2	23.86	108.58
						ST4	31.85	42.80

						SE1	28.54	81.19
						SE2	28.54	96.99
2	28.50	0.00	HL-93	52.66	HL-93	ST1	23.82	129.60
						ST2	23.82	108.54
						ST4	31.80	42.75
						SE1	28.50	81.16
						SE2	28.50	96.95

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.50	-0.0147	-0.0654	0.0000	-0.0273	-0.0810	-0.0273	-0.1082
1	2	5.00	-0.0290	-0.1287	0.0000	-0.0536	-0.1594	-0.0536	-0.2130
1	3	7.50	-0.0423	-0.1881	0.0000	-0.0784	-0.2330	-0.0784	-0.3113
1	4	10.00	-0.0545	-0.2419	0.0000	-0.1008	-0.2996	-0.1008	-0.4004
1	5	12.50	-0.0650	-0.2887	0.0000	-0.1202	-0.3576	-0.1202	-0.4778
1	6	15.00	-0.0737	-0.3276	0.0000	-0.1363	-0.4058	-0.1363	-0.5421
1	7	17.50	-0.0807	-0.3587	0.0000	-0.1491	-0.4442	-0.1491	-0.5933
1	8	20.00	-0.0858	-0.3813	0.0000	-0.1585	-0.4722	-0.1585	-0.6307
1	9	22.50	-0.0889	-0.3951	0.0000	-0.1642	-0.4892	-0.1642	-0.6534
1	10	25.00	-0.0900	-0.3997	0.0000	-0.1661	-0.4949	-0.1661	-0.6610
1	11	27.50	-0.0889	-0.3951	0.0000	-0.1642	-0.4892	-0.1642	-0.6534
1	12	30.00	-0.0858	-0.3813	0.0000	-0.1585	-0.4721	-0.1585	-0.6307
1	13	32.50	-0.0807	-0.3587	0.0000	-0.1491	-0.4441	-0.1491	-0.5933
1	14	35.00	-0.0737	-0.3276	0.0000	-0.1363	-0.4057	-0.1363	-0.5420
1	15	37.50	-0.0650	-0.2887	0.0000	-0.1202	-0.3575	-0.1202	-0.4777
1	16	40.00	-0.0545	-0.2419	0.0000	-0.1008	-0.2995	-0.1008	-0.4002
1	17	42.50	-0.0423	-0.1881	0.0000	-0.0784	-0.2328	-0.0784	-0.3112
1	18	45.00	-0.0290	-0.1287	0.0000	-0.0536	-0.1593	-0.0536	-0.2129
1	19	47.50	-0.0147	-0.0654	0.0000	-0.0273	-0.0809	-0.0273	-0.1082
1	20	50.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	NONCOMPOSITE (in) STEEL DEFL.	CAMBER SIZE	DEAD LOADS (in) SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.5	-0.015	0 1/16	-0.066	0 1/ 8	-0.027	0 1/16	-0.108	0 1/ 8

1	2	5.0	-0.029	0	1/16	-0.130	0	3/16	-0.054	0	1/16	-0.213	0	1/ 4
1	3	7.5	-0.042	0	1/16	-0.191	0	1/ 4	-0.078	0	1/ 8	-0.311	0	5/16
1	4	10.0	-0.054	0	1/16	-0.245	0	1/ 4	-0.101	0	1/ 8	-0.400	0	7/16
1	5	12.5	-0.065	0	1/ 8	-0.293	0	5/16	-0.120	0	1/ 8	-0.478	0	1/ 2
1	6	15.0	-0.074	0	1/ 8	-0.332	0	3/ 8	-0.136	0	3/16	-0.542	0	9/16
1	7	17.5	-0.081	0	1/ 8	-0.363	0	3/ 8	-0.149	0	3/16	-0.593	0	5/ 8
1	8	20.0	-0.086	0	1/ 8	-0.386	0	7/16	-0.159	0	3/16	-0.631	0	11/16
1	9	22.5	-0.089	0	1/ 8	-0.400	0	7/16	-0.164	0	3/16	-0.653	0	11/16
1	10	25.0	-0.090	0	1/ 8	-0.405	0	7/16	-0.166	0	3/16	-0.661	0	11/16
1	11	27.5	-0.089	0	1/ 8	-0.400	0	7/16	-0.164	0	3/16	-0.653	0	11/16
1	12	30.0	-0.086	0	1/ 8	-0.386	0	7/16	-0.159	0	3/16	-0.631	0	11/16
1	13	32.5	-0.081	0	1/ 8	-0.363	0	3/ 8	-0.149	0	3/16	-0.593	0	5/ 8
1	14	35.0	-0.074	0	1/ 8	-0.332	0	3/ 8	-0.136	0	3/16	-0.542	0	9/16
1	15	37.5	-0.065	0	1/ 8	-0.292	0	5/16	-0.120	0	1/ 8	-0.478	0	1/ 2
1	16	40.0	-0.054	0	1/16	-0.245	0	1/ 4	-0.101	0	1/ 8	-0.400	0	7/16
1	17	42.5	-0.042	0	1/16	-0.190	0	1/ 4	-0.078	0	1/ 8	-0.311	0	5/16
1	18	45.0	-0.029	0	1/16	-0.130	0	3/16	-0.054	0	1/16	-0.213	0	1/ 4
1	19	47.5	-0.015	0	1/16	-0.066	0	1/ 8	-0.027	0	1/16	-0.108	0	1/ 8
1	20	50.0	0.000	--		0.000	--		0.000	--		0.000	--	

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. DEFLECTION FACTOR (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.00	3	0.425	-0.279 MAX HL-93	0.75	0.00270
				0.029 MIN		
				-0.071 MAX LANE		
				0.029 MIN LANE		
		4	0.433	-0.284 MAX HL-93	0.75	0.00270
				0.029 MIN		
				-0.072 MAX LANE		
				0.029 MIN LANE		
		3	0.425	-0.173 MAX FA		
				0.000 MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	-0.25	0.25	-1.12	1.12	-1.36	1.36
1	2	5.00	-0.47	0.47	-2.12	2.12	-2.59	2.59
1	3	7.50	-0.67	0.67	-3.00	3.00	-3.67	3.67
1	4	10.00	-0.84	0.84	-3.77	3.77	-4.60	4.60
1	5	12.50	-0.98	0.98	-4.42	4.42	-5.40	5.40
1	6	15.00	-1.03	0.85	-4.65	3.81	-5.68	4.66
1	7	17.50	-1.12	0.92	-5.03	4.13	-6.15	5.05
1	8	20.00	-1.18	0.97	-5.31	4.35	-6.49	5.32
1	9	22.50	-1.22	1.00	-5.47	4.49	-6.69	5.49
1	10	25.00	-1.23	1.01	-5.53	4.54	-6.76	5.54
1	11	27.50	-1.22	1.00	-5.48	4.49	-6.69	5.49
1	12	30.00	-1.18	0.97	-5.32	4.36	-6.50	5.33
1	13	32.50	-1.12	0.92	-5.04	4.13	-6.16	5.05
1	14	35.00	-1.03	0.85	-4.65	3.81	-5.68	4.66
1	15	37.50	-0.98	0.98	-4.40	4.40	-5.38	5.38
1	16	40.00	-0.84	0.84	-3.76	3.76	-4.59	4.59
1	17	42.50	-0.67	0.67	-2.99	2.99	-3.66	3.66
1	18	45.00	-0.47	0.47	-2.11	2.11	-2.58	2.58
1	19	47.50	-0.25	0.25	-1.11	1.11	-1.36	1.36
1	20	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN D NO	FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	-0.02	-0.25	0.68	-0.02	-1.62	2.04
1	2	5.00	-0.03	-0.48	1.28	-0.03	-3.07	3.87
1	3	7.50	-0.05	-0.68	1.81	-0.05	-4.35	5.48
1	4	10.00	-0.06	-0.86	2.28	-0.06	-5.46	6.88
1	5	12.50	-0.07	-1.00	2.67	-0.07	-6.40	8.07
1	6	15.00	-0.07	-1.12	2.33	-0.07	-6.80	7.00
1	7	17.50	-0.08	-1.21	2.53	-0.08	-7.37	7.58
1	8	20.00	-0.08	-1.28	2.67	-0.08	-7.77	7.99
1	9	22.50	-0.09	-1.32	2.75	-0.09	-8.01	8.24
1	10	25.00	-0.09	-1.33	2.78	-0.09	-8.09	8.32
1	11	27.50	-0.09	-1.32	2.75	-0.09	-8.01	8.24
1	12	30.00	-0.08	-1.28	2.67	-0.08	-7.77	8.00
1	13	32.50	-0.08	-1.21	2.53	-0.08	-7.37	7.58
1	14	35.00	-0.07	-1.12	2.33	-0.07	-6.80	6.99
1	15	37.50	-0.07	-1.00	2.67	-0.07	-6.39	8.05

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1 16	40.00	-0.06	-0.86	2.28	-0.06	-5.45	6.87
1 17	42.50	-0.05	-0.68	1.81	-0.05	-4.34	5.47
1 18	45.00	-0.03	-0.48	1.28	-0.03	-3.06	3.86
1 19	47.50	-0.02	-0.25	0.68	-0.02	-1.62	2.04
1 20	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	-0.12	-0.31	2.78	0.00	0.00	0.00
1	2	5.00	-0.23	-0.59	5.22	0.00	0.00	0.00
1	3	7.50	-0.32	-0.82	7.30	0.00	0.00	0.00
1	4	10.00	-0.40	-1.01	9.02	0.00	0.00	0.00
1	5	12.50	-0.46	-1.16	10.35	0.00	0.00	0.00
1	6	15.00	-0.45	-1.46	8.96	0.00	0.00	0.00
1	7	17.50	-0.49	-1.57	9.62	0.00	0.00	0.00
1	8	20.00	-0.51	-1.64	10.03	0.00	0.00	0.00
1	9	22.50	-0.51	-1.65	10.11	0.00	0.00	0.00
1	10	25.00	-0.51	-1.66	10.14	0.00	0.00	0.00
1	11	27.50	-0.51	-1.65	10.11	0.00	0.00	0.00
1	12	30.00	-0.51	-1.64	10.03	0.00	0.00	0.00
1	13	32.50	-0.49	-1.57	9.62	0.00	0.00	0.00
1	14	35.00	-0.45	-1.46	8.96	0.00	0.00	0.00
1	15	37.50	-0.46	-1.16	10.35	0.00	0.00	0.00
1	16	40.00	-0.40	-1.01	9.02	0.00	0.00	0.00
1	17	42.50	-0.32	-0.82	7.30	0.00	0.00	0.00
1	18	45.00	-0.23	-0.59	5.22	0.00	0.00	0.00
1	19	47.50	-0.12	-0.31	2.78	0.00	0.00	0.00
1	20	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	2.50	0.1	0.0	0.1	C	HL	1.0	0.0	1.0	T	HL
1	2	5.00	0.2	0.0	0.2	C	HL	1.8	0.0	1.8	T	HL
1	3	7.50	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T	HL
1	4	10.00	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	5	12.50	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	6	15.00	0.5	0.0	0.5	C	HL	2.9	0.0	2.9	T	HL
1	7	17.50	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	8	20.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL
1	9	22.50	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	10	25.00	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	11	27.50	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	12	30.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL

1 13	32.50	0.5	0.0	0.5 C	HL	3.1	0.0	3.1 T	HL
1 14	35.00	0.5	0.0	0.5 C	HL	2.9	0.0	2.9 T	HL
1 15	37.50	0.4	0.0	0.4 C	HL	3.4	0.0	3.4 T	HL
1 16	40.00	0.3	0.0	0.3 C	HL	3.0	0.0	3.0 T	HL
1 17	42.50	0.3	0.0	0.3 C	HL	2.5	0.0	2.5 T	HL
1 18	45.00	0.2	0.0	0.2 C	HL	1.8	0.0	1.8 T	HL
1 19	47.50	0.1	0.0	0.1 C	HL	1.0	0.0	1.0 T	HL
1 20	50.00	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.14	-1.93	4.82	-0.02	-1.62	2.04
1	2	5.00	33.	33.	-0.26	-3.65	9.08	-0.03	-3.07	3.87
1	3	7.50	33.	33.	-0.37	-5.17	12.78	-0.05	-4.35	5.48
1	4	10.00	33.	33.	-0.46	-6.47	15.90	-0.06	-5.46	6.88
1	5	12.50	33.	33.	-0.53	-7.56	18.42	-0.07	-6.40	8.07
1	6	15.00	33.	33.	-0.53	-8.27	15.96	-0.07	-6.80	7.00
1	7	17.50	33.	33.	-0.57	-8.94	17.19	-0.08	-7.37	7.58
1	8	20.00	33.	33.	-0.59	-9.41	18.02	-0.08	-7.77	7.99
1	9	22.50	33.	33.	-0.60	-9.66	18.35	-0.09	-8.01	8.24
1	10	25.00	33.	33.	-0.60	-9.75	18.46	-0.09	-8.09	8.32
1	11	27.50	33.	33.	-0.60	-9.67	18.36	-0.09	-8.01	8.24
1	12	30.00	33.	33.	-0.59	-9.41	18.03	-0.08	-7.77	8.00
1	13	32.50	33.	33.	-0.57	-8.94	17.19	-0.08	-7.37	7.58
1	14	35.00	33.	33.	-0.53	-8.26	15.95	-0.07	-6.80	6.99
1	15	37.50	33.	33.	-0.53	-7.55	18.40	-0.07	-6.39	8.05
1	16	40.00	33.	33.	-0.46	-6.46	15.89	-0.06	-5.45	6.87
1	17	42.50	33.	33.	-0.37	-5.16	12.77	-0.05	-4.34	5.47
1	18	45.00	33.	33.	-0.26	-3.65	9.08	-0.03	-3.06	3.86
1	19	47.50	33.	33.	-0.14	-1.93	4.82	-0.02	-1.62	2.04
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.18	-2.02	5.66	-0.02	-1.62	2.04
1	2	5.00	33.	33.	-0.33	-3.83	10.65	-0.03	-3.07	3.87
1	3	7.50	33.	33.	-0.47	-5.41	14.96	-0.05	-4.35	5.48
1	4	10.00	33.	33.	-0.58	-6.78	18.61	-0.06	-5.46	6.88

1	5	12.50	33.	33.	-0.66	-7.91	21.52	-0.07	-6.40	8.07
1	6	15.00	33.	33.	-0.67	-8.71	18.65	-0.07	-6.80	7.00
1	7	17.50	33.	33.	-0.71	-9.41	20.08	-0.08	-7.37	7.58
1	8	20.00	33.	33.	-0.75	-9.90	21.03	-0.08	-7.77	7.99
1	9	22.50	33.	33.	-0.75	-10.16	21.39	-0.09	-8.01	8.24
1	10	25.00	33.	33.	-0.76	-10.25	21.51	-0.09	-8.09	8.32
1	11	27.50	33.	33.	-0.75	-10.16	21.39	-0.09	-8.01	8.24
1	12	30.00	33.	33.	-0.75	-9.91	21.04	-0.08	-7.77	8.00
1	13	32.50	33.	33.	-0.71	-9.41	20.08	-0.08	-7.37	7.58
1	14	35.00	33.	33.	-0.67	-8.70	18.64	-0.07	-6.80	6.99
1	15	37.50	33.	33.	-0.66	-7.90	21.51	-0.07	-6.39	8.05
1	16	40.00	33.	33.	-0.58	-6.76	18.60	-0.06	-5.45	6.87
1	17	42.50	33.	33.	-0.47	-5.40	14.96	-0.05	-4.34	5.47
1	18	45.00	33.	33.	-0.33	-3.82	10.64	-0.03	-3.06	3.86
1	19	47.50	33.	33.	-0.18	-2.02	5.66	-0.02	-1.62	2.04
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.24	-2.61	7.53	-0.03	-2.06	2.66
1	2	5.00	33.	33.	-0.45	-4.94	14.17	-0.05	-3.91	5.04
1	3	7.50	33.	33.	-0.63	-6.98	19.91	-0.07	-5.54	7.14
1	4	10.00	33.	33.	-0.78	-8.73	24.76	-0.09	-6.96	8.97
1	5	12.50	33.	33.	-0.90	-10.20	28.62	-0.10	-8.16	10.51
1	6	15.00	33.	33.	-0.90	-11.25	24.80	-0.10	-8.68	9.12
1	7	17.50	33.	33.	-0.97	-12.15	26.70	-0.11	-9.40	9.88
1	8	20.00	33.	33.	-1.01	-12.78	27.97	-0.12	-9.92	10.42
1	9	22.50	33.	33.	-1.02	-13.12	28.44	-0.12	-10.23	10.74
1	10	25.00	33.	33.	-1.02	-13.23	28.60	-0.12	-10.33	10.85
1	11	27.50	33.	33.	-1.02	-13.12	28.44	-0.12	-10.23	10.74
1	12	30.00	33.	33.	-1.01	-12.79	27.98	-0.12	-9.92	10.42
1	13	32.50	33.	33.	-0.97	-12.16	26.71	-0.11	-9.41	9.88
1	14	35.00	33.	33.	-0.90	-11.24	24.80	-0.10	-8.68	9.11
1	15	37.50	33.	33.	-0.90	-10.18	28.61	-0.10	-8.14	10.49
1	16	40.00	33.	33.	-0.78	-8.72	24.74	-0.09	-6.95	8.95
1	17	42.50	33.	33.	-0.63	-6.97	19.90	-0.07	-5.53	7.13
1	18	45.00	33.	33.	-0.45	-4.93	14.16	-0.05	-3.90	5.03
1	19	47.50	33.	33.	-0.24	-2.61	7.53	-0.03	-2.06	2.65
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.23	-1.96	6.60	-0.01	-1.42	1.73
1	2	5.00	33.	33.	-0.43	-3.71	12.40	-0.03	-2.68	3.27
1	3	7.50	33.	33.	-0.60	-5.24	17.41	-0.04	-3.80	4.64

1	4	10.00	33.	33.	-0.74	-6.55	21.62	-0.05	-4.78	5.83
1	5	12.50	33.	33.	-0.85	-7.63	24.94	-0.05	-5.60	6.83
1	6	15.00	33.	33.	-0.85	-8.51	21.60	-0.05	-5.94	5.92
1	7	17.50	33.	33.	-0.91	-9.18	23.24	-0.06	-6.44	6.41
1	8	20.00	33.	33.	-0.95	-9.65	24.32	-0.06	-6.79	6.76
1	9	22.50	33.	33.	-0.96	-9.89	24.67	-0.06	-7.00	6.97
1	10	25.00	33.	33.	-0.97	-9.97	24.79	-0.06	-7.07	7.05
1	11	27.50	33.	33.	-0.96	-9.89	24.68	-0.06	-7.00	6.98
1	12	30.00	33.	33.	-0.95	-9.66	24.32	-0.06	-6.79	6.77
1	13	32.50	33.	33.	-0.91	-9.19	23.24	-0.06	-6.44	6.41
1	14	35.00	33.	33.	-0.85	-8.50	21.60	-0.05	-5.94	5.92
1	15	37.50	33.	33.	-0.85	-7.62	24.93	-0.05	-5.59	6.82
1	16	40.00	33.	33.	-0.74	-6.54	21.61	-0.05	-4.76	5.82
1	17	42.50	33.	33.	-0.60	-5.23	17.40	-0.04	-3.80	4.63
1	18	45.00	33.	33.	-0.43	-3.70	12.40	-0.03	-2.68	3.27
1	19	47.50	33.	33.	-0.23	-1.96	6.60	-0.01	-1.41	1.72
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.19	-2.49	6.42	-0.03	-2.06	2.66
1	2	5.00	33.	33.	-0.36	-4.70	12.08	-0.05	-3.91	5.04
1	3	7.50	33.	33.	-0.50	-6.65	16.99	-0.07	-5.54	7.14
1	4	10.00	33.	33.	-0.62	-8.33	21.15	-0.09	-6.96	8.97
1	5	12.50	33.	33.	-0.72	-9.73	24.48	-0.10	-8.16	10.51
1	6	15.00	33.	33.	-0.72	-10.66	21.22	-0.10	-8.68	9.12
1	7	17.50	33.	33.	-0.77	-11.52	22.86	-0.11	-9.40	9.88
1	8	20.00	33.	33.	-0.81	-12.13	23.96	-0.12	-9.92	10.42
1	9	22.50	33.	33.	-0.82	-12.46	24.39	-0.12	-10.23	10.74
1	10	25.00	33.	33.	-0.82	-12.57	24.54	-0.12	-10.33	10.85
1	11	27.50	33.	33.	-0.82	-12.46	24.40	-0.12	-10.23	10.74
1	12	30.00	33.	33.	-0.81	-12.14	23.96	-0.12	-9.92	10.42
1	13	32.50	33.	33.	-0.77	-11.53	22.86	-0.11	-9.41	9.88
1	14	35.00	33.	33.	-0.72	-10.65	21.21	-0.10	-8.68	9.11
1	15	37.50	33.	33.	-0.72	-9.71	24.47	-0.10	-8.14	10.49
1	16	40.00	33.	33.	-0.62	-8.31	21.13	-0.09	-6.95	8.95
1	17	42.50	33.	33.	-0.50	-6.64	16.98	-0.07	-5.53	7.13
1	18	45.00	33.	33.	-0.36	-4.69	12.07	-0.05	-3.90	5.03
1	19	47.50	33.	33.	-0.19	-2.48	6.41	-0.03	-2.06	2.65
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.50	33.	33.	-0.18	-1.84	5.49	-0.01	-1.42	1.73
1	2	5.00	33.	33.	-0.34	-3.47	10.32	-0.03	-2.68	3.27

1	3	7.50	33.	33.	-0.47	-4.91	14.49	-0.04	-3.80	4.64
1	4	10.00	33.	33.	-0.58	-6.14	18.01	-0.05	-4.78	5.83
1	5	12.50	33.	33.	-0.67	-7.17	20.80	-0.05	-5.60	6.83
1	6	15.00	33.	33.	-0.67	-7.92	18.02	-0.05	-5.94	5.92
1	7	17.50	33.	33.	-0.72	-8.56	19.39	-0.06	-6.44	6.41
1	8	20.00	33.	33.	-0.75	-9.00	20.30	-0.06	-6.79	6.76
1	9	22.50	33.	33.	-0.76	-9.23	20.63	-0.06	-7.00	6.97
1	10	25.00	33.	33.	-0.76	-9.31	20.74	-0.06	-7.07	7.05
1	11	27.50	33.	33.	-0.76	-9.23	20.63	-0.06	-7.00	6.98
1	12	30.00	33.	33.	-0.75	-9.00	20.31	-0.06	-6.79	6.77
1	13	32.50	33.	33.	-0.72	-8.56	19.40	-0.06	-6.44	6.41
1	14	35.00	33.	33.	-0.67	-7.91	18.02	-0.05	-5.94	5.92
1	15	37.50	33.	33.	-0.67	-7.16	20.79	-0.05	-5.59	6.82
1	16	40.00	33.	33.	-0.58	-6.13	18.00	-0.05	-4.76	5.82
1	17	42.50	33.	33.	-0.47	-4.90	14.48	-0.04	-3.80	4.63
1	18	45.00	33.	33.	-0.34	-3.47	10.31	-0.03	-2.68	3.27
1	19	47.50	33.	33.	-0.18	-1.83	5.48	-0.01	-1.41	1.72
1	20	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	13.50	169.	-5.7	112.	-1.1	480.	-1.2	0.	0.0	-7.92
											1.21
	LB			5.7		2.8		10.8		0.0	19.23
											10.78
	RT			-5.3		-1.0		-1.4		0.0	-7.75
											1.38
	RB			4.4		2.2		8.5		0.0	15.01
											8.46
1	LT	36.50	169.	-5.3	112.	-1.0	480.	-1.4	0.	0.0	-7.74
											1.38
	LB			4.4		2.2		8.5		0.0	15.00
											8.46
	RT			-5.6		-1.1		-1.2		0.0	-7.90
											1.21
	RB			5.6		2.8		10.8		0.0	19.22
											10.78

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK
1	13.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	23.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.4375

```
 3  13.50    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

 ---------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   50.0
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  ------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------

  1  13.50  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

  2  23.00  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

  3  13.50  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

 ---------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-----------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  1   2.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  2   5.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  3   7.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  4  10.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  5  12.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  6  15.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  7  17.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  8  20.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  9  22.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
```

1	10	25.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	11	27.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	12	30.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	13	32.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	14	35.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	15	37.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	16	40.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	17	42.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	18	45.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	19	47.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	20	50.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.50	0	1	31.42	0.550	57.13	168.97	2
1	2	5.00	0	1	31.42	0.550	57.13	168.97	2
1	3	7.50	0	1	31.42	0.550	57.13	168.97	2
1	4	10.00	0	1	31.42	0.550	57.13	168.97	2
1	5	12.50	0	1	31.42	0.550	57.13	168.97	2
1	6	15.00	0	1	31.42	0.550	63.98	168.97	2
1	7	17.50	0	1	31.42	0.550	63.98	168.97	2
1	8	20.00	0	1	31.42	0.550	63.98	168.97	2
1	9	22.50	0	1	31.42	0.550	63.98	168.97	2
1	10	25.00	0	1	31.42	0.550	63.98	168.97	2
1	11	27.50	0	1	31.42	0.550	63.98	168.97	2
1	12	30.00	0	1	31.42	0.550	63.98	168.97	2
1	13	32.50	0	1	31.42	0.550	63.98	168.97	2
1	14	35.00	0	1	31.42	0.550	63.98	168.97	2
1	15	37.50	0	1	31.42	0.550	57.13	168.97	2
1	16	40.00	0	1	31.42	0.550	57.13	168.97	2
1	17	42.50	0	1	31.42	0.550	57.13	168.97	2
1	18	45.00	0	1	31.42	0.550	57.13	168.97	2
1	19	47.50	0	1	31.42	0.550	57.13	168.97	2
1	20	50.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ------------------------------------------------------------

  1  0   0.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  1   2.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  2   5.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  3   7.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  4  10.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  5  12.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  6  15.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  7  17.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  20.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  22.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  25.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  27.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  30.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  32.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  35.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  37.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 16  40.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 17  42.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 18  45.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 19  47.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 20  50.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
```

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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
```

LATERAL BRACING NO	LEFT ----(FT)---- FROM	SUPT TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	12.3	0.000	8.008	4.628	1.248	1.594
			0.000	0.000	0.000	0.000	1.000
2	12.3	31.1	8.008	9.509	9.944	10.378	1.000
			0.000	0.000	0.000	0.000	1.000
3	31.1	50.0	0.000	9.509	6.565	3.621	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	Lb (ft)	Lp (ft)	Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	1		2.50	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	2		5.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	3		7.50	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	4		10.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	5		12.50	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	6		15.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	7		17.50	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	8		20.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	9		22.50	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	10		25.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	11		27.50	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	12		30.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	13		32.50	1	7.77	11.26	19.84	0	18.85	6.98	26.22	2	2
1	14		35.00	1	7.77	11.26	19.84	0	18.85	6.98	26.22	2	2
1	15		37.50	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	16		40.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	17		42.50	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	18		45.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	19		47.50	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	20		50.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.59	33.00	33.00	33.00
1	1		2.50	1	1.000	1.000	1.59	33.00	33.00	33.00
1	2		5.00	1	1.000	1.000	1.59	33.00	33.00	33.00
1	3		7.50	1	1.000	1.000	1.59	33.00	33.00	33.00
1	4		10.00	1	1.000	1.000	1.59	33.00	33.00	33.00
1	5		12.50	1	1.000	1.000	1.00	33.00	27.04	27.04
1	6		15.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	7		17.50	1	1.000	1.000	1.00	33.00	26.90	26.90
1	8		20.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	9		22.50	1	1.000	1.000	1.00	33.00	26.90	26.90
1	10		25.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	11		27.50	1	1.000	1.000	1.00	33.00	26.90	26.90
1	12		30.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	13		32.50	1	1.000	1.000	1.00	33.00	26.89	26.89
1	14		35.00	1	1.000	1.000	1.00	33.00	26.89	26.89
1	15		37.50	1	1.000	1.000	1.00	33.00	27.04	27.04
1	16		40.00	1	1.000	1.000	1.00	33.00	27.04	27.04
1	17		42.50	1	1.000	1.000	1.00	33.00	27.04	27.04
1	18		45.00	1	1.000	1.000	1.00	33.00	27.04	27.04
1	19		47.50	1	1.000	1.000	1.00	33.00	27.04	27.04
1	20		50.00	1	1.000	1.000	1.00	33.00	27.04	27.04

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	6.34	17.35
1	1		2.50	1	6.34	17.35
1	2		5.00	1	6.34	17.35
1	3		7.50	1	6.34	17.35
1	4		10.00	1	6.34	17.35
1	5		12.50	1	6.34	17.35
1	6		15.00	1	8.50	17.53
1	7		17.50	1	8.50	17.53
1	8		20.00	1	8.50	17.53
1	9		22.50	1	8.50	17.53
1	10		25.00	1	8.50	17.53
1	11		27.50	1	8.50	17.53
1	12		30.00	1	8.50	17.53
1	13		32.50	1	8.50	17.53
1	14		35.00	1	8.50	17.53
1	15		37.50	1	6.34	17.35

1 16	40.00	1	6.34	17.35
1 17	42.50	1	6.34	17.35
1 18	45.00	1	6.34	17.35
1 19	47.50	1	6.34	17.35
1 20	50.00	1	6.34	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	2.50	1	0	2	0		2	0
1	2	5.00	1	0	2	0		2	0
1	3	7.50	1	0	2	0		2	0
1	4	10.00	1	0	2	0		2	0
1	5	12.50	1	0	2	0		2	0
1	6	15.00	1	0	2	0		2	0
1	7	17.50	1	0	2	0		2	0
1	8	20.00	1	0	2	0		2	0
1	9	22.50	1	0	2	0		2	0
1	10	25.00	1	0	2	0		2	0
1	11	27.50	1	0	2	0		2	0
1	12	30.00	1	0	2	0		2	0
1	13	32.50	1	0	2	0		2	0
1	14	35.00	1	0	2	0		2	0
1	15	37.50	1	0	2	0		2	0
1	16	40.00	1	0	2	0		2	0
1	17	42.50	1	0	2	0		2	0
1	18	45.00	1	0	2	0		2	0
1	19	47.50	1	0	2	0		2	0
1	20	50.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for

5969 Span 2-S15 Exterior Strength Mid M.RES
composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	2.50	0.0	19.8	2.0	33.0	2.0	33.0	2.0	33.0	0
			0.0	-	2.0	-	2.0	33.0	-	-	0
1	2	5.00	0.0	19.8	3.9	33.0	3.9	33.0	3.9	33.0	0
			0.0	-	3.9	-	3.9	33.0	-	-	0
1	3	7.50	0.0	19.8	5.5	33.0	5.5	33.0	5.5	33.0	0
			0.0	-	5.5	-	5.5	33.0	-	-	0
1	4	10.00	0.0	19.8	6.9	33.0	6.9	33.0	6.9	33.0	0
			0.0	-	6.9	-	6.9	33.0	-	-	0
1	5	12.50	0.0	19.8	8.1	33.0	8.1	33.0	8.1	27.0	0
			0.0	-	8.1	-	8.1	33.0	-	-	0
1	6	15.00	0.0	19.8	8.5	33.0	8.5	33.0	8.5	26.9	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	7	17.50	0.0	19.8	9.2	33.0	9.2	33.0	9.2	26.9	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	8	20.00	0.0	19.8	9.7	33.0	9.7	33.0	9.7	26.9	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	9	22.50	0.0	19.8	10.0	33.0	10.0	33.0	10.0	26.9	0
			0.0	-	8.2	-	8.2	33.0	-	-	0
1	10	25.00	0.0	19.8	10.1	33.0	10.1	33.0	10.1	26.9	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	11	27.50	0.0	19.8	10.0	33.0	10.0	33.0	10.0	26.9	0
			0.0	-	8.2	-	8.2	33.0	-	-	0
1	12	30.00	0.0	19.8	9.7	33.0	9.7	33.0	9.7	26.9	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	13	32.50	0.0	19.8	9.2	33.0	9.2	33.0	9.2	26.9	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	14	35.00	0.0	19.8	8.5	33.0	8.5	33.0	8.5	26.9	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	15	37.50	0.0	19.8	8.1	33.0	8.1	33.0	8.1	27.0	0
			0.0	-	8.1	-	8.1	33.0	-	-	0
1	16	40.00	0.0	19.8	6.9	33.0	6.9	33.0	6.9	27.0	0
			0.0	-	6.9	-	6.9	33.0	-	-	0
1	17	42.50	0.0	19.8	5.5	33.0	5.5	33.0	5.5	27.0	0
			0.0	-	5.5	-	5.5	33.0	-	-	0
1	18	45.00	0.0	19.8	3.9	33.0	3.9	33.0	3.9	27.0	0
			0.0	-	3.9	-	3.9	33.0	-	-	0
1	19	47.50	0.0	19.8	2.0	33.0	2.0	33.0	2.0	27.0	0
			0.0	-	2.0	-	2.0	33.0	-	-	0
1	20	50.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,

the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.50	0.000	0.062	0.062	0.062	0.062	2
					0.062		0.062	3
1	2	5.00	0.000	0.118	0.118	0.118	0.118	2
					0.118		0.118	3
1	3	7.50	0.000	0.167	0.167	0.167	0.167	2
					0.167		0.167	3
1	4	10.00	0.000	0.209	0.209	0.209	0.209	2
					0.209		0.209	3
1	5	12.50	0.000	0.245	0.245	0.299	0.299	4
					0.245		0.245	3
1	6	15.00	0.000	0.258	0.258	0.317	0.317	4
					0.212		0.212	3
1	7	17.50	0.000	0.280	0.280	0.343	0.343	4
					0.229		0.229	3
1	8	20.00	0.000	0.295	0.295	0.362	0.362	4
					0.242		0.242	3
1	9	22.50	0.000	0.304	0.304	0.373	0.373	4
					0.249		0.249	3
1	10	25.00	0.000	0.307	0.307	0.377	0.377	4
					0.252		0.252	3
1	11	27.50	0.000	0.304	0.304	0.373	0.373	4
					0.250		0.250	3
1	12	30.00	0.000	0.295	0.295	0.362	0.362	4
					0.242		0.242	3
1	13	32.50	0.000	0.280	0.280	0.343	0.343	4
					0.229		0.229	3
1	14	35.00	0.000	0.258	0.258	0.317	0.317	4
					0.212		0.212	3
1	15	37.50	0.000	0.245	0.245	0.299	0.299	4
					0.245		0.245	3
1	16	40.00	0.000	0.209	0.209	0.255	0.255	4
					0.209		0.209	3
1	17	42.50	0.000	0.166	0.166	0.203	0.203	4
					0.166		0.166	3
1	18	45.00	0.000	0.117	0.117	0.143	0.143	4
					0.117		0.117	3
1	19	47.50	0.000	0.062	0.062	0.076	0.076	4
					0.062		0.062	3
1	20	50.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0
1	1		2.50	0	306.2	1969.3	-	-	0.0	19.8	-	-	0--0
1	2		5.00	0	575.5	1969.3	-	-	0.0	19.8	-	-	0--0
1	3		7.50	0	808.1	1969.3	-	-	0.0	19.8	-	-	0--0
1	4		10.00	0	1003.9	1969.3	-	-	0.0	19.8	-	-	0--0
1	5		12.50	0	1159.2	1969.3	-	-	0.0	19.8	-	-	0--0
1	6		15.00	0	1284.5	2282.6	-	-	0.0	19.8	-	-	0--0
1	7		17.50	0	1382.3	2282.6	-	-	0.0	19.8	-	-	0--0
1	8		20.00	0	1446.9	2282.6	-	-	0.0	19.8	-	-	0--0
1	9		22.50	0	1469.2	2282.6	-	-	0.0	19.8	-	-	0--0
1	10		25.00	0	1476.6	2282.6	-	-	0.0	19.8	-	-	0--0
1	11		27.50	0	1469.3	2282.6	-	-	0.0	19.8	-	-	0--0
1	12		30.00	0	1447.1	2282.6	-	-	0.0	19.8	-	-	0--0
1	13		32.50	0	1382.4	2282.6	-	-	0.0	19.8	-	-	0--0
1	14		35.00	0	1284.3	2282.6	-	-	0.0	19.8	-	-	0--0
1	15		37.50	0	1158.6	1969.3	-	-	0.0	19.8	-	-	0--0
1	16		40.00	0	1003.4	1969.3	-	-	0.0	19.8	-	-	0--0
1	17		42.50	0	807.8	1969.3	-	-	0.0	19.8	-	-	0--0
1	18		45.00	0	575.3	1969.3	-	-	0.0	19.8	-	-	0--0
1	19		47.50	0	306.1	1969.3	-	-	0.0	19.8	-	-	0--0
1	20		50.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.

1	0	0.00	0	0.000	0.000	1
1	1	2.50	0	0.155	0.155	1
1	2	5.00	0	0.292	0.292	1
1	3	7.50	0	0.410	0.410	1
1	4	10.00	0	0.510	0.510	1
1	5	12.50	0	0.589	0.589	1
1	6	15.00	0	0.563	0.563	1
1	7	17.50	0	0.606	0.606	1
1	8	20.00	0	0.634	0.634	1
1	9	22.50	0	0.644	0.644	1
1	10	25.00	0	0.647	0.647	1
1	11	27.50	0	0.644	0.644	1
1	12	30.00	0	0.634	0.634	1
1	13	32.50	0	0.606	0.606	1
1	14	35.00	0	0.563	0.563	1
1	15	37.50	0	0.588	0.588	1
1	16	40.00	0	0.510	0.510	1
1	17	42.50	0	0.410	0.410	1
1	18	45.00	0	0.292	0.292	1
1	19	47.50	0	0.155	0.155	1
1	20	50.00	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D L	FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0		0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1		2.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2		5.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3		7.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4		10.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5		12.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6		15.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7		17.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8		20.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9		22.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10		25.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11		27.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12		30.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13		32.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14		35.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15		37.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16		40.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

1 17	42.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 18	45.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 19	47.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 20	50.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	129.6	<	330.8	0	0
1	1	2.50	0	119.7	<	330.8	0	0
1	2	5.00	0	109.8	<	330.8	0	0
1	3	7.50	0	100.0	<	330.8	0	0
1	4	10.00	0	90.3	<	330.8	0	0
1	5	12.50	0	80.4	<	330.8	0	0
1	6	15.00	0	70.9	<	330.8	0	0
1	7	17.50	0	61.3	<	330.8	0	0
1	8	20.00	0	52.0	<	330.8	0	0
1	9	22.50	0	42.9	<	330.8	0	0
1	10	25.00	0	34.0	<	330.8	0	0

1 11	27.50	0	42.9	<	330.8	0	0
1 12	30.00	0	51.9	<	330.8	0	0
1 13	32.50	0	61.4	<	330.8	0	0
1 14	35.00	0	71.0	<	330.8	0	0
1 15	37.50	0	80.6	<	330.8	0	0
1 16	40.00	0	90.2	<	330.8	0	0
1 17	42.50	0	100.0	<	330.8	0	0
1 18	45.00	0	109.8	<	330.8	0	0
1 19	47.50	0	119.6	<	330.8	0	0
1 20	50.00	0	129.6	<	330.8	0	0

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      NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

               For compact section  (composite sections in positive flexure)

                 w/o long. stiff.

                 D/tw <= 150 (Eq. 6.10.2.1.1-1)

                 w / long. stiff.

                 D/tw <= 300 (Eq. 6.10.2.1.2-1)

                 2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

               For compact or non-compact sections specified in Art. 6.10.8

                 (composite sections in negative flexure and
                  non-composite sections)

                 2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


            [2]: SHEAR CAPACITY ... web crippling control

               Vn = CVp .......... AASHTO Eq.6.10.9.2-1

               Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

     Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

               Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

     Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

               Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

             * For the detailed description of shear capacity,
               please refer to AASHTO Art.6.10.9.2 and .3

             * STRENGTH CATEGORY; 0 = compact section
                                  2 = braced non-compact section
                                  3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************

              YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
 SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
 NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
        (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
 ---------------------------------------------------------------------------
```

1	0	0.00	33.	129.65	330.76	0
1	1	2.50	33.	119.68	330.76	0
1	2	5.00	33.	109.80	330.76	0
1	3	7.50	33.	100.00	330.76	0
1	4	10.00	33.	90.27	330.76	0
1	5	12.50	33.	80.45	330.76	0
1	6	15.00	33.	70.88	330.76	0
1	7	17.50	33.	61.31	330.76	0
1	8	20.00	33.	51.98	330.76	0
1	9	22.50	33.	42.90	330.76	0
1	10	25.00	33.	34.02	330.76	0
1	11	27.50	33.	42.86	330.76	0
1	12	30.00	33.	51.93	330.76	0
1	13	32.50	33.	61.44	330.76	0
1	14	35.00	33.	71.01	330.76	0
1	15	37.50	33.	80.58	330.76	0
1	16	40.00	33.	90.23	330.76	0
1	17	42.50	33.	99.95	330.76	0
1	18	45.00	33.	109.76	330.76	0
1	19	47.50	33.	119.64	330.76	0
1	20	50.00	33.	129.60	330.76	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1] ff+fl/2 (ksi)	BOT. FLANGE [2] (ksi)	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0		0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1		2.50	2.0	-	31.4	5.7	31.4	-	-	00-
1	2		5.00	3.8	-	31.4	10.6	31.4	-	-	00-
1	3		7.50	5.4	-	31.4	15.0	31.4	-	-	00-
1	4		10.00	6.8	-	31.4	18.6	31.4	-	-	00-
1	5		12.50	7.9	-	31.4	21.5	31.4	-	-	00-
1	6		15.00	8.7	-	31.4	18.6	31.4	-	-	00-
1	7		17.50	9.4	-	31.4	20.1	31.4	-	-	00-
1	8		20.00	9.9	-	31.4	21.0	31.4	-	-	00-
1	9		22.50	10.2	-	31.4	21.4	31.4	-	-	00-
1	10		25.00	10.2	-	31.4	21.5	31.4	-	-	00-
1	11		27.50	10.2	-	31.4	21.4	31.4	-	-	00-
1	12		30.00	9.9	-	31.4	21.0	31.4	-	-	00-
1	13		32.50	9.4	-	31.4	20.1	31.4	-	-	00-
1	14		35.00	8.7	-	31.4	18.6	31.4	-	-	00-
1	15		37.50	7.9	-	31.4	21.5	31.4	-	-	00-
1	16		40.00	6.8	-	31.4	18.6	31.4	-	-	00-
1	17		42.50	5.4	-	31.4	15.0	31.4	-	-	00-
1	18		45.00	3.8	-	31.4	10.6	31.4	-	-	00-
1	19		47.50	2.0	-	31.4	5.7	31.4	-	-	00-
1	20		50.00	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient

= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)

where:

Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.50	TR	-0.2	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	2	5.00	TR	-0.3	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	3	7.50	TR	-0.4	A B B^C^C D E E^	TR	3.7	A B B^C^C D E
1	4	10.00	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	5	12.50	TR	-0.6	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1		13.50	SCG					
1	6	15.00	TR	-0.7	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1	7	17.50	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	8	20.00	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9	22.50	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	10	25.00	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	11	27.50	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	12	30.00	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	13	32.50	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	14	35.00	TR	-0.7	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1		36.50	SCG					
1	15	37.50	TR	-0.6	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1	16	40.00	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	17	42.50	TR	-0.4	A B B^C^C D E E^	TR	3.7	A B B^C^C D E
1	18	45.00	TR	-0.3	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	19	47.50	TR	-0.2	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	20	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.50	TR	-0.1	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	2	5.00	TR	-0.2	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	3	7.50	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	4	10.00	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	5	12.50	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1		13.50	SCG					
1	6	15.00	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	7	17.50	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	8	20.00	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	9	22.50	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	10	25.00	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	11	27.50	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	12	30.00	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	13	32.50	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	14	35.00	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1		36.50	SCG					
1	15	37.50	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	16	40.00	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	17	42.50	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	18	45.00	TR	-0.2	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	19	47.50	TR	-0.1	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	20	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]

5969 Span 2-S15 Exterior Strength Mid M.RES

1	0	0.00	31.5	.28983E-01	0.9	TR	3.09	3	10.17
1	1	2.50	30.0	.28983E-01	0.9	TR	3.09	3	10.66
1	2	5.00	28.7	.28983E-01	0.8	TR	3.09	3	11.15
1	3	7.50	27.5	.28983E-01	0.8	TR	3.09	3	11.63
1	4	10.00	26.4	.28983E-01	0.8	TR	3.09	3	12.12
1	5	12.50	25.1	.28983E-01	0.7	TR	3.09	3	12.76
1	6	15.00	24.3	.27497E-01	0.7	TR	3.09	3	13.88
1	7	17.50	24.4	.27497E-01	0.7	TR	3.09	3	13.84
1	8	20.00	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	9	22.50	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	10	25.00	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	11	27.50	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	12	30.00	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	13	32.50	24.4	.27497E-01	0.7	TR	3.09	3	13.84
1	14	35.00	24.3	.27497E-01	0.7	TR	3.09	3	13.88
1	15	37.50	25.1	.28983E-01	0.7	TR	3.09	3	12.76
1	16	40.00	26.4	.28983E-01	0.8	TR	3.09	3	12.12
1	17	42.50	27.5	.28983E-01	0.8	TR	3.09	3	11.63
1	18	45.00	28.7	.28983E-01	0.8	TR	3.09	3	11.15
1	19	47.50	30.0	.28983E-01	0.9	TR	3.09	3	10.66
1	20	50.00	31.5	.28983E-01	0.9	TR	3.09	3	10.17

Please read NOTE on the following page

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr <--Input	AASHTO LRFD Eqn. 6.10.10.2-2 [5], input	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	15.7	.28983E-01	0.5	TR	1.36	3	8.97
1	1	2.50	15.0	.28983E-01	0.4	TR	1.36	3	9.40
1	2	5.00	14.4	.28983E-01	0.4	TR	1.36	3	9.83
1	3	7.50	13.8	.28983E-01	0.4	TR	1.36	3	10.25
1	4	10.00	13.2	.28983E-01	0.4	TR	1.36	3	10.69
1	5	12.50	12.5	.28983E-01	0.4	TR	1.36	3	11.25
1	6	15.00	12.2	.27497E-01	0.3	TR	1.36	3	12.24
1	7	17.50	12.2	.27497E-01	0.3	TR	1.36	3	12.20
1	8	20.00	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	9	22.50	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	10	25.00	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	11	27.50	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	12	30.00	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	13	32.50	12.2	.27497E-01	0.3	TR	1.36	3	12.20
1	14	35.00	12.2	.27497E-01	0.3	TR	1.36	3	12.24
1	15	37.50	12.5	.28983E-01	0.4	TR	1.36	3	11.25
1	16	40.00	13.2	.28983E-01	0.4	TR	1.36	3	10.69
1	17	42.50	13.8	.28983E-01	0.4	TR	1.36	3	10.25
1	18	45.00	14.4	.28983E-01	0.4	TR	1.36	3	9.83
1	19	47.50	15.0	.28983E-01	0.4	TR	1.36	3	9.40
1	20	50.00	15.7	.28983E-01	0.5	TR	1.36	3	8.97

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D L (ft)	FROM SUPT 0=NEG.	MOMENT REGION 1=POS.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) FATIG. CRI.	<=>	SEE NOTE N1	STATUS N2 BLANK=OK CHECK=**
1	10	25.00	1	1276.3	1204.9			93	>	63	
1	20	50.00	1	1276.3	1204.9			93	>	63	

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange

bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	25.000	10.000
1	25.000	50.000	10.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	50.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.

OK 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

OK 1.2.22.4=DEPTH RATIOS

OK 1.2.22.10=CONSTRUCBILITY CHECK

OK 1.2.22.14=STRENGTH LIMIT STATE CHECK

OK 1.2.22.17=TRANSVERSE STIFFENER SPACING

OK 1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	POS. MAX. STRESS	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.50	1969.3	40.8	27.0	124.1	0.0	9.99	HL-93	8.66	0
1	2	5.00	1969.3	77.3	51.2	232.4	0.0	5.74	HL-93	4.43	0
1	3	7.50	1969.3	109.6	72.5	325.1	0.0	3.94	HL-93	3.04	0
1	4	10.00	1969.3	137.6	91.0	402.0	0.0	3.08	HL-93	2.37	0
1	5	12.50	1969.3	161.4	106.6	461.2	0.0	2.60	HL-93	2.00	0
1	6	15.00	2282.6	180.5	119.4	508.8	0.0	2.75	HL-93	2.12	0
1	7	17.50	2282.6	195.5	129.4	546.0	0.0	2.52	HL-93	1.94	0
1	8	20.00	2282.6	206.1	136.5	569.5	0.0	2.38	HL-93	1.84	0
1	9	22.50	2282.6	212.5	140.8	574.2	0.0	2.35	HL-93	1.81	0
1	10	25.00	2282.6	214.7	142.2	575.8	0.0	2.33	HL-93	1.80	0
1	11	27.50	2282.6	212.6	140.8	574.2	0.0	2.35	HL-93	1.81	0
1	12	30.00	2282.6	206.3	136.5	569.5	0.0	2.38	HL-93	1.84	0
1	13	32.50	2282.6	195.5	129.4	546.0	0.0	2.52	HL-93	1.94	0
1	14	35.00	2282.6	180.4	119.4	508.8	0.0	2.75	HL-93	2.12	0
1	15	37.50	1969.3	160.9	106.6	461.2	0.0	2.60	HL-93	2.00	0
1	16	40.00	1969.3	137.2	91.0	402.0	0.0	3.08	HL-93	2.37	0
1	17	42.50	1969.3	109.3	72.5	325.1	0.0	3.94	HL-93	3.04	0
1	18	45.00	1969.3	77.1	51.2	232.4	0.0	5.74	HL-93	4.43	0
1	19	47.50	1969.3	40.7	27.0	124.1	0.0	9.99	HL-93	8.66	0
1	20	50.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi	BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi	BOT	MAX. NEG. --- STEEL--- TOP;ksi	BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	2.50	31.4	-1.62	2.04	-0.31	2.78	0.00	0.00	9.99	HL-93	7.68
1	2	5.00	31.4	-3.07	3.87	-0.59	5.22	0.00	0.00	5.27	HL-93	4.05
1	3	7.50	31.4	-4.35	5.48	-0.82	7.30	0.00	0.00	3.55	HL-93	2.73
1	4	10.00	31.4	-5.46	6.88	-1.01	9.02	0.00	0.00	2.71	HL-93	2.09
1	5	12.50	31.4	-6.40	8.07	-1.16	10.35	0.00	0.00	2.25	HL-93	1.73
1	6	15.00	31.4	-6.80	7.00	-1.46	8.96	0.00	0.00	2.72	HL-93	2.09
1	7	17.50	31.4	-7.37	7.58	-1.57	9.62	0.00	0.00	2.47	HL-93	1.90
1	8	20.00	31.4	-7.77	7.99	-1.64	10.03	0.00	0.00	2.33	HL-93	1.79
1	9	22.50	31.4	-8.01	8.24	-1.65	10.11	0.00	0.00	2.29	HL-93	1.76
1	10	25.00	31.4	-8.09	8.32	-1.66	10.14	0.00	0.00	2.27	HL-93	1.75
1	11	27.50	31.4	-8.01	8.24	-1.65	10.11	0.00	0.00	2.28	HL-93	1.76
1	12	30.00	31.4	-7.77	8.00	-1.64	10.03	0.00	0.00	2.33	HL-93	1.79
1	13	32.50	31.4	-7.37	7.58	-1.57	9.62	0.00	0.00	2.47	HL-93	1.90
1	14	35.00	31.4	-6.80	6.99	-1.46	8.96	0.00	0.00	2.72	HL-93	2.09
1	15	37.50	31.4	-6.39	8.05	-1.16	10.35	0.00	0.00	2.25	HL-93	1.73
1	16	40.00	31.4	-5.45	6.87	-1.01	9.02	0.00	0.00	2.71	HL-93	2.09
1	17	42.50	31.4	-4.34	5.47	-0.82	7.30	0.00	0.00	3.55	HL-93	2.73
1	18	45.00	31.4	-3.06	3.86	-0.59	5.22	0.00	0.00	5.27	HL-93	4.05
1	19	47.50	31.4	-1.62	2.04	-0.31	2.78	0.00	0.00	9.99	HL-93	7.68
1	20	50.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

5969 Span 2-S15 Exterior Strength Mid M.RES

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	50.00	25.00	STRENGTH L.S.	2.33	1.80	HL-93
1	50.00	12.50	SERVICEBILITY	2.25	1.73	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.250
Inventory= 1.730

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	50.00	25.00	STRENGTH L.S.	3.28	553.13
1	50.00	12.50	SERVICEBILITY	2.98	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.984

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	17.2	11.4	52.7	0.0	4.13	HL-93	3.18
1	1	2.50	330.8	15.5	10.2	49.1	-1.2	4.48	HL-93	3.46
1	2	5.00	330.8	13.8	9.1	45.6	-2.5	4.89	HL-93	3.77
1	3	7.50	330.8	12.1	8.0	42.1	-4.3	5.36	HL-93	4.13
1	4	10.00	330.8	10.4	6.8	38.7	-6.3	5.90	HL-93	4.55
1	5	12.50	330.8	8.5	5.7	35.3	-8.4	6.55	HL-93	5.05
1	6	15.00	330.8	6.8	4.5	32.0	-10.5	7.32	HL-93	5.65
1	7	17.50	330.8	5.1	3.4	28.6	-12.7	8.27	HL-93	6.38
1	8	20.00	330.8	3.4	2.3	25.4	-14.9	9.42	HL-93	7.27

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1	9	22.50	330.8	1.7	1.1	22.4	-17.1	9.99	HL-93	8.35
1	10	25.00	330.8	0.0	0.0	19.4	-19.4	9.99	HL-93	9.73
1	11	27.50	330.8	-1.7	-1.1	17.1	-22.4	9.99	HL-93	8.35
1	12	30.00	330.8	-3.4	-2.3	14.9	-25.4	9.42	HL-93	7.27
1	13	32.50	330.8	-5.2	-3.4	12.7	-28.6	8.27	HL-93	6.38
1	14	35.00	330.8	-6.9	-4.6	10.5	-32.0	7.32	HL-93	5.64
1	15	37.50	330.8	-8.6	-5.7	8.4	-35.3	6.55	HL-93	5.05
1	16	40.00	330.8	-10.3	-6.8	6.3	-38.7	5.90	HL-93	4.55
1	17	42.50	330.8	-12.0	-8.0	4.3	-42.1	5.36	HL-93	4.13
1	18	45.00	330.8	-13.7	-9.1	2.5	-45.6	4.89	HL-93	3.77
1	19	47.50	330.8	-15.4	-10.2	1.2	-49.1	4.48	HL-93	3.46
1	20	50.00	330.8	-17.1	-11.4	0.0	-52.7	4.13	HL-93	3.18

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING INVENTORY RATING	OPERATING INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	50.00	0.00	STRENGTH L.S.	4.13	3.18	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.125
Inventory= 3.182

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	50.00	0.00	STRENGTH L.S.	5.15	56.90

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.154

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	25.00	3 0.425	-0.28	HL-93	2.69
		4 0.433	-0.28		2.64

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5969 Span 2-S15 Exterior Strength Mid M.RES

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95NB Span 2 S15 Ext. Strength CP 8/19/13


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       15           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
50.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	STANDARD SECTN DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.0000	10.00	0.4375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	13.50				33.	33.	33.
2	2	2	23.00				33.	33.	33.
3	1	1	13.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.56	2 2 2 2 2
1	0.00	0	0.00	0.40	0.00	0.00	0 0 0 0 0	0.56	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied
to the indicated loading type.

1 = The special distribution factor defined is applied to
the indicated loading type of calculation of all
moment, shear and deflection.

2 = The special distribution factor defined is only applied
to the loading type for calculating moment.

3 = The special distribution factor defined is only applied
to the loading type for calculating shear.

4 = The special distribution factor defined is only applied
to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	50.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those
of normal weight concrete (W=145 pcf) of various design strength shall
be as follows:

fc' = unit ultimate compressive strength of concrete as determined
by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	12.31
2	2	Diaphragm	0.000	0.00	0.00	0.14	31.15
3	1	Railing	0.087	0.00	50.00	0.00	0.00
4	1	Curb	0.076	0.00	50.00	0.00	0.00
5	0	Pavement	0.292	0.00	50.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi) (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	12.31	33.0	33.0	12.31
12.31	31.15	33.0	33.0	18.84
31.15	50.00	33.0	33.0	18.85

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.555       0.1249     0.6799


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      50.00

     1   DC2     0.087       0.000     50.000

     2   DC2     0.076       0.000     50.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1214.1	45.0	182.1	37.5	219.59

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.562	0.562	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.561				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.399				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN D NO	FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.33	5900.0	32.9	16.45	358.7	358.7
1	2	6.67	5900.0	32.9	16.45	358.7	358.7
1	3	10.00	5900.0	32.9	16.45	358.7	358.7
1	4	13.33	5900.0	32.9	16.45	358.7	358.7
1	5	16.67	6981.8	32.9	15.02	464.8	381.2
1	6	20.00	6981.8	32.9	15.02	464.8	381.2
1	7	23.33	6981.8	32.9	15.02	464.8	381.2
1	8	26.67	6981.8	32.9	15.02	464.8	381.2
1	9	30.00	6981.8	32.9	15.02	464.8	381.2
1	10	33.33	6981.8	32.9	15.02	464.8	381.2
1	11	36.67	5900.0	32.9	16.45	358.7	358.7
1	12	40.00	5900.0	32.9	16.45	358.7	358.7

1	13	43.33	5900.0	32.9	16.45	358.7	358.7
1	14	46.67	5900.0	32.9	16.45	358.7	358.7
1	15	50.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11472.1	0.022815	479.7	1276.7	659.5
1	1	3.33	11472.1	0.022815	479.7	1276.7	659.5
1	2	6.67	11472.1	0.022815	479.7	1276.7	659.5
1	3	10.00	11472.1	0.022815	479.7	1276.7	659.5
1	4	13.33	11472.1	0.022815	479.7	1276.7	659.5
1	5	16.67	13830.3	0.021457	613.8	1279.9	719.7
1	6	20.00	13830.3	0.021457	613.8	1279.9	719.7
1	7	23.33	13830.3	0.021457	613.8	1279.9	719.7
1	8	26.67	13830.3	0.021457	613.8	1279.9	719.7
1	9	30.00	13830.3	0.021457	613.8	1279.9	719.7
1	10	33.33	13830.3	0.021457	613.8	1279.9	719.7
1	11	36.67	11472.1	0.022815	479.7	1276.7	659.5
1	12	40.00	11472.1	0.022815	479.7	1276.7	659.5
1	13	43.33	11472.1	0.022815	479.7	1276.7	659.5
1	14	46.67	11472.1	0.022815	479.7	1276.7	659.5
1	15	50.00	11472.1	0.022815	479.7	1276.7	659.5

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15815.0	0.028983	534.7	4762.1	1348.1
1	1	3.33	15815.0	0.028983	534.7	4762.1	1348.1
1	2	6.67	15815.0	0.028983	534.7	4762.1	1348.1
1	3	10.00	15815.0	0.028983	534.7	4762.1	1348.1
1	4	13.33	15815.0	0.028983	534.7	4762.1	1348.1
1	5	16.67	19524.5	0.027497	681.3	4170.7	1491.4
1	6	20.00	19524.5	0.027497	681.3	4170.7	1491.4
1	7	23.33	19524.5	0.027497	681.3	4170.7	1491.4
1	8	26.67	19524.5	0.027497	681.3	4170.7	1491.4
1	9	30.00	19524.5	0.027497	681.3	4170.7	1491.4
1	10	33.33	19524.5	0.027497	681.3	4170.7	1491.4
1	11	36.67	15815.0	0.028983	534.7	4762.1	1348.1
1	12	40.00	15815.0	0.028983	534.7	4762.1	1348.1
1	13	43.33	15815.0	0.028983	534.7	4762.1	1348.1

1 14	46.67	15815.0	0.028983	534.7	4762.1	1348.1
1 15	50.00	15815.0	0.028983	534.7	4762.1	1348.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	3.33	9.7	43.2	0.5	0.0	53.4	
1	2	6.67	18.0	80.2	1.1	0.0	99.3	
1	3	10.00	25.0	111.0	1.6	0.0	137.6	
1	4	13.33	30.5	135.7	2.0	0.0	168.2	
1	5	16.67	34.7	154.2	2.1	0.0	191.0	
1	6	20.00	37.5	166.5	2.1	0.0	206.1	
1	7	23.33	38.9	172.7	2.2	0.0	213.7	
1	8	26.67	38.9	172.7	2.3	0.0	213.8	
1	9	30.00	37.5	166.5	2.3	0.0	206.3	
1	10	33.33	34.7	154.2	2.1	0.0	191.0	
1	11	36.67	30.5	135.7	1.7	0.0	167.9	
1	12	40.00	25.0	111.0	1.3	0.0	137.2	
1	13	43.33	18.0	80.2	0.8	0.0	99.0	
1	14	46.67	9.7	43.2	0.4	0.0	53.3	
1	15	50.00	0.0	0.0	0.0	0.0	0.0	

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.33	22.7	12.7	0.0	0.0	0.0	35.4
1	2	6.67	42.2	23.5	0.0	0.0	0.0	65.7
1	3	10.00	58.4	32.6	0.0	0.0	0.0	91.0
1	4	13.33	71.4	39.8	0.0	0.0	0.0	111.2
1	5	16.67	81.1	45.3	0.0	0.0	0.0	126.4
1	6	20.00	87.6	48.9	0.0	0.0	0.0	136.5
1	7	23.33	90.8	50.7	0.0	0.0	0.0	141.6
1	8	26.67	90.8	50.7	0.0	0.0	0.0	141.6
1	9	30.00	87.6	48.9	0.0	0.0	0.0	136.5
1	10	33.33	81.1	45.3	0.0	0.0	0.0	126.4
1	11	36.67	71.4	39.8	0.0	0.0	0.0	111.2
1	12	40.00	58.4	32.6	0.0	0.0	0.0	91.0
1	13	43.33	42.2	23.5	0.0	0.0	0.0	65.7
1	14	46.67	22.7	12.7	0.0	0.0	0.0	35.4
1	15	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.33	0.0	0.0	161.9	HL-93	0.0	HL-93
1	2	6.67	0.0	0.0	295.9	HL-93	0.0	HL-93
1	3	10.00	0.0	0.0	402.0	HL-93	0.0	HL-93
1	4	13.33	0.0	0.0	478.3	HL-93	0.0	HL-93
1	5	16.67	0.0	0.0	535.1	HL-93	0.0	HL-93
1	6	20.00	0.0	0.0	569.5	HL-93	0.0	HL-93
1	7	23.33	0.0	0.0	575.1	HL-93	0.0	HL-93
1	8	26.67	0.0	0.0	575.1	HL-93	0.0	HL-93
1	9	30.00	0.0	0.0	569.5	HL-93	0.0	HL-93
1	10	33.33	0.0	0.0	535.1	HL-93	0.0	HL-93
1	11	36.67	0.0	0.0	478.3	HL-93	0.0	HL-93
1	12	40.00	0.0	0.0	402.0	HL-93	0.0	HL-93
1	13	43.33	0.0	0.0	295.9	HL-93	0.0	HL-93
1	14	46.67	0.0	0.0	161.9	HL-93	0.0	HL-93
1	15	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.33	57.	0.	57.
1	2	6.67	102.	0.	102.
1	3	10.00	135.	0.	135.
1	4	13.33	155.	0.	155.
1	5	16.67	172.	0.	172.
1	6	20.00	183.	0.	183.
1	7	23.33	186.	0.	186.
1	8	26.67	186.	0.	186.
1	9	30.00	183.	0.	183.
1	10	33.33	172.	0.	172.
1	11	36.67	155.	0.	155.
1	12	40.00	135.	0.	135.
1	13	43.33	102.	0.	102.
1	14	46.67	57.	0.	57.
1	15	50.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.33	53.4	35.4	161.9	HL-93	0.0	HL-93	250.7	88.8
1	2	6.67	99.3	65.7	295.9	HL-93	0.0	HL-93	460.9	165.0
1	3	10.00	137.6	91.0	402.0	HL-93	0.0	HL-93	630.6	228.6
1	4	13.33	168.2	111.2	478.3	HL-93	0.0	HL-93	757.8	279.4
1	5	16.67	191.0	126.4	535.1	HL-93	0.0	HL-93	852.4	317.3
1	6	20.00	206.1	136.5	569.5	HL-93	0.0	HL-93	912.2	342.6
1	7	23.33	213.7	141.6	575.1	HL-93	0.0	HL-93	930.4	355.3
1	8	26.67	213.8	141.6	575.1	HL-93	0.0	HL-93	930.5	355.4
1	9	30.00	206.3	136.5	569.5	HL-93	0.0	HL-93	912.3	342.8
1	10	33.33	191.0	126.4	535.1	HL-93	0.0	HL-93	852.4	317.3
1	11	36.67	167.9	111.2	478.3	HL-93	0.0	HL-93	757.4	279.1
1	12	40.00	137.2	91.0	402.0	HL-93	0.0	HL-93	630.2	228.2
1	13	43.33	99.0	65.7	295.9	HL-93	0.0	HL-93	460.7	164.8
1	14	46.67	53.3	35.4	161.9	HL-93	0.0	HL-93	250.6	88.7
1	15	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)

1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.33	250.7	299.3	400.1	335.3	133.2	42.9
1	2	6.67	460.9	549.7	734.7	616.3	247.5	76.3
1	3	10.00	630.6	751.2	1003.9	843.1	342.9	101.0
1	4	13.33	757.8	901.3	1204.2	1012.9	419.2	116.5
1	5	16.67	852.4	1013.0	1353.4	1139.3	476.0	129.1
1	6	20.00	912.2	1083.0	1446.9	1219.0	513.9	137.3
1	7	23.33	930.4	1103.0	1473.3	1243.2	532.9	139.8
1	8	26.67	930.5	1103.0	1473.4	1243.3	533.0	139.8
1	9	30.00	912.3	1083.2	1447.1	1219.3	514.2	137.3
1	10	33.33	852.4	1013.0	1353.4	1139.3	476.0	129.1
1	11	36.67	757.4	900.9	1203.8	1012.5	418.6	116.5
1	12	40.00	630.2	750.9	1003.4	842.6	342.4	101.0
1	13	43.33	460.7	549.5	734.4	616.0	247.2	76.3
1	14	46.67	250.6	299.2	399.9	335.1	133.0	42.9
1	15	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.1	13.9	0.2	0.0	17.2
1	1	3.33	2.7	12.0	0.2	0.0	14.9
1	2	6.67	2.3	10.2	0.2	0.0	12.6
1	3	10.00	1.9	8.3	0.2	0.0	10.4
1	4	13.33	1.5	6.5	0.0	0.0	8.0
1	5	16.67	1.0	4.6	0.0	0.0	5.7
1	6	20.00	0.6	2.8	0.0	0.0	3.4
1	7	23.33	0.2	0.9	0.0	0.0	1.2
1	8	26.67	-0.2	-0.9	0.0	0.0	-1.1
1	9	30.00	-0.6	-2.8	0.0	0.0	-3.4
1	10	33.33	-1.0	-4.6	-0.1	0.0	-5.8
1	11	36.67	-1.5	-6.5	-0.1	0.0	-8.1
1	12	40.00	-1.9	-8.3	-0.1	0.0	-10.3
1	13	43.33	-2.3	-10.2	-0.1	0.0	-12.6
1	14	46.67	-2.7	-12.0	-0.1	0.0	-14.9
1	15	50.00	-3.1	-13.9	-0.1	0.0	-17.1

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.3	4.1	0.0	0.0	11.4	17.2	28.5
1	1	3.33	6.3	3.5	0.0	0.0	9.9	14.9	24.8
1	2	6.67	5.4	3.0	0.0	0.0	8.3	12.6	21.0
1	3	10.00	4.4	2.4	0.0	0.0	6.8	10.4	17.2

1	4	13.33	3.4	1.9	0.0	0.0	5.3	8.0	13.3
1	5	16.67	2.4	1.4	0.0	0.0	3.8	5.7	9.5
1	6	20.00	1.5	0.8	0.0	0.0	2.3	3.4	5.7
1	7	23.33	0.5	0.3	0.0	0.0	0.8	1.2	1.9
1	8	26.67	-0.5	-0.3	0.0	0.0	-0.8	-1.1	-1.9
1	9	30.00	-1.5	-0.8	0.0	0.0	-2.3	-3.4	-5.7
1	10	33.33	-2.4	-1.4	0.0	0.0	-3.8	-5.8	-9.6
1	11	36.67	-3.4	-1.9	0.0	0.0	-5.3	-8.1	-13.4
1	12	40.00	-4.4	-2.4	0.0	0.0	-6.8	-10.3	-17.1
1	13	43.33	-5.4	-3.0	0.0	0.0	-8.3	-12.6	-20.9
1	14	46.67	-6.3	-3.5	0.0	0.0	-9.9	-14.9	-24.7
1	15	50.00	-7.3	-4.1	0.0	0.0	-11.4	-17.1	-28.5

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	52.7	HL-93	0.0	HL-93
1	1	3.33	0.0	0.0	47.9	HL-93	-1.6	HL-93
1	2	6.67	0.0	0.0	43.3	HL-93	-3.6	HL-93
1	3	10.00	0.0	0.0	38.7	HL-93	-6.3	HL-93
1	4	13.33	0.0	0.0	34.2	HL-93	-9.1	HL-93
1	5	16.67	0.0	0.0	29.7	HL-93	-11.9	HL-93
1	6	20.00	0.0	0.0	25.4	HL-93	-14.9	HL-93
1	7	23.33	0.0	0.0	21.4	HL-93	-17.9	HL-93
1	8	26.67	0.0	0.0	17.9	HL-93	-21.4	HL-93
1	9	30.00	0.0	0.0	14.9	HL-93	-25.4	HL-93
1	10	33.33	0.0	0.0	11.9	HL-93	-29.7	HL-93
1	11	36.67	0.0	0.0	9.1	HL-93	-34.2	HL-93
1	12	40.00	0.0	0.0	6.3	HL-93	-38.7	HL-93
1	13	43.33	0.0	0.0	3.6	HL-93	-43.3	HL-93
1	14	46.67	0.0	0.0	1.6	HL-93	-47.9	HL-93
1	15	50.00	0.0	0.0	0.0	HL-93	-52.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	21.	0.	21.
1	1	3.33	19.	-1.	20.
1	2	6.67	17.	-2.	19.
1	3	10.00	15.	-3.	18.
1	4	13.33	13.	-4.	17.
1	5	16.67	11.	-5.	16.
1	6	20.00	10.	-6.	16.
1	7	23.33	9.	-8.	16.
1	8	26.67	8.	-9.	16.
1	9	30.00	6.	-10.	16.
1	10	33.33	5.	-11.	16.
1	11	36.67	4.	-13.	17.
1	12	40.00	3.	-15.	18.
1	13	43.33	2.	-17.	19.

```
 1 14  46.67    1.   -19.    20.
 1 15  50.00    0.   -21.    21.


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 TABLE 1.2.6.3B=MINIMUM WELD SIZE
                *****************

                          TOP FLANGE            BOTTOM FLANGE
                    ----------------------- -----------------------
 SP IN D FROM   Vr      Q/I    SHEAR  WELD      Q/I    SHEAR  WELD
 NO NO L SUPT  [1]      [2]    FLOW   SIZE             FLOW   SIZE
         (ft) (Kip)   (1/in)    [3]   (in)    (1/in)          (in)
 ---------------------------------------------------------------------
  1  0   0.00  21.0 .30967E-01  0.32 0.2500*.14525E-01  0.15 0.2500*
  1  1   3.33  19.7 .30967E-01  0.31 0.2500*.14525E-01  0.14 0.2500*
  1  2   6.67  18.6 .30967E-01  0.29 0.2500*.14525E-01  0.14 0.2500*
  1  3  10.00  17.6 .30967E-01  0.27 0.2500*.14525E-01  0.13 0.2500*
  1  4  13.33  16.5 .30967E-01  0.26 0.2500*.14525E-01  0.12 0.2500*
  1  5  16.67  16.2 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1  6  20.00  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1  7  23.33  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1  8  26.67  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1  9  30.00  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1 10  33.33  16.2 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
  1 11  36.67  16.5 .30967E-01  0.26 0.2500*.14525E-01  0.12 0.2500*
  1 12  40.00  17.6 .30967E-01  0.27 0.2500*.14525E-01  0.13 0.2500*
  1 13  43.33  18.6 .30967E-01  0.29 0.2500*.14525E-01  0.14 0.2500*
  1 14  46.67  19.7 .30967E-01  0.31 0.2500*.14525E-01  0.14 0.2500*
  1 15  50.00  21.0 .30967E-01  0.32 0.2500*.14525E-01  0.15 0.2500*


 ---------------------------------------------------------------------
   NOTE:  [1] Vr = range of shear due to live loads and impact

          [2] For non-composite construction:

              Q/I = (At * Dt) / Inc -- top flange
              Inc = moment of inertia of non-composite section
              At = area of top flange
              Dt = distance between the center of top flange and neutral axis
              Q/I = (Ab * Db) / Inc -- bottom flange
              Ab = area of bottom flange
              Db = distance between the center of bottom flange and neutral axis

              For composite construction:

              Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
              Q = statical moment about the neutral axis of the
                  composite section of the transformed compressive
                  concrete area or the area of reinforcement embedded
                  in the concrete for negative moment
              Ic = moment of inertia for composite section
              Q/I = (Ab * Db) / Ic -- bottom flange

          [3] shear flow = (Vr * Q) / (2 * I)

          * -- minimum weld size governs
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 TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
               ****************************************

                TOTAL     TOTAL L+I , LOAD TYPE = HL - 93
 SP IN D FROM ---------- -----------------------------------    MAX. SHEAR
 NO NO L SUPT DEAD LOADS POSITIVE GOVERN.  NEGATIVE GOVERN.       (kips)
```

		(ft)	(k)	(k)	LOAD TYPE	(k)	LOAD TYPE	POS.	NEG.
1	0	0.00	28.5	52.7	HL-93	0.0	HL-93	81.2	28.5
1	1	3.33	24.8	47.9	HL-93	-1.6	HL-93	72.7	23.1
1	2	6.67	21.0	43.3	HL-93	-3.6	HL-93	64.2	17.3
1	3	10.00	17.2	38.7	HL-93	-6.3	HL-93	55.9	10.9
1	4	13.33	13.3	34.2	HL-93	-9.1	HL-93	47.4	4.2
1	5	16.67	9.5	29.7	HL-93	-11.9	HL-93	39.2	-2.5
1	6	20.00	5.7	25.4	HL-93	-14.9	HL-93	31.1	-9.2
1	7	23.33	1.9	21.4	HL-93	-17.9	HL-93	23.3	-16.0
1	8	26.67	-1.9	17.9	HL-93	-21.4	HL-93	16.0	-23.3
1	9	30.00	-5.7	14.9	HL-93	-25.4	HL-93	9.2	-31.1
1	10	33.33	-9.6	11.9	HL-93	-29.7	HL-93	2.4	-39.3
1	11	36.67	-13.4	9.1	HL-93	-34.2	HL-93	-4.3	-47.5
1	12	40.00	-17.1	6.3	HL-93	-38.7	HL-93	-10.8	-55.8
1	13	43.33	-20.9	3.6	HL-93	-43.3	HL-93	-17.3	-64.2
1	14	46.67	-24.7	1.6	HL-93	-47.9	HL-93	-23.1	-72.6
1	15	50.00	-28.5	0.0	HL-93	-52.7	HL-93	-28.5	-81.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	81.2	97.0	129.6	108.6	42.8	15.7
1	1	3.33	72.7	87.0	116.4	97.2	37.1	14.8
1	2	6.67	64.2	77.2	103.3	86.0	31.5	14.0
1	3	10.00	55.9	67.5	90.3	74.8	25.8	13.2
1	4	13.33	47.4	57.7	77.2	63.6	19.9	12.4
1	5	16.67	39.2	48.1	64.5	52.6	14.2	12.2
1	6	20.00	31.1	38.7	52.0	41.8	8.5	12.2
1	7	23.33	23.3	29.7	39.9	31.4	2.9	12.2
1	8	26.67	-23.3	-29.7	-39.9	-31.3	-2.8	12.2
1	9	30.00	-31.1	-38.7	-51.9	-41.8	-8.5	12.2
1	10	33.33	-39.3	-48.2	-64.6	-52.7	-14.4	12.2
1	11	36.67	-47.5	-57.8	-77.4	-63.7	-20.0	12.4
1	12	40.00	-55.8	-67.4	-90.2	-74.8	-25.7	13.2
1	13	43.33	-64.2	-77.2	-103.2	-85.9	-31.4	14.0
1	14	46.67	-72.6	-87.0	-116.3	-97.2	-37.1	14.8
1	15	50.00	-81.2	-97.0	-129.6	-108.5	-42.7	15.7

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	28.54	0.00	HL-93	52.66	HL-93	ST1	23.86	129.65
						ST2	23.86	108.58
						ST4	31.85	42.80
						SE1	28.54	81.19

```
                                                    SE2    28.54         96.99

  2     28.50       0.00  HL-93      52.66  HL-93 ST1    23.82        129.60
                                                    ST2    23.82        108.54
                                                    ST4    31.80         42.75
                                                    SE1    28.50         81.16
                                                    SE2    28.50         96.95

 ------------------------------------------------------------------------------
 NOTE: [1] " - " Indicates Uplift

          ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.
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 TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
               ***************************************************

               NONCOMPOSITE DL     COMPOSITE DL          T O T A L
 SP IN D FROM ---------------- -------------------- --------------------------
 NO NO L SUPT   BEAM    SLAB   CONCENTRATED UNIFORM NONCOMPOSITE+COMPOSITE = DL
        (ft)    (in)    (in)       (in)      (in)       (in)       (in)     (in)
 ------------------------------------------------------------------------------

  1  0   0.00  0.0000  0.0000     0.0000     0.0000     0.0000   0.0000   0.0000
  1  1   3.33 -0.0195 -0.0868     0.0000    -0.0362    -0.1075  -0.0362  -0.1437
  1  2   6.67 -0.0380 -0.1688     0.0000    -0.0704    -0.2091  -0.0704  -0.2795
  1  3  10.00 -0.0545 -0.2419     0.0000    -0.1008    -0.2996  -0.1008  -0.4004
  1  4  13.33 -0.0681 -0.3026     0.0000    -0.1259    -0.3748  -0.1259  -0.5007
  1  5  16.67 -0.0786 -0.3492     0.0000    -0.1452    -0.4325  -0.1452  -0.5778
  1  6  20.00 -0.0858 -0.3813     0.0000    -0.1585    -0.4722  -0.1585  -0.6307
  1  7  23.33 -0.0895 -0.3976     0.0000    -0.1653    -0.4924  -0.1653  -0.6577
  1  8  26.67 -0.0895 -0.3976     0.0000    -0.1653    -0.4924  -0.1653  -0.6576
  1  9  30.00 -0.0858 -0.3813     0.0000    -0.1585    -0.4721  -0.1585  -0.6307
  1 10  33.33 -0.0786 -0.3492     0.0000    -0.1452    -0.4324  -0.1452  -0.5777
  1 11  36.67 -0.0681 -0.3026     0.0000    -0.1259    -0.3746  -0.1259  -0.5006
  1 12  40.00 -0.0545 -0.2419     0.0000    -0.1008    -0.2995  -0.1008  -0.4002
  1 13  43.33 -0.0380 -0.1688     0.0000    -0.0704    -0.2090  -0.0704  -0.2793
  1 14  46.67 -0.0195 -0.0868     0.0000    -0.0362    -0.1074  -0.0362  -0.1436
  1 15  50.00  0.0000  0.0000     0.0000     0.0000     0.0000   0.0000   0.0000


 ------------------------------------------------------------------------------
  NOTE: " - " Indicates downward deflections

  NOTE: The total noncomposite DL deflection is the sum of the deflections
        due to beam, slab, arbitrary DL uniform load and arbitrary DL
        concentrated load.

  NOTE: Due to space limit only beam deflections and slab deflections are
        printed out.
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 TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
               *****************

                NONCOMPOSITE  DEAD  LOADS       COMPOSITE DL      T O T A L
 SP IN D FROM --------------(in)-------------- ------(in)------ ------(in)------
 NO NO L SUPT STEEL  CAMBER    SLAB   CAMBER    DEFL.  CAMBER    DEFL.  CAMBER
        (ft)  DEFL.   SIZE     DEFL.   SIZE              SIZE             SIZE
 ------------------------------------------------------------------------------
  1  0   0.0  0.000    --      0.000    --      0.000    --      0.000    --
  1  1   3.3 -0.020  0   1/16 -0.088  0   1/ 8 -0.036  0   1/16 -0.144  0   3/16
  1  2   6.7 -0.038  0   1/16 -0.171  0   3/16 -0.070  0   1/ 8 -0.279  0   5/16
  1  3  10.0 -0.054  0   1/16 -0.245  0   1/ 4 -0.101  0   1/ 8 -0.400  0   7/16
  1  4  13.3 -0.068  0   1/ 8 -0.307  0   5/16 -0.126  0   3/16 -0.501  0   9/16
  1  5  16.7 -0.079  0   1/ 8 -0.354  0   3/ 8 -0.145  0   3/16 -0.578  0   5/ 8
  1  6  20.0 -0.086  0   1/ 8 -0.386  0   7/16 -0.159  0   3/16 -0.631  0  11/16
  1  7  23.3 -0.089  0   1/ 8 -0.403  0   7/16 -0.165  0   3/16 -0.658  0  11/16
```

1	8	26.7	-0.089	0	1/ 8	-0.403	0	7/16	-0.165	0	3/16	-0.658	0	11/16
1	9	30.0	-0.086	0	1/ 8	-0.386	0	7/16	-0.159	0	3/16	-0.631	0	11/16
1	10	33.3	-0.079	0	1/ 8	-0.354	0	3/ 8	-0.145	0	3/16	-0.578	0	5/ 8
1	11	36.7	-0.068	0	1/ 8	-0.307	0	5/16	-0.126	0	3/16	-0.501	0	9/16
1	12	40.0	-0.054	0	1/16	-0.245	0	1/ 4	-0.101	0	1/ 8	-0.400	0	7/16
1	13	43.3	-0.038	0	1/16	-0.171	0	3/16	-0.070	0	1/ 8	-0.279	0	5/16
1	14	46.7	-0.020	0	1/16	-0.088	0	1/ 8	-0.036	0	1/16	-0.144	0	3/16
1	15	50.0	0.000	--		0.000	--		0.000	--		0.000	--	

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.00	3 0.425	-0.279	MAX HL-93	0.75	0.00270
			0.029	MIN		
			-0.071	MAX LANE		
			0.029	MIN LANE		
		4 0.433	-0.284	MAX HL-93	0.75	0.00270
			0.029	MIN		
			-0.072	MAX LANE		
			0.029	MIN LANE		
		3 0.425	-0.173	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	-0.33	0.33	-1.46	1.46	-1.79	1.79
1	2	6.67	-0.60	0.60	-2.72	2.72	-3.32	3.32
1	3	10.00	-0.84	0.84	-3.77	3.77	-4.60	4.60
1	4	13.33	-1.02	1.02	-4.61	4.61	-5.63	5.63
1	5	16.67	-1.09	0.90	-4.92	4.03	-6.01	4.93
1	6	20.00	-1.18	0.97	-5.31	4.35	-6.49	5.32
1	7	23.33	-1.22	1.00	-5.51	4.51	-6.73	5.52
1	8	26.67	-1.22	1.00	-5.51	4.52	-6.73	5.52
1	9	30.00	-1.18	0.97	-5.32	4.36	-6.50	5.33
1	10	33.33	-1.09	0.90	-4.92	4.03	-6.01	4.93
1	11	36.67	-1.02	1.02	-4.59	4.59	-5.62	5.62
1	12	40.00	-0.84	0.84	-3.76	3.76	-4.59	4.59
1	13	43.33	-0.60	0.60	-2.71	2.71	-3.31	3.31
1	14	46.67	-0.33	0.33	-1.46	1.46	-1.78	1.78
1	15	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	-0.02	-0.33	0.89	-0.02	-2.12	2.67
1	2	6.67	-0.04	-0.62	1.64	-0.04	-3.94	4.97
1	3	10.00	-0.06	-0.86	2.28	-0.06	-5.46	6.88
1	4	13.33	-0.07	-1.05	2.78	-0.07	-6.67	8.41
1	5	16.67	-0.08	-1.19	2.47	-0.08	-7.20	7.40
1	6	20.00	-0.08	-1.28	2.67	-0.08	-7.77	7.99
1	7	23.33	-0.09	-1.33	2.77	-0.09	-8.06	8.29
1	8	26.67	-0.09	-1.33	2.77	-0.09	-8.06	8.29
1	9	30.00	-0.08	-1.28	2.67	-0.08	-7.77	8.00
1	10	33.33	-0.08	-1.19	2.47	-0.08	-7.20	7.40
1	11	36.67	-0.07	-1.05	2.78	-0.07	-6.66	8.40
1	12	40.00	-0.06	-0.86	2.28	-0.06	-5.45	6.87
1	13	43.33	-0.04	-0.62	1.64	-0.04	-3.93	4.96
1	14	46.67	-0.02	-0.33	0.89	-0.02	-2.12	2.67
1	15	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)

MAXIMUM POSITIVE,(MPa) MAXIMUM NEGATIVE,(ksi)

SP NO	IN NO	D FROM L SUPT (ft)	CONCR. TOP	STEEL TOP	STEEL BOT	CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	-0.16	-0.41	3.63	0.00	0.00	0.00
1	2	6.67	-0.29	-0.75	6.64	0.00	0.00	0.00
1	3	10.00	-0.40	-1.01	9.02	0.00	0.00	0.00
1	4	13.33	-0.47	-1.21	10.74	0.00	0.00	0.00
1	5	16.67	-0.48	-1.54	9.42	0.00	0.00	0.00
1	6	20.00	-0.51	-1.64	10.03	0.00	0.00	0.00
1	7	23.33	-0.51	-1.65	10.13	0.00	0.00	0.00
1	8	26.67	-0.51	-1.65	10.13	0.00	0.00	0.00
1	9	30.00	-0.51	-1.64	10.03	0.00	0.00	0.00
1	10	33.33	-0.48	-1.54	9.42	0.00	0.00	0.00
1	11	36.67	-0.47	-1.21	10.74	0.00	0.00	0.00
1	12	40.00	-0.40	-1.01	9.02	0.00	0.00	0.00
1	13	43.33	-0.29	-0.75	6.64	0.00	0.00	0.00
1	14	46.67	-0.16	-0.41	3.63	0.00	0.00	0.00
1	15	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.33	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	2	6.67	0.3	0.0	0.3	C	HL	2.3	0.0	2.3	T	HL
1	3	10.00	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	4	13.33	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	5	16.67	0.5	0.0	0.5	C	HL	3.0	0.0	3.0	T	HL
1	6	20.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL
1	7	23.33	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	8	26.67	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	9	30.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL
1	10	33.33	0.5	0.0	0.5	C	HL	3.0	0.0	3.0	T	HL
1	11	36.67	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	12	40.00	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	13	43.33	0.3	0.0	0.3	C	HL	2.3	0.0	2.3	T	HL
1	14	46.67	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	15	50.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.18	-2.53	6.31	-0.02	-2.12	2.67
1	2	6.67	33.	33.	-0.34	-4.69	11.61	-0.04	-3.94	4.97
1	3	10.00	33.	33.	-0.46	-6.47	15.90	-0.06	-5.46	6.88
1	4	13.33	33.	33.	-0.55	-7.88	19.15	-0.07	-6.67	8.41
1	5	16.67	33.	33.	-0.56	-8.74	16.83	-0.08	-7.20	7.40
1	6	20.00	33.	33.	-0.59	-9.41	18.02	-0.08	-7.77	7.99
1	7	23.33	33.	33.	-0.60	-9.71	18.41	-0.09	-8.06	8.29
1	8	26.67	33.	33.	-0.60	-9.71	18.42	-0.09	-8.06	8.29
1	9	30.00	33.	33.	-0.59	-9.41	18.03	-0.08	-7.77	8.00
1	10	33.33	33.	33.	-0.56	-8.74	16.83	-0.08	-7.20	7.40
1	11	36.67	33.	33.	-0.55	-7.87	19.13	-0.07	-6.66	8.40
1	12	40.00	33.	33.	-0.46	-6.46	15.89	-0.06	-5.45	6.87
1	13	43.33	33.	33.	-0.34	-4.68	11.60	-0.04	-3.93	4.96
1	14	46.67	33.	33.	-0.18	-2.52	6.30	-0.02	-2.12	2.67
1	15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.23	-2.65	7.40	-0.02	-2.12	2.67
1	2	6.67	33.	33.	-0.42	-4.91	13.60	-0.04	-3.94	4.97
1	3	10.00	33.	33.	-0.58	-6.78	18.61	-0.06	-5.46	6.88
1	4	13.33	33.	33.	-0.69	-8.24	22.37	-0.07	-6.67	8.41
1	5	16.67	33.	33.	-0.70	-9.20	19.65	-0.08	-7.20	7.40
1	6	20.00	33.	33.	-0.75	-9.90	21.03	-0.08	-7.77	7.99
1	7	23.33	33.	33.	-0.76	-10.21	21.45	-0.09	-8.06	8.29
1	8	26.67	33.	33.	-0.76	-10.21	21.46	-0.09	-8.06	8.29
1	9	30.00	33.	33.	-0.75	-9.91	21.04	-0.08	-7.77	8.00
1	10	33.33	33.	33.	-0.70	-9.20	19.65	-0.08	-7.20	7.40
1	11	36.67	33.	33.	-0.69	-8.23	22.35	-0.07	-6.66	8.40
1	12	40.00	33.	33.	-0.58	-6.76	18.60	-0.06	-5.45	6.87
1	13	43.33	33.	33.	-0.42	-4.90	13.59	-0.04	-3.93	4.96
1	14	46.67	33.	33.	-0.23	-2.65	7.39	-0.02	-2.12	2.67
1	15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.31	-3.42	9.84	-0.03	-2.70	3.48
1	2	6.67	33.	33.	-0.57	-6.33	18.09	-0.06	-5.02	6.47
1	3	10.00	33.	33.	-0.78	-8.73	24.76	-0.09	-6.96	8.97
1	4	13.33	33.	33.	-0.93	-10.62	29.75	-0.11	-8.51	10.96
1	5	16.67	33.	33.	-0.95	-11.88	26.14	-0.11	-9.19	9.65

1	6	20.00	33.	33.	-1.01	-12.78	27.97	-0.12	-9.92	10.42
1	7	23.33	33.	33.	-1.02	-13.18	28.53	-0.12	-10.28	10.80
1	8	26.67	33.	33.	-1.02	-13.18	28.53	-0.12	-10.29	10.80
1	9	30.00	33.	33.	-1.01	-12.79	27.98	-0.12	-9.92	10.42
1	10	33.33	33.	33.	-0.95	-11.88	26.14	-0.11	-9.19	9.65
1	11	36.67	33.	33.	-0.93	-10.60	29.73	-0.11	-8.50	10.94
1	12	40.00	33.	33.	-0.78	-8.72	24.74	-0.09	-6.95	8.95
1	13	43.33	33.	33.	-0.57	-6.32	18.08	-0.06	-5.01	6.46
1	14	46.67	33.	33.	-0.31	-3.41	9.84	-0.03	-2.70	3.48
1	15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.30	-2.57	8.62	-0.02	-1.85	2.26
1	2	6.67	33.	33.	-0.54	-4.75	15.83	-0.03	-3.45	4.21
1	3	10.00	33.	33.	-0.74	-6.55	21.62	-0.05	-4.78	5.83
1	4	13.33	33.	33.	-0.88	-7.95	25.91	-0.06	-5.84	7.12
1	5	16.67	33.	33.	-0.89	-8.98	22.76	-0.06	-6.29	6.26
1	6	20.00	33.	33.	-0.95	-9.65	24.32	-0.06	-6.79	6.76
1	7	23.33	33.	33.	-0.96	-9.93	24.74	-0.06	-7.04	7.01
1	8	26.67	33.	33.	-0.96	-9.94	24.74	-0.06	-7.04	7.01
1	9	30.00	33.	33.	-0.95	-9.66	24.32	-0.06	-6.79	6.77
1	10	33.33	33.	33.	-0.89	-8.98	22.76	-0.06	-6.29	6.26
1	11	36.67	33.	33.	-0.88	-7.94	25.90	-0.06	-5.83	7.11
1	12	40.00	33.	33.	-0.74	-6.54	21.61	-0.05	-4.76	5.82
1	13	43.33	33.	33.	-0.54	-4.74	15.82	-0.03	-3.44	4.20
1	14	46.67	33.	33.	-0.30	-2.56	8.62	-0.02	-1.85	2.26
1	15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.25	-3.25	8.39	-0.03	-2.70	3.48
1	2	6.67	33.	33.	-0.46	-6.03	15.44	-0.06	-5.02	6.47
1	3	10.00	33.	33.	-0.62	-8.33	21.15	-0.09	-6.96	8.97
1	4	13.33	33.	33.	-0.74	-10.14	25.45	-0.11	-8.51	10.96
1	5	16.67	33.	33.	-0.76	-11.26	22.37	-0.11	-9.19	9.65
1	6	20.00	33.	33.	-0.81	-12.13	23.96	-0.12	-9.92	10.42
1	7	23.33	33.	33.	-0.82	-12.52	24.48	-0.12	-10.28	10.80
1	8	26.67	33.	33.	-0.82	-12.52	24.48	-0.12	-10.29	10.80
1	9	30.00	33.	33.	-0.81	-12.14	23.96	-0.12	-9.92	10.42
1	10	33.33	33.	33.	-0.76	-11.26	22.37	-0.11	-9.19	9.65
1	11	36.67	33.	33.	-0.74	-10.12	25.44	-0.11	-8.50	10.94
1	12	40.00	33.	33.	-0.62	-8.31	21.13	-0.09	-6.95	8.95
1	13	43.33	33.	33.	-0.46	-6.02	15.43	-0.06	-5.01	6.46
1	14	46.67	33.	33.	-0.25	-3.25	8.38	-0.03	-2.70	3.48

1 15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.33	33.	33.	-0.23	-2.41	7.17	-0.02	-1.85	2.26
1	2	6.67	33.	33.	-0.43	-4.45	13.17	-0.03	-3.45	4.21
1	3	10.00	33.	33.	-0.58	-6.14	18.01	-0.05	-4.78	5.83
1	4	13.33	33.	33.	-0.69	-7.47	21.62	-0.06	-5.84	7.12
1	5	16.67	33.	33.	-0.70	-8.37	18.99	-0.06	-6.29	6.26
1	6	20.00	33.	33.	-0.75	-9.00	20.30	-0.06	-6.79	6.76
1	7	23.33	33.	33.	-0.76	-9.27	20.69	-0.06	-7.04	7.01
1	8	26.67	33.	33.	-0.76	-9.27	20.69	-0.06	-7.04	7.01
1	9	30.00	33.	33.	-0.75	-9.00	20.31	-0.06	-6.79	6.77
1	10	33.33	33.	33.	-0.70	-8.37	18.99	-0.06	-6.29	6.26
1	11	36.67	33.	33.	-0.69	-7.46	21.61	-0.06	-5.83	7.11
1	12	40.00	33.	33.	-0.58	-6.13	18.00	-0.05	-4.76	5.82
1	13	43.33	33.	33.	-0.43	-4.45	13.16	-0.03	-3.44	4.20
1	14	46.67	33.	33.	-0.23	-2.40	7.17	-0.02	-1.85	2.26
1	15	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V 10.4
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — COMPOSITE
UNIVERSITY OF MARYLAND — LRF -- 2010
ANALYSIS

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	13.50	169.	-5.7	112.	-1.1	481.	-1.2	0.	0.0	-7.93
											1.21
	LB			5.7		2.8		10.8		0.0	19.27
											10.80
	RT			-5.3		-1.0		-1.4		0.0	-7.77
											1.38
	RB			4.4		2.2		8.5		0.0	15.04
											8.47
1	LT	36.50	169.	-5.3	112.	-1.0	481.	-1.4	0.	0.0	-7.76
											1.38
	LB			4.4		2.2		8.5		0.0	15.03
											8.47
	RT			-5.7		-1.1		-1.2		0.0	-7.92
											1.21
	RB			5.7		2.8		10.8		0.0	19.26
											10.80

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V 10.4
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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	13.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	23.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.4375
3	13.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 50.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1]) [3]/(12*[1])	<=>	LIMITATION 1/25 1/30	STATUS
1	13.50	6.75	32.90	[2]= 41.31	0.06885	>	0.04000	OK
				[3]= 32.90	0.05483	>	0.03300	OK
2	23.00	6.75	32.90	[2]= 41.31	0.06885	>	0.04000	OK
				[3]= 32.90	0.05483	>	0.03300	OK
3	13.50	6.75	32.90	[2]= 41.31	0.06885	>	0.04000	OK
				[3]= 32.90	0.05483	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0		0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	1		3.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	2		6.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	3		10.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	4		13.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	5		16.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	6		20.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	7		23.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	8		26.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	9		30.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	10		33.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	11		36.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	12		40.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	13		43.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	14		46.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	15		50.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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MERLIN V 10.4
COMPOSITE
LRF -- 2010
CODE CHECK

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0		0.00	0	1	31.42	0.550	57.13	168.97	2
1	1		3.33	0	1	31.42	0.550	57.13	168.97	2
1	2		6.67	0	1	31.42	0.550	57.13	168.97	2
1	3		10.00	0	1	31.42	0.550	57.13	168.97	2
1	4		13.33	0	1	31.42	0.550	57.13	168.97	2
1	5		16.67	0	1	31.42	0.550	63.98	168.97	2
1	6		20.00	0	1	31.42	0.550	63.98	168.97	2
1	7		23.33	0	1	31.42	0.550	63.98	168.97	2
1	8		26.67	0	1	31.42	0.550	63.98	168.97	2
1	9		30.00	0	1	31.42	0.550	63.98	168.97	2
1	10		33.33	0	1	31.42	0.550	63.98	168.97	2
1	11		36.67	0	1	31.42	0.550	57.13	168.97	2
1	12		40.00	0	1	31.42	0.550	57.13	168.97	2
1	13		43.33	0	1	31.42	0.550	57.13	168.97	2
1	14		46.67	0	1	31.42	0.550	57.13	168.97	2
1	15		50.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners

```
          1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ------------------------------------------------------------

  1  0   0.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  1   3.33   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  2   6.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  3  10.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  4  13.33   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  5  16.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  6  20.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  7  23.33   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  26.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  30.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  33.33   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  36.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  40.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  43.33   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  46.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  50.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                  ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
```

NO	FROM	TO	(ksi)	(ksi)	(ksi)	(ksi)	Cb
1	0.0	12.3	0.000	7.971	4.630	1.289	1.588
			0.000	0.000	0.000	0.000	1.000
2	12.3	31.1	7.971	9.492	9.920	10.348	1.000
			0.000	0.000	0.000	0.000	1.000
3	31.1	50.0	0.000	9.492	6.557	3.621	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	Lb (ft)	Lp (ft)	Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	1	3.33	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	2	6.67	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	3	10.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	4	13.33	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	5	16.67	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	6	20.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	7	23.33	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	8	26.67	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	9	30.00	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	10	33.33	1	7.77	11.26	19.84	0	18.85	6.98	26.22	2	2
1	11	36.67	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	12	40.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	13	43.33	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	14	46.67	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	15	50.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.59	33.00	33.00	33.00
1	1		3.33	1	1.000	1.000	1.59	33.00	33.00	33.00
1	2		6.67	1	1.000	1.000	1.59	33.00	33.00	33.00
1	3		10.00	1	1.000	1.000	1.59	33.00	33.00	33.00
1	4		13.33	1	1.000	1.000	1.00	33.00	27.04	27.04
1	5		16.67	1	1.000	1.000	1.00	33.00	26.90	26.90
1	6		20.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	7		23.33	1	1.000	1.000	1.00	33.00	26.90	26.90
1	8		26.67	1	1.000	1.000	1.00	33.00	26.90	26.90
1	9		30.00	1	1.000	1.000	1.00	33.00	26.90	26.90
1	10		33.33	1	1.000	1.000	1.00	33.00	26.89	26.89
1	11		36.67	1	1.000	1.000	1.00	33.00	27.04	27.04
1	12		40.00	1	1.000	1.000	1.00	33.00	27.04	27.04
1	13		43.33	1	1.000	1.000	1.00	33.00	27.04	27.04
1	14		46.67	1	1.000	1.000	1.00	33.00	27.04	27.04
1	15		50.00	1	1.000	1.000	1.00	33.00	27.04	27.04

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	6.34	17.35
1	1		3.33	1	6.34	17.35
1	2		6.67	1	6.34	17.35
1	3		10.00	1	6.34	17.35
1	4		13.33	1	6.34	17.35
1	5		16.67	1	8.50	17.53
1	6		20.00	1	8.50	17.53
1	7		23.33	1	8.50	17.53
1	8		26.67	1	8.50	17.53
1	9		30.00	1	8.50	17.53
1	10		33.33	1	8.50	17.53
1	11		36.67	1	6.34	17.35
1	12		40.00	1	6.34	17.35
1	13		43.33	1	6.34	17.35
1	14		46.67	1	6.34	17.35
1	15		50.00	1	6.34	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

5969 Span 2-S15 Exterior Strength CP.RES
S T R E N G T H C A T E G O R Y, Category

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0		0.00	1	0	2	0		2	0
1	1		3.33	1	0	2	0		2	0
1	2		6.67	1	0	2	0		2	0
1	3		10.00	1	0	2	0		2	0
1	4		13.33	1	0	2	0		2	0
1	5		16.67	1	0	2	0		2	0
1	6		20.00	1	0	2	0		2	0
1	7		23.33	1	0	2	0		2	0
1	8		26.67	1	0	2	0		2	0
1	9		30.00	1	0	2	0		2	0
1	10		33.33	1	0	2	0		2	0
1	11		36.67	1	0	2	0		2	0
1	12		40.00	1	0	2	0		2	0
1	13		43.33	1	0	2	0		2	0
1	14		46.67	1	0	2	0		2	0
1	15		50.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.33	0.0	19.8	2.7	33.0	2.7	33.0	2.7	33.0	0
				0.0	-	2.7	-	2.7	33.0	-	-	0
1	2		6.67	0.0	19.8	5.0	33.0	5.0	33.0	5.0	33.0	0

			0.0	-	5.0	-	5.0	33.0	-	-	0
1	3	10.00	0.0	19.8	6.9	33.0	6.9	33.0	6.9	33.0	0
			0.0	-	6.9	-	6.9	33.0	-	-	0
1	4	13.33	0.0	19.8	8.4	33.0	8.4	33.0	8.4	27.0	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	5	16.67	0.0	19.8	9.0	33.0	9.0	33.0	9.0	26.9	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	6	20.00	0.0	19.8	9.7	33.0	9.7	33.0	9.7	26.9	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	7	23.33	0.0	19.8	10.1	33.0	10.1	33.0	10.1	26.9	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	8	26.67	0.0	19.8	10.1	33.0	10.1	33.0	10.1	26.9	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	9	30.00	0.0	19.8	9.7	33.0	9.7	33.0	9.7	26.9	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	10	33.33	0.0	19.8	9.0	33.0	9.0	33.0	9.0	26.9	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	11	36.67	0.0	19.8	8.4	33.0	8.4	33.0	8.4	27.0	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	12	40.00	0.0	19.8	6.9	33.0	6.9	33.0	6.9	27.0	0
			0.0	-	6.9	-	6.9	33.0	-	-	0
1	13	43.33	0.0	19.8	5.0	33.0	5.0	33.0	5.0	27.0	0
			0.0	-	5.0	-	5.0	33.0	-	-	0
1	14	46.67	0.0	19.8	2.7	33.0	2.7	33.0	2.7	27.0	0
			0.0	-	2.7	-	2.7	33.0	-	-	0
1	15	50.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.33	0.000	0.081	0.081	0.081	0.081	2
					0.081		0.081	3
1	2	6.67	0.000	0.151	0.151	0.151	0.151	2
					0.151		0.151	3
1	3	10.00	0.000	0.209	0.209	0.209	0.209	2
					0.209		0.209	3
1	4	13.33	0.000	0.256	0.256	0.312	0.312	4
					0.256		0.256	3
1	5	16.67	0.000	0.273	0.273	0.335	0.335	4
					0.224		0.224	3
1	6	20.00	0.000	0.295	0.295	0.362	0.362	4
					0.242		0.242	3
1	7	23.33	0.000	0.306	0.306	0.375	0.375	4
					0.251		0.251	3

1	8	26.67	0.000	0.306	0.306	0.375	0.375	4
					0.251		0.251	3
1	9	30.00	0.000	0.295	0.295	0.362	0.362	4
					0.242		0.242	3
1	10	33.33	0.000	0.273	0.273	0.335	0.335	4
					0.224		0.224	3
1	11	36.67	0.000	0.255	0.255	0.312	0.312	4
					0.255		0.255	3
1	12	40.00	0.000	0.209	0.209	0.255	0.255	4
					0.209		0.209	3
1	13	43.33	0.000	0.151	0.151	0.184	0.184	4
					0.151		0.151	3
1	14	46.67	0.000	0.081	0.081	0.099	0.099	4
					0.081		0.081	3
1	15	50.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0
1	1		3.33	0	400.1	1969.3	-	-	0.0	19.8	-	-	0--0
1	2		6.67	0	734.7	1969.3	-	-	0.0	19.8	-	-	0--0
1	3		10.00	0	1003.9	1969.3	-	-	0.0	19.8	-	-	0--0
1	4		13.33	0	1204.2	1969.3	-	-	0.0	19.8	-	-	0--0
1	5		16.67	0	1353.4	2282.6	-	-	0.0	19.8	-	-	0--0
1	6		20.00	0	1446.9	2282.6	-	-	0.0	19.8	-	-	0--0
1	7		23.33	0	1473.3	2282.6	-	-	0.0	19.8	-	-	0--0
1	8		26.67	0	1473.4	2282.6	-	-	0.0	19.8	-	-	0--0
1	9		30.00	0	1447.1	2282.6	-	-	0.0	19.8	-	-	0--0
1	10		33.33	0	1353.4	2282.6	-	-	0.0	19.8	-	-	0--0
1	11		36.67	0	1203.8	1969.3	-	-	0.0	19.8	-	-	0--0
1	12		40.00	0	1003.4	1969.3	-	-	0.0	19.8	-	-	0--0
1	13		43.33	0	734.4	1969.3	-	-	0.0	19.8	-	-	0--0
1	14		46.67	0	399.9	1969.3	-	-	0.0	19.8	-	-	0--0
1	15		50.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange
of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections

in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	3.33	0	0.203				0.203	1
1	2	6.67	0	0.373				0.373	1
1	3	10.00	0	0.510				0.510	1
1	4	13.33	0	0.612				0.612	1
1	5	16.67	0	0.593				0.593	1
1	6	20.00	0	0.634				0.634	1
1	7	23.33	0	0.645				0.645	1
1	8	26.67	0	0.645				0.645	1
1	9	30.00	0	0.634				0.634	1
1	10	33.33	0	0.593				0.593	1
1	11	36.67	0	0.611				0.611	1
1	12	40.00	0	0.510				0.510	1
1	13	43.33	0	0.373				0.373	1
1	14	46.67	0	0.203				0.203	1
1	15	50.00	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	10.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	13.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	16.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	20.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	23.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	26.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	30.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	33.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	36.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	40.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	43.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	46.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	50.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
            = 5 for unstiffened beams and girders
            = 5 for stiffened girders when (d0/D) > 3, or
                                           (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	129.6	<	330.8	0	0
1	1	3.33	0	116.4	<	330.8	0	0
1	2	6.67	0	103.3	<	330.8	0	0
1	3	10.00	0	90.3	<	330.8	0	0
1	4	13.33	0	77.2	<	330.8	0	0
1	5	16.67	0	64.5	<	330.8	0	0
1	6	20.00	0	52.0	<	330.8	0	0
1	7	23.33	0	39.9	<	330.8	0	0
1	8	26.67	0	39.9	<	330.8	0	0
1	9	30.00	0	51.9	<	330.8	0	0
1	10	33.33	0	64.6	<	330.8	0	0
1	11	36.67	0	77.4	<	330.8	0	0
1	12	40.00	0	90.2	<	330.8	0	0
1	13	43.33	0	103.2	<	330.8	0	0
1	14	46.67	0	116.3	<	330.8	0	0
1	15	50.00	0	129.6	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	129.65	330.76	0	
1	1	3.33	33.	116.38	330.76	0	
1	2	6.67	33.	103.26	330.76	0	
1	3	10.00	33.	90.27	330.76	0	
1	4	13.33	33.	77.25	330.76	0	
1	5	16.67	33.	64.47	330.76	0	
1	6	20.00	33.	51.98	330.76	0	
1	7	23.33	33.	39.92	330.76	0	
1	8	26.67	33.	39.87	330.76	0	
1	9	30.00	33.	51.93	330.76	0	
1	10	33.33	33.	64.60	330.76	0	
1	11	36.67	33.	77.38	330.76	0	
1	12	40.00	33.	90.23	330.76	0	
1	13	43.33	33.	103.21	330.76	0	
1	14	46.67	33.	116.33	330.76	0	
1	15	50.00	33.	129.60	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	BOT. FLANGE ff+fl/2 [1] (ksi)	BOT. FLANGE ff+fl/2 [2] (ksi)	COMP. FLG fc (ksi)	FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	3.33	2.7	-	31.4	7.4	31.4	-	-	00-
1	2	6.67	4.9	-	31.4	13.6	31.4	-	-	00-
1	3	10.00	6.8	-	31.4	18.6	31.4	-	-	00-
1	4	13.33	8.2	-	31.4	22.4	31.4	-	-	00-
1	5	16.67	9.2	-	31.4	19.7	31.4	-	-	00-
1	6	20.00	9.9	-	31.4	21.0	31.4	-	-	00-
1	7	23.33	10.2	-	31.4	21.5	31.4	-	-	00-
1	8	26.67	10.2	-	31.4	21.5	31.4	-	-	00-
1	9	30.00	9.9	-	31.4	21.0	31.4	-	-	00-
1	10	33.33	9.2	-	31.4	19.7	31.4	-	-	00-
1	11	36.67	8.2	-	31.4	22.4	31.4	-	-	00-
1	12	40.00	6.8	-	31.4	18.6	31.4	-	-	00-
1	13	43.33	4.9	-	31.4	13.6	31.4	-	-	00-
1	14	46.67	2.6	-	31.4	7.4	31.4	-	-	00-
1	15	50.00	0.0	-	31.4	0.0	31.4	-	-	00-

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```
NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
            but not to exceed the smaller of RhFyc and Fyw/0.7
            k = bending-buckling coefficient
              = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
            where:
            Dc = depth of the web in compression in the elastic
                 range. For composite sections, Dc shall be
                 determined as specified in Article D6.3.1

      "-" is N.A.

      Flag check - 0 = OK; 1 = NG
      T = Top Flange; B = Bottom Flange; C = Comp. Flange

      The values of fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------
```

SP NO	IN NO	D L FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.33	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	2	6.67	TR	-0.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	3	10.00	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	4	13.33	TR	-0.6	A B B^C^C D E E^	TR	5.2	A B B^C^C D
1		13.50	SCG					
1	5	16.67	TR	-0.7	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	6	20.00	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	7	23.33	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	8	26.67	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	9	30.00	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	10	33.33	TR	-0.7	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1		36.50	SCG					
1	11	36.67	TR	-0.6	A B B^C^C D E E^	TR	5.2	A B B^C^C D
1	12	40.00	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	13	43.33	TR	-0.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	14	46.67	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	15	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

```
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
             POC = Dead load point of contraflexure

             I:P = Point where INT coincides with POC
             I:C = Point where INT coincides with SCG
             S:P = Point where SCG coincides with POC
```

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.33	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	2	6.67	TR	-0.2	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	3	10.00	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	4	13.33	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1		13.50	SCG					
1	5	16.67	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	6	20.00	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	7	23.33	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	8	26.67	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	9	30.00	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	10	33.33	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1		36.50	SCG					
1	11	36.67	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	12	40.00	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	13	43.33	TR	-0.2	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	14	46.67	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	15	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	31.5	.28983E-01	0.9	TR	3.09	3	10.17
1	1	3.33	29.6	.28983E-01	0.9	TR	3.09	3	10.82
1	2	6.67	27.9	.28983E-01	0.8	TR	3.09	3	11.46

1	3	10.00	26.4	.28983E-01	0.8	TR	3.09	3	12.12
1	4	13.33	24.8	.28983E-01	0.7	TR	3.09	3	12.93
1	5	16.67	24.4	.27497E-01	0.7	TR	3.09	3	13.85
1	6	20.00	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	7	23.33	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	8	26.67	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	9	30.00	24.4	.27497E-01	0.7	TR	3.09	3	13.82
1	10	33.33	24.4	.27497E-01	0.7	TR	3.09	3	13.85
1	11	36.67	24.8	.28983E-01	0.7	TR	3.09	3	12.93
1	12	40.00	26.4	.28983E-01	0.8	TR	3.09	3	12.12
1	13	43.33	27.9	.28983E-01	0.8	TR	3.09	3	11.46
1	14	46.67	29.6	.28983E-01	0.9	TR	3.09	3	10.82
1	15	50.00	31.5	.28983E-01	0.9	TR	3.09	3	10.17

Please read NOTE on the following page

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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                   I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
           *** If ADTT Single Lane is less than 960, use Fatigue II Table.
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                  *********************************************

                 AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-2     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
```

		(ft)	[1]	[2]	[3]	[4],Zr <--Input	[5], input	(in),[6]
1	0	0.00	15.7	.28983E-01	0.5	TR	1.36	3	8.97
1	1	3.33	14.8	.28983E-01	0.4	TR	1.36	3	9.54
1	2	6.67	14.0	.28983E-01	0.4	TR	1.36	3	10.11
1	3	10.00	13.2	.28983E-01	0.4	TR	1.36	3	10.69
1	4	13.33	12.4	.28983E-01	0.4	TR	1.36	3	11.40
1	5	16.67	12.2	.27497E-01	0.3	TR	1.36	3	12.21
1	6	20.00	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	7	23.33	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	8	26.67	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	9	30.00	12.2	.27497E-01	0.3	TR	1.36	3	12.18
1	10	33.33	12.2	.27497E-01	0.3	TR	1.36	3	12.21
1	11	36.67	12.4	.28983E-01	0.4	TR	1.36	3	11.40
1	12	40.00	13.2	.28983E-01	0.4	TR	1.36	3	10.69
1	13	43.33	14.0	.28983E-01	0.4	TR	1.36	3	10.11
1	14	46.67	14.8	.28983E-01	0.4	TR	1.36	3	9.54
1	15	50.00	15.7	.28983E-01	0.5	TR	1.36	3	8.97

Please read NOTE on the following page

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D L (ft)	FROM SUPT	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) <=>	SEE NOTE N1	SEE NOTE N2	STATUS BLANK=OK CHECK=**
1	8	26.67		1	1276.3	1204.9		96	>	63	
1	15	50.00		1	1276.3	1204.9		96	>	63	

Please read NOTE on the Following Page

1

Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment

and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	26.667	10.000
1	26.667	50.000	10.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	50.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN D NO L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.33	1969.3	53.4	35.4	161.9	0.0	8.48	HL-93	6.54	0
1	2	6.67	1969.3	99.3	65.7	295.9	0.0	4.39	HL-93	3.38	0
1	3	10.00	1969.3	137.6	91.0	402.0	0.0	3.08	HL-93	2.37	0
1	4	13.33	1969.3	168.2	111.2	478.3	0.0	2.48	HL-93	1.91	0
1	5	16.67	2282.6	191.0	126.4	535.1	0.0	2.58	HL-93	1.99	0
1	6	20.00	2282.6	206.1	136.5	569.5	0.0	2.38	HL-93	1.84	0
1	7	23.33	2282.6	213.7	141.6	575.1	0.0	2.34	HL-93	1.80	0
1	8	26.67	2282.6	213.8	141.6	575.1	0.0	2.34	HL-93	1.80	0
1	9	30.00	2282.6	206.3	136.5	569.5	0.0	2.38	HL-93	1.84	0
1	10	33.33	2282.6	191.0	126.4	535.1	0.0	2.58	HL-93	1.99	0
1	11	36.67	1969.3	167.9	111.2	478.3	0.0	2.48	HL-93	1.91	0
1	12	40.00	1969.3	137.2	91.0	402.0	0.0	3.08	HL-93	2.37	0
1	13	43.33	1969.3	99.0	65.7	295.9	0.0	4.39	HL-93	3.38	0
1	14	46.67	1969.3	53.3	35.4	161.9	0.0	8.48	HL-93	6.54	0
1	15	50.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

Please read NOTES on the following page

1

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.33	31.4	-2.12	2.67	-0.41	3.63	0.00	0.00	7.89	HL-93	6.07
1	2	6.67	31.4	-3.94	4.97	-0.75	6.64	0.00	0.00	3.97	HL-93	3.06
1	3	10.00	31.4	-5.46	6.88	-1.01	9.02	0.00	0.00	2.71	HL-93	2.09
1	4	13.33	31.4	-6.67	8.41	-1.21	10.74	0.00	0.00	2.14	HL-93	1.64
1	5	16.67	31.4	-7.20	7.40	-1.54	9.42	0.00	0.00	2.54	HL-93	1.95
1	6	20.00	31.4	-7.77	7.99	-1.64	10.03	0.00	0.00	2.33	HL-93	1.79
1	7	23.33	31.4	-8.06	8.29	-1.65	10.13	0.00	0.00	2.28	HL-93	1.75
1	8	26.67	31.4	-8.06	8.29	-1.65	10.13	0.00	0.00	2.28	HL-93	1.75
1	9	30.00	31.4	-7.77	8.00	-1.64	10.03	0.00	0.00	2.33	HL-93	1.79
1	10	33.33	31.4	-7.20	7.40	-1.54	9.42	0.00	0.00	2.54	HL-93	1.95
1	11	36.67	31.4	-6.66	8.40	-1.21	10.74	0.00	0.00	2.14	HL-93	1.64
1	12	40.00	31.4	-5.45	6.87	-1.01	9.02	0.00	0.00	2.71	HL-93	2.09
1	13	43.33	31.4	-3.93	4.96	-0.75	6.64	0.00	0.00	3.97	HL-93	3.06
1	14	46.67	31.4	-2.12	2.67	-0.41	3.63	0.00	0.00	7.89	HL-93	6.07
1	15	50.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

1

NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH	CRITICAL LOCATION D FROM L SUPT	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

	(ft)	(ft)				
1	50.00	26.67	STRENGTH L.S.	2.34	1.80	HL-93
1	50.00	13.33	SERVICEBILITY	2.14	1.64	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.137
Inventory= 1.644

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	50.00	26.67	STRENGTH L.S.	3.29	551.91
1	50.00	13.33	SERVICEBILITY	2.85	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.850

1

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	17.2	11.4	52.7	0.0	4.13	HL-93	3.18
1	1	3.33	330.8	14.9	9.9	47.9	-1.6	4.61	HL-93	3.56
1	2	6.67	330.8	12.6	8.3	43.3	-3.6	5.19	HL-93	4.01
1	3	10.00	330.8	10.4	6.8	38.7	-6.3	5.90	HL-93	4.55
1	4	13.33	330.8	8.0	5.3	34.2	-9.1	6.79	HL-93	5.24
1	5	16.67	330.8	5.7	3.8	29.7	-11.9	7.93	HL-93	6.12
1	6	20.00	330.8	3.4	2.3	25.4	-14.9	9.42	HL-93	7.27
1	7	23.33	330.8	1.2	0.8	21.4	-17.9	9.99	HL-93	8.77
1	8	26.67	330.8	-1.1	-0.8	17.9	-21.4	9.99	HL-93	8.78
1	9	30.00	330.8	-3.4	-2.3	14.9	-25.4	9.42	HL-93	7.27
1	10	33.33	330.8	-5.8	-3.8	11.9	-29.7	7.93	HL-93	6.12
1	11	36.67	330.8	-8.1	-5.3	9.1	-34.2	6.79	HL-93	5.24
1	12	40.00	330.8	-10.3	-6.8	6.3	-38.7	5.90	HL-93	4.55
1	13	43.33	330.8	-12.6	-8.3	3.6	-43.3	5.19	HL-93	4.01
1	14	46.67	330.8	-14.9	-9.9	1.6	-47.9	4.61	HL-93	3.56
1	15	50.00	330.8	-17.1	-11.4	0.0	-52.7	4.13	HL-93	3.18

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN CRITICAL LOCATION GOVERNING GOVERNING

NO.	LENGTH (ft)	D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	LIVE LOAD TYPE
1	50.00	0.00	STRENGTH L.S.	4.13	3.18	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.125
Inventory= 3.182

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	50.00	0.00	STRENGTH L.S.	5.15	56.90

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.154

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	25.00	3 0.425	-0.28	HL-93	2.69
		4 0.433	-0.28		2.64

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 2 S15 Exterior Fatigue 8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       13           1       AASHTO  2010     0     2            3
 ----------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
50.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	STANDARD SECTN DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.0000	10.00	0.4375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB -->TYPE<-- 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	13.50				33.	33.	33.
2	2	2	23.00				33.	33.	33.
3	1	1	13.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)		DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	OPTIONAL DISTRIBUTION FACTORS DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.56	2 2 2 2 2
1	0.00	0	0.00	0.40	0.00	0.00	0 0 0 0 0	0.56	3 3 3 3 3

(Columns DF M- ST/SE through A D M G C: OPTIONAL CALCULATION OF FACTOR LOADING TYPES)

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	POUR NO	IDENTIFICATION DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	MODULAR RATIO N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	POSITION TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	50.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	12.31
2	2	Diaphragm	0.000	0.00	0.00	0.14	31.15
3	1	Railing	0.087	0.00	50.00	0.00	0.00
4	1	Curb	0.076	0.00	50.00	0.00	0.00
5	0	Pavement	0.292	0.00	50.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi) (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	12.31	33.0	33.0	12.31
12.31	31.15	33.0	33.0	18.84
31.15	50.00	33.0	33.0	18.85

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.555       0.1249     0.6799


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      50.00

     1   DC2     0.087       0.000     50.000

     2   DC2     0.076       0.000     50.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1214.1	45.0	182.1	37.5	219.59

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.562	0.562	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.561				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.399				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.85	5900.0	32.9	16.45	358.7	358.7
1	2	7.69	5900.0	32.9	16.45	358.7	358.7
1	3	11.54	5900.0	32.9	16.45	358.7	358.7
1	4	15.38	6981.8	32.9	15.02	464.8	381.2
1	5	19.23	6981.8	32.9	15.02	464.8	381.2
1	6	23.08	6981.8	32.9	15.02	464.8	381.2
1	7	26.92	6981.8	32.9	15.02	464.8	381.2
1	8	30.77	6981.8	32.9	15.02	464.8	381.2
1	9	34.62	6981.8	32.9	15.02	464.8	381.2
1	10	38.46	5900.0	32.9	16.45	358.7	358.7
1	11	42.31	5900.0	32.9	16.45	358.7	358.7
1	12	46.15	5900.0	32.9	16.45	358.7	358.7

1	13	50.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11472.1	0.022815	479.7	1276.7	659.5
1	1	3.85	11472.1	0.022815	479.7	1276.7	659.5
1	2	7.69	11472.1	0.022815	479.7	1276.7	659.5
1	3	11.54	11472.1	0.022815	479.7	1276.7	659.5
1	4	15.38	13830.3	0.021457	613.8	1279.9	719.7
1	5	19.23	13830.3	0.021457	613.8	1279.9	719.7
1	6	23.08	13830.3	0.021457	613.8	1279.9	719.7
1	7	26.92	13830.3	0.021457	613.8	1279.9	719.7
1	8	30.77	13830.3	0.021457	613.8	1279.9	719.7
1	9	34.62	13830.3	0.021457	613.8	1279.9	719.7
1	10	38.46	11472.1	0.022815	479.7	1276.7	659.5
1	11	42.31	11472.1	0.022815	479.7	1276.7	659.5
1	12	46.15	11472.1	0.022815	479.7	1276.7	659.5
1	13	50.00	11472.1	0.022815	479.7	1276.7	659.5

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15815.0	0.028983	534.7	4762.1	1348.1
1	1	3.85	15815.0	0.028983	534.7	4762.1	1348.1
1	2	7.69	15815.0	0.028983	534.7	4762.1	1348.1
1	3	11.54	15815.0	0.028983	534.7	4762.1	1348.1
1	4	15.38	19524.5	0.027497	681.3	4170.7	1491.4
1	5	19.23	19524.5	0.027497	681.3	4170.7	1491.4
1	6	23.08	19524.5	0.027497	681.3	4170.7	1491.4
1	7	26.92	19524.5	0.027497	681.3	4170.7	1491.4
1	8	30.77	19524.5	0.027497	681.3	4170.7	1491.4
1	9	34.62	19524.5	0.027497	681.3	4170.7	1491.4
1	10	38.46	15815.0	0.028983	534.7	4762.1	1348.1
1	11	42.31	15815.0	0.028983	534.7	4762.1	1348.1
1	12	46.15	15815.0	0.028983	534.7	4762.1	1348.1
1	13	50.00	15815.0	0.028983	534.7	4762.1	1348.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.85	11.1	49.3	0.6	0.0		61.0
1	2	7.69	20.3	90.3	1.3	0.0		111.9
1	3	11.54	27.7	123.2	1.9	0.0		152.7
1	4	15.38	33.3	147.8	2.1	0.0		183.1
1	5	19.23	37.0	164.2	2.1	0.0		203.3
1	6	23.08	38.8	172.4	2.2	0.0		213.4
1	7	26.92	38.8	172.4	2.3	0.0		213.5
1	8	30.77	37.0	164.2	2.4	0.0		203.5
1	9	34.62	33.3	147.8	1.9	0.0		183.0
1	10	38.46	27.7	123.2	1.4	0.0		152.3
1	11	42.31	20.3	90.3	1.0	0.0		111.6
1	12	46.15	11.1	49.3	0.5	0.0		60.8
1	13	50.00	0.0	0.0	0.0	0.0		0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.85	25.9	14.5	0.0	0.0	0.0	40.4
1	2	7.69	47.5	26.5	0.0	0.0	0.0	74.0
1	3	11.54	64.8	36.2	0.0	0.0	0.0	101.0
1	4	15.38	77.8	43.4	0.0	0.0	0.0	121.2
1	5	19.23	86.4	48.2	0.0	0.0	0.0	134.6
1	6	23.08	90.7	50.6	0.0	0.0	0.0	141.3
1	7	26.92	90.7	50.6	0.0	0.0	0.0	141.3
1	8	30.77	86.4	48.2	0.0	0.0	0.0	134.6
1	9	34.62	77.8	43.4	0.0	0.0	0.0	121.2
1	10	38.46	64.8	36.2	0.0	0.0	0.0	101.0
1	11	42.31	47.5	26.5	0.0	0.0	0.0	74.0
1	12	46.15	25.9	14.5	0.0	0.0	0.0	40.4
1	13	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.85	0.0	0.0	184.4	HL-93	0.0	HL-93
1	2	7.69	0.0	0.0	331.5	HL-93	0.0	HL-93
1	3	11.54	0.0	0.0	439.8	HL-93	0.0	HL-93
1	4	15.38	0.0	0.0	515.4	HL-93	0.0	HL-93
1	5	19.23	0.0	0.0	563.7	HL-93	0.0	HL-93
1	6	23.08	0.0	0.0	574.9	HL-93	0.0	HL-93
1	7	26.92	0.0	0.0	574.9	HL-93	0.0	HL-93
1	8	30.77	0.0	0.0	563.7	HL-93	0.0	HL-93
1	9	34.62	0.0	0.0	515.4	HL-93	0.0	HL-93
1	10	38.46	0.0	0.0	439.8	HL-93	0.0	HL-93
1	11	42.31	0.0	0.0	331.5	HL-93	0.0	HL-93
1	12	46.15	0.0	0.0	184.4	HL-93	0.0	HL-93
1	13	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.85	65.	0.	65.
1	2	7.69	113.	0.	113.
1	3	11.54	145.	0.	145.
1	4	15.38	166.	0.	166.
1	5	19.23	181.	0.	181.
1	6	23.08	186.	0.	186.

1	7	26.92	186.	0.	186.
1	8	30.77	181.	0.	181.
1	9	34.62	166.	0.	166.
1	10	38.46	145.	0.	145.
1	11	42.31	113.	0.	113.
1	12	46.15	65.	0.	65.
1	13	50.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.85	61.0	40.4	184.4	HL-93	0.0	HL-93	285.7	101.4
1	2	7.69	111.9	74.0	331.5	HL-93	0.0	HL-93	517.5	185.9
1	3	11.54	152.7	101.0	439.8	HL-93	0.0	HL-93	693.5	253.7
1	4	15.38	183.1	121.2	515.4	HL-93	0.0	HL-93	819.7	304.3
1	5	19.23	203.3	134.6	563.7	HL-93	0.0	HL-93	901.6	337.9
1	6	23.08	213.4	141.3	574.9	HL-93	0.0	HL-93	929.7	354.8
1	7	26.92	213.5	141.3	574.9	HL-93	0.0	HL-93	929.7	354.8
1	8	30.77	203.5	134.6	563.7	HL-93	0.0	HL-93	901.9	338.1
1	9	34.62	183.0	121.2	515.4	HL-93	0.0	HL-93	819.5	304.1
1	10	38.46	152.3	101.0	439.8	HL-93	0.0	HL-93	693.1	253.3
1	11	42.31	111.6	74.0	331.5	HL-93	0.0	HL-93	517.2	185.6
1	12	46.15	60.8	40.4	184.4	HL-93	0.0	HL-93	285.6	101.2
1	13	50.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.85	285.7	341.0	455.8	382.1	152.0	48.7
1	2	7.69	517.5	616.9	824.5	691.9	278.9	84.7
1	3	11.54	693.5	825.4	1102.9	927.0	380.6	108.6
1	4	15.38	819.7	974.3	1301.7	1095.6	456.4	124.5
1	5	19.23	901.6	1070.8	1430.5	1205.0	506.9	135.9
1	6	23.08	929.7	1102.1	1472.2	1242.2	532.2	139.7
1	7	26.92	929.7	1102.2	1472.3	1242.3	532.3	139.7
1	8	30.77	901.9	1071.0	1430.8	1205.3	507.2	135.9
1	9	34.62	819.5	974.2	1301.6	1095.4	456.2	124.5
1	10	38.46	693.1	825.0	1102.4	926.5	379.9	108.6
1	11	42.31	517.2	616.6	824.1	691.5	278.5	84.7
1	12	46.15	285.6	340.9	455.6	381.9	151.8	48.7
1	13	50.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.1	13.9	0.2	0.0	17.2
1	1	3.85	2.6	11.7	0.2	0.0	14.5
1	2	7.69	2.2	9.6	0.2	0.0	11.9
1	3	11.54	1.7	7.5	0.2	0.0	9.3
1	4	15.38	1.2	5.3	0.0	0.0	6.6
1	5	19.23	0.7	3.2	0.0	0.0	3.9
1	6	23.08	0.2	1.1	0.0	0.0	1.3
1	7	26.92	-0.2	-1.1	0.0	0.0	-1.3
1	8	30.77	-0.7	-3.2	0.0	0.0	-3.9
1	9	34.62	-1.2	-5.3	-0.1	0.0	-6.7
1	10	38.46	-1.7	-7.5	-0.1	0.0	-9.3
1	11	42.31	-2.2	-9.6	-0.1	0.0	-11.9
1	12	46.15	-2.6	-11.7	-0.1	0.0	-14.5
1	13	50.00	-3.1	-13.9	-0.1	0.0	-17.1

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.3	4.1	0.0	0.0	11.4	17.2	28.5
1	1	3.85	6.2	3.4	0.0	0.0	9.6	14.5	24.2
1	2	7.69	5.1	2.8	0.0	0.0	7.9	11.9	19.8
1	3	11.54	3.9	2.2	0.0	0.0	6.1	9.3	15.4
1	4	15.38	2.8	1.6	0.0	0.0	4.4	6.6	10.9
1	5	19.23	1.7	0.9	0.0	0.0	2.6	3.9	6.6
1	6	23.08	0.6	0.3	0.0	0.0	0.9	1.3	2.2
1	7	26.92	-0.6	-0.3	0.0	0.0	-0.9	-1.3	-2.2
1	8	30.77	-1.7	-0.9	0.0	0.0	-2.6	-3.9	-6.5
1	9	34.62	-2.8	-1.6	0.0	0.0	-4.4	-6.7	-11.0
1	10	38.46	-3.9	-2.2	0.0	0.0	-6.1	-9.3	-15.4
1	11	42.31	-5.1	-2.8	0.0	0.0	-7.9	-11.9	-19.8
1	12	46.15	-6.2	-3.4	0.0	0.0	-9.6	-14.5	-24.1
1	13	50.00	-7.3	-4.1	0.0	0.0	-11.4	-17.1	-28.5

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	52.7	HL-93	0.0	HL-93
1	1	3.85	0.0	0.0	47.2	HL-93	-1.9	HL-93
1	2	7.69	0.0	0.0	41.8	HL-93	-4.5	HL-93
1	3	11.54	0.0	0.0	36.6	HL-93	-7.6	HL-93
1	4	15.38	0.0	0.0	31.4	HL-93	-10.8	HL-93
1	5	19.23	0.0	0.0	26.4	HL-93	-14.2	HL-93
1	6	23.08	0.0	0.0	21.7	HL-93	-17.6	HL-93
1	7	26.92	0.0	0.0	17.6	HL-93	-21.7	HL-93
1	8	30.77	0.0	0.0	14.2	HL-93	-26.4	HL-93
1	9	34.62	0.0	0.0	10.8	HL-93	-31.4	HL-93
1	10	38.46	0.0	0.0	7.6	HL-93	-36.6	HL-93
1	11	42.31	0.0	0.0	4.5	HL-93	-41.8	HL-93
1	12	46.15	0.0	0.0	1.9	HL-93	-47.2	HL-93
1	13	50.00	0.0	0.0	0.0	HL-93	-52.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	21.	0.	21.
1	1	3.85	18.	-1.	20.
1	2	7.69	16.	-2.	18.
1	3	11.54	14.	-3.	17.
1	4	15.38	12.	-5.	16.
1	5	19.23	10.	-6.	16.
1	6	23.08	9.	-7.	16.
1	7	26.92	7.	-9.	16.
1	8	30.77	6.	-10.	16.
1	9	34.62	5.	-12.	16.
1	10	38.46	3.	-14.	17.
1	11	42.31	2.	-16.	18.
1	12	46.15	1.	-18.	20.
1	13	50.00	0.	-21.	21.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	21.0	.30967E-01	0.32	0.2500*	.14525E-01	0.15	0.2500*
1	1	3.85	19.5	.30967E-01	0.30	0.2500*	.14525E-01	0.14	0.2500*
1	2	7.69	18.3	.30967E-01	0.28	0.2500*	.14525E-01	0.13	0.2500*
1	3	11.54	17.0	.30967E-01	0.26	0.2500*	.14525E-01	0.12	0.2500*
1	4	15.38	16.2	.29686E-01	0.24	0.2500*	.11183E-01	0.09	0.2500*

```
 1  5  19.23  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
 1  6  23.08  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
 1  7  26.92  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
 1  8  30.77  16.3 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
 1  9  34.62  16.2 .29686E-01  0.24 0.2500*.11183E-01  0.09 0.2500*
 1 10  38.46  17.0 .30967E-01  0.26 0.2500*.14525E-01  0.12 0.2500*
 1 11  42.31  18.3 .30967E-01  0.28 0.2500*.14525E-01  0.13 0.2500*
 1 12  46.15  19.5 .30967E-01  0.30 0.2500*.14525E-01  0.14 0.2500*
 1 13  50.00  21.0 .30967E-01  0.32 0.2500*.14525E-01  0.15 0.2500*


 ----------------------------------------------------------------
   NOTE:  [1] Vr = range of shear due to live loads and impact

          [2] For non-composite construction:

              Q/I = (At * Dt) / Inc -- top flange
              Inc = moment of inertia of non-composite section
              At = area of top flange
              Dt = distance between the center of top flange and neutral axis
              Q/I = (Ab * Db) / Inc -- bottom flange
              Ab = area of bottom flange
              Db = distance between the center of bottom flange and neutral axis

              For composite construction:

              Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
              Q = statical moment about the neutral axis of the
                  composite section of the transformed compressive
                  concrete area or the area of reinforcement embedded
                  in the concrete for negative moment
              Ic = moment of inertia for composite section
              Q/I = (Ab * Db) / Ic -- bottom flange

          [3] shear flow = (Vr * Q) / (2 * I)

          * -- minimum weld size governs
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 TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
               ****************************************

                 TOTAL     TOTAL L+I , LOAD TYPE = HL - 93
 SP IN D FROM ---------- ----------------------------------    MAX. SHEAR
 NO NO L SUPT DEAD LOADS POSITIVE GOVERN.  NEGATIVE GOVERN.      (kips)
        (ft)     (k)       (k)   LOAD TYPE  (k)    LOAD TYPE   POS.    NEG.
 ---------------------------------------------------------------------------

  1  0   0.00     28.5     52.7   HL-93       0.0  HL-93       81.2    28.5
  1  1   3.85     24.2     47.2   HL-93      -1.9  HL-93       71.4    22.3
  1  2   7.69     19.8     41.8   HL-93      -4.5  HL-93       61.6    15.4
  1  3  11.54     15.4     36.6   HL-93      -7.6  HL-93       52.0     7.9
  1  4  15.38     10.9     31.4   HL-93     -10.8  HL-93       42.4     0.1
  1  5  19.23      6.6     26.4   HL-93     -14.2  HL-93       33.0    -7.6
  1  6  23.08      2.2     21.7   HL-93     -17.6  HL-93       23.9   -15.4
  1  7  26.92     -2.2     17.6   HL-93     -21.7  HL-93       15.5   -23.8
  1  8  30.77     -6.5     14.2   HL-93     -26.4  HL-93        7.6   -32.9
  1  9  34.62    -11.0     10.8   HL-93     -31.4  HL-93       -0.2   -42.5
  1 10  38.46    -15.4      7.6   HL-93     -36.6  HL-93       -7.8   -52.0
  1 11  42.31    -19.8      4.5   HL-93     -41.8  HL-93      -15.3   -61.6
  1 12  46.15    -24.1      1.9   HL-93     -47.2  HL-93      -22.2   -71.3
  1 13  50.00    -28.5      0.0   HL-93     -52.7  HL-93      -28.5   -81.2


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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	81.2	97.0	129.6	108.6	42.8	15.7
1	1	3.85	71.4	85.5	114.4	95.5	36.3	14.7
1	2	7.69	61.6	74.2	99.2	82.5	29.7	13.7
1	3	11.54	52.0	63.0	84.3	69.7	23.2	12.8
1	4	15.38	42.4	51.8	69.4	56.8	16.4	12.2
1	5	19.23	33.0	40.9	54.8	44.3	9.8	12.2
1	6	23.08	23.9	30.4	40.8	32.2	3.3	12.2
1	7	26.92	-23.8	-30.4	-40.8	-32.1	-3.2	12.2
1	8	30.77	-32.9	-40.8	-54.8	-44.2	-9.8	12.2
1	9	34.62	-42.5	-51.9	-69.5	-56.9	-16.6	12.2
1	10	38.46	-52.0	-63.0	-84.3	-69.6	-23.1	12.8
1	11	42.31	-61.6	-74.2	-99.2	-82.5	-29.7	13.7
1	12	46.15	-71.3	-85.5	-114.3	-95.4	-36.2	14.7
1	13	50.00	-81.2	-97.0	-129.6	-108.5	-42.7	15.7

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE		TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM
1	28.54	0.00	HL-93	52.66	HL-93	ST1	23.86	129.65
						ST2	23.86	108.58
						ST4	31.85	42.80
						SE1	28.54	81.19
						SE2	28.54	96.99
2	28.50	0.00	HL-93	52.66	HL-93	ST1	23.82	129.60
						ST2	23.82	108.54
						ST4	31.80	42.75
						SE1	28.50	81.16
						SE2	28.50	96.95

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.85	-0.0225	-0.0998	0.0000	-0.0416	-0.1237	-0.0416	-0.1653
1	2	7.69	-0.0433	-0.1924	0.0000	-0.0802	-0.2384	-0.0802	-0.3185

1	3	11.54	-0.0611	-0.2716	0.0000	-0.1131	-0.3364	-0.1131	-0.4496
1	4	15.38	-0.0749	-0.3329	0.0000	-0.1385	-0.4123	-0.1385	-0.5508
1	5	19.23	-0.0845	-0.3753	0.0000	-0.1560	-0.4647	-0.1560	-0.6207
1	6	23.08	-0.0893	-0.3969	0.0000	-0.1650	-0.4915	-0.1650	-0.6565
1	7	26.92	-0.0893	-0.3969	0.0000	-0.1650	-0.4915	-0.1650	-0.6565
1	8	30.77	-0.0845	-0.3753	0.0000	-0.1560	-0.4647	-0.1560	-0.6207
1	9	34.62	-0.0749	-0.3329	0.0000	-0.1385	-0.4122	-0.1385	-0.5507
1	10	38.46	-0.0611	-0.2716	0.0000	-0.1131	-0.3363	-0.1131	-0.4494
1	11	42.31	-0.0433	-0.1924	0.0000	-0.0802	-0.2382	-0.0802	-0.3184
1	12	46.15	-0.0225	-0.0998	0.0000	-0.0416	-0.1236	-0.0416	-0.1652
1	13	50.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.8	-0.022	0 1/16	-0.101	0 1/ 8	-0.042	0 1/16	-0.165	0 3/16
1	2	7.7	-0.043	0 1/16	-0.195	0 1/ 4	-0.080	0 1/ 8	-0.319	0 3/ 8
1	3	11.5	-0.061	0 1/16	-0.275	0 5/16	-0.113	0 1/ 8	-0.450	0 1/ 2
1	4	15.4	-0.075	0 1/ 8	-0.337	0 3/ 8	-0.138	0 3/16	-0.551	0 9/16
1	5	19.2	-0.084	0 1/ 8	-0.380	0 7/16	-0.156	0 3/16	-0.621	0 5/ 8
1	6	23.1	-0.089	0 1/ 8	-0.402	0 7/16	-0.165	0 3/16	-0.657	0 11/16
1	7	26.9	-0.089	0 1/ 8	-0.402	0 7/16	-0.165	0 3/16	-0.657	0 11/16
1	8	30.8	-0.084	0 1/ 8	-0.380	0 7/16	-0.156	0 3/16	-0.621	0 5/ 8
1	9	34.6	-0.075	0 1/ 8	-0.337	0 3/ 8	-0.138	0 3/16	-0.551	0 9/16
1	10	38.5	-0.061	0 1/16	-0.275	0 5/16	-0.113	0 1/ 8	-0.449	0 1/ 2
1	11	42.3	-0.043	0 1/16	-0.195	0 1/ 4	-0.080	0 1/ 8	-0.318	0 3/ 8
1	12	46.2	-0.022	0 1/16	-0.101	0 1/ 8	-0.042	0 1/16	-0.165	0 3/16
1	13	50.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.00	3 0.425	-0.279	MAX HL-93	0.75	0.00270
			0.029	MIN		
			-0.071	MAX LANE		
			0.029	MIN LANE		
		4 0.433	-0.284	MAX HL-93	0.75	0.00270

```
                              5969 Span 2-S15 Exterior Fatigue.RES
                                    0.029 MIN
                                   -0.072 MAX LANE
                                    0.029 MIN LANE

                    3    0.425     -0.173 MAX FA
                                    0.000 MIN FA

    ---------------------------------------------------------------------
    NOTE: [1] " - " indicates downward deflection

          [2] The distribution factor for LL+I deflection is defined as

              DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

              where NL= no. of traffic lanes
                    Ng= no. of girders
                    (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

          [3] This table is based upon the optional criteria specified
              in AASHTO LRFD Art. 3.6.1.3.2

          [4] The number of traffic lanes is determined according to
              AASHTO LRFD Art.3.6.1.1.1.
              The 1st line is for the most probable number of lanes
              and the 2nd line is for the next probable number of lanes.

          [5] Max rotations at left (1st line) & right (2nd line)
              supports of the span without averaging, factor and impact

          [6] If ADTT is between 100 and 1000, multi-presence
              factor of 0.95 is applied. If ADTT is below 100, factor
              is 0.9 (AASHTO LRFD C3.6.1.1.2).

          [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
              ********************************************

              STEEL DEAD LOAD    OTHER DEAD LOAD   TOTAL DEAD LOAD
 SP IN D FROM ----------------  ----------------  ----------------
 NO NO L SUPT    STEEL BEAM        STEEL BEAM        STEEL BEAM
               TOP      BOT      TOP      BOT      TOP      BOT
        (ft)       (ksi)             (ksi)             (ksi)
 ---------------------------------------------------------------

  1  0   0.00    0.00    0.00      0.00    0.00      0.00    0.00
  1  1   3.85   -0.37    0.37     -1.67    1.67     -2.04    2.04
  1  2   7.69   -0.68    0.68     -3.06    3.06     -3.74    3.74
  1  3  11.54   -0.93    0.93     -4.18    4.18     -5.11    5.11
  1  4  15.38   -1.05    0.86     -4.72    3.87     -5.76    4.73
  1  5  19.23   -1.16    0.95     -5.24    4.29     -6.40    5.25
  1  6  23.08   -1.22    1.00     -5.50    4.51     -6.72    5.51
  1  7  26.92   -1.22    1.00     -5.50    4.51     -6.72    5.51
  1  8  30.77   -1.16    0.95     -5.24    4.30     -6.41    5.25
  1  9  34.62   -1.05    0.86     -4.71    3.87     -5.76    4.72
  1 10  38.46   -0.93    0.93     -4.17    4.17     -5.10    5.10
  1 11  42.31   -0.68    0.68     -3.05    3.05     -3.73    3.73
  1 12  46.15   -0.37    0.37     -1.66    1.66     -2.04    2.04
  1 13  50.00    0.00    0.00      0.00    0.00      0.00    0.00


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 TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
              ****************************************************
```

(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	-0.03	-0.38	1.01	-0.03	-2.42	3.05
1	2	7.69	-0.05	-0.70	1.85	-0.05	-4.44	5.60
1	3	11.54	-0.07	-0.95	2.53	-0.07	-6.06	7.64
1	4	15.38	-0.07	-1.14	2.37	-0.07	-6.90	7.10
1	5	19.23	-0.08	-1.26	2.63	-0.08	-7.66	7.88
1	6	23.08	-0.09	-1.33	2.76	-0.09	-8.04	8.27
1	7	26.92	-0.09	-1.33	2.76	-0.09	-8.05	8.28
1	8	30.77	-0.08	-1.26	2.63	-0.08	-7.67	7.89
1	9	34.62	-0.07	-1.14	2.37	-0.07	-6.90	7.09
1	10	38.46	-0.07	-0.95	2.53	-0.07	-6.05	7.62
1	11	42.31	-0.05	-0.70	1.85	-0.05	-4.43	5.59
1	12	46.15	-0.03	-0.38	1.01	-0.03	-2.41	3.05
1	13	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	-0.18	-0.46	4.14	0.00	0.00	0.00
1	2	7.69	-0.33	-0.84	7.44	0.00	0.00	0.00
1	3	11.54	-0.43	-1.11	9.87	0.00	0.00	0.00
1	4	15.38	-0.46	-1.48	9.08	0.00	0.00	0.00
1	5	19.23	-0.50	-1.62	9.93	0.00	0.00	0.00
1	6	23.08	-0.51	-1.65	10.13	0.00	0.00	0.00
1	7	26.92	-0.51	-1.65	10.13	0.00	0.00	0.00
1	8	30.77	-0.50	-1.62	9.93	0.00	0.00	0.00
1	9	34.62	-0.46	-1.48	9.08	0.00	0.00	0.00
1	10	38.46	-0.43	-1.11	9.87	0.00	0.00	0.00
1	11	42.31	-0.33	-0.84	7.44	0.00	0.00	0.00
1	12	46.15	-0.18	-0.46	4.14	0.00	0.00	0.00
1	13	50.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK ONLY	GOVERNING STRESS RANGE	LOAD TYPE	BOT OF BOT FLANGE TRUCK ONLY	GOVERNING STRESS RANGE	LOAD TYPE
1	0	0.00	0.0	0.0	0.0 T HL	0.0	0.0	0.0 T HL
1	1	3.85	0.2	0.0	0.2 C HL	1.5	0.0	1.5 T HL

1	2	7.69	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T HL
1	3	11.54	0.4	0.0	0.4	C	HL	3.2	0.0	3.2	T HL
1	4	15.38	0.5	0.0	0.5	C	HL	2.9	0.0	2.9	T HL
1	5	19.23	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T HL
1	6	23.08	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T HL
1	7	26.92	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T HL
1	8	30.77	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T HL
1	9	34.62	0.5	0.0	0.5	C	HL	2.9	0.0	2.9	T HL
1	10	38.46	0.4	0.0	0.4	C	HL	3.2	0.0	3.2	T HL
1	11	42.31	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T HL
1	12	46.15	0.2	0.0	0.2	C	HL	1.5	0.0	1.5	T HL
1	13	50.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.21	-2.88	7.19	-0.03	-2.42	3.05
1	2	7.69	33.	33.	-0.38	-5.27	13.04	-0.05	-4.44	5.60
1	3	11.54	33.	33.	-0.50	-7.17	17.51	-0.07	-6.06	7.64
1	4	15.38	33.	33.	-0.54	-8.38	16.17	-0.07	-6.90	7.10
1	5	19.23	33.	33.	-0.59	-9.28	17.81	-0.08	-7.66	7.88
1	6	23.08	33.	33.	-0.60	-9.70	18.40	-0.09	-8.04	8.27
1	7	26.92	33.	33.	-0.60	-9.70	18.40	-0.09	-8.05	8.28
1	8	30.77	33.	33.	-0.59	-9.29	17.81	-0.08	-7.67	7.89
1	9	34.62	33.	33.	-0.54	-8.38	16.17	-0.07	-6.90	7.09
1	10	38.46	33.	33.	-0.50	-7.15	17.49	-0.07	-6.05	7.62
1	11	42.31	33.	33.	-0.38	-5.27	13.03	-0.05	-4.43	5.59
1	12	46.15	33.	33.	-0.21	-2.88	7.18	-0.03	-2.41	3.05
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.26	-3.02	8.43	-0.03	-2.42	3.05
1	2	7.69	33.	33.	-0.48	-5.53	15.27	-0.05	-4.44	5.60
1	3	11.54	33.	33.	-0.63	-7.50	20.47	-0.07	-6.06	7.64
1	4	15.38	33.	33.	-0.67	-8.83	18.90	-0.07	-6.90	7.10
1	5	19.23	33.	33.	-0.74	-9.77	20.79	-0.08	-7.66	7.88
1	6	23.08	33.	33.	-0.76	-10.19	21.44	-0.09	-8.04	8.27
1	7	26.92	33.	33.	-0.76	-10.20	21.44	-0.09	-8.05	8.28

1	8	30.77	33.	33.	-0.74	-9.78	20.79	-0.08	-7.67	7.89
1	9	34.62	33.	33.	-0.67	-8.82	18.89	-0.07	-6.90	7.09
1	10	38.46	33.	33.	-0.63	-7.49	20.45	-0.07	-6.05	7.62
1	11	42.31	33.	33.	-0.48	-5.52	15.26	-0.05	-4.43	5.59
1	12	46.15	33.	33.	-0.26	-3.02	8.42	-0.03	-2.41	3.05
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.36	-3.90	11.22	-0.04	-3.09	3.97
1	2	7.69	33.	33.	-0.64	-7.12	20.31	-0.07	-5.66	7.29
1	3	11.54	33.	33.	-0.86	-9.67	27.22	-0.10	-7.73	9.95
1	4	15.38	33.	33.	-0.91	-11.40	25.14	-0.11	-8.81	9.25
1	5	19.23	33.	33.	-1.00	-12.62	27.65	-0.12	-9.78	10.27
1	6	23.08	33.	33.	-1.02	-13.16	28.50	-0.12	-10.27	10.79
1	7	26.92	33.	33.	-1.02	-13.17	28.51	-0.12	-10.27	10.79
1	8	30.77	33.	33.	-1.00	-12.63	27.65	-0.12	-9.79	10.28
1	9	34.62	33.	33.	-0.91	-11.40	25.13	-0.11	-8.80	9.25
1	10	38.46	33.	33.	-0.86	-9.65	27.21	-0.10	-7.71	9.93
1	11	42.31	33.	33.	-0.64	-7.11	20.30	-0.07	-5.65	7.28
1	12	46.15	33.	33.	-0.36	-3.89	11.21	-0.04	-3.08	3.97
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.34	-2.93	9.82	-0.02	-2.12	2.58
1	2	7.69	33.	33.	-0.61	-5.35	17.76	-0.04	-3.88	4.74
1	3	11.54	33.	33.	-0.81	-7.24	23.74	-0.05	-5.30	6.47
1	4	15.38	33.	33.	-0.86	-8.62	21.89	-0.06	-6.03	6.01
1	5	19.23	33.	33.	-0.94	-9.53	24.05	-0.06	-6.69	6.67
1	6	23.08	33.	33.	-0.96	-9.92	24.72	-0.06	-7.03	7.00
1	7	26.92	33.	33.	-0.96	-9.92	24.72	-0.06	-7.03	7.00
1	8	30.77	33.	33.	-0.94	-9.54	24.05	-0.06	-6.70	6.68
1	9	34.62	33.	33.	-0.86	-8.62	21.89	-0.06	-6.02	6.00
1	10	38.46	33.	33.	-0.81	-7.23	23.73	-0.05	-5.29	6.45
1	11	42.31	33.	33.	-0.61	-5.34	17.75	-0.04	-3.88	4.73
1	12	46.15	33.	33.	-0.34	-2.93	9.82	-0.02	-2.11	2.58
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.28	-3.71	9.56	-0.04	-3.09	3.97
1	2	7.69	33.	33.	-0.51	-6.79	17.34	-0.07	-5.66	7.29
1	3	11.54	33.	33.	-0.68	-9.22	23.28	-0.10	-7.73	9.95
1	4	15.38	33.	33.	-0.73	-10.81	21.50	-0.11	-8.81	9.25
1	5	19.23	33.	33.	-0.80	-11.97	23.68	-0.12	-9.78	10.27
1	6	23.08	33.	33.	-0.82	-12.50	24.45	-0.12	-10.27	10.79
1	7	26.92	33.	33.	-0.82	-12.50	24.46	-0.12	-10.27	10.79
1	8	30.77	33.	33.	-0.80	-11.98	23.68	-0.12	-9.79	10.28
1	9	34.62	33.	33.	-0.73	-10.80	21.50	-0.11	-8.80	9.25
1	10	38.46	33.	33.	-0.68	-9.20	23.26	-0.10	-7.71	9.93
1	11	42.31	33.	33.	-0.51	-6.78	17.32	-0.07	-5.65	7.28
1	12	46.15	33.	33.	-0.28	-3.71	9.55	-0.04	-3.08	3.97
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.85	33.	33.	-0.27	-2.74	8.17	-0.02	-2.12	2.58
1	2	7.69	33.	33.	-0.48	-5.01	14.78	-0.04	-3.88	4.74
1	3	11.54	33.	33.	-0.64	-6.80	19.79	-0.05	-5.30	6.47
1	4	15.38	33.	33.	-0.68	-8.03	18.26	-0.06	-6.03	6.01
1	5	19.23	33.	33.	-0.74	-8.88	20.07	-0.06	-6.69	6.67
1	6	23.08	33.	33.	-0.76	-9.26	20.67	-0.06	-7.03	7.00
1	7	26.92	33.	33.	-0.76	-9.26	20.67	-0.06	-7.03	7.00
1	8	30.77	33.	33.	-0.74	-8.89	20.08	-0.06	-6.70	6.68
1	9	34.62	33.	33.	-0.68	-8.03	18.26	-0.06	-6.02	6.00
1	10	38.46	33.	33.	-0.64	-6.78	19.78	-0.05	-5.29	6.45
1	11	42.31	33.	33.	-0.48	-5.00	14.78	-0.04	-3.88	4.73
1	12	46.15	33.	33.	-0.27	-2.74	8.16	-0.02	-2.11	2.58
1	13	50.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	13.50	168.	-5.6	111.	-1.0	478.	-1.2	0.	0.0	-7.88

```
                                                                                  1.21
        LB                  5.6          2.8         10.7          0.0   19.15
                                                                                 10.74
        RT                 -5.3         -1.0         -1.4          0.0   -7.72
                                                                                  1.38
        RB                  4.3          2.2          8.4          0.0   14.94
                                                                                  8.42
    1   LT   36.50   168.  -5.3   111.  -1.0   478.  -1.4     0.   0.0   -7.71
                                                                                  1.38
        LB                  4.3          2.2          8.4          0.0   14.94
                                                                                  8.42
        RT                 -5.6         -1.0         -1.2          0.0   -7.87
                                                                                  1.21
        RB                  5.6          2.8         10.7          0.0   19.14
                                                                                 10.74
    ------------------------------------------------------------------------------
    NOTE:  LT = left-hand-side point, top of top flange
           LB = left-hand-side point, bottom of bottom flange
           RT = right-hand-side point, top of top flange
           RB = right-hand-side point, bottom of bottom flange
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 TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
               **********************************

                                                          COVER PLATE -- (in)
  MEMBER          TOP FLANGE     WEB       BOT FLANGE  -------------------------
 --------- STEEL ----------- ------------ ------------     TOP          BOT
 NO LENGTH  TYPE WIDTH THICK DEPTH THICK  WIDTH THICK  -----------  ------------
     (ft)           (in)        (in)          (in)     WIDTH THICK  WIDTH THICK
 ------------------------------------------------------------------------------

  1  13.50    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

  2  23.00    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 10.00 0.4375

  3  13.50    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

 ------------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
               ************

  ** [1] = Span length (ft) =   50.0
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ------------------------------------------------------------------------

  1  13.50  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

  2  23.00  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

  3  13.50  6.75   32.90 [2]=   41.31     0.06885      >   0.04000    OK

                         [3]=   32.90     0.05483      >   0.03300    OK

 ------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths
```

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth
of girder (concrete slab plus haunch & girder) preferably should not
be less than 0.040*Length of span (or 0.032*L if continuous spans).
and the depth of steel girder alone preferably should not be
less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability
during construction and the limitation for the live load
deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0		0.00	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	1		3.85	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	2		7.69	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	3		11.54	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	4		15.38	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	5		19.23	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	6		23.08	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	7		26.92	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	8		30.77	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	9		34.62	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	10		38.46	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	11		42.31	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	12		46.15	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0
1	13		50.00	0	1	31.42	0.550	57.1	150.0		0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2
to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category

1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	3.85	0	1	31.42	0.550	57.13	168.97	2
1	2	7.69	0	1	31.42	0.550	57.13	168.97	2
1	3	11.54	0	1	31.42	0.550	57.13	168.97	2
1	4	15.38	0	1	31.42	0.550	63.98	168.97	2
1	5	19.23	0	1	31.42	0.550	63.98	168.97	2
1	6	23.08	0	1	31.42	0.550	63.98	168.97	2
1	7	26.92	0	1	31.42	0.550	63.98	168.97	2
1	8	30.77	0	1	31.42	0.550	63.98	168.97	2
1	9	34.62	0	1	31.42	0.550	63.98	168.97	2
1	10	38.46	0	1	31.42	0.550	57.13	168.97	2
1	11	42.31	0	1	31.42	0.550	57.13	168.97	2
1	12	46.15	0	1	31.42	0.550	57.13	168.97	2
1	13	50.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf	[1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	1	3.85	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	2	7.69	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	3	11.54	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	4	15.38	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	5	19.23	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	6	23.08	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	7	26.92	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	8	30.77	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	9	34.62	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	10	38.46	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	11	42.31	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	12	46.15	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	13	50.00	7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	12.3	0.000	7.863	4.594	1.325	1.582
			0.000	0.000	0.000	0.000	1.000
2	12.3	31.1	7.863	9.514	9.911	10.308	1.000
			0.000	0.000	0.000	0.000	1.000
3	31.1	50.0	0.000	9.514	6.521	3.528	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0		0.00	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	1		3.85	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	2		7.69	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	3		11.54	1	7.77	11.26	19.84	0	12.31	7.09	26.62	2	2
1	4		15.38	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	5		19.23	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	6		23.08	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	7		26.92	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	8		30.77	1	7.77	11.26	19.84	0	18.84	6.98	26.22	2	2
1	9		34.62	1	7.77	11.26	19.84	0	18.85	6.98	26.22	2	2
1	10		38.46	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	11		42.31	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	12		46.15	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2
1	13		50.00	1	7.77	11.26	19.84	0	18.85	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange

residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.58	33.00	33.00	33.00
1	1		3.85	1	1.000	1.000	1.58	33.00	33.00	33.00
1	2		7.69	1	1.000	1.000	1.58	33.00	33.00	33.00
1	3		11.54	1	1.000	1.000	1.58	33.00	33.00	33.00
1	4		15.38	1	1.000	1.000	1.00	33.00	26.90	26.90
1	5		19.23	1	1.000	1.000	1.00	33.00	26.90	26.90
1	6		23.08	1	1.000	1.000	1.00	33.00	26.90	26.90
1	7		26.92	1	1.000	1.000	1.00	33.00	26.90	26.90
1	8		30.77	1	1.000	1.000	1.00	33.00	26.90	26.90
1	9		34.62	1	1.000	1.000	1.00	33.00	26.89	26.89
1	10		38.46	1	1.000	1.000	1.00	33.00	27.04	27.04
1	11		42.31	1	1.000	1.000	1.00	33.00	27.04	27.04
1	12		46.15	1	1.000	1.000	1.00	33.00	27.04	27.04
1	13		50.00	1	1.000	1.000	1.00	33.00	27.04	27.04

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	6.34	17.35
1	1		3.85	1	6.34	17.35
1	2		7.69	1	6.34	17.35
1	3		11.54	1	6.34	17.35
1	4		15.38	1	8.50	17.53
1	5		19.23	1	8.50	17.53
1	6		23.08	1	8.50	17.53
1	7		26.92	1	8.50	17.53
1	8		30.77	1	8.50	17.53
1	9		34.62	1	8.50	17.53
1	10		38.46	1	6.34	17.35
1	11		42.31	1	6.34	17.35
1	12		46.15	1	6.34	17.35
1	13		50.00	1	6.34	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.85	1	0	2	0		2	0
1	2	7.69	1	0	2	0		2	0
1	3	11.54	1	0	2	0		2	0
1	4	15.38	1	0	2	0		2	0
1	5	19.23	1	0	2	0		2	0
1	6	23.08	1	0	2	0		2	0
1	7	26.92	1	0	2	0		2	0
1	8	30.77	1	0	2	0		2	0
1	9	34.62	1	0	2	0		2	0
1	10	38.46	1	0	2	0		2	0
1	11	42.31	1	0	2	0		2	0
1	12	46.15	1	0	2	0		2	0
1	13	50.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1]	fbu+fl (ksi)	[2]	fbu+1/3fl (ksi)	[3]	FLAG (ksi)

1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	3.85	0.0	19.8	3.1	33.0	3.1	33.0	3.1	33.0	0
			0.0	-	3.1	-	3.1	33.0	-	-	0
1	2	7.69	0.0	19.8	5.6	33.0	5.6	33.0	5.6	33.0	0
			0.0	-	5.6	-	5.6	33.0	-	-	0
1	3	11.54	0.0	19.8	7.7	33.0	7.7	33.0	7.7	33.0	0
			0.0	-	7.7	-	7.7	33.0	-	-	0
1	4	15.38	0.0	19.8	8.6	33.0	8.6	33.0	8.6	26.9	0
			0.0	-	7.1	-	7.1	33.0	-	-	0
1	5	19.23	0.0	19.8	9.6	33.0	9.6	33.0	9.6	26.9	0
			0.0	-	7.9	-	7.9	33.0	-	-	0
1	6	23.08	0.0	19.8	10.1	33.0	10.1	33.0	10.1	26.9	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	7	26.92	0.0	19.8	10.1	33.0	10.1	33.0	10.1	26.9	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	8	30.77	0.0	19.8	9.6	33.0	9.6	33.0	9.6	26.9	0
			0.0	-	7.9	-	7.9	33.0	-	-	0
1	9	34.62	0.0	19.8	8.6	33.0	8.6	33.0	8.6	26.9	0
			0.0	-	7.1	-	7.1	33.0	-	-	0
1	10	38.46	0.0	19.8	7.6	33.0	7.6	33.0	7.6	27.0	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	11	42.31	0.0	19.8	5.6	33.0	5.6	33.0	5.6	27.0	0
			0.0	-	5.6	-	5.6	33.0	-	-	0
1	12	46.15	0.0	19.8	3.1	33.0	3.1	33.0	3.1	27.0	0
			0.0	-	3.1	-	3.1	33.0	-	-	0
1	13	50.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.85	0.000	0.093	0.093	0.093	0.093	2
					0.093		0.093	3
1	2	7.69	0.000	0.170	0.170	0.170	0.170	2
					0.170		0.170	3
1	3	11.54	0.000	0.232	0.232	0.232	0.232	2
					0.232		0.232	3
1	4	15.38	0.000	0.262	0.262	0.321	0.321	4
					0.215		0.215	3
1	5	19.23	0.000	0.291	0.291	0.357	0.357	4
					0.239		0.239	3
1	6	23.08	0.000	0.305	0.305	0.375	0.375	4

					0.250		0.250	3
1	7	26.92	0.000	0.306	0.306	0.375	0.375	4
					0.251		0.251	3
1	8	30.77	0.000	0.291	0.291	0.357	0.357	4
					0.239		0.239	3
1	9	34.62	0.000	0.262	0.262	0.321	0.321	4
					0.215		0.215	3
1	10	38.46	0.000	0.232	0.232	0.283	0.283	4
					0.232		0.232	3
1	11	42.31	0.000	0.170	0.170	0.207	0.207	4
					0.170		0.170	3
1	12	46.15	0.000	0.093	0.093	0.113	0.113	4
					0.093		0.093	3
1	13	50.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0
1	1		3.85	0	455.8	1969.3	-	-	0.0	19.8	-	-	0--0
1	2		7.69	0	824.5	1969.3	-	-	0.0	19.8	-	-	0--0
1	3		11.54	0	1102.9	1969.3	-	-	0.0	19.8	-	-	0--0
1	4		15.38	0	1301.7	2282.6	-	-	0.0	19.8	-	-	0--0
1	5		19.23	0	1430.5	2282.6	-	-	0.0	19.8	-	-	0--0
1	6		23.08	0	1472.2	2282.6	-	-	0.0	19.8	-	-	0--0
1	7		26.92	0	1472.3	2282.6	-	-	0.0	19.8	-	-	0--0
1	8		30.77	0	1430.8	2282.6	-	-	0.0	19.8	-	-	0--0
1	9		34.62	0	1301.6	2282.6	-	-	0.0	19.8	-	-	0--0
1	10		38.46	0	1102.4	1969.3	-	-	0.0	19.8	-	-	0--0
1	11		42.31	0	824.1	1969.3	-	-	0.0	19.8	-	-	0--0
1	12		46.15	0	455.6	1969.3	-	-	0.0	19.8	-	-	0--0
1	13		50.00	0	0.0	1969.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange
of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections
in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in
positive moment region, the 1st line is for top
flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000			0.000	1
1	1	3.85	0	0.231			0.231	1
1	2	7.69	0	0.419			0.419	1
1	3	11.54	0	0.560			0.560	1
1	4	15.38	0	0.570			0.570	1
1	5	19.23	0	0.627			0.627	1
1	6	23.08	0	0.645			0.645	1
1	7	26.92	0	0.645			0.645	1
1	8	30.77	0	0.627			0.627	1
1	9	34.62	0	0.570			0.570	1
1	10	38.46	0	0.560			0.560	1
1	11	42.31	0	0.418			0.418	1
1	12	46.15	0	0.231			0.231	1
1	13	50.00	0	0.000			0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.85	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	7.69	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	11.54	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	15.38	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	19.23	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	23.08	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	26.92	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	30.77	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	34.62	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	38.46	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	42.31	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	46.15	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	50.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP	IN	D FROM	WEB STABILITY	LFD/LRFD	UNSTIFFENED	OVERALL

5969 Span 2-S15 Exterior Fatigue.RES

NO	NO	L (ft)	SUPT UNSTIFFENED, ---- D/tw -----	[1] MAX. SHEAR (kip)	<=>	SHEAR CAPA-CITY,	CATEGORY [2]	WEB
1	0	0.00	0	129.6	<	330.8	0	0
1	1	3.85	0	114.4	<	330.8	0	0
1	2	7.69	0	99.2	<	330.8	0	0
1	3	11.54	0	84.3	<	330.8	0	0
1	4	15.38	0	69.4	<	330.8	0	0
1	5	19.23	0	54.8	<	330.8	0	0
1	6	23.08	0	40.8	<	330.8	0	0
1	7	26.92	0	40.8	<	330.8	0	0
1	8	30.77	0	54.8	<	330.8	0	0
1	9	34.62	0	69.5	<	330.8	0	0
1	10	38.46	0	84.3	<	330.8	0	0
1	11	42.31	0	99.2	<	330.8	0	0
1	12	46.15	0	114.3	<	330.8	0	0
1	13	50.00	0	129.6	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	129.65	330.76	0	
1	1	3.85	33.	114.35	330.76	0	
1	2	7.69	33.	99.25	330.76	0	
1	3	11.54	33.	84.33	330.76	0	
1	4	15.38	33.	69.39	330.76	0	
1	5	19.23	33.	54.82	330.76	0	
1	6	23.08	33.	40.84	330.76	0	
1	7	26.92	33.	40.79	330.76	0	
1	8	30.77	33.	54.77	330.76	0	
1	9	34.62	33.	69.52	330.76	0	
1	10	38.46	33.	84.28	330.76	0	
1	11	42.31	33.	99.20	330.76	0	
1	12	46.15	33.	114.31	330.76	0	
1	13	50.00	33.	129.60	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	BOT. FLANGE [1] ff+fl/2 (ksi)	BOT. FLANGE [2] (ksi)	COMP. FLG fc (ksi)	COMP. FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	3.85	3.0	-	31.4	8.4	31.4	-	-	00-
1	2	7.69	5.5	-	31.4	15.3	31.4	-	-	00-
1	3	11.54	7.5	-	31.4	20.5	31.4	-	-	00-
1	4	15.38	8.8	-	31.4	18.9	31.4	-	-	00-
1	5	19.23	9.8	-	31.4	20.8	31.4	-	-	00-
1	6	23.08	10.2	-	31.4	21.4	31.4	-	-	00-
1	7	26.92	10.2	-	31.4	21.4	31.4	-	-	00-
1	8	30.77	9.8	-	31.4	20.8	31.4	-	-	00-
1	9	34.62	8.8	-	31.4	18.9	31.4	-	-	00-
1	10	38.46	7.5	-	31.4	20.5	31.4	-	-	00-
1	11	42.31	5.5	-	31.4	15.3	31.4	-	-	00-
1	12	46.15	3.0	-	31.4	8.4	31.4	-	-	00-
1	13	50.00	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.85	TR	-0.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	2	7.69	TR	-0.4	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	3	11.54	TR	-0.5	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1		13.50	SCG					
1	4	15.38	TR	-0.7	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	5	19.23	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	6	23.08	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	7	26.92	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	8	30.77	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9	34.62	TR	-0.7	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1		36.50	SCG					
1	10	38.46	TR	-0.5	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	11	42.31	TR	-0.4	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	12	46.15	TR	-0.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	13	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.85	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2	7.69	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	3	11.54	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1		13.50	SCG					
1	4	15.38	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	5	19.23	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	6	23.08	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	7	26.92	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	8	30.77	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	9	34.62	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1		36.50	SCG					
1	10	38.46	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	11	42.31	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	12	46.15	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	13	50.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	31.5	.28983E-01	0.9	TR 3.09	3	10.17
1	1	3.85	29.3	.28983E-01	0.8	TR 3.09	3	10.92
1	2	7.69	27.5	.28983E-01	0.8	TR 3.09	3	11.66
1	3	11.54	25.5	.28983E-01	0.7	TR 3.09	3	12.54
1	4	15.38	24.3	.27497E-01	0.7	TR 3.09	3	13.87
1	5	19.23	24.4	.27497E-01	0.7	TR 3.09	3	13.82
1	6	23.08	24.4	.27497E-01	0.7	TR 3.09	3	13.82
1	7	26.92	24.4	.27497E-01	0.7	TR 3.09	3	13.82
1	8	30.77	24.4	.27497E-01	0.7	TR 3.09	3	13.82
1	9	34.62	24.3	.27497E-01	0.7	TR 3.09	3	13.87
1	10	38.46	25.5	.28983E-01	0.7	TR 3.09	3	12.54
1	11	42.31	27.5	.28983E-01	0.8	TR 3.09	3	11.66
1	12	46.15	29.3	.28983E-01	0.8	TR 3.09	3	10.92
1	13	50.00	31.5	.28983E-01	0.9	TR 3.09	3	10.17

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	15.7	.28983E-01	0.5	TR 1.36	3	8.97
1	1	3.85	14.7	.28983E-01	0.4	TR 1.36	3	9.63
1	2	7.69	13.7	.28983E-01	0.4	TR 1.36	3	10.28
1	3	11.54	12.8	.28983E-01	0.4	TR 1.36	3	11.06
1	4	15.38	12.2	.27497E-01	0.3	TR 1.36	3	12.23
1	5	19.23	12.2	.27497E-01	0.3	TR 1.36	3	12.19
1	6	23.08	12.2	.27497E-01	0.3	TR 1.36	3	12.18
1	7	26.92	12.2	.27497E-01	0.3	TR 1.36	3	12.18
1	8	30.77	12.2	.27497E-01	0.3	TR 1.36	3	12.19
1	9	34.62	12.2	.27497E-01	0.3	TR 1.36	3	12.23
1	10	38.46	12.8	.28983E-01	0.4	TR 1.36	3	11.06
1	11	42.31	13.7	.28983E-01	0.4	TR 1.36	3	10.28
1	12	46.15	14.7	.28983E-01	0.4	TR 1.36	3	9.63
1	13	50.00	15.7	.28983E-01	0.5	TR 1.36	3	8.97

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN D NO L (ft)	FROM SUPT 1=POS. 0=NEG.	MOMENT REGION	P2p,1n (k)	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	N2	STATUS BLANK=OK CHECK=**
1	7	26.92	1	1276.3	1204.9		99	>	63		
1	13	50.00	1	1276.3	1204.9		99	>	63		

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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
             Qr = factored shear resistance of one shear connector
                = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
             Qn = nominal shear resistance
             (Phi)sc = resistance factor for shear connectors

      [3] The total horizontal shear force between the point of
           maximum positive moment and each adjacent point of 0.0 moment
           shall be the lesser either:

                P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
           or
             P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
           Where
             f'c = specified 28-day compressive strength of the concrete
             bs = effective width of the slab
             bfc = width of compression flange
             bft = width of tension flange
             ts = slab thickness
             Fyw = specified minimum yield strength of the web
             Fyt = specified minimum yield strength of the tension flange
             Fyc = specified minimum yield strength of the compression flange
             D = web depth
             tft = thickness of tension flange
             tfc = thickness of compression flange
             tw = web thickness


      [4] For continuous-span composite sections, the total horizontal
           shear force between each adjacent point of 0.0 moment
           and the centerline of an interior support shall be taken as:

                P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
           Where
             bs = effective width of the concrete deck
             ts = thickness of a concrete deck

      [5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
          If ADTT Single Lane is less than 960, the pitch should use Fatigue II.
1
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 TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
                 ****************************************

                SPAN     CURRENT SPAN     REQUIRED
```

NO.	FROM (ft)	TO (ft)	PITCH (in)
1	0.000	26.923	10.000
1	26.923	50.000	10.000

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	50.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

UNFACTORED UNFACTORED L+I

5969 Span 2-S15 Exterior Fatigue.RES

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	DEAD LOAD MOMENT D1; k-ft	D2 ksi	MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.85	1969.3	61.0	40.4	184.4	0.0	7.38	HL-93	5.69	0
1	2	7.69	1969.3	111.9	74.0	331.5	0.0	3.85	HL-93	2.97	0
1	3	11.54	1969.3	152.7	101.0	439.8	0.0	2.76	HL-93	2.13	0
1	4	15.38	2282.6	183.1	121.2	515.4	0.0	2.71	HL-93	2.09	0
1	5	19.23	2282.6	203.3	134.6	563.7	0.0	2.42	HL-93	1.86	0
1	6	23.08	2282.6	213.4	141.3	574.9	0.0	2.34	HL-93	1.81	0
1	7	26.92	2282.6	213.5	141.3	574.9	0.0	2.34	HL-93	1.81	0
1	8	30.77	2282.6	203.5	134.6	563.7	0.0	2.42	HL-93	1.86	0
1	9	34.62	2282.6	183.0	121.2	515.4	0.0	2.71	HL-93	2.09	0
1	10	38.46	1969.3	152.3	101.0	439.8	0.0	2.76	HL-93	2.13	0
1	11	42.31	1969.3	111.6	74.0	331.5	0.0	3.85	HL-93	2.97	0
1	12	46.15	1969.3	60.8	40.4	184.4	0.0	7.38	HL-93	5.69	0
1	13	50.00	1969.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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1

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS STEEL TOP;ksi	BOT	UNFACTORED L+I STRESS MAX. POS. STEEL TOP;ksi	BOT	MAX. NEG. STEEL TOP;ksi	BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.85	31.4	-2.42	3.05	-0.46	4.14	0.00	0.00	6.84	HL-93	5.26
1	2	7.69	31.4	-4.44	5.60	-0.84	7.44	0.00	0.00	3.46	HL-93	2.66
1	3	11.54	31.4	-6.06	7.64	-1.11	9.87	0.00	0.00	2.40	HL-93	1.85
1	4	15.38	31.4	-6.90	7.10	-1.48	9.08	0.00	0.00	2.67	HL-93	2.06

1	5	19.23	31.4	-7.66	7.88	-1.62	9.93	0.00	0.00	2.36	HL-93	1.82
1	6	23.08	31.4	-8.04	8.27	-1.65	10.13	0.00	0.00	2.28	HL-93	1.75
1	7	26.92	31.4	-8.05	8.28	-1.65	10.13	0.00	0.00	2.28	HL-93	1.75
1	8	30.77	31.4	-7.67	7.89	-1.62	9.93	0.00	0.00	2.36	HL-93	1.82
1	9	34.62	31.4	-6.90	7.09	-1.48	9.08	0.00	0.00	2.67	HL-93	2.06
1	10	38.46	31.4	-6.05	7.62	-1.11	9.87	0.00	0.00	2.40	HL-93	1.85
1	11	42.31	31.4	-4.43	5.59	-0.84	7.44	0.00	0.00	3.46	HL-93	2.66
1	12	46.15	31.4	-2.41	3.05	-0.46	4.14	0.00	0.00	6.84	HL-93	5.26
1	13	50.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	50.00	26.92	STRENGTH L.S.	2.34	1.81	HL-93
1	50.00	26.92	SERVICEBILITY	2.28	1.75	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.279
Inventory= 1.753

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	50.00	26.92	STRENGTH L.S.	3.29	551.51
1	50.00	26.92	SERVICEBILITY	3.05	

```
-------------------------------------------------------------
    NOTE: control Rating Factor for Current Bridge:
          SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
               Legal Load Rating= 3.046
1
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                                                                   RATING
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 TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
               *********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT.
 SP IN D FROM   SHEAR        SHEAR       MAX.  SHEAR    RATING  LIVE  RATING
 NO NO L SUPT   CAP.                     --------------- FACTOR  LOAD  FACTOR
       (ft)  (kips)       D1; kips  D2     POS.     NEG.  [3]   TYPE
 ------------------------------------------------------------------------------

  1  0   0.00    330.8     17.2     11.4     52.7      0.0  4.13  HL-93  3.18
  1  1   3.85    330.8     14.5      9.6     47.2     -1.9  4.69  HL-93  3.62
  1  2   7.69    330.8     11.9      7.9     41.8     -4.5  5.39  HL-93  4.16
  1  3  11.54    330.8      9.3      6.1     36.6     -7.6  6.28  HL-93  4.85
  1  4  15.38    330.8      6.6      4.4     31.4    -10.8  7.45  HL-93  5.75
  1  5  19.23    330.8      3.9      2.6     26.4    -14.2  9.04  HL-93  6.97
  1  6  23.08    330.8      1.3      0.9     21.7    -17.6  9.99  HL-93  8.64
  1  7  26.92    330.8     -1.3     -0.9     17.6    -21.7  9.99  HL-93  8.64
  1  8  30.77    330.8     -3.9     -2.6     14.2    -26.4  9.04  HL-93  6.97
  1  9  34.62    330.8     -6.7     -4.4     10.8    -31.4  7.45  HL-93  5.75
  1 10  38.46    330.8     -9.3     -6.1      7.6    -36.6  6.29  HL-93  4.85
  1 11  42.31    330.8    -11.9     -7.9      4.5    -41.8  5.40  HL-93  4.16
  1 12  46.15    330.8    -14.5     -9.6      1.9    -47.2  4.69  HL-93  3.62
  1 13  50.00    330.8    -17.1    -11.4      0.0    -52.7  4.13  HL-93  3.18


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                                                                   RATING
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 TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
                ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                        GOVERNING
 ------------- ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    RATING     TYPE
       (ft)         (ft)
 ------------------------------------------------------------------------------

   1    50.00         0.00       STRENGTH L.S.  4.13      3.18     HL-93

 ------------------------------------------------------------------------------
    NOTE: Control Rating Factor for Current Bridge:
          STRENGTH L.S.
               Operating= 4.125
               Inventory= 3.182


 TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK     FA
                ***************************************************

     SPAN       CRITICAL LOCATION GOVERNING
 ------------- ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    LL SHEAR
       (ft)         (ft)                                  (kips)
 ---------------------------------------------------------------

   1    50.00         0.00       STRENGTH L.S.  5.15         56.90

 -------------------------------------------------------------
```

5969 Span 2-S15 Exterior Fatigue.RES

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.154

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	25.00	3 0.425	-0.28	HL-93	2.69
		4 0.433	-0.28		2.64

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

1

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95NB Span 2 S16 Int. Strength CP 8/20/13


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Interior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       17           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
1
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                                                                INPUT
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
52.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT (lb/ft)	WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.0000	10.00	0.7500		

STANDARD SECTN PLATE GIRDER

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	9.61				33.	33.	33.
2	2	2	33.00				33.	33.	33.
3	1	1	9.61				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
	OPTIONAL CALCULATION OF FACTOR					OPTIONAL LOADING TYPES	OPTIONAL DISTRIBUTION FACTORS	
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied
to the indicated loading type.

1 = The special distribution factor defined is applied to
the indicated loading type of calculation of all
moment, shear and deflection.

2 = The special distribution factor defined is only applied
to the loading type for calculating moment.

3 = The special distribution factor defined is only applied
to the loading type for calculating shear.

4 = The special distribution factor defined is only applied
to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	POUR NO	IDENTIFICATION DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	MODULAR RATIO N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	52.23

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those
of normal weight concrete (W=145 pcf) of various design strength shall
be as follows:

fc' = unit ultimate compressive strength of concrete as determined
by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	17.45
2	2	Diaphragm	0.000	0.00	0.00	0.29	36.29
3	1	Railing	0.087	0.00	52.23	0.00	0.00
4	1	Curb	0.076	0.00	52.23	0.00	0.00
5	0	Pavement	0.292	0.00	52.23	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi) (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	17.45	33.0	33.0	17.45
17.45	36.29	33.0	33.0	18.84
36.29	52.23	33.0	33.0	15.94

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.727       0.1342     0.8612


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      52.23

     1   DC2     0.087       0.000     52.230

     2   DC2     0.076       0.000     52.230


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1268.2	47.0	190.2	42.1	232.29

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.714	0.714	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.07	5900.0	32.9	16.45	358.7	358.7
1	2	6.14	5900.0	32.9	16.45	358.7	358.7
1	3	9.22	5900.0	32.9	16.45	358.7	358.7
1	4	12.29	7650.6	32.9	14.21	538.4	393.5
1	5	15.36	7650.6	32.9	14.21	538.4	393.5
1	6	18.43	7650.6	32.9	14.21	538.4	393.5
1	7	21.51	7650.6	32.9	14.21	538.4	393.5
1	8	24.58	7650.6	32.9	14.21	538.4	393.5
1	9	27.65	7650.6	32.9	14.21	538.4	393.5
1	10	30.72	7650.6	32.9	14.21	538.4	393.5
1	11	33.80	7650.6	32.9	14.21	538.4	393.5
1	12	36.87	7650.6	32.9	14.21	538.4	393.5

1	13	39.94	7650.6	32.9	14.21	538.4	393.5
1	14	43.01	5900.0	32.9	16.45	358.7	358.7
1	15	46.09	5900.0	32.9	16.45	358.7	358.7
1	16	49.16	5900.0	32.9	16.45	358.7	358.7
1	17	52.23	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12575.9	0.024742	496.1	1666.1	788.1
1	1	3.07	12575.9	0.024742	496.1	1666.1	788.1
1	2	6.14	12575.9	0.024742	496.1	1666.1	788.1
1	3	9.22	12575.9	0.024742	496.1	1666.1	788.1
1	4	12.29	16991.1	0.022596	731.4	1631.0	902.5
1	5	15.36	16991.1	0.022596	731.4	1631.0	902.5
1	6	18.43	16991.1	0.022596	731.4	1631.0	902.5
1	7	21.51	16991.1	0.022596	731.4	1631.0	902.5
1	8	24.58	16991.1	0.022596	731.4	1631.0	902.5
1	9	27.65	16991.1	0.022596	731.4	1631.0	902.5
1	10	30.72	16991.1	0.022596	731.4	1631.0	902.5
1	11	33.80	16991.1	0.022596	731.4	1631.0	902.5
1	12	36.87	16991.1	0.022596	731.4	1631.0	902.5
1	13	39.94	16991.1	0.022596	731.4	1631.0	902.5
1	14	43.01	12575.9	0.024742	496.1	1666.1	788.1
1	15	46.09	12575.9	0.024742	496.1	1666.1	788.1
1	16	49.16	12575.9	0.024742	496.1	1666.1	788.1
1	17	52.23	12575.9	0.024742	496.1	1666.1	788.1

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	16953.0	0.029821	547.3	8815.3	1640.6
1	1	3.07	16953.0	0.029821	547.3	8815.3	1640.6
1	2	6.14	16953.0	0.029821	547.3	8815.3	1640.6
1	3	9.22	16953.0	0.029821	547.3	8815.3	1640.6
1	4	12.29	23858.7	0.027580	802.7	6075.2	1933.9
1	5	15.36	23858.7	0.027580	802.7	6075.2	1933.9
1	6	18.43	23858.7	0.027580	802.7	6075.2	1933.9
1	7	21.51	23858.7	0.027580	802.7	6075.2	1933.9
1	8	24.58	23858.7	0.027580	802.7	6075.2	1933.9
1	9	27.65	23858.7	0.027580	802.7	6075.2	1933.9

1 10	30.72	23858.7	0.027580	802.7	6075.2	1933.9
1 11	33.80	23858.7	0.027580	802.7	6075.2	1933.9
1 12	36.87	23858.7	0.027580	802.7	6075.2	1933.9
1 13	39.94	23858.7	0.027580	802.7	6075.2	1933.9
1 14	43.01	16953.0	0.029821	547.3	8815.3	1640.6
1 15	46.09	16953.0	0.029821	547.3	8815.3	1640.6
1 16	49.16	16953.0	0.029821	547.3	8815.3	1640.6
1 17	52.23	16953.0	0.029821	547.3	8815.3	1640.6

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.07	10.1	54.9	0.9	0.0		65.9
1	2	6.14	19.0	102.9	1.7	0.0		123.7
1	3	9.22	26.6	144.1	2.6	0.0		173.3
1	4	12.29	32.9	178.4	3.4	0.0		214.8
1	5	15.36	38.0	205.9	4.3	0.0		248.2
1	6	18.43	41.8	226.5	4.9	0.0		273.1
1	7	21.51	44.3	240.2	4.8	0.0		289.4
1	8	24.58	45.6	247.0	4.8	0.0		297.5
1	9	27.65	45.6	247.0	4.8	0.0		297.4
1	10	30.72	44.3	240.2	4.8	0.0		289.3
1	11	33.80	41.8	226.5	4.7	0.0		273.0
1	12	36.87	38.0	205.9	4.5	0.0		248.4
1	13	39.94	32.9	178.4	3.6	0.0		215.0
1	14	43.01	26.6	144.1	2.7	0.0		173.4
1	15	46.09	19.0	102.9	1.8	0.0		123.8

1 16	49.16	10.1	54.9	0.9	0.0	65.9
1 17	52.23	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.07	22.1	12.3	0.0	0.0	0.0	34.4
1	2	6.14	41.3	23.1	0.0	0.0	0.0	64.4
1	3	9.22	57.9	32.3	0.0	0.0	0.0	90.2
1	4	12.29	71.7	40.0	0.0	0.0	0.0	111.7
1	5	15.36	82.7	46.2	0.0	0.0	0.0	128.8
1	6	18.43	91.0	50.8	0.0	0.0	0.0	141.7
1	7	21.51	96.5	53.9	0.0	0.0	0.0	150.3
1	8	24.58	99.2	55.4	0.0	0.0	0.0	154.6
1	9	27.65	99.2	55.4	0.0	0.0	0.0	154.6
1	10	30.72	96.5	53.9	0.0	0.0	0.0	150.3
1	11	33.80	91.0	50.8	0.0	0.0	0.0	141.7
1	12	36.87	82.7	46.2	0.0	0.0	0.0	128.8
1	13	39.94	71.7	40.0	0.0	0.0	0.0	111.7
1	14	43.01	57.9	32.3	0.0	0.0	0.0	90.2
1	15	46.09	41.3	23.1	0.0	0.0	0.0	64.4
1	16	49.16	22.1	12.3	0.0	0.0	0.0	34.4
1	17	52.23	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.07	0.0	0.0	190.3	HL-93	0.0	HL-93
1	2	6.14	0.0	0.0	352.3	HL-93	0.0	HL-93
1	3	9.22	0.0	0.0	485.8	HL-93	0.0	HL-93
1	4	12.29	0.0	0.0	588.6	HL-93	0.0	HL-93
1	5	15.36	0.0	0.0	667.2	HL-93	0.0	HL-93
1	6	18.43	0.0	0.0	726.0	HL-93	0.0	HL-93
1	7	21.51	0.0	0.0	757.5	HL-93	0.0	HL-93
1	8	24.58	0.0	0.0	764.3	HL-93	0.0	HL-93
1	9	27.65	0.0	0.0	764.3	HL-93	0.0	HL-93
1	10	30.72	0.0	0.0	757.5	HL-93	0.0	HL-93
1	11	33.80	0.0	0.0	725.9	HL-93	0.0	HL-93
1	12	36.87	0.0	0.0	667.2	HL-93	0.0	HL-93
1	13	39.94	0.0	0.0	588.6	HL-93	0.0	HL-93
1	14	43.01	0.0	0.0	485.8	HL-93	0.0	HL-93
1	15	46.09	0.0	0.0	352.3	HL-93	0.0	HL-93
1	16	49.16	0.0	0.0	190.3	HL-93	0.0	HL-93
1	17	52.23	0.0	0.0	0.0	HL-93	0.0	HL-93

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 TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N =  9.0 (k-ft) (UNFACTORED)
                 ************************************************

 SP IN D FROM     TRUCK   ONLY
 NO NO L SUPT -------------------
        (ft)    POS    NEG  RANGE
 --------------------------------
  1  0   0.00    0.     0.     0.
  1  1   3.07   65.     0.    65.
  1  2   6.14  116.     0.   116.
  1  3   9.22  156.     0.   156.
  1  4  12.29  187.     0.   187.
  1  5  15.36  208.     0.   208.
  1  6  18.43  217.     0.   217.
  1  7  21.51  225.     0.   225.
  1  8  24.58  230.     0.   230.
  1  9  27.65  230.     0.   230.
  1 10  30.72  225.     0.   225.
  1 11  33.80  217.     0.   217.
  1 12  36.87  208.     0.   208.
  1 13  39.94  187.     0.   187.
  1 14  43.01  156.     0.   156.
  1 15  46.09  116.     0.   116.
  1 16  49.16   65.     0.    65.
  1 17  52.23    0.     0.     0.


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 TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
                ****************************************


                   DEAD  LOAD   LL+I:N= 9.0 LOAD TYPE= HL - 93   TOTAL MAXIMUM
               ---------------- ------------------------------ -----------------
 SP IN D FROM NON COMP. COMP.  MAXIMUM  GOVERN MAXIMUM GOVERN POSITIVE NEGATIVE
 NO NO L SUPT  N=Infin. N=27.0 POSITIVE  LOAD  NEGATIVE LOAD  -------- --------
        (ft)   (k-ft)   (k-ft)  (k-ft)   TYPE   (k-ft)  TYPE   (k-ft)   (k-ft)
 ------------------------------------------------------------------------------

  1  0   0.00     0.0     0.0     0.0  HL-93      0.0  HL-93      0.0      0.0
  1  1   3.07    65.9    34.4   190.3  HL-93      0.0  HL-93    290.6    100.2
  1  2   6.14   123.7    64.4   352.3  HL-93      0.0  HL-93    540.3    188.1
  1  3   9.22   173.3    90.2   485.8  HL-93      0.0  HL-93    749.3    263.5
  1  4  12.29   214.8   111.7   588.6  HL-93      0.0  HL-93    915.1    326.5
  1  5  15.36   248.2   128.8   667.2  HL-93      0.0  HL-93   1044.2    377.0
  1  6  18.43   273.1   141.7   726.0  HL-93      0.0  HL-93   1140.8    414.8
  1  7  21.51   289.4   150.3   757.5  HL-93      0.0  HL-93   1197.2    439.7
  1  8  24.58   297.5   154.6   764.3  HL-93      0.0  HL-93   1216.4    452.1
  1  9  27.65   297.4   154.6   764.3  HL-93      0.0  HL-93   1216.4    452.0
  1 10  30.72   289.3   150.3   757.5  HL-93      0.0  HL-93   1197.1    439.6
  1 11  33.80   273.0   141.7   725.9  HL-93      0.0  HL-93   1140.7    414.7
  1 12  36.87   248.4   128.8   667.2  HL-93      0.0  HL-93   1044.5    377.3
  1 13  39.94   215.0   111.7   588.6  HL-93      0.0  HL-93    915.3    326.7
  1 14  43.01   173.4    90.2   485.8  HL-93      0.0  HL-93    749.4    263.6
  1 15  46.09   123.8    64.4   352.3  HL-93      0.0  HL-93    540.4    188.2
  1 16  49.16    65.9    34.4   190.3  HL-93      0.0  HL-93    290.6    100.3
  1 17  52.23     0.0     0.0     0.0  HL-93      0.0  HL-93      0.0      0.0


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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.07	290.6	347.7	463.9	387.7	150.4	48.4
1	2	6.14	540.3	646.0	861.9	721.0	282.1	87.2
1	3	9.22	749.3	895.0	1194.0	999.7	395.2	117.2
1	4	12.29	915.1	1091.7	1456.1	1220.6	489.7	139.9
1	5	15.36	1044.2	1244.4	1659.6	1392.7	565.5	155.8
1	6	18.43	1140.8	1358.6	1811.7	1521.3	622.3	163.1
1	7	21.51	1197.2	1424.4	1899.4	1596.4	659.5	168.6
1	8	24.58	1216.4	1445.7	1927.4	1621.7	678.1	172.4
1	9	27.65	1216.4	1445.6	1927.4	1621.7	678.1	172.4
1	10	30.72	1197.1	1424.4	1899.3	1596.3	659.4	168.6
1	11	33.80	1140.7	1358.5	1811.6	1521.2	622.1	163.1
1	12	36.87	1044.5	1244.7	1659.9	1393.0	565.9	155.8
1	13	39.94	915.3	1091.9	1456.3	1220.9	490.0	139.9
1	14	43.01	749.4	895.2	1194.2	999.9	395.4	117.2
1	15	46.09	540.4	646.1	862.0	721.1	282.3	87.2
1	16	49.16	290.6	347.7	463.9	387.8	150.4	48.4
1	17	52.23	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.5	19.0	0.3	0.0	22.8
1	1	3.07	3.1	16.8	0.3	0.0	20.1
1	2	6.14	2.7	14.5	0.3	0.0	17.5
1	3	9.22	2.3	12.3	0.3	0.0	14.8
1	4	12.29	1.9	10.1	0.3	0.0	12.2
1	5	15.36	1.4	7.8	0.3	0.0	9.5
1	6	18.43	1.0	5.6	0.0	0.0	6.6
1	7	21.51	0.6	3.4	0.0	0.0	4.0
1	8	24.58	0.2	1.1	0.0	0.0	1.3
1	9	27.65	-0.2	-1.1	0.0	0.0	-1.3
1	10	30.72	-0.6	-3.4	0.0	0.0	-4.0
1	11	33.80	-1.0	-5.6	0.0	0.0	-6.6
1	12	36.87	-1.4	-7.8	-0.3	0.0	-9.6
1	13	39.94	-1.9	-10.1	-0.3	0.0	-12.2
1	14	43.01	-2.3	-12.3	-0.3	0.0	-14.8
1	15	46.09	-2.7	-14.5	-0.3	0.0	-17.5
1	16	49.16	-3.1	-16.8	-0.3	0.0	-20.1
1	17	52.23	-3.5	-19.0	-0.3	0.0	-22.8

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN D NO	L FROM SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	DC2	CONCENTRATED LOADS DW	DC2	TOTAL COMPOSITE (k)	NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.6	4.3	0.0	0.0	11.9	22.8	34.7
1	1	3.07	6.7	3.8	0.0	0.0	10.5	20.1	30.6
1	2	6.14	5.8	3.3	0.0	0.0	9.1	17.5	26.6
1	3	9.22	4.9	2.8	0.0	0.0	7.7	14.8	22.5
1	4	12.29	4.0	2.3	0.0	0.0	6.3	12.2	18.5
1	5	15.36	3.1	1.8	0.0	0.0	4.9	9.5	14.4
1	6	18.43	2.2	1.3	0.0	0.0	3.5	6.6	10.1
1	7	21.51	1.3	0.8	0.0	0.0	2.1	4.0	6.1
1	8	24.58	0.4	0.3	0.0	0.0	0.7	1.3	2.0
1	9	27.65	-0.4	-0.3	0.0	0.0	-0.7	-1.3	-2.0
1	10	30.72	-1.3	-0.8	0.0	0.0	-2.1	-4.0	-6.1
1	11	33.80	-2.2	-1.3	0.0	0.0	-3.5	-6.6	-10.1
1	12	36.87	-3.1	-1.8	0.0	0.0	-4.9	-9.6	-14.4
1	13	39.94	-4.0	-2.3	0.0	0.0	-6.3	-12.2	-18.5
1	14	43.01	-4.9	-2.8	0.0	0.0	-7.7	-14.8	-22.5
1	15	46.09	-5.8	-3.3	0.0	0.0	-9.1	-17.5	-26.6
1	16	49.16	-6.7	-3.8	0.0	0.0	-10.5	-20.1	-30.6
1	17	52.23	-7.6	-4.3	0.0	0.0	-11.9	-22.8	-34.7

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN D NO	L FROM SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	83.7	HL-93	0.0	HL-93
1	1	3.07	0.0	0.0	77.1	HL-93	-2.2	HL-93
1	2	6.14	0.0	0.0	70.6	HL-93	-4.8	HL-93
1	3	9.22	0.0	0.0	64.2	HL-93	-8.5	HL-93
1	4	12.29	0.0	0.0	57.8	HL-93	-12.3	HL-93
1	5	15.36	0.0	0.0	51.6	HL-93	-16.2	HL-93
1	6	18.43	0.0	0.0	45.5	HL-93	-20.2	HL-93
1	7	21.51	0.0	0.0	39.5	HL-93	-24.3	HL-93
1	8	24.58	0.0	0.0	33.8	HL-93	-28.5	HL-93
1	9	27.65	0.0	0.0	28.5	HL-93	-33.8	HL-93
1	10	30.72	0.0	0.0	24.3	HL-93	-39.5	HL-93
1	11	33.80	0.0	0.0	20.2	HL-93	-45.5	HL-93
1	12	36.87	0.0	0.0	16.2	HL-93	-51.6	HL-93
1	13	39.94	0.0	0.0	12.3	HL-93	-57.8	HL-93
1	14	43.01	0.0	0.0	8.5	HL-93	-64.2	HL-93
1	15	46.09	0.0	0.0	4.8	HL-93	-70.6	HL-93
1	16	49.16	0.0	0.0	2.2	HL-93	-77.1	HL-93
1	17	52.23	0.0	0.0	0.0	HL-93	-83.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	33.	0.	33.
1	1	3.07	30.	-1.	31.
1	2	6.14	27.	-3.	30.
1	3	9.22	24.	-4.	28.
1	4	12.29	22.	-5.	27.
1	5	15.36	19.	-7.	26.
1	6	18.43	16.	-8.	24.
1	7	21.51	14.	-10.	24.
1	8	24.58	13.	-12.	25.
1	9	27.65	12.	-13.	25.
1	10	30.72	10.	-14.	24.
1	11	33.80	8.	-16.	24.
1	12	36.87	7.	-19.	26.
1	13	39.94	5.	-22.	27.
1	14	43.01	4.	-24.	28.
1	15	46.09	3.	-27.	30.
1	16	49.16	1.	-30.	31.
1	17	52.23	0.	-33.	33.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	33.0	.31015E-01	0.51	0.2500*	.14206E-01	0.23	0.2500*
1	1	3.07	31.3	.31015E-01	0.49	0.2500*	.14206E-01	0.22	0.2500*
1	2	6.14	29.7	.31015E-01	0.46	0.2500*	.14206E-01	0.21	0.2500*
1	3	9.22	28.2	.31015E-01	0.44	0.2500*	.14206E-01	0.20	0.2500*
1	4	12.29	26.9	.29123E-01	0.39	0.2500*	.93999E-02	0.13	0.2500*
1	5	15.36	25.7	.29123E-01	0.37	0.2500*	.93999E-02	0.12	0.2500*
1	6	18.43	24.3	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	7	21.51	24.2	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	8	24.58	25.0	.29123E-01	0.36	0.2500*	.93999E-02	0.12	0.2500*
1	9	27.65	25.0	.29123E-01	0.36	0.2500*	.93999E-02	0.12	0.2500*
1	10	30.72	24.2	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	11	33.80	24.3	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	12	36.87	25.7	.29123E-01	0.37	0.2500*	.93999E-02	0.12	0.2500*
1	13	39.94	26.9	.29123E-01	0.39	0.2500*	.93999E-02	0.13	0.2500*
1	14	43.01	28.2	.31015E-01	0.44	0.2500*	.14206E-01	0.20	0.2500*
1	15	46.09	29.7	.31015E-01	0.46	0.2500*	.14206E-01	0.21	0.2500*
1	16	49.16	31.3	.31015E-01	0.49	0.2500*	.14206E-01	0.22	0.2500*
1	17	52.23	33.0	.31015E-01	0.51	0.2500*	.14206E-01	0.23	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	34.7	83.7	HL-93	0.0	HL-93	118.4	34.7
1	1	3.07	30.6	77.1	HL-93	-2.2	HL-93	107.7	28.4
1	2	6.14	26.6	70.6	HL-93	-4.8	HL-93	97.1	21.7
1	3	9.22	22.5	64.2	HL-93	-8.5	HL-93	86.7	14.0
1	4	12.29	18.5	57.8	HL-93	-12.3	HL-93	76.3	6.2
1	5	15.36	14.4	51.6	HL-93	-16.2	HL-93	66.1	-1.8
1	6	18.43	10.1	45.5	HL-93	-20.2	HL-93	55.6	-10.1
1	7	21.51	6.1	39.5	HL-93	-24.3	HL-93	45.6	-18.2
1	8	24.58	2.0	33.8	HL-93	-28.5	HL-93	35.8	-26.5
1	9	27.65	-2.0	28.5	HL-93	-33.8	HL-93	26.4	-35.8
1	10	30.72	-6.1	24.3	HL-93	-39.5	HL-93	18.2	-45.6
1	11	33.80	-10.1	20.2	HL-93	-45.5	HL-93	10.1	-55.6
1	12	36.87	-14.4	16.2	HL-93	-51.6	HL-93	1.7	-66.1
1	13	39.94	-18.5	12.3	HL-93	-57.8	HL-93	-6.2	-76.3
1	14	43.01	-22.5	8.5	HL-93	-64.2	HL-93	-14.0	-86.7
1	15	46.09	-26.6	4.8	HL-93	-70.6	HL-93	-21.8	-97.1
1	16	49.16	-30.6	2.2	HL-93	-77.1	HL-93	-28.4	-107.7
1	17	52.23	-34.7	0.0	HL-93	-83.7	HL-93	-34.7	-118.4

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	118.4	143.5	191.7	158.2	52.0	24.8
1	1	3.07	107.7	130.8	174.8	144.0	45.9	23.5
1	2	6.14	97.1	118.3	158.2	129.9	39.8	22.3
1	3	9.22	86.7	105.9	141.6	116.0	33.8	21.1
1	4	12.29	76.3	93.7	125.3	102.2	27.7	20.1
1	5	15.36	66.1	81.5	109.2	88.5	21.6	19.3
1	6	18.43	55.6	69.2	92.7	74.6	15.2	18.2
1	7	21.51	45.6	57.4	77.0	61.2	9.1	18.1
1	8	24.58	35.8	45.9	61.7	48.2	3.0	18.8
1	9	27.65	-35.8	-45.9	-61.7	-48.2	-3.0	18.8

1 10	30.72	-45.6	-57.4	-77.1	-61.3	-9.1	18.1
1 11	33.80	-55.6	-69.2	-92.8	-74.6	-15.2	18.2
1 12	36.87	-66.1	-81.6	-109.2	-88.5	-21.7	19.3
1 13	39.94	-76.3	-93.7	-125.3	-102.2	-27.7	20.1
1 14	43.01	-86.7	-105.9	-141.7	-116.0	-33.8	21.1
1 15	46.09	-97.1	-118.3	-158.2	-130.0	-39.9	22.3
1 16	49.16	-107.7	-130.8	-174.9	-144.0	-45.9	23.5
1 17	52.23	-118.4	-143.5	-191.7	-158.3	-52.0	24.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	34.65	0.00	HL-93	83.71	HL-93	ST1	29.28	191.71
						ST2	29.28	158.23
						ST4	40.54	51.98
						SE1	34.65	118.36
						SE2	34.65	143.47
2	34.67	0.00	HL-93	83.71	HL-93	ST1	29.29	191.73
						ST2	29.29	158.25
						ST4	40.56	52.00
						SE1	34.67	118.38
						SE2	34.67	143.49

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE+COMPOSITE = DL (in)	(in)	(in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.07	-0.0199	-0.1078	0.0000	-0.0307	-0.1299	-0.0307	-0.1605
1	2	6.14	-0.0389	-0.2105	0.0000	-0.0598	-0.2535	-0.0598	-0.3133
1	3	9.22	-0.0560	-0.3034	0.0000	-0.0861	-0.3654	-0.0861	-0.4515
1	4	12.29	-0.0709	-0.3840	0.0000	-0.1088	-0.4624	-0.1088	-0.5712
1	5	15.36	-0.0833	-0.4514	0.0000	-0.1278	-0.5437	-0.1278	-0.6715
1	6	18.43	-0.0930	-0.5037	0.0000	-0.1425	-0.6068	-0.1425	-0.7493
1	7	21.51	-0.0996	-0.5395	0.0000	-0.1526	-0.6498	-0.1526	-0.8024
1	8	24.58	-0.1029	-0.5576	0.0000	-0.1577	-0.6716	-0.1577	-0.8294
1	9	27.65	-0.1029	-0.5576	0.0000	-0.1577	-0.6717	-0.1577	-0.8294
1	10	30.72	-0.0996	-0.5395	0.0000	-0.1526	-0.6498	-0.1526	-0.8024
1	11	33.80	-0.0930	-0.5037	0.0000	-0.1425	-0.6068	-0.1425	-0.7493
1	12	36.87	-0.0833	-0.4514	0.0000	-0.1278	-0.5438	-0.1278	-0.6715
1	13	39.94	-0.0709	-0.3840	0.0000	-0.1088	-0.4625	-0.1088	-0.5713
1	14	43.01	-0.0560	-0.3034	0.0000	-0.0861	-0.3655	-0.0861	-0.4516
1	15	46.09	-0.0389	-0.2105	0.0000	-0.0598	-0.2535	-0.0598	-0.3133
1	16	49.16	-0.0199	-0.1078	0.0000	-0.0307	-0.1299	-0.0307	-0.1605
1	17	52.23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.1	-0.020	0 1/16	-0.110	0 1/ 8	-0.031	0 1/16	-0.161	0 3/16
1	2	6.1	-0.039	0 1/16	-0.215	0 1/ 4	-0.060	0 1/16	-0.313	0 3/ 8
1	3	9.2	-0.056	0 1/16	-0.309	0 5/16	-0.086	0 1/ 8	-0.451	0 1/ 2
1	4	12.3	-0.071	0 1/ 8	-0.392	0 7/16	-0.109	0 1/ 8	-0.571	0 5/ 8
1	5	15.4	-0.083	0 1/ 8	-0.460	0 1/ 2	-0.128	0 3/16	-0.671	0 11/16
1	6	18.4	-0.093	0 1/ 8	-0.514	0 9/16	-0.143	0 3/16	-0.749	0 3/ 4
1	7	21.5	-0.100	0 1/ 8	-0.550	0 9/16	-0.153	0 3/16	-0.802	0 13/16
1	8	24.6	-0.103	0 1/ 8	-0.569	0 5/ 8	-0.158	0 3/16	-0.829	0 7/ 8
1	9	27.7	-0.103	0 1/ 8	-0.569	0 5/ 8	-0.158	0 3/16	-0.829	0 7/ 8
1	10	30.7	-0.100	0 1/ 8	-0.550	0 9/16	-0.153	0 3/16	-0.802	0 13/16
1	11	33.8	-0.093	0 1/ 8	-0.514	0 9/16	-0.143	0 3/16	-0.749	0 3/ 4
1	12	36.9	-0.083	0 1/ 8	-0.460	0 1/ 2	-0.128	0 3/16	-0.672	0 11/16
1	13	39.9	-0.071	0 1/ 8	-0.392	0 7/16	-0.109	0 1/ 8	-0.571	0 5/ 8
1	14	43.0	-0.056	0 1/16	-0.309	0 5/16	-0.086	0 1/ 8	-0.452	0 1/ 2
1	15	46.1	-0.039	0 1/16	-0.215	0 1/ 4	-0.060	0 1/16	-0.313	0 3/ 8
1	16	49.2	-0.020	0 1/16	-0.110	0 1/ 8	-0.031	0 1/16	-0.161	0 3/16
1	17	52.2	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	26.11	3	0.425	-0.260	MAX HL-93	0.78	0.00242
				0.025	MIN		
				-0.068	MAX LANE		
				0.025	MIN LANE		
		4	0.433	-0.265	MAX HL-93	0.78	0.00242
				0.026	MIN		
				-0.069	MAX LANE		
				0.026	MIN LANE		
		3	0.425	-0.166	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	-0.34	0.34	-1.87	1.87	-2.20	2.20
1	2	6.14	-0.64	0.64	-3.50	3.50	-4.14	4.14
1	3	9.22	-0.89	0.89	-4.91	4.91	-5.80	5.80
1	4	12.29	-1.00	0.73	-5.54	4.05	-6.55	4.79
1	5	15.36	-1.16	0.85	-6.41	4.68	-7.57	5.53
1	6	18.43	-1.27	0.93	-7.05	5.16	-8.33	6.09
1	7	21.51	-1.35	0.99	-7.47	5.46	-8.82	6.45
1	8	24.58	-1.39	1.02	-7.68	5.61	-9.07	6.63
1	9	27.65	-1.39	1.02	-7.68	5.61	-9.07	6.63
1	10	30.72	-1.35	0.99	-7.47	5.46	-8.82	6.45
1	11	33.80	-1.27	0.93	-7.05	5.15	-8.32	6.08
1	12	36.87	-1.16	0.85	-6.42	4.69	-7.57	5.54
1	13	39.94	-1.00	0.73	-5.55	4.06	-6.56	4.79
1	14	43.01	-0.89	0.89	-4.91	4.91	-5.80	5.80
1	15	46.09	-0.64	0.64	-3.50	3.50	-4.14	4.14
1	16	49.16	-0.34	0.34	-1.87	1.87	-2.21	2.21
1	17	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SUPERIMPOSED DEAD LOAD,(ksi) TOTAL DEAD LOAD,(ksi)

5969 Span 2-S16 Interior Strength CP.RES

SP NO	IN NO	D FROM L SUPT (ft)	CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	-0.02	-0.25	0.83	-0.02	-2.45	3.04
1	2	6.14	-0.04	-0.46	1.56	-0.04	-4.60	5.70
1	3	9.22	-0.05	-0.65	2.18	-0.05	-6.45	7.98
1	4	12.29	-0.05	-0.82	1.83	-0.05	-7.37	6.62
1	5	15.36	-0.06	-0.95	2.11	-0.06	-8.51	7.65
1	6	18.43	-0.07	-1.04	2.33	-0.07	-9.37	8.41
1	7	21.51	-0.07	-1.11	2.47	-0.07	-9.93	8.92
1	8	24.58	-0.08	-1.14	2.54	-0.08	-10.21	9.17
1	9	27.65	-0.08	-1.14	2.54	-0.08	-10.21	9.17
1	10	30.72	-0.07	-1.11	2.47	-0.07	-9.93	8.91
1	11	33.80	-0.07	-1.04	2.33	-0.07	-9.37	8.41
1	12	36.87	-0.06	-0.95	2.11	-0.06	-8.52	7.65
1	13	39.94	-0.05	-0.82	1.83	-0.05	-7.38	6.62
1	14	43.01	-0.05	-0.65	2.18	-0.05	-6.45	7.98
1	15	46.09	-0.04	-0.46	1.56	-0.04	-4.60	5.70
1	16	49.16	-0.02	-0.25	0.83	-0.02	-2.45	3.04
1	17	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	-0.15	-0.26	4.17	0.00	0.00	0.00
1	2	6.14	-0.29	-0.48	7.72	0.00	0.00	0.00
1	3	9.22	-0.39	-0.66	10.65	0.00	0.00	0.00
1	4	12.29	-0.41	-1.16	8.80	0.00	0.00	0.00
1	5	15.36	-0.46	-1.32	9.97	0.00	0.00	0.00
1	6	18.43	-0.50	-1.43	10.85	0.00	0.00	0.00
1	7	21.51	-0.52	-1.50	11.32	0.00	0.00	0.00
1	8	24.58	-0.53	-1.51	11.43	0.00	0.00	0.00
1	9	27.65	-0.53	-1.51	11.43	0.00	0.00	0.00
1	10	30.72	-0.52	-1.50	11.32	0.00	0.00	0.00
1	11	33.80	-0.50	-1.43	10.85	0.00	0.00	0.00
1	12	36.87	-0.46	-1.32	9.97	0.00	0.00	0.00
1	13	39.94	-0.41	-1.16	8.80	0.00	0.00	0.00
1	14	43.01	-0.39	-0.66	10.65	0.00	0.00	0.00
1	15	46.09	-0.29	-0.48	7.72	0.00	0.00	0.00
1	16	49.16	-0.15	-0.26	4.17	0.00	0.00	0.00
1	17	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT	TOP OF TOP FLANGE	GOVERNING STRESS	GOVERNING LOAD	BOT OF BOT FLANGE	GOVERNING STRESS	GOVERNING LOAD

		(ft)	TRUCK	ONLY	RANGE	TP	TYPE	TRUCK	ONLY	RANGE	TP	TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.07	0.1	0.0	0.1	C	HL	1.4	0.0	1.4	T	HL
1	2	6.14	0.2	0.0	0.2	C	HL	2.5	0.0	2.5	T	HL
1	3	9.22	0.2	0.0	0.2	C	HL	3.4	0.0	3.4	T	HL
1	4	12.29	0.4	0.0	0.4	C	HL	2.8	0.0	2.8	T	HL
1	5	15.36	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	6	18.43	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	7	21.51	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	8	24.58	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	9	27.65	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	10	30.72	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	11	33.80	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	12	36.87	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	13	39.94	0.4	0.0	0.4	C	HL	2.8	0.0	2.8	T	HL
1	14	43.01	0.2	0.0	0.2	C	HL	3.4	0.0	3.4	T	HL
1	15	46.09	0.2	0.0	0.2	C	HL	2.5	0.0	2.5	T	HL
1	16	49.16	0.1	0.0	0.1	C	HL	1.4	0.0	1.4	T	HL
1	17	52.23	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.17	-2.71	7.21	-0.02	-2.45	3.04
1	2	6.14	33.	33.	-0.32	-5.08	13.42	-0.04	-4.60	5.70
1	3	9.22	33.	33.	-0.45	-7.11	18.63	-0.05	-6.45	7.98
1	4	12.29	33.	33.	-0.46	-8.53	15.42	-0.05	-7.37	6.62
1	5	15.36	33.	33.	-0.52	-9.83	17.62	-0.06	-8.51	7.65
1	6	18.43	33.	33.	-0.57	-10.80	19.27	-0.07	-9.37	8.41
1	7	21.51	33.	33.	-0.60	-11.43	20.24	-0.07	-9.93	8.92
1	8	24.58	33.	33.	-0.60	-11.72	20.59	-0.08	-10.21	9.17
1	9	27.65	33.	33.	-0.60	-11.72	20.59	-0.08	-10.21	9.17
1	10	30.72	33.	33.	-0.60	-11.42	20.24	-0.07	-9.93	8.91
1	11	33.80	33.	33.	-0.57	-10.80	19.26	-0.07	-9.37	8.41
1	12	36.87	33.	33.	-0.52	-9.84	17.63	-0.06	-8.52	7.65
1	13	39.94	33.	33.	-0.46	-8.54	15.42	-0.05	-7.38	6.62
1	14	43.01	33.	33.	-0.45	-7.11	18.64	-0.05	-6.45	7.98
1	15	46.09	33.	33.	-0.32	-5.08	13.42	-0.04	-4.60	5.70
1	16	49.16	33.	33.	-0.17	-2.71	7.21	-0.02	-2.45	3.04
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

YIELD STRESS — TOTAL POSITIVE ,(ksi) — TOTAL NEGATIVE ,(ksi)

5969 Span 2-S16 Interior Strength CP.RES

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi) TOP	Fy (ksi) BOT	CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.22	-2.79	8.46	-0.02	-2.45	3.04
1	2	6.14	33.	33.	-0.41	-5.22	15.74	-0.04	-4.60	5.70
1	3	9.22	33.	33.	-0.56	-7.31	21.83	-0.05	-6.45	7.98
1	4	12.29	33.	33.	-0.58	-8.88	18.06	-0.05	-7.37	6.62
1	5	15.36	33.	33.	-0.66	-10.23	20.61	-0.06	-8.51	7.65
1	6	18.43	33.	33.	-0.72	-11.23	22.52	-0.07	-9.37	8.41
1	7	21.51	33.	33.	-0.75	-11.87	23.64	-0.07	-9.93	8.92
1	8	24.58	33.	33.	-0.76	-12.17	24.02	-0.08	-10.21	9.17
1	9	27.65	33.	33.	-0.76	-12.17	24.02	-0.08	-10.21	9.17
1	10	30.72	33.	33.	-0.75	-11.87	23.64	-0.07	-9.93	8.91
1	11	33.80	33.	33.	-0.72	-11.23	22.52	-0.07	-9.37	8.41
1	12	36.87	33.	33.	-0.66	-10.24	20.62	-0.06	-8.52	7.65
1	13	39.94	33.	33.	-0.58	-8.89	18.06	-0.05	-7.38	6.62
1	14	43.01	33.	33.	-0.56	-7.31	21.83	-0.05	-6.45	7.98
1	15	46.09	33.	33.	-0.41	-5.23	15.74	-0.04	-4.60	5.70
1	16	49.16	33.	33.	-0.22	-2.79	8.46	-0.02	-2.45	3.04
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.30	-3.56	11.23	-0.03	-3.10	3.93
1	2	6.14	33.	33.	-0.55	-6.66	20.89	-0.05	-5.83	7.37
1	3	9.22	33.	33.	-0.76	-9.32	28.97	-0.07	-8.16	10.32
1	4	12.29	33.	33.	-0.79	-11.38	23.97	-0.08	-9.35	8.57
1	5	15.36	33.	33.	-0.89	-13.10	27.35	-0.09	-10.80	9.90
1	6	18.43	33.	33.	-0.97	-14.39	29.88	-0.10	-11.88	10.89
1	7	21.51	33.	33.	-1.02	-15.21	31.36	-0.10	-12.59	11.54
1	8	24.58	33.	33.	-1.03	-15.58	31.86	-0.11	-12.94	11.87
1	9	27.65	33.	33.	-1.03	-15.58	31.86	-0.11	-12.94	11.86
1	10	30.72	33.	33.	-1.02	-15.20	31.36	-0.10	-12.59	11.54
1	11	33.80	33.	33.	-0.97	-14.39	29.88	-0.10	-11.88	10.89
1	12	36.87	33.	33.	-0.89	-13.11	27.36	-0.09	-10.81	9.90
1	13	39.94	33.	33.	-0.79	-11.39	23.97	-0.08	-9.35	8.57
1	14	43.01	33.	33.	-0.76	-9.33	28.97	-0.07	-8.17	10.33
1	15	46.09	33.	33.	-0.55	-6.67	20.89	-0.05	-5.83	7.37
1	16	49.16	33.	33.	-0.30	-3.56	11.23	-0.03	-3.11	3.93
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.28	-2.62	9.90	-0.01	-2.17	2.60
1	2	6.14	33.	33.	-0.53	-4.91	18.39	-0.03	-4.07	4.88
1	3	9.22	33.	33.	-0.73	-6.86	25.47	-0.04	-5.70	6.83
1	4	12.29	33.	33.	-0.75	-8.54	21.06	-0.04	-6.50	5.66
1	5	15.36	33.	33.	-0.85	-9.82	24.00	-0.05	-7.51	6.54
1	6	18.43	33.	33.	-0.93	-10.78	26.19	-0.05	-8.27	7.20
1	7	21.51	33.	33.	-0.97	-11.38	27.45	-0.05	-8.76	7.63
1	8	24.58	33.	33.	-0.98	-11.65	27.84	-0.06	-9.00	7.84
1	9	27.65	33.	33.	-0.98	-11.65	27.84	-0.06	-9.00	7.84
1	10	30.72	33.	33.	-0.97	-11.37	27.44	-0.05	-8.76	7.63
1	11	33.80	33.	33.	-0.93	-10.77	26.19	-0.05	-8.26	7.20
1	12	36.87	33.	33.	-0.85	-9.82	24.00	-0.05	-7.52	6.55
1	13	39.94	33.	33.	-0.75	-8.54	21.07	-0.04	-6.51	5.67
1	14	43.01	33.	33.	-0.73	-6.86	25.48	-0.04	-5.70	6.84
1	15	46.09	33.	33.	-0.53	-4.91	18.40	-0.03	-4.07	4.88
1	16	49.16	33.	33.	-0.28	-2.62	9.90	-0.01	-2.17	2.60
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.24	-3.45	9.56	-0.03	-3.10	3.93
1	2	6.14	33.	33.	-0.44	-6.47	17.80	-0.05	-5.83	7.37
1	3	9.22	33.	33.	-0.60	-9.06	24.70	-0.07	-8.16	10.32
1	4	12.29	33.	33.	-0.63	-10.91	20.45	-0.08	-9.35	8.57
1	5	15.36	33.	33.	-0.71	-12.57	23.36	-0.09	-10.80	9.90
1	6	18.43	33.	33.	-0.77	-13.82	25.54	-0.10	-11.88	10.89
1	7	21.51	33.	33.	-0.81	-14.61	26.83	-0.10	-12.59	11.54
1	8	24.58	33.	33.	-0.82	-14.98	27.29	-0.11	-12.94	11.87
1	9	27.65	33.	33.	-0.82	-14.98	27.29	-0.11	-12.94	11.86
1	10	30.72	33.	33.	-0.81	-14.61	26.83	-0.10	-12.59	11.54
1	11	33.80	33.	33.	-0.77	-13.81	25.54	-0.10	-11.88	10.89
1	12	36.87	33.	33.	-0.71	-12.58	23.37	-0.09	-10.81	9.90
1	13	39.94	33.	33.	-0.63	-10.92	20.45	-0.08	-9.35	8.57
1	14	43.01	33.	33.	-0.60	-9.06	24.71	-0.07	-8.17	10.33
1	15	46.09	33.	33.	-0.44	-6.48	17.80	-0.05	-5.83	7.37
1	16	49.16	33.	33.	-0.24	-3.46	9.56	-0.03	-3.11	3.93
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.07	33.	33.	-0.22	-2.52	8.23	-0.01	-2.17	2.60
1	2	6.14	33.	33.	-0.41	-4.71	15.30	-0.03	-4.07	4.88
1	3	9.22	33.	33.	-0.57	-6.59	21.21	-0.04	-5.70	6.83
1	4	12.29	33.	33.	-0.59	-8.07	17.54	-0.04	-6.50	5.66

1	5	15.36	33.	33.	-0.67	-9.29	20.01	-0.05	-7.51	6.54
1	6	18.43	33.	33.	-0.73	-10.20	21.85	-0.05	-8.27	7.20
1	7	21.51	33.	33.	-0.76	-10.78	22.92	-0.05	-8.76	7.63
1	8	24.58	33.	33.	-0.77	-11.04	23.27	-0.06	-9.00	7.84
1	9	27.65	33.	33.	-0.77	-11.04	23.27	-0.06	-9.00	7.84
1	10	30.72	33.	33.	-0.76	-10.78	22.91	-0.05	-8.76	7.63
1	11	33.80	33.	33.	-0.73	-10.20	21.85	-0.05	-8.26	7.20
1	12	36.87	33.	33.	-0.67	-9.30	20.01	-0.05	-7.52	6.55
1	13	39.94	33.	33.	-0.59	-8.08	17.55	-0.04	-6.51	5.67
1	14	43.01	33.	33.	-0.57	-6.60	21.22	-0.04	-5.70	6.84
1	15	46.09	33.	33.	-0.41	-4.72	15.31	-0.03	-4.07	4.88
1	16	49.16	33.	33.	-0.22	-2.52	8.23	-0.01	-2.17	2.60
1	17	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	9.61	179.	-6.0	93.	-0.7	499.	-0.7	0.	0.0	-7.33
											0.68
	LB			6.0		2.2		10.9		0.0	19.17
											10.94
	RT			-5.4		-0.7		-1.0		0.0	-7.12
											0.99
	RB			4.0		1.5		7.5		0.0	12.97
											7.46
1	LT	42.61	179.	-5.5	93.	-0.7	499.	-1.0	0.	0.0	-7.12
											0.99
	LB			4.0		1.5		7.5		0.0	12.97
											7.46
	RT			-6.0		-0.7		-0.7		0.0	-7.33
											0.68
	RB			6.0		2.2		10.9		0.0	19.18
											10.94

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	9.61	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	33.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.7500
3	9.61	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   52.2
                                                                LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])              1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])              1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------------

  1   9.61  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

  2  33.00  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

  3   9.61  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                          [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  1   3.07  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  2   6.14  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  3   9.22  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  4  12.29  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  5  15.36  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  6  18.43  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  7  21.51  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  8  24.58  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  9  27.65  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 10  30.72  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 11  33.80  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 12  36.87  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 13  39.94  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 14  43.01  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
```

```
  1 15  46.09  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1 16  49.16  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1 17  52.23  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0


-------------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE  1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
                 ********************************

 SP IN D FROM  Lo Co       Web                   Web
 NO NO L SUPT  ng mp depth thick     2Dc/tw      Category
              [1]                  [2]   [3]
-------------------------------------------------------
  1  0   0.00  0  1  31.42 0.550  57.13 168.97    2
  1  1   3.07  0  1  31.42 0.550  57.13 168.97    2
  1  2   6.14  0  1  31.42 0.550  57.13 168.97    2
  1  3   9.22  0  1  31.42 0.550  57.13 168.97    2
  1  4  12.29  0  1  31.42 0.550  68.10 168.97    2
  1  5  15.36  0  1  31.42 0.550  68.10 168.97    2
  1  6  18.43  0  1  31.42 0.550  68.10 168.97    2
  1  7  21.51  0  1  31.42 0.550  68.10 168.97    2
  1  8  24.58  0  1  31.42 0.550  68.10 168.97    2
  1  9  27.65  0  1  31.42 0.550  68.10 168.97    2
  1 10  30.72  0  1  31.42 0.550  68.10 168.97    2
  1 11  33.80  0  1  31.42 0.550  68.10 168.97    2
  1 12  36.87  0  1  31.42 0.550  68.10 168.97    2
  1 13  39.94  0  1  31.42 0.550  68.10 168.97    2
  1 14  43.01  0  1  31.42 0.550  57.13 168.97    2
  1 15  46.09  0  1  31.42 0.550  57.13 168.97    2
  1 16  49.16  0  1  31.42 0.550  57.13 168.97    2
  1 17  52.23  0  1  31.42 0.550  57.13 168.97    2


-------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************
```

SP NO	D NO	FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	1	3.07	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	2	6.14	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	3	9.22	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	4	12.29	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	5	15.36	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	6	18.43	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	7	21.51	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	8	24.58	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	9	27.65	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	10	30.72	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	11	33.80	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	12	36.87	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	13	39.94	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	14	43.01	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	15	46.09	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	16	49.16	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	17	52.23	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	17.5	0.000	12.126	8.297	4.469	1.000
			0.000	0.000	0.000	0.000	1.000
2	17.5	36.3	11.573	12.126	13.604	15.082	1.000
			0.000	0.000	0.000	0.000	1.000
3	36.3	52.2	0.000	11.573	7.692	3.811	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length

from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0		0.00	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	1		3.07	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	2		6.14	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	3		9.22	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	4		12.29	1	7.77	11.26	19.84	0	17.45	6.92	25.99	2	2
1	5		15.36	1	7.77	11.26	19.84	0	17.45	6.92	25.99	2	2
1	6		18.43	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	7		21.51	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	8		24.58	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	9		27.65	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	10		30.72	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	11		33.80	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	12		36.87	1	7.77	11.26	19.84	0	15.94	6.92	25.99	2	2
1	13		39.94	1	7.77	11.26	19.84	0	15.94	6.92	25.99	2	2
1	14		43.01	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	15		46.09	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	16		49.16	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	17		52.23	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)

1	0	0.00	1	1.000	1.000	1.00	33.00	27.75	27.75
1	1	3.07	1	1.000	1.000	1.00	33.00	27.75	27.75
1	2	6.14	1	1.000	1.000	1.00	33.00	27.75	27.75
1	3	9.22	1	1.000	1.000	1.00	33.00	27.75	27.75
1	4	12.29	1	1.000	1.000	1.00	33.00	27.54	27.54
1	5	15.36	1	1.000	1.000	1.00	33.00	27.54	27.54
1	6	18.43	1	1.000	1.000	1.00	33.00	26.81	26.81
1	7	21.51	1	1.000	1.000	1.00	33.00	26.81	26.81
1	8	24.58	1	1.000	1.000	1.00	33.00	26.81	26.81
1	9	27.65	1	1.000	1.000	1.00	33.00	26.81	26.81
1	10	30.72	1	1.000	1.000	1.00	33.00	26.81	26.81
1	11	33.80	1	1.000	1.000	1.00	33.00	26.81	26.81
1	12	36.87	1	1.000	1.000	1.00	33.00	28.32	28.32
1	13	39.94	1	1.000	1.000	1.00	33.00	28.32	28.32
1	14	43.01	1	1.000	1.000	1.00	33.00	28.51	28.51
1	15	46.09	1	1.000	1.000	1.00	33.00	28.51	28.51
1	16	49.16	1	1.000	1.000	1.00	33.00	28.51	28.51
1	17	52.23	1	1.000	1.000	1.44	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	4.70	17.35
1	1		3.07	1	4.70	17.35
1	2		6.14	1	4.70	17.35
1	3		9.22	1	4.70	17.35
1	4		12.29	1	5.72	17.67
1	5		15.36	1	5.72	17.67
1	6		18.43	1	5.72	17.67
1	7		21.51	1	5.72	17.67
1	8		24.58	1	5.72	17.67
1	9		27.65	1	5.72	17.67
1	10		30.72	1	5.72	17.67
1	11		33.80	1	5.72	17.67
1	12		36.87	1	5.72	17.67
1	13		39.94	1	5.72	17.67
1	14		43.01	1	4.70	17.35
1	15		46.09	1	4.70	17.35
1	16		49.16	1	4.70	17.35
1	17		52.23	1	4.70	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Category Non-Comp.	Category Comp.
1	0		0.00	1	0	2	0		2	0
1	1		3.07	1	0	2	0		2	0

1	2	6.14	1	0	2	0	2	0
1	3	9.22	1	0	2	0	2	0
1	4	12.29	1	0	2	0	2	0
1	5	15.36	1	0	2	0	2	0
1	6	18.43	1	0	2	0	2	0
1	7	21.51	1	0	2	0	2	0
1	8	24.58	1	0	2	0	2	0
1	9	27.65	1	0	2	0	2	0
1	10	30.72	1	0	2	0	2	0
1	11	33.80	1	0	2	0	2	0
1	12	36.87	1	0	2	0	2	0
1	13	39.94	1	0	2	0	2	0
1	14	43.01	1	0	2	0	2	0
1	15	46.09	1	0	2	0	2	0
1	16	49.16	1	0	2	0	2	0
1	17	52.23	1	0	2	0	2	0

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.7	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.07	0.0	19.8	3.3	33.0	3.3	33.0	3.3	27.7	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	2		6.14	0.0	19.8	6.2	33.0	6.2	33.0	6.2	27.7	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	3		9.22	0.0	19.8	8.7	33.0	8.7	33.0	8.7	27.7	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	4		12.29	0.0	19.8	9.8	33.0	9.8	33.0	9.8	27.5	0
				0.0	-	7.2	-	7.2	33.0	-	-	0
1	5		15.36	0.0	19.8	11.4	33.0	11.4	33.0	11.4	27.5	0
				0.0	-	8.3	-	8.3	33.0	-	-	0

1	6	18.43	0.0	19.8	12.5	33.0	12.5	33.0	12.5	26.8	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	7	21.51	0.0	19.8	13.2	33.0	13.2	33.0	13.2	26.8	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	8	24.58	0.0	19.8	13.6	33.0	13.6	33.0	13.6	26.8	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	9	27.65	0.0	19.8	13.6	33.0	13.6	33.0	13.6	26.8	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	10	30.72	0.0	19.8	13.2	33.0	13.2	33.0	13.2	26.8	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	11	33.80	0.0	19.8	12.5	33.0	12.5	33.0	12.5	26.8	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	12	36.87	0.0	19.8	11.4	33.0	11.4	33.0	11.4	28.3	0
			0.0	-	8.3	-	8.3	33.0	-	-	0
1	13	39.94	0.0	19.8	9.8	33.0	9.8	33.0	9.8	28.3	0
			0.0	-	7.2	-	7.2	33.0	-	-	0
1	14	43.01	0.0	19.8	8.7	33.0	8.7	33.0	8.7	28.5	0
			0.0	-	8.7	-	8.7	33.0	-	-	0
1	15	46.09	0.0	19.8	6.2	33.0	6.2	33.0	6.2	28.5	0
			0.0	-	6.2	-	6.2	33.0	-	-	0
1	16	49.16	0.0	19.8	3.3	33.0	3.3	33.0	3.3	28.5	0
			0.0	-	3.3	-	3.3	33.0	-	-	0
1	17	52.23	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.07	0.000	0.100	0.100	0.119	0.119	4
					0.100		0.100	3
1	2	6.14	0.000	0.188	0.188	0.224	0.224	4
					0.188		0.188	3
1	3	9.22	0.000	0.264	0.264	0.313	0.313	4
					0.264		0.264	3
1	4	12.29	0.000	0.298	0.298	0.357	0.357	4
					0.218		0.218	3
1	5	15.36	0.000	0.344	0.344	0.412	0.412	4
					0.251		0.251	3
1	6	18.43	0.000	0.379	0.379	0.466	0.466	4
					0.277		0.277	3
1	7	21.51	0.000	0.401	0.401	0.494	0.494	4
					0.293		0.293	3
1	8	24.58	0.000	0.412	0.412	0.507	0.507	4
					0.301		0.301	3
1	9	27.65	0.000	0.412	0.412	0.507	0.507	4

				0.301		0.301	3
1 10	30.72	0.000	0.401	0.401	0.493	0.493	4
				0.293		0.293	3
1 11	33.80	0.000	0.378	0.378	0.466	0.466	4
				0.277		0.277	3
1 12	36.87	0.000	0.344	0.344	0.401	0.401	4
				0.252		0.252	3
1 13	39.94	0.000	0.298	0.298	0.347	0.347	4
				0.218		0.218	3
1 14	43.01	0.000	0.264	0.264	0.305	0.305	4
				0.264		0.264	3
1 15	46.09	0.000	0.188	0.188	0.218	0.218	4
				0.188		0.188	3
1 16	49.16	0.000	0.100	0.100	0.116	0.116	4
				0.100		0.100	3
1 17	52.23	0.000	0.000	0.000	0.000	0.000	1
				0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	1		3.07	0	463.9	2103.1	-	-	0.0	19.8	-	-	0--0
1	2		6.14	0	861.9	2103.1	-	-	0.0	19.8	-	-	0--0
1	3		9.22	0	1194.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	4		12.29	0	1456.1	2803.1	-	-	0.0	19.8	-	-	0--0
1	5		15.36	0	1659.6	2803.1	-	-	0.0	19.8	-	-	0--0
1	6		18.43	0	1811.7	2803.1	-	-	0.0	19.8	-	-	0--0
1	7		21.51	0	1899.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	8		24.58	0	1927.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	9		27.65	0	1927.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	10		30.72	0	1899.3	2803.1	-	-	0.0	19.8	-	-	0--0
1	11		33.80	0	1811.6	2803.1	-	-	0.0	19.8	-	-	0--0
1	12		36.87	0	1659.9	2803.1	-	-	0.0	19.8	-	-	0--0
1	13		39.94	0	1456.3	2803.1	-	-	0.0	19.8	-	-	0--0
1	14		43.01	0	1194.2	2103.1	-	-	0.0	19.8	-	-	0--0
1	15		46.09	0	862.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	16		49.16	0	463.9	2103.1	-	-	0.0	19.8	-	-	0--0
1	17		52.23	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)

[4] = (PHI)f * Fnt for non-compact tension flange
of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections
in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in
positive moment region, the 1st line is for top
flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	3.07	0	0.221				0.221	1
1	2	6.14	0	0.410				0.410	1
1	3	9.22	0	0.568				0.568	1
1	4	12.29	0	0.519				0.519	1
1	5	15.36	0	0.592				0.592	1
1	6	18.43	0	0.646				0.646	1
1	7	21.51	0	0.678				0.678	1
1	8	24.58	0	0.688				0.688	1
1	9	27.65	0	0.688				0.688	1
1	10	30.72	0	0.678				0.678	1
1	11	33.80	0	0.646				0.646	1
1	12	36.87	0	0.592				0.592	1
1	13	39.94	0	0.520				0.520	1
1	14	43.01	0	0.568				0.568	1
1	15	46.09	0	0.410				0.410	1
1	16	49.16	0	0.221				0.221	1
1	17	52.23	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange
of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections
in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.07	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.14	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	9.22	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	12.29	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	15.36	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	18.43	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	21.51	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	24.58	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	27.65	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	30.72	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	33.80	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	36.87	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	39.94	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	43.01	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	46.09	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	49.16	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	17	52.23	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-

field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	191.7	<	330.8	0	0
1	1	3.07	0	174.8	<	330.8	0	0
1	2	6.14	0	158.2	<	330.8	0	0
1	3	9.22	0	141.6	<	330.8	0	0
1	4	12.29	0	125.3	<	330.8	0	0
1	5	15.36	0	109.2	<	330.8	0	0
1	6	18.43	0	92.7	<	330.8	0	0
1	7	21.51	0	77.0	<	330.8	0	0
1	8	24.58	0	61.7	<	330.8	0	0
1	9	27.65	0	61.7	<	330.8	0	0
1	10	30.72	0	77.1	<	330.8	0	0
1	11	33.80	0	92.8	<	330.8	0	0
1	12	36.87	0	109.2	<	330.8	0	0
1	13	39.94	0	125.3	<	330.8	0	0
1	14	43.01	0	141.7	<	330.8	0	0
1	15	46.09	0	158.2	<	330.8	0	0
1	16	49.16	0	174.9	<	330.8	0	0
1	17	52.23	0	191.7	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	191.71	330.76	0	
1	1	3.07	33.	174.85	330.76	0	
1	2	6.14	33.	158.16	330.76	0	
1	3	9.22	33.	141.65	330.76	0	
1	4	12.29	33.	125.31	330.76	0	
1	5	15.36	33.	109.16	330.76	0	
1	6	18.43	33.	92.75	330.76	0	
1	7	21.51	33.	77.04	330.76	0	
1	8	24.58	33.	61.71	330.76	0	
1	9	27.65	33.	61.73	330.76	0	
1	10	30.72	33.	77.06	330.76	0	
1	11	33.80	33.	92.77	330.76	0	
1	12	36.87	33.	109.18	330.76	0	
1	13	39.94	33.	125.33	330.76	0	
1	14	43.01	33.	141.67	330.76	0	
1	15	46.09	33.	158.18	330.76	0	
1	16	49.16	33.	174.87	330.76	0	
1	17	52.23	33.	191.73	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	BOT. FLANGE [1] ff+fl/2 (ksi)	[2] (ksi)	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3] TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	- 00-
1	1	3.07	2.8	-	31.4	8.5	31.4	-	- 00-
1	2	6.14	5.2	-	31.4	15.7	31.4	-	- 00-
1	3	9.22	7.3	-	31.4	21.8	31.4	-	- 00-
1	4	12.29	8.9	-	31.4	18.1	31.4	-	- 00-
1	5	15.36	10.2	-	31.4	20.6	31.4	-	- 00-
1	6	18.43	11.2	-	31.4	22.5	31.4	-	- 00-
1	7	21.51	11.9	-	31.4	23.6	31.4	-	- 00-
1	8	24.58	12.2	-	31.4	24.0	31.4	-	- 00-
1	9	27.65	12.2	-	31.4	24.0	31.4	-	- 00-
1	10	30.72	11.9	-	31.4	23.6	31.4	-	- 00-
1	11	33.80	11.2	-	31.4	22.5	31.4	-	- 00-
1	12	36.87	10.2	-	31.4	20.6	31.4	-	- 00-
1	13	39.94	8.9	-	31.4	18.1	31.4	-	- 00-
1	14	43.01	7.3	-	31.4	21.8	31.4	-	- 00-

```
 1 15  46.09   5.2    -     31.4    15.7  31.4   -     -   00-
 1 16  49.16   2.8    -     31.4     8.5  31.4   -     -   00-
 1 17  52.23   0.0    -     31.4     0.0  31.4   -     -   00-


    ------------------------------------------------------------
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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ---------------------------- ----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.07    TR     -0.1 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
  1  2   6.14    TR     -0.2 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
  1  3   9.22    TR     -0.3 A B B^C^C D E E^  TR      5.1 A B B^C^C D
  1      9.61  SCG
  1  4  12.29    TR     -0.6 A B B^C^C D E E^  TR      4.2 A B B^C^C D E
  1  5  15.36    TR     -0.6 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1  6  18.43    TR     -0.6 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1  7  21.51    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1  8  24.58    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
  1  9  27.65    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
  1 10  30.72    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1 11  33.80    TR     -0.6 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1 12  36.87    TR     -0.6 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1 13  39.94    TR     -0.6 A B B^C^C D E E^  TR      4.2 A B B^C^C D E
  1     42.61  SCG
  1 14  43.01    TR     -0.3 A B B^C^C D E E^  TR      5.1 A B B^C^C D
  1 15  46.09    TR     -0.2 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
  1 16  49.16    TR     -0.1 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
  1 17  52.23    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
```

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0		0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1		3.07	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2		6.14	TR	-0.1	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	3		9.22	TR	-0.2	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1			9.61	SCG					
1	4		12.29	TR	-0.3	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	5		15.36	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	6		18.43	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	7		21.51	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	8		24.58	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	9		27.65	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	10		30.72	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	11		33.80	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	12		36.87	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	13		39.94	TR	-0.3	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1			42.61	SCG					
1	14		43.01	TR	-0.2	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	15		46.09	TR	-0.1	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	16		49.16	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	17		52.23	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor. Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input [5], input	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	49.6	.29821E-01	1.5	TR	3.09	3	6.28
1	1	3.07	47.0	.29821E-01	1.4	TR	3.09	3	6.62
1	2	6.14	44.6	.29821E-01	1.3	TR	3.09	3	6.98
1	3	9.22	42.3	.29821E-01	1.3	TR	3.09	3	7.36
1	4	12.29	40.3	.27580E-01	1.1	TR	3.09	3	8.35
1	5	15.36	38.5	.27580E-01	1.1	TR	3.09	3	8.74
1	6	18.43	36.4	.27580E-01	1.0	TR	3.09	3	9.24
1	7	21.51	36.2	.27580E-01	1.0	TR	3.09	3	9.29
1	8	24.58	37.6	.27580E-01	1.0	TR	3.09	3	8.96
1	9	27.65	37.6	.27580E-01	1.0	TR	3.09	3	8.96
1	10	30.72	36.2	.27580E-01	1.0	TR	3.09	3	9.29
1	11	33.80	36.4	.27580E-01	1.0	TR	3.09	3	9.24
1	12	36.87	38.5	.27580E-01	1.1	TR	3.09	3	8.74
1	13	39.94	40.3	.27580E-01	1.1	TR	3.09	3	8.35
1	14	43.01	42.3	.29821E-01	1.3	TR	3.09	3	7.36
1	15	46.09	44.6	.29821E-01	1.3	TR	3.09	3	6.98
1	16	49.16	47.0	.29821E-01	1.4	TR	3.09	3	6.62
1	17	52.23	49.6	.29821E-01	1.5	TR	3.09	3	6.28

Please read NOTE on the following page

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon
AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	<--Input [5], input	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	24.8	.29821E-01	0.7	TR	1.36	3	5.54
1	1	3.07	23.5	.29821E-01	0.7	TR	1.36	3	5.84
1	2	6.14	22.3	.29821E-01	0.7	TR	1.36	3	6.15
1	3	9.22	21.1	.29821E-01	0.6	TR	1.36	3	6.49
1	4	12.29	20.1	.27580E-01	0.6	TR	1.36	3	7.36
1	5	15.36	19.3	.27580E-01	0.5	TR	1.36	3	7.71
1	6	18.43	18.2	.27580E-01	0.5	TR	1.36	3	8.14
1	7	21.51	18.1	.27580E-01	0.5	TR	1.36	3	8.19
1	8	24.58	18.8	.27580E-01	0.5	TR	1.36	3	7.90
1	9	27.65	18.8	.27580E-01	0.5	TR	1.36	3	7.90
1	10	30.72	18.1	.27580E-01	0.5	TR	1.36	3	8.19
1	11	33.80	18.2	.27580E-01	0.5	TR	1.36	3	8.14
1	12	36.87	19.3	.27580E-01	0.5	TR	1.36	3	7.71
1	13	39.94	20.1	.27580E-01	0.6	TR	1.36	3	7.36
1	14	43.01	21.1	.29821E-01	0.6	TR	1.36	3	6.49
1	15	46.09	22.3	.29821E-01	0.7	TR	1.36	3	6.15
1	16	49.16	23.5	.29821E-01	0.7	TR	1.36	3	5.84
1	17	52.23	24.8	.29821E-01	0.7	TR	1.36	3	5.54

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken

as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D FROM L (ft)	SUPT 1=POS. 0=NEG.	MOMENT REGION LRFD 6.10.10.4.2	P2p,1n (k)	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	SEE NOTE N2	STATUS BLANK=OK CHECK=**
1	8	24.58	1	1379.4	1626.6			150	>	72		
1	17	52.23	1	1379.4	1626.6			150	>	72		

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where

N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment

N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support

P = total nominal horizontal shear force

Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)

Qn = nominal shear resistance

(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
or
P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	24.579	6.000
1	24.579	52.230	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	TCR
1	0.0	52.2	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.07	2103.1	65.9	34.4	190.3	0.0	7.68	HL-93	5.92	0
1	2	6.14	2103.1	123.7	64.4	352.3	0.0	3.91	HL-93	3.01	0
1	3	9.22	2103.1	173.3	90.2	485.8	0.0	2.68	HL-93	2.07	0
1	4	12.29	2803.1	214.8	111.7	588.6	0.0	2.99	HL-93	2.31	0
1	5	15.36	2803.1	248.2	128.8	667.2	0.0	2.57	HL-93	1.98	0
1	6	18.43	2803.1	273.1	141.7	726.0	0.0	2.31	HL-93	1.78	0
1	7	21.51	2803.1	289.4	150.3	757.5	0.0	2.18	HL-93	1.68	0
1	8	24.58	2803.1	297.5	154.6	764.3	0.0	2.14	HL-93	1.65	0
1	9	27.65	2803.1	297.4	154.6	764.3	0.0	2.14	HL-93	1.65	0
1	10	30.72	2803.1	289.3	150.3	757.5	0.0	2.18	HL-93	1.68	0
1	11	33.80	2803.1	273.0	141.7	725.9	0.0	2.31	HL-93	1.78	0
1	12	36.87	2803.1	248.4	128.8	667.2	0.0	2.57	HL-93	1.98	0
1	13	39.94	2803.1	215.0	111.7	588.6	0.0	2.99	HL-93	2.31	0
1	14	43.01	2103.1	173.4	90.2	485.8	0.0	2.68	HL-93	2.07	0
1	15	46.09	2103.1	123.8	64.4	352.3	0.0	3.91	HL-93	3.01	0
1	16	49.16	2103.1	65.9	34.4	190.3	0.0	7.68	HL-93	5.92	0
1	17	52.23	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS TOP;ksi	UNFACTORED DEAD LOAD STRESS BOT	UNFACTORED L+I STRESS MAX. POS. STEEL TOP;ksi	UNFACTORED L+I STRESS MAX. POS. STEEL BOT	UNFACTORED L+I STRESS MAX. NEG. STEEL TOP;ksi	UNFACTORED L+I STRESS MAX. NEG. STEEL BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.07	31.4	-2.45	3.04	-0.26	4.17	0.00	0.00	6.79	HL-93	5.22
1	2	6.14	31.4	-4.60	5.70	-0.48	7.72	0.00	0.00	3.32	HL-93	2.56
1	3	9.22	31.4	-6.45	7.98	-0.66	10.65	0.00	0.00	2.19	HL-93	1.69
1	4	12.29	31.4	-7.37	6.62	-1.16	8.80	0.00	0.00	2.81	HL-93	2.16
1	5	15.36	31.4	-8.51	7.65	-1.32	9.97	0.00	0.00	2.38	HL-93	1.83
1	6	18.43	31.4	-9.37	8.41	-1.43	10.85	0.00	0.00	2.11	HL-93	1.63
1	7	21.51	31.4	-9.93	8.92	-1.50	11.32	0.00	0.00	1.98	HL-93	1.52
1	8	24.58	31.4	-10.21	9.17	-1.51	11.43	0.00	0.00	1.94	HL-93	1.49
1	9	27.65	31.4	-10.21	9.17	-1.51	11.43	0.00	0.00	1.94	HL-93	1.49
1	10	30.72	31.4	-9.93	8.91	-1.50	11.32	0.00	0.00	1.98	HL-93	1.52
1	11	33.80	31.4	-9.37	8.41	-1.43	10.85	0.00	0.00	2.11	HL-93	1.63
1	12	36.87	31.4	-8.52	7.65	-1.32	9.97	0.00	0.00	2.38	HL-93	1.83
1	13	39.94	31.4	-7.38	6.62	-1.16	8.80	0.00	0.00	2.81	HL-93	2.16
1	14	43.01	31.4	-6.45	7.98	-0.66	10.65	0.00	0.00	2.19	HL-93	1.69
1	15	46.09	31.4	-4.60	5.70	-0.48	7.72	0.00	0.00	3.32	HL-93	2.55
1	16	49.16	31.4	-2.45	3.04	-0.26	4.17	0.00	0.00	6.78	HL-93	5.22
1	17	52.23	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	52.23	24.58	STRENGTH L.S.	2.14	1.65	HL-93
1	52.23	24.58	SERVICEBILITY	1.94	1.49	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.941
Inventory= 1.493

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	52.23	24.58	STRENGTH L.S.	3.08	719.48
1	52.23	24.58	SERVICEBILITY	2.64	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.645

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	22.8	11.9	83.7	0.0	2.53	HL-93	1.95
1	1	3.07	330.8	20.1	10.5	77.1	-2.2	2.79	HL-93	2.16
1	2	6.14	330.8	17.5	9.1	70.6	-4.8	3.11	HL-93	2.40
1	3	9.22	330.8	14.8	7.7	64.2	-8.5	3.48	HL-93	2.68
1	4	12.29	330.8	12.2	6.3	57.8	-12.3	3.93	HL-93	3.03
1	5	15.36	330.8	9.5	4.9	51.6	-16.2	4.48	HL-93	3.45
1	6	18.43	330.8	6.6	3.5	45.5	-20.2	5.17	HL-93	3.99

1	7	21.51	330.8	4.0	2.1	39.5	-24.3	6.05	HL-93	4.67
1	8	24.58	330.8	1.3	0.7	33.8	-28.5	7.20	HL-93	5.55
1	9	27.65	330.8	-1.3	-0.7	28.5	-33.8	7.20	HL-93	5.55
1	10	30.72	330.8	-4.0	-2.1	24.3	-39.5	6.05	HL-93	4.67
1	11	33.80	330.8	-6.6	-3.5	20.2	-45.5	5.17	HL-93	3.99
1	12	36.87	330.8	-9.6	-4.9	16.2	-51.6	4.48	HL-93	3.45
1	13	39.94	330.8	-12.2	-6.3	12.3	-57.8	3.93	HL-93	3.03
1	14	43.01	330.8	-14.8	-7.7	8.5	-64.2	3.48	HL-93	2.68
1	15	46.09	330.8	-17.5	-9.1	4.8	-70.6	3.11	HL-93	2.40
1	16	49.16	330.8	-20.1	-10.5	2.2	-77.1	2.79	HL-93	2.16
1	17	52.23	330.8	-22.8	-11.9	0.0	-83.7	2.53	HL-93	1.95

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	52.23	52.23	STRENGTH L.S.	2.53	1.95	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.527
Inventory= 1.949

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	52.23	52.23	STRENGTH L.S.	3.13	91.23

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.130

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	26.11	3 0.425	-0.26	HL-93	3.02
		4 0.433	-0.26		2.96

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span2 S16 Interior Fat & Mid M	8/20/13

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT DESIGN TYPE OPTION	PROGRAM FLOW CONTROL
1	20	1	AASHTO	2010	0	2 3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
52.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	STANDARD SECTN DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.7500		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	9.61				33.	33.	33.
2	2	2	33.00				33.	33.	33.
3	1	1	9.61				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

	OPTIONAL CALCULATION OF FACTOR					LOADING TYPES	OPTIONAL DISTRIBUTION FACTORS	
SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied
to the indicated loading type.

1 = The special distribution factor defined is applied to
the indicated loading type of calculation of all
moment, shear and deflection.

2 = The special distribution factor defined is only applied
to the loading type for calculating moment.

3 = The special distribution factor defined is only applied
to the loading type for calculating shear.

4 = The special distribution factor defined is only applied
to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	POUR NO	IDENTIFICATION DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	MODULAR RATIO N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	52.23

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those
of normal weight concrete (W=145 pcf) of various design strength shall
be as follows:

fc' = unit ultimate compressive strength of concrete as determined
by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

5969 Span 2-S16 Interior Fatigue and Mid M.RES

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	17.45
2	2	Diaphragm	0.000	0.00	0.00	0.29	36.29
3	1	Railing	0.087	0.00	52.23	0.00	0.00
4	1	Curb	0.076	0.00	52.23	0.00	0.00
5	0	Pavement	0.292	0.00	52.23	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN.	DIA. SEC (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi) (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	17.45	33.0	33.0	17.45
17.45	36.29	33.0	33.0	18.84
36.29	52.23	33.0	33.0	15.94

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13 GAMMA for Load DC maximum = 1.25

14 GAMMA for Load DC minimum = 0.90

15 GAMMA for Load DW minimum = 1.50

16 GAMMA for Load DW minimum = 0.65

17 GAMMA for LL Load Strength I = 1.75

18 GAMMA for LL Load Strength II = 1.35

19 GAMMA for LL Load Service I = 1.00

20 GAMMA for LL Load Service II = 1.30

21 GAMMA for LL Load Fatigue = 0.75

22 ETA for Service Limit State = 1.00

23 ETA for Strength Limit State = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.727	0.1342	0.8612

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	52.23
1	DC2	0.087	0.000	52.230
2	DC2	0.076	0.000	52.230

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1268.2	47.0	190.2	42.1	232.29

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.714	0.714	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.61	5900.0	32.9	16.45	358.7	358.7
1	2	5.22	5900.0	32.9	16.45	358.7	358.7
1	3	7.83	5900.0	32.9	16.45	358.7	358.7
1	4	10.45	7650.6	32.9	14.21	538.4	393.5
1	5	13.06	7650.6	32.9	14.21	538.4	393.5
1	6	15.67	7650.6	32.9	14.21	538.4	393.5
1	7	18.28	7650.6	32.9	14.21	538.4	393.5
1	8	20.89	7650.6	32.9	14.21	538.4	393.5
1	9	23.50	7650.6	32.9	14.21	538.4	393.5
1	10	26.11	7650.6	32.9	14.21	538.4	393.5
1	11	28.73	7650.6	32.9	14.21	538.4	393.5
1	12	31.34	7650.6	32.9	14.21	538.4	393.5

1 13	33.95	7650.6	32.9	14.21	538.4	393.5
1 14	36.56	7650.6	32.9	14.21	538.4	393.5
1 15	39.17	7650.6	32.9	14.21	538.4	393.5
1 16	41.78	7650.6	32.9	14.21	538.4	393.5
1 17	44.40	5900.0	32.9	16.45	358.7	358.7
1 18	47.01	5900.0	32.9	16.45	358.7	358.7
1 19	49.62	5900.0	32.9	16.45	358.7	358.7
1 20	52.23	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12575.9	0.024742	496.1	1666.1	788.1
1	1	2.61	12575.9	0.024742	496.1	1666.1	788.1
1	2	5.22	12575.9	0.024742	496.1	1666.1	788.1
1	3	7.83	12575.9	0.024742	496.1	1666.1	788.1
1	4	10.45	16991.1	0.022596	731.4	1631.0	902.5
1	5	13.06	16991.1	0.022596	731.4	1631.0	902.5
1	6	15.67	16991.1	0.022596	731.4	1631.0	902.5
1	7	18.28	16991.1	0.022596	731.4	1631.0	902.5
1	8	20.89	16991.1	0.022596	731.4	1631.0	902.5
1	9	23.50	16991.1	0.022596	731.4	1631.0	902.5
1	10	26.11	16991.1	0.022596	731.4	1631.0	902.5
1	11	28.73	16991.1	0.022596	731.4	1631.0	902.5
1	12	31.34	16991.1	0.022596	731.4	1631.0	902.5
1	13	33.95	16991.1	0.022596	731.4	1631.0	902.5
1	14	36.56	16991.1	0.022596	731.4	1631.0	902.5
1	15	39.17	16991.1	0.022596	731.4	1631.0	902.5
1	16	41.78	16991.1	0.022596	731.4	1631.0	902.5
1	17	44.40	12575.9	0.024742	496.1	1666.1	788.1
1	18	47.01	12575.9	0.024742	496.1	1666.1	788.1
1	19	49.62	12575.9	0.024742	496.1	1666.1	788.1
1	20	52.23	12575.9	0.024742	496.1	1666.1	788.1

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	16953.0	0.029821	547.3	8815.3	1640.6
1	1	2.61	16953.0	0.029821	547.3	8815.3	1640.6
1	2	5.22	16953.0	0.029821	547.3	8815.3	1640.6
1	3	7.83	16953.0	0.029821	547.3	8815.3	1640.6

1	4	10.45	23858.7	0.027580	802.7	6075.2	1933.9
1	5	13.06	23858.7	0.027580	802.7	6075.2	1933.9
1	6	15.67	23858.7	0.027580	802.7	6075.2	1933.9
1	7	18.28	23858.7	0.027580	802.7	6075.2	1933.9
1	8	20.89	23858.7	0.027580	802.7	6075.2	1933.9
1	9	23.50	23858.7	0.027580	802.7	6075.2	1933.9
1	10	26.11	23858.7	0.027580	802.7	6075.2	1933.9
1	11	28.73	23858.7	0.027580	802.7	6075.2	1933.9
1	12	31.34	23858.7	0.027580	802.7	6075.2	1933.9
1	13	33.95	23858.7	0.027580	802.7	6075.2	1933.9
1	14	36.56	23858.7	0.027580	802.7	6075.2	1933.9
1	15	39.17	23858.7	0.027580	802.7	6075.2	1933.9
1	16	41.78	23858.7	0.027580	802.7	6075.2	1933.9
1	17	44.40	16953.0	0.029821	547.3	8815.3	1640.6
1	18	47.01	16953.0	0.029821	547.3	8815.3	1640.6
1	19	49.62	16953.0	0.029821	547.3	8815.3	1640.6
1	20	52.23	16953.0	0.029821	547.3	8815.3	1640.6

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	2.61	8.7	47.1	0.7	0.0	56.5	
1	2	5.22	16.5	89.2	1.5	0.0	107.2	
1	3	7.83	23.3	126.4	2.2	0.0	152.0	
1	4	10.45	29.3	158.7	2.9	0.0	190.9	
1	5	13.06	34.3	185.9	3.6	0.0	223.9	
1	6	15.67	38.4	208.2	4.4	0.0	251.0	

1	7	18.28	41.6	225.6	4.9	0.0	272.1
1	8	20.89	43.9	238.0	4.8	0.0	286.8
1	9	23.50	45.3	245.4	4.8	0.0	295.5
1	10	26.11	45.8	247.9	4.8	0.0	298.5
1	11	28.73	45.3	245.4	4.8	0.0	295.5
1	12	31.34	43.9	238.0	4.7	0.0	286.7
1	13	33.95	41.6	225.6	4.7	0.0	272.0
1	14	36.56	38.4	208.2	4.6	0.0	251.3
1	15	39.17	34.3	185.9	3.9	0.0	224.1
1	16	41.78	29.3	158.7	3.1	0.0	191.0
1	17	44.40	23.3	126.4	2.3	0.0	152.1
1	18	47.01	16.5	89.2	1.5	0.0	107.3
1	19	49.62	8.7	47.1	0.8	0.0	56.6
1	20	52.23	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.61	18.9	10.6	0.0	0.0	0.0	29.5
1	2	5.22	35.8	20.0	0.0	0.0	0.0	55.9
1	3	7.83	50.8	28.3	0.0	0.0	0.0	79.1
1	4	10.45	63.7	35.6	0.0	0.0	0.0	99.3
1	5	13.06	74.7	41.7	0.0	0.0	0.0	116.4
1	6	15.67	83.6	46.7	0.0	0.0	0.0	130.3
1	7	18.28	90.6	50.6	0.0	0.0	0.0	141.2
1	8	20.89	95.6	53.4	0.0	0.0	0.0	148.9
1	9	23.50	98.6	55.0	0.0	0.0	0.0	153.6
1	10	26.11	99.6	55.6	0.0	0.0	0.0	155.2
1	11	28.73	98.6	55.0	0.0	0.0	0.0	153.6
1	12	31.34	95.6	53.4	0.0	0.0	0.0	148.9
1	13	33.95	90.6	50.6	0.0	0.0	0.0	141.2
1	14	36.56	83.6	46.7	0.0	0.0	0.0	130.3
1	15	39.17	74.7	41.7	0.0	0.0	0.0	116.4
1	16	41.78	63.7	35.6	0.0	0.0	0.0	99.3
1	17	44.40	50.8	28.3	0.0	0.0	0.0	79.1
1	18	47.01	35.8	20.0	0.0	0.0	0.0	55.9
1	19	49.62	18.9	10.6	0.0	0.0	0.0	29.5
1	20	52.23	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.61	0.0	0.0	163.6	HL-93	0.0	HL-93
1	2	5.22	0.0	0.0	306.7	HL-93	0.0	HL-93

1	3	7.83	0.0	0.0	429.2	HL-93	0.0	HL-93
1	4	10.45	0.0	0.0	531.3	HL-93	0.0	HL-93
1	5	13.06	0.0	0.0	610.3	HL-93	0.0	HL-93
1	6	15.67	0.0	0.0	673.9	HL-93	0.0	HL-93
1	7	18.28	0.0	0.0	723.6	HL-93	0.0	HL-93
1	8	20.89	0.0	0.0	755.3	HL-93	0.0	HL-93
1	9	23.50	0.0	0.0	762.7	HL-93	0.0	HL-93
1	10	26.11	0.0	0.0	765.2	HL-93	0.0	HL-93
1	11	28.73	0.0	0.0	762.7	HL-93	0.0	HL-93
1	12	31.34	0.0	0.0	755.3	HL-93	0.0	HL-93
1	13	33.95	0.0	0.0	723.6	HL-93	0.0	HL-93
1	14	36.56	0.0	0.0	673.9	HL-93	0.0	HL-93
1	15	39.17	0.0	0.0	610.3	HL-93	0.0	HL-93
1	16	41.78	0.0	0.0	531.3	HL-93	0.0	HL-93
1	17	44.40	0.0	0.0	429.2	HL-93	0.0	HL-93
1	18	47.01	0.0	0.0	306.7	HL-93	0.0	HL-93
1	19	49.62	0.0	0.0	163.6	HL-93	0.0	HL-93
1	20	52.23	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.61	56.	0.	56.
1	2	5.22	102.	0.	102.
1	3	7.83	139.	0.	139.
1	4	10.45	170.	0.	170.
1	5	13.06	193.	0.	193.
1	6	15.67	209.	0.	209.
1	7	18.28	217.	0.	217.
1	8	20.89	223.	0.	223.
1	9	23.50	229.	0.	229.
1	10	26.11	231.	0.	231.
1	11	28.73	229.	0.	229.
1	12	31.34	223.	0.	223.
1	13	33.95	217.	0.	217.
1	14	36.56	209.	0.	209.
1	15	39.17	193.	0.	193.
1	16	41.78	170.	0.	170.
1	17	44.40	139.	0.	139.
1	18	47.01	102.	0.	102.
1	19	49.62	56.	0.	56.
1	20	52.23	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

1	1	2.61	56.5	29.5	163.6	HL-93	0.0	HL-93	249.6	86.0
1	2	5.22	107.2	55.9	306.7	HL-93	0.0	HL-93	469.7	163.0
1	3	7.83	152.0	79.1	429.2	HL-93	0.0	HL-93	660.3	231.1
1	4	10.45	190.9	99.3	531.3	HL-93	0.0	HL-93	821.4	290.2
1	5	13.06	223.9	116.4	610.3	HL-93	0.0	HL-93	950.5	340.3
1	6	15.67	251.0	130.3	673.9	HL-93	0.0	HL-93	1055.3	381.4
1	7	18.28	272.1	141.2	723.6	HL-93	0.0	HL-93	1136.9	413.3
1	8	20.89	286.8	148.9	755.3	HL-93	0.0	HL-93	1191.0	435.7
1	9	23.50	295.5	153.6	762.7	HL-93	0.0	HL-93	1211.8	449.1
1	10	26.11	298.5	155.2	765.2	HL-93	0.0	HL-93	1218.8	453.6
1	11	28.73	295.5	153.6	762.7	HL-93	0.0	HL-93	1211.8	449.1
1	12	31.34	286.7	148.9	755.3	HL-93	0.0	HL-93	1191.0	435.6
1	13	33.95	272.0	141.2	723.6	HL-93	0.0	HL-93	1136.7	413.2
1	14	36.56	251.3	130.3	673.9	HL-93	0.0	HL-93	1055.6	381.6
1	15	39.17	224.1	116.4	610.3	HL-93	0.0	HL-93	950.7	340.5
1	16	41.78	191.0	99.3	531.3	HL-93	0.0	HL-93	821.6	290.3
1	17	44.40	152.1	79.1	429.2	HL-93	0.0	HL-93	660.4	231.2
1	18	47.01	107.3	55.9	306.7	HL-93	0.0	HL-93	469.8	163.1
1	19	49.62	56.6	29.5	163.6	HL-93	0.0	HL-93	249.6	86.0
1	20	52.23	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.61	249.6	298.7	398.5	333.1	129.0	41.8
1	2	5.22	469.7	561.7	749.4	626.7	244.5	76.9
1	3	7.83	660.3	789.1	1052.7	881.0	346.6	104.6
1	4	10.45	821.4	980.8	1308.4	1095.9	435.2	127.1
1	5	13.06	950.5	1133.6	1511.9	1267.8	510.4	144.5
1	6	15.67	1055.3	1257.5	1677.0	1407.4	572.1	157.0
1	7	18.28	1136.9	1353.9	1805.5	1516.1	619.9	162.8
1	8	20.89	1191.0	1417.7	1890.4	1588.2	653.6	167.4
1	9	23.50	1211.8	1440.7	1920.8	1615.7	673.7	171.5
1	10	26.11	1218.8	1448.3	1930.9	1624.9	680.4	172.9
1	11	28.73	1211.8	1440.6	1920.7	1615.7	673.7	171.5
1	12	31.34	1191.0	1417.6	1890.3	1588.1	653.4	167.4
1	13	33.95	1136.7	1353.8	1805.4	1515.9	619.7	162.8
1	14	36.56	1055.6	1257.7	1677.3	1407.7	572.5	157.0
1	15	39.17	950.7	1133.8	1512.2	1268.1	510.7	144.5
1	16	41.78	821.6	981.0	1308.6	1096.1	435.5	127.1
1	17	44.40	660.4	789.2	1052.8	881.2	346.8	104.6
1	18	47.01	469.8	561.8	749.5	626.8	244.7	76.9
1	19	49.62	249.6	298.7	398.5	333.1	129.1	41.8
1	20	52.23	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT	STEEL BEAM	CONC. SLAB	CONCENT. LOAD	OTHER UNIFORM LOAD	TOTAL NONCOMPOSITE DEAD LOAD

		(ft)	(kip)	(kip)	(kip)	(kip)	(kip)
1	0	0.00	3.5	19.0	0.3	0.0	22.8
1	1	2.61	3.2	17.1	0.3	0.0	20.5
1	2	5.22	2.8	15.2	0.3	0.0	18.3
1	3	7.83	2.5	13.3	0.3	0.0	16.0
1	4	10.45	2.1	11.4	0.3	0.0	13.8
1	5	13.06	1.8	9.5	0.3	0.0	11.5
1	6	15.67	1.4	7.6	0.3	0.0	9.3
1	7	18.28	1.1	5.7	0.0	0.0	6.7
1	8	20.89	0.7	3.8	0.0	0.0	4.5
1	9	23.50	0.4	1.9	0.0	0.0	2.2
1	10	26.11	0.0	0.0	0.0	0.0	0.0
1	11	28.73	-0.4	-1.9	0.0	0.0	-2.3
1	12	31.34	-0.7	-3.8	0.0	0.0	-4.5
1	13	33.95	-1.1	-5.7	0.0	0.0	-6.8
1	14	36.56	-1.4	-7.6	-0.3	0.0	-9.3
1	15	39.17	-1.8	-9.5	-0.3	0.0	-11.5
1	16	41.78	-2.1	-11.4	-0.3	0.0	-13.8
1	17	44.40	-2.5	-13.3	-0.3	0.0	-16.0
1	18	47.01	-2.8	-15.2	-0.3	0.0	-18.3
1	19	49.62	-3.2	-17.1	-0.3	0.0	-20.5
1	20	52.23	-3.5	-19.0	-0.3	0.0	-22.8

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	DC2	CONCENTRATED LOADS DW	DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.6	4.3	0.0	0.0	11.9	22.8	34.7
1	1	2.61	6.9	3.8	0.0	0.0	10.7	20.5	31.2
1	2	5.22	6.1	3.4	0.0	0.0	9.5	18.3	27.8
1	3	7.83	5.3	3.0	0.0	0.0	8.3	16.0	24.3
1	4	10.45	4.6	2.6	0.0	0.0	7.1	13.8	20.9
1	5	13.06	3.8	2.1	0.0	0.0	5.9	11.5	17.5
1	6	15.67	3.1	1.7	0.0	0.0	4.8	9.3	14.0
1	7	18.28	2.3	1.3	0.0	0.0	3.6	6.7	10.3
1	8	20.89	1.5	0.9	0.0	0.0	2.4	4.5	6.9
1	9	23.50	0.8	0.4	0.0	0.0	1.2	2.2	3.4
1	10	26.11	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	11	28.73	-0.8	-0.4	0.0	0.0	-1.2	-2.3	-3.4
1	12	31.34	-1.5	-0.9	0.0	0.0	-2.4	-4.5	-6.9
1	13	33.95	-2.3	-1.3	0.0	0.0	-3.6	-6.8	-10.3
1	14	36.56	-3.1	-1.7	0.0	0.0	-4.8	-9.3	-14.0
1	15	39.17	-3.8	-2.1	0.0	0.0	-5.9	-11.5	-17.5
1	16	41.78	-4.6	-2.6	0.0	0.0	-7.1	-13.8	-20.9
1	17	44.40	-5.3	-3.0	0.0	0.0	-8.3	-16.0	-24.4
1	18	47.01	-6.1	-3.4	0.0	0.0	-9.5	-18.3	-27.8
1	19	49.62	-6.9	-3.8	0.0	0.0	-10.7	-20.5	-31.2
1	20	52.23	-7.6	-4.3	0.0	0.0	-11.9	-22.8	-34.7

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)

5969 Span 2-S16 Interior Fatigue and Mid M.RES
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	83.7	HL-93	0.0	HL-93
1	1	2.61	0.0	0.0	78.1	HL-93	-1.9	HL-93
1	2	5.22	0.0	0.0	72.5	HL-93	-3.9	HL-93
1	3	7.83	0.0	0.0	67.0	HL-93	-6.8	HL-93
1	4	10.45	0.0	0.0	61.6	HL-93	-10.0	HL-93
1	5	13.06	0.0	0.0	56.3	HL-93	-13.3	HL-93
1	6	15.67	0.0	0.0	51.0	HL-93	-16.6	HL-93
1	7	18.28	0.0	0.0	45.8	HL-93	-20.0	HL-93
1	8	20.89	0.0	0.0	40.7	HL-93	-23.5	HL-93
1	9	23.50	0.0	0.0	35.7	HL-93	-27.0	HL-93
1	10	26.11	0.0	0.0	31.0	HL-93	-31.0	HL-93
1	11	28.73	0.0	0.0	27.0	HL-93	-35.7	HL-93
1	12	31.34	0.0	0.0	23.5	HL-93	-40.7	HL-93
1	13	33.95	0.0	0.0	20.0	HL-93	-45.8	HL-93
1	14	36.56	0.0	0.0	16.6	HL-93	-51.0	HL-93
1	15	39.17	0.0	0.0	13.3	HL-93	-56.3	HL-93
1	16	41.78	0.0	0.0	10.0	HL-93	-61.6	HL-93
1	17	44.40	0.0	0.0	6.8	HL-93	-67.0	HL-93
1	18	47.01	0.0	0.0	3.9	HL-93	-72.5	HL-93
1	19	49.62	0.0	0.0	1.9	HL-93	-78.1	HL-93
1	20	52.23	0.0	0.0	0.0	HL-93	-83.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	33.	0.	33.
1	1	2.61	30.	-1.	32.
1	2	5.22	28.	-2.	30.
1	3	7.83	26.	-3.	29.
1	4	10.45	23.	-5.	28.
1	5	13.06	21.	-6.	27.
1	6	15.67	19.	-7.	26.
1	7	18.28	16.	-8.	24.
1	8	20.89	14.	-10.	24.
1	9	23.50	14.	-11.	25.
1	10	26.11	13.	-13.	25.
1	11	28.73	11.	-14.	25.
1	12	31.34	10.	-14.	24.
1	13	33.95	8.	-16.	24.
1	14	36.56	7.	-19.	26.
1	15	39.17	6.	-21.	27.
1	16	41.78	5.	-23.	28.
1	17	44.40	3.	-26.	29.
1	18	47.01	2.	-28.	30.
1	19	49.62	1.	-30.	32.
1	20	52.23	0.	-33.	33.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	33.0	.31015E-01	0.51	0.2500*	.14206E-01	0.23	0.2500*
1	1		2.61	31.6	.31015E-01	0.49	0.2500*	.14206E-01	0.22	0.2500*
1	2		5.22	30.2	.31015E-01	0.47	0.2500*	.14206E-01	0.21	0.2500*
1	3		7.83	28.9	.31015E-01	0.45	0.2500*	.14206E-01	0.21	0.2500*
1	4		10.45	27.6	.29123E-01	0.40	0.2500*	.93999E-02	0.13	0.2500*
1	5		13.06	26.5	.29123E-01	0.39	0.2500*	.93999E-02	0.12	0.2500*
1	6		15.67	25.6	.29123E-01	0.37	0.2500*	.93999E-02	0.12	0.2500*
1	7		18.28	24.3	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	8		20.89	23.9	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	9		23.50	24.8	.29123E-01	0.36	0.2500*	.93999E-02	0.12	0.2500*
1	10		26.11	25.2	.29123E-01	0.37	0.2500*	.93999E-02	0.12	0.2500*
1	11		28.73	24.8	.29123E-01	0.36	0.2500*	.93999E-02	0.12	0.2500*
1	12		31.34	23.9	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	13		33.95	24.3	.29123E-01	0.35	0.2500*	.93999E-02	0.11	0.2500*
1	14		36.56	25.6	.29123E-01	0.37	0.2500*	.93999E-02	0.12	0.2500*
1	15		39.17	26.5	.29123E-01	0.39	0.2500*	.93999E-02	0.12	0.2500*
1	16		41.78	27.6	.29123E-01	0.40	0.2500*	.93999E-02	0.13	0.2500*
1	17		44.40	28.9	.31015E-01	0.45	0.2500*	.14206E-01	0.21	0.2500*
1	18		47.01	30.2	.31015E-01	0.47	0.2500*	.14206E-01	0.21	0.2500*
1	19		49.62	31.6	.31015E-01	0.49	0.2500*	.14206E-01	0.22	0.2500*
1	20		52.23	33.0	.31015E-01	0.51	0.2500*	.14206E-01	0.23	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0		0.00	34.7	83.7	HL-93	0.0	HL-93	118.4	34.7
1	1		2.61	31.2	78.1	HL-93	-1.9	HL-93	109.3	29.3
1	2		5.22	27.8	72.5	HL-93	-3.9	HL-93	100.3	23.9
1	3		7.83	24.3	67.0	HL-93	-6.8	HL-93	91.4	17.5
1	4		10.45	20.9	61.6	HL-93	-10.0	HL-93	82.5	10.9
1	5		13.06	17.5	56.3	HL-93	-13.3	HL-93	73.7	4.2

1	6	15.67	14.0	51.0	HL-93	-16.6	HL-93	65.0	-2.6
1	7	18.28	10.3	45.8	HL-93	-20.0	HL-93	56.1	-9.7
1	8	20.89	6.9	40.7	HL-93	-23.5	HL-93	47.6	-16.6
1	9	23.50	3.4	35.7	HL-93	-27.0	HL-93	39.2	-23.6
1	10	26.11	0.0	31.0	HL-93	-31.0	HL-93	31.0	-31.0
1	11	28.73	-3.4	27.0	HL-93	-35.7	HL-93	23.6	-39.2
1	12	31.34	-6.9	23.5	HL-93	-40.7	HL-93	16.6	-47.6
1	13	33.95	-10.3	20.0	HL-93	-45.8	HL-93	9.7	-56.1
1	14	36.56	-14.0	16.6	HL-93	-51.0	HL-93	2.5	-65.0
1	15	39.17	-17.5	13.3	HL-93	-56.3	HL-93	-4.2	-73.8
1	16	41.78	-20.9	10.0	HL-93	-61.6	HL-93	-10.9	-82.5
1	17	44.40	-24.4	6.8	HL-93	-67.0	HL-93	-17.5	-91.4
1	18	47.01	-27.8	3.9	HL-93	-72.5	HL-93	-23.9	-100.3
1	19	49.62	-31.2	1.9	HL-93	-78.1	HL-93	-29.3	-109.3
1	20	52.23	-34.7	0.0	HL-93	-83.7	HL-93	-34.7	-118.4

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	118.4	143.5	191.7	158.2	52.0	24.8
1	1	2.61	109.3	132.7	177.4	146.1	46.8	23.7
1	2	5.22	100.3	122.0	163.1	134.1	41.7	22.7
1	3	7.83	91.4	111.5	149.1	122.2	36.5	21.6
1	4	10.45	82.5	101.0	135.1	110.4	31.4	20.7
1	5	13.06	73.7	90.6	121.3	98.8	26.2	19.9
1	6	15.67	65.0	80.3	107.6	87.2	21.0	19.2
1	7	18.28	56.1	69.8	93.5	75.2	15.5	18.3
1	8	20.89	47.6	59.8	80.2	63.9	10.3	17.9
1	9	23.50	39.2	49.9	67.0	52.7	5.1	18.6
1	10	26.11	-31.0	-40.3	-54.3	-41.9	0.0	18.9
1	11	28.73	-39.2	-49.9	-67.0	-52.7	-5.2	18.6
1	12	31.34	-47.6	-59.8	-80.2	-63.9	-10.3	17.9
1	13	33.95	-56.1	-69.8	-93.6	-75.3	-15.5	18.3
1	14	36.56	-65.0	-80.3	-107.6	-87.2	-21.1	19.2
1	15	39.17	-73.8	-90.6	-121.3	-98.8	-26.2	19.9
1	16	41.78	-82.5	-101.0	-135.1	-110.5	-31.4	20.7
1	17	44.40	-91.4	-111.5	-149.1	-122.3	-36.5	21.6
1	18	47.01	-100.3	-122.1	-163.2	-134.2	-41.7	22.7
1	19	49.62	-109.3	-132.7	-177.4	-146.2	-46.8	23.7
1	20	52.23	-118.4	-143.5	-191.7	-158.3	-52.0	24.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	34.65	0.00	HL-93	83.71	HL-93	ST1	29.28	191.71
						ST2	29.28	158.23
						ST4	40.54	51.98

						SE1	34.65	118.36
						SE2	34.65	143.47
2	34.67	0.00	HL-93	83.71	HL-93	ST1	29.29	191.73
						ST2	29.29	158.25
						ST4	40.56	52.00
						SE1	34.67	118.38
						SE2	34.67	143.49

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.61	-0.0170	-0.0919	0.0000	-0.0261	-0.1106	-0.0261	-0.1368
1	2	5.22	-0.0333	-0.1805	0.0000	-0.0513	-0.2174	-0.0513	-0.2687
1	3	7.83	-0.0486	-0.2631	0.0000	-0.0747	-0.3168	-0.0747	-0.3915
1	4	10.45	-0.0622	-0.3371	0.0000	-0.0956	-0.4059	-0.0956	-0.5015
1	5	13.06	-0.0742	-0.4021	0.0000	-0.1139	-0.4843	-0.1139	-0.5982
1	6	15.67	-0.0844	-0.4573	0.0000	-0.1295	-0.5509	-0.1295	-0.6803
1	7	18.28	-0.0926	-0.5015	0.0000	-0.1419	-0.6041	-0.1419	-0.7460
1	8	20.89	-0.0985	-0.5337	0.0000	-0.1510	-0.6429	-0.1510	-0.7939
1	9	23.50	-0.1021	-0.5533	0.0000	-0.1565	-0.6665	-0.1565	-0.8230
1	10	26.11	-0.1033	-0.5599	0.0000	-0.1584	-0.6744	-0.1584	-0.8327
1	11	28.73	-0.1021	-0.5533	0.0000	-0.1565	-0.6665	-0.1565	-0.8230
1	12	31.34	-0.0985	-0.5337	0.0000	-0.1510	-0.6429	-0.1510	-0.7939
1	13	33.95	-0.0926	-0.5015	0.0000	-0.1419	-0.6041	-0.1419	-0.7460
1	14	36.56	-0.0844	-0.4573	0.0000	-0.1295	-0.5509	-0.1295	-0.6804
1	15	39.17	-0.0742	-0.4021	0.0000	-0.1139	-0.4844	-0.1139	-0.5983
1	16	41.78	-0.0622	-0.3371	0.0000	-0.0956	-0.4060	-0.0956	-0.5016
1	17	44.40	-0.0486	-0.2631	0.0000	-0.0747	-0.3169	-0.0747	-0.3916
1	18	47.01	-0.0333	-0.1805	0.0000	-0.0513	-0.2174	-0.0513	-0.2687
1	19	49.62	-0.0170	-0.0919	0.0000	-0.0261	-0.1107	-0.0261	-0.1368
1	20	52.23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE (in) STEEL DEFL.	CAMBER SIZE	DEAD LOADS (in) SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.6	-0.017	0 1/16	-0.094	0 1/ 8	-0.026	0 1/16	-0.137	0 3/16

1	2	5.2	-0.033	0	1/16	-0.184	0	3/16	-0.051	0	1/16	-0.269	0	5/16
1	3	7.8	-0.049	0	1/16	-0.268	0	5/16	-0.075	0	1/ 8	-0.391	0	7/16
1	4	10.4	-0.062	0	1/16	-0.344	0	3/ 8	-0.096	0	1/ 8	-0.502	0	9/16
1	5	13.1	-0.074	0	1/ 8	-0.410	0	7/16	-0.114	0	1/ 8	-0.598	0	5/ 8
1	6	15.7	-0.084	0	1/ 8	-0.466	0	1/ 2	-0.129	0	3/16	-0.680	0	11/16
1	7	18.3	-0.093	0	1/ 8	-0.512	0	9/16	-0.142	0	3/16	-0.746	0	3/ 4
1	8	20.9	-0.099	0	1/ 8	-0.544	0	9/16	-0.151	0	3/16	-0.794	0	13/16
1	9	23.5	-0.102	0	1/ 8	-0.564	0	5/ 8	-0.157	0	3/16	-0.823	0	7/ 8
1	10	26.1	-0.103	0	1/ 8	-0.571	0	5/ 8	-0.158	0	3/16	-0.833	0	7/ 8
1	11	28.7	-0.102	0	1/ 8	-0.564	0	5/ 8	-0.157	0	3/16	-0.823	0	7/ 8
1	12	31.3	-0.099	0	1/ 8	-0.544	0	9/16	-0.151	0	3/16	-0.794	0	13/16
1	13	33.9	-0.093	0	1/ 8	-0.512	0	9/16	-0.142	0	3/16	-0.746	0	3/ 4
1	14	36.6	-0.084	0	1/ 8	-0.467	0	1/ 2	-0.129	0	3/16	-0.680	0	11/16
1	15	39.2	-0.074	0	1/ 8	-0.410	0	7/16	-0.114	0	1/ 8	-0.598	0	5/ 8
1	16	41.8	-0.062	0	1/16	-0.344	0	3/ 8	-0.096	0	1/ 8	-0.502	0	9/16
1	17	44.4	-0.049	0	1/16	-0.268	0	5/16	-0.075	0	1/ 8	-0.392	0	7/16
1	18	47.0	-0.033	0	1/16	-0.184	0	3/16	-0.051	0	1/16	-0.269	0	5/16
1	19	49.6	-0.017	0	1/16	-0.094	0	1/ 8	-0.026	0	1/16	-0.137	0	3/16
1	20	52.2	0.000	--		0.000	--		0.000	--		0.000	--	

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	26.11	3	0.425	-0.260	MAX HL-93	0.78	0.00242
				0.025	MIN		
				-0.068	MAX LANE		
				0.025	MIN LANE		
		4	0.433	-0.265	MAX HL-93	0.78	0.00242
				0.026	MIN		
				-0.069	MAX LANE		
				0.026	MIN LANE		
		3	0.425	-0.166	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	-0.29	0.29	-1.60	1.60	-1.89	1.89
1	2	5.22	-0.55	0.55	-3.03	3.03	-3.59	3.59
1	3	7.83	-0.78	0.78	-4.30	4.30	-5.08	5.08
1	4	10.45	-0.89	0.65	-4.93	3.60	-5.82	4.25
1	5	13.06	-1.05	0.76	-5.78	4.23	-6.83	4.99
1	6	15.67	-1.17	0.86	-6.48	4.74	-7.65	5.60
1	7	18.28	-1.27	0.93	-7.03	5.14	-8.30	6.06
1	8	20.89	-1.34	0.98	-7.40	5.41	-8.74	6.39
1	9	23.50	-1.38	1.01	-7.63	5.58	-9.01	6.59
1	10	26.11	-1.40	1.02	-7.71	5.63	-9.10	6.65
1	11	28.73	-1.38	1.01	-7.63	5.58	-9.01	6.59
1	12	31.34	-1.34	0.98	-7.40	5.41	-8.74	6.39
1	13	33.95	-1.27	0.93	-7.02	5.13	-8.29	6.06
1	14	36.56	-1.17	0.86	-6.49	4.74	-7.66	5.60
1	15	39.17	-1.05	0.76	-5.79	4.23	-6.83	4.99
1	16	41.78	-0.89	0.65	-4.93	3.60	-5.82	4.26
1	17	44.40	-0.78	0.78	-4.31	4.31	-5.09	5.09
1	18	47.01	-0.55	0.55	-3.04	3.04	-3.59	3.59
1	19	49.62	-0.29	0.29	-1.60	1.60	-1.89	1.89
1	20	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN D NO L	FROM SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	-0.02	-0.21	0.71	-0.02	-2.10	2.60
1	2	5.22	-0.03	-0.40	1.35	-0.03	-3.99	4.94
1	3	7.83	-0.04	-0.57	1.91	-0.04	-5.65	7.00
1	4	10.45	-0.05	-0.73	1.63	-0.05	-6.55	5.88
1	5	13.06	-0.06	-0.86	1.91	-0.06	-7.68	6.90
1	6	15.67	-0.06	-0.96	2.14	-0.06	-8.61	7.73
1	7	18.28	-0.07	-1.04	2.32	-0.07	-9.34	8.38
1	8	20.89	-0.07	-1.10	2.44	-0.07	-9.84	8.84
1	9	23.50	-0.08	-1.13	2.52	-0.08	-10.14	9.11
1	10	26.11	-0.08	-1.14	2.55	-0.08	-10.24	9.20
1	11	28.73	-0.08	-1.13	2.52	-0.08	-10.14	9.11
1	12	31.34	-0.07	-1.10	2.44	-0.07	-9.84	8.83
1	13	33.95	-0.07	-1.04	2.32	-0.07	-9.33	8.38
1	14	36.56	-0.06	-0.96	2.14	-0.06	-8.62	7.74
1	15	39.17	-0.06	-0.86	1.91	-0.06	-7.69	6.90

1 16	41.78	-0.05	-0.73	1.63	-0.05	-6.56	5.89
1 17	44.40	-0.04	-0.57	1.91	-0.04	-5.66	7.00
1 18	47.01	-0.03	-0.40	1.35	-0.03	-3.99	4.94
1 19	49.62	-0.02	-0.21	0.71	-0.02	-2.10	2.61
1 20	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	-0.13	-0.22	3.59	0.00	0.00	0.00
1	2	5.22	-0.25	-0.42	6.72	0.00	0.00	0.00
1	3	7.83	-0.35	-0.58	9.41	0.00	0.00	0.00
1	4	10.45	-0.37	-1.05	7.94	0.00	0.00	0.00
1	5	13.06	-0.42	-1.21	9.12	0.00	0.00	0.00
1	6	15.67	-0.46	-1.33	10.07	0.00	0.00	0.00
1	7	18.28	-0.50	-1.43	10.82	0.00	0.00	0.00
1	8	20.89	-0.52	-1.49	11.29	0.00	0.00	0.00
1	9	23.50	-0.53	-1.51	11.40	0.00	0.00	0.00
1	10	26.11	-0.53	-1.51	11.44	0.00	0.00	0.00
1	11	28.73	-0.53	-1.51	11.40	0.00	0.00	0.00
1	12	31.34	-0.52	-1.49	11.29	0.00	0.00	0.00
1	13	33.95	-0.50	-1.43	10.82	0.00	0.00	0.00
1	14	36.56	-0.46	-1.33	10.07	0.00	0.00	0.00
1	15	39.17	-0.42	-1.21	9.12	0.00	0.00	0.00
1	16	41.78	-0.37	-1.05	7.94	0.00	0.00	0.00
1	17	44.40	-0.35	-0.58	9.41	0.00	0.00	0.00
1	18	47.01	-0.25	-0.42	6.72	0.00	0.00	0.00
1	19	49.62	-0.13	-0.22	3.59	0.00	0.00	0.00
1	20	52.23	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	2.61	0.1	0.0	0.1	C	HL	1.2	0.0	1.2	T	HL
1	2	5.22	0.1	0.0	0.1	C	HL	2.2	0.0	2.2	T	HL
1	3	7.83	0.2	0.0	0.2	C	HL	3.1	0.0	3.1	T	HL
1	4	10.45	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T	HL
1	5	13.06	0.4	0.0	0.4	C	HL	2.9	0.0	2.9	T	HL
1	6	15.67	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	7	18.28	0.4	0.0	0.4	C	HL	3.2	0.0	3.2	T	HL
1	8	20.89	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	9	23.50	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	10	26.11	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	11	28.73	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	12	31.34	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL

1 13	33.95	0.4	0.0	0.4 C	HL	3.2	0.0	3.2 T	HL
1 14	36.56	0.4	0.0	0.4 C	HL	3.1	0.0	3.1 T	HL
1 15	39.17	0.4	0.0	0.4 C	HL	2.9	0.0	2.9 T	HL
1 16	41.78	0.3	0.0	0.3 C	HL	2.5	0.0	2.5 T	HL
1 17	44.40	0.2	0.0	0.2 C	HL	3.1	0.0	3.1 T	HL
1 18	47.01	0.1	0.0	0.1 C	HL	2.2	0.0	2.2 T	HL
1 19	49.62	0.1	0.0	0.1 C	HL	1.2	0.0	1.2 T	HL
1 20	52.23	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.15	-2.33	6.19	-0.02	-2.10	2.60
1	2	5.22	33.	33.	-0.28	-4.41	11.66	-0.03	-3.99	4.94
1	3	7.83	33.	33.	-0.39	-6.24	16.41	-0.04	-5.65	7.00
1	4	10.45	33.	33.	-0.42	-7.60	13.83	-0.05	-6.55	5.88
1	5	13.06	33.	33.	-0.48	-8.89	16.02	-0.06	-7.68	6.90
1	6	15.67	33.	33.	-0.53	-9.94	17.81	-0.06	-8.61	7.73
1	7	18.28	33.	33.	-0.57	-10.76	19.20	-0.07	-9.34	8.38
1	8	20.89	33.	33.	-0.59	-11.33	20.13	-0.07	-9.84	8.84
1	9	23.50	33.	33.	-0.60	-11.65	20.51	-0.08	-10.14	9.11
1	10	26.11	33.	33.	-0.60	-11.75	20.64	-0.08	-10.24	9.20
1	11	28.73	33.	33.	-0.60	-11.65	20.51	-0.08	-10.14	9.11
1	12	31.34	33.	33.	-0.59	-11.33	20.13	-0.07	-9.84	8.83
1	13	33.95	33.	33.	-0.57	-10.76	19.20	-0.07	-9.33	8.38
1	14	36.56	33.	33.	-0.53	-9.95	17.81	-0.06	-8.62	7.74
1	15	39.17	33.	33.	-0.48	-8.89	16.03	-0.06	-7.69	6.90
1	16	41.78	33.	33.	-0.42	-7.60	13.83	-0.05	-6.56	5.89
1	17	44.40	33.	33.	-0.39	-6.24	16.41	-0.04	-5.66	7.00
1	18	47.01	33.	33.	-0.28	-4.41	11.66	-0.03	-3.99	4.94
1	19	49.62	33.	33.	-0.15	-2.33	6.19	-0.02	-2.10	2.61
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.19	-2.39	7.27	-0.02	-2.10	2.60
1	2	5.22	33.	33.	-0.36	-4.53	13.68	-0.03	-3.99	4.94
1	3	7.83	33.	33.	-0.50	-6.41	19.23	-0.04	-5.65	7.00
1	4	10.45	33.	33.	-0.53	-7.91	16.21	-0.05	-6.55	5.88

1	5	13.06	33.	33.	-0.60	-9.25	18.76	-0.06	-7.68	6.90
1	6	15.67	33.	33.	-0.67	-10.34	20.83	-0.06	-8.61	7.73
1	7	18.28	33.	33.	-0.72	-11.19	22.44	-0.07	-9.34	8.38
1	8	20.89	33.	33.	-0.75	-11.78	23.51	-0.07	-9.84	8.84
1	9	23.50	33.	33.	-0.76	-12.10	23.93	-0.08	-10.14	9.11
1	10	26.11	33.	33.	-0.76	-12.21	24.07	-0.08	-10.24	9.20
1	11	28.73	33.	33.	-0.76	-12.10	23.93	-0.08	-10.14	9.11
1	12	31.34	33.	33.	-0.75	-11.78	23.51	-0.07	-9.84	8.83
1	13	33.95	33.	33.	-0.72	-11.19	22.44	-0.07	-9.33	8.38
1	14	36.56	33.	33.	-0.67	-10.35	20.84	-0.06	-8.62	7.74
1	15	39.17	33.	33.	-0.60	-9.26	18.76	-0.06	-7.69	6.90
1	16	41.78	33.	33.	-0.53	-7.92	16.21	-0.05	-6.56	5.89
1	17	44.40	33.	33.	-0.50	-6.42	19.24	-0.04	-5.66	7.00
1	18	47.01	33.	33.	-0.36	-4.53	13.68	-0.03	-3.99	4.94
1	19	49.62	33.	33.	-0.19	-2.39	7.27	-0.02	-2.10	2.61
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.26	-3.05	9.65	-0.02	-2.66	3.37
1	2	5.22	33.	33.	-0.48	-5.78	18.15	-0.04	-5.05	6.39
1	3	7.83	33.	33.	-0.67	-8.18	25.52	-0.06	-7.16	9.05
1	4	10.45	33.	33.	-0.71	-10.14	21.51	-0.07	-8.31	7.62
1	5	13.06	33.	33.	-0.82	-11.85	24.90	-0.08	-9.74	8.93
1	6	15.67	33.	33.	-0.90	-13.25	27.64	-0.09	-10.92	10.01
1	7	18.28	33.	33.	-0.97	-14.34	29.78	-0.10	-11.84	10.85
1	8	20.89	33.	33.	-1.01	-15.09	31.20	-0.10	-12.48	11.44
1	9	23.50	33.	33.	-1.03	-15.50	31.74	-0.11	-12.86	11.79
1	10	26.11	33.	33.	-1.03	-15.63	31.92	-0.11	-12.99	11.91
1	11	28.73	33.	33.	-1.03	-15.49	31.74	-0.11	-12.86	11.79
1	12	31.34	33.	33.	-1.01	-15.08	31.19	-0.10	-12.47	11.43
1	13	33.95	33.	33.	-0.97	-14.33	29.77	-0.10	-11.83	10.84
1	14	36.56	33.	33.	-0.90	-13.26	27.65	-0.09	-10.93	10.02
1	15	39.17	33.	33.	-0.82	-11.86	24.90	-0.08	-9.75	8.94
1	16	41.78	33.	33.	-0.71	-10.15	21.52	-0.07	-8.31	7.62
1	17	44.40	33.	33.	-0.67	-8.19	25.53	-0.06	-7.16	9.06
1	18	47.01	33.	33.	-0.48	-5.78	18.16	-0.04	-5.05	6.39
1	19	49.62	33.	33.	-0.26	-3.05	9.65	-0.02	-2.67	3.37
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.24	-2.25	8.51	-0.01	-1.86	2.23
1	2	5.22	33.	33.	-0.46	-4.26	15.99	-0.02	-3.52	4.23
1	3	7.83	33.	33.	-0.64	-6.02	22.46	-0.03	-5.00	5.99

1	4	10.45	33.	33.	-0.68	-7.61	18.93	-0.04	-5.78	5.03
1	5	13.06	33.	33.	-0.78	-8.89	21.87	-0.04	-6.78	5.90
1	6	15.67	33.	33.	-0.86	-9.93	24.25	-0.05	-7.60	6.62
1	7	18.28	33.	33.	-0.92	-10.74	26.10	-0.05	-8.24	7.17
1	8	20.89	33.	33.	-0.97	-11.29	27.32	-0.05	-8.68	7.56
1	9	23.50	33.	33.	-0.98	-11.58	27.75	-0.06	-8.95	7.79
1	10	26.11	33.	33.	-0.98	-11.68	27.89	-0.06	-9.03	7.87
1	11	28.73	33.	33.	-0.98	-11.58	27.74	-0.06	-8.95	7.79
1	12	31.34	33.	33.	-0.97	-11.29	27.32	-0.05	-8.68	7.56
1	13	33.95	33.	33.	-0.92	-10.73	26.10	-0.05	-8.23	7.17
1	14	36.56	33.	33.	-0.86	-9.94	24.25	-0.05	-7.61	6.62
1	15	39.17	33.	33.	-0.78	-8.89	21.87	-0.04	-6.78	5.91
1	16	41.78	33.	33.	-0.68	-7.62	18.94	-0.04	-5.78	5.04
1	17	44.40	33.	33.	-0.64	-6.02	22.47	-0.03	-5.00	6.00
1	18	47.01	33.	33.	-0.46	-4.26	16.00	-0.02	-3.53	4.23
1	19	49.62	33.	33.	-0.24	-2.25	8.51	-0.01	-1.86	2.23
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.20	-2.96	8.21	-0.02	-2.66	3.37
1	2	5.22	33.	33.	-0.38	-5.61	15.47	-0.04	-5.05	6.39
1	3	7.83	33.	33.	-0.53	-7.95	21.76	-0.06	-7.16	9.05
1	4	10.45	33.	33.	-0.56	-9.72	18.34	-0.07	-8.31	7.62
1	5	13.06	33.	33.	-0.65	-11.37	21.25	-0.08	-9.74	8.93
1	6	15.67	33.	33.	-0.72	-12.72	23.61	-0.09	-10.92	10.01
1	7	18.28	33.	33.	-0.77	-13.77	25.45	-0.10	-11.84	10.85
1	8	20.89	33.	33.	-0.81	-14.49	26.68	-0.10	-12.48	11.44
1	9	23.50	33.	33.	-0.82	-14.89	27.18	-0.11	-12.86	11.79
1	10	26.11	33.	33.	-0.82	-15.03	27.35	-0.11	-12.99	11.91
1	11	28.73	33.	33.	-0.82	-14.89	27.18	-0.11	-12.86	11.79
1	12	31.34	33.	33.	-0.81	-14.49	26.68	-0.10	-12.47	11.43
1	13	33.95	33.	33.	-0.77	-13.76	25.45	-0.10	-11.83	10.84
1	14	36.56	33.	33.	-0.72	-12.73	23.62	-0.09	-10.93	10.02
1	15	39.17	33.	33.	-0.65	-11.38	21.25	-0.08	-9.75	8.94
1	16	41.78	33.	33.	-0.56	-9.73	18.34	-0.07	-8.31	7.62
1	17	44.40	33.	33.	-0.53	-7.95	21.77	-0.06	-7.16	9.06
1	18	47.01	33.	33.	-0.38	-5.62	15.47	-0.04	-5.05	6.39
1	19	49.62	33.	33.	-0.20	-2.97	8.21	-0.02	-2.67	3.37
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.61	33.	33.	-0.19	-2.16	7.07	-0.01	-1.86	2.23
1	2	5.22	33.	33.	-0.36	-4.09	13.30	-0.02	-3.52	4.23

1	3	7.83	33.	33.	-0.50	-5.79	18.70	-0.03	-5.00	5.99
1	4	10.45	33.	33.	-0.53	-7.19	15.76	-0.04	-5.78	5.03
1	5	13.06	33.	33.	-0.61	-8.40	18.22	-0.04	-6.78	5.90
1	6	15.67	33.	33.	-0.67	-9.40	20.22	-0.05	-7.60	6.62
1	7	18.28	33.	33.	-0.72	-10.16	21.77	-0.05	-8.24	7.17
1	8	20.89	33.	33.	-0.76	-10.69	22.80	-0.05	-8.68	7.56
1	9	23.50	33.	33.	-0.77	-10.98	23.18	-0.06	-8.95	7.79
1	10	26.11	33.	33.	-0.77	-11.08	23.31	-0.06	-9.03	7.87
1	11	28.73	33.	33.	-0.77	-10.98	23.18	-0.06	-8.95	7.79
1	12	31.34	33.	33.	-0.76	-10.69	22.80	-0.05	-8.68	7.56
1	13	33.95	33.	33.	-0.72	-10.16	21.77	-0.05	-8.23	7.17
1	14	36.56	33.	33.	-0.67	-9.40	20.22	-0.05	-7.61	6.62
1	15	39.17	33.	33.	-0.61	-8.41	18.22	-0.04	-6.78	5.91
1	16	41.78	33.	33.	-0.53	-7.20	15.76	-0.04	-5.78	5.04
1	17	44.40	33.	33.	-0.50	-5.79	18.70	-0.03	-5.00	6.00
1	18	47.01	33.	33.	-0.36	-4.09	13.31	-0.02	-3.53	4.23
1	19	49.62	33.	33.	-0.19	-2.16	7.07	-0.01	-1.86	2.23
1	20	52.23	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	9.61	178.	-6.0	93.	-0.7	499.	-0.7	0.	0.0	-7.32
											0.68
	LB			6.0		2.2		10.9		0.0	19.16
											10.94
	RT			-5.4		-0.7		-1.0		0.0	-7.11
											0.99
	RB			4.0		1.5		7.5		0.0	12.96
											7.46
1	LT	42.61	179.	-5.4	93.	-0.7	499.	-1.0	0.	0.0	-7.12
											0.99
	LB			4.0		1.5		7.5		0.0	12.96
											7.46
	RT			-6.0		-0.7		-0.7		0.0	-7.32
											0.68
	RB			6.0		2.2		10.9		0.0	19.16
											10.94

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK
1	9.61	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	33.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.7500

```
 3   9.61    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

 ---------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   52.2
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  ------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------------

  1   9.61  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

  2  33.00  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

  3   9.61  6.75   32.90 [2]=   41.31     0.06591      >   0.04000    OK

                         [3]=   32.90     0.05249      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-----------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  1   2.61  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  2   5.22  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  3   7.83  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  4  10.45  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  5  13.06  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  6  15.67  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  7  18.28  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  8  20.89  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  9  23.50  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
```

1	10	26.11	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	11	28.73	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	12	31.34	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	13	33.95	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	14	36.56	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	15	39.17	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	16	41.78	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	17	44.40	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	18	47.01	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	19	49.62	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	20	52.23	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.61	0	1	31.42	0.550	57.13	168.97	2
1	2	5.22	0	1	31.42	0.550	57.13	168.97	2
1	3	7.83	0	1	31.42	0.550	57.13	168.97	2
1	4	10.45	0	1	31.42	0.550	68.10	168.97	2
1	5	13.06	0	1	31.42	0.550	68.10	168.97	2
1	6	15.67	0	1	31.42	0.550	68.10	168.97	2
1	7	18.28	0	1	31.42	0.550	68.10	168.97	2
1	8	20.89	0	1	31.42	0.550	68.10	168.97	2
1	9	23.50	0	1	31.42	0.550	68.10	168.97	2
1	10	26.11	0	1	31.42	0.550	68.10	168.97	2
1	11	28.73	0	1	31.42	0.550	68.10	168.97	2
1	12	31.34	0	1	31.42	0.550	68.10	168.97	2
1	13	33.95	0	1	31.42	0.550	68.10	168.97	2
1	14	36.56	0	1	31.42	0.550	68.10	168.97	2
1	15	39.17	0	1	31.42	0.550	68.10	168.97	2
1	16	41.78	0	1	31.42	0.550	68.10	168.97	2
1	17	44.40	0	1	31.42	0.550	57.13	168.97	2
1	18	47.01	0	1	31.42	0.550	57.13	168.97	2
1	19	49.62	0	1	31.42	0.550	57.13	168.97	2
1	20	52.23	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ------------------------------------------------------------

  1  0   0.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  1   2.61   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  2   5.22   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  3   7.83   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  4  10.45   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  5  13.06   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  6  15.67   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  7  18.28   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  20.89   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  23.50   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  26.11   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  28.73   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  31.34   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  33.95   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  36.56   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  39.17   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 16  41.78   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 17  44.40   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 18  47.01   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 19  49.62   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 20  52.23   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
```

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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
```

LATERAL BRACING NO	LEFT ----(FT)---- FROM	SUPT TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	17.5	0.000	12.139	8.002	3.866	1.000
			0.000	0.000	0.000	0.000	1.000
2	17.5	36.3	11.592	12.139	13.612	15.085	1.000
			0.000	0.000	0.000	0.000	1.000
3	36.3	52.2	0.000	11.592	7.690	3.787	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	Lb (ft)	Lp (ft)	Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	1		2.61	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	2		5.22	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	3		7.83	1	7.77	11.26	19.84	0	17.45	7.09	26.62	2	2
1	4		10.45	1	7.77	11.26	19.84	0	17.45	6.92	25.99	2	2
1	5		13.06	1	7.77	11.26	19.84	0	17.45	6.92	25.99	2	2
1	6		15.67	1	7.77	11.26	19.84	0	17.45	6.92	25.99	2	2
1	7		18.28	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	8		20.89	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	9		23.50	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	10		26.11	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	11		28.73	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	12		31.34	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	13		33.95	1	7.77	11.26	19.84	0	18.84	6.92	25.99	2	2
1	14		36.56	1	7.77	11.26	19.84	0	15.94	6.92	25.99	2	2
1	15		39.17	1	7.77	11.26	19.84	0	15.94	6.92	25.99	2	2
1	16		41.78	1	7.77	11.26	19.84	0	15.94	6.92	25.99	2	2
1	17		44.40	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	18		47.01	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	19		49.62	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2
1	20		52.23	1	7.77	11.26	19.84	0	15.94	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.00	33.00	27.75	27.75
1	1		2.61	1	1.000	1.000	1.00	33.00	27.75	27.75
1	2		5.22	1	1.000	1.000	1.00	33.00	27.75	27.75
1	3		7.83	1	1.000	1.000	1.00	33.00	27.75	27.75
1	4		10.45	1	1.000	1.000	1.00	33.00	27.54	27.54
1	5		13.06	1	1.000	1.000	1.00	33.00	27.54	27.54
1	6		15.67	1	1.000	1.000	1.00	33.00	27.54	27.54
1	7		18.28	1	1.000	1.000	1.00	33.00	26.81	26.81
1	8		20.89	1	1.000	1.000	1.00	33.00	26.81	26.81
1	9		23.50	1	1.000	1.000	1.00	33.00	26.81	26.81
1	10		26.11	1	1.000	1.000	1.00	33.00	26.81	26.81
1	11		28.73	1	1.000	1.000	1.00	33.00	26.81	26.81
1	12		31.34	1	1.000	1.000	1.00	33.00	26.81	26.81
1	13		33.95	1	1.000	1.000	1.00	33.00	26.81	26.81
1	14		36.56	1	1.000	1.000	1.00	33.00	28.32	28.32
1	15		39.17	1	1.000	1.000	1.00	33.00	28.32	28.32
1	16		41.78	1	1.000	1.000	1.00	33.00	28.32	28.32
1	17		44.40	1	1.000	1.000	1.00	33.00	28.51	28.51
1	18		47.01	1	1.000	1.000	1.00	33.00	28.51	28.51
1	19		49.62	1	1.000	1.000	1.00	33.00	28.51	28.51
1	20		52.23	1	1.000	1.000	1.00	33.00	28.51	28.51

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	4.70	17.35
1	1		2.61	1	4.70	17.35
1	2		5.22	1	4.70	17.35
1	3		7.83	1	4.70	17.35
1	4		10.45	1	5.72	17.67
1	5		13.06	1	5.72	17.67
1	6		15.67	1	5.72	17.67
1	7		18.28	1	5.72	17.67
1	8		20.89	1	5.72	17.67
1	9		23.50	1	5.72	17.67
1	10		26.11	1	5.72	17.67
1	11		28.73	1	5.72	17.67
1	12		31.34	1	5.72	17.67
1	13		33.95	1	5.72	17.67
1	14		36.56	1	5.72	17.67
1	15		39.17	1	5.72	17.67

1 16	41.78	1	5.72	17.67
1 17	44.40	1	4.70	17.35
1 18	47.01	1	4.70	17.35
1 19	49.62	1	4.70	17.35
1 20	52.23	1	4.70	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0		0.00	1	0	2	0		2	0
1	1		2.61	1	0	2	0		2	0
1	2		5.22	1	0	2	0		2	0
1	3		7.83	1	0	2	0		2	0
1	4		10.45	1	0	2	0		2	0
1	5		13.06	1	0	2	0		2	0
1	6		15.67	1	0	2	0		2	0
1	7		18.28	1	0	2	0		2	0
1	8		20.89	1	0	2	0		2	0
1	9		23.50	1	0	2	0		2	0
1	10		26.11	1	0	2	0		2	0
1	11		28.73	1	0	2	0		2	0
1	12		31.34	1	0	2	0		2	0
1	13		33.95	1	0	2	0		2	0
1	14		36.56	1	0	2	0		2	0
1	15		39.17	1	0	2	0		2	0
1	16		41.78	1	0	2	0		2	0
1	17		44.40	1	0	2	0		2	0
1	18		47.01	1	0	2	0		2	0
1	19		49.62	1	0	2	0		2	0
1	20		52.23	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for

5969 Span 2-S16 Interior Fatigue and Mid M.RES
composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.7	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	2.61	0.0	19.8	2.8	33.0	2.8	33.0	2.8	27.7	0
			0.0	-	2.8	-	2.8	33.0	-	-	0
1	2	5.22	0.0	19.8	5.4	33.0	5.4	33.0	5.4	27.7	0
			0.0	-	5.4	-	5.4	33.0	-	-	0
1	3	7.83	0.0	19.8	7.6	33.0	7.6	33.0	7.6	27.7	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	4	10.45	0.0	19.8	8.7	33.0	8.7	33.0	8.7	27.5	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	5	13.06	0.0	19.8	10.2	33.0	10.2	33.0	10.2	27.5	0
			0.0	-	7.5	-	7.5	33.0	-	-	0
1	6	15.67	0.0	19.8	11.5	33.0	11.5	33.0	11.5	27.5	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	7	18.28	0.0	19.8	12.4	33.0	12.4	33.0	12.4	26.8	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	8	20.89	0.0	19.8	13.1	33.0	13.1	33.0	13.1	26.8	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	9	23.50	0.0	19.8	13.5	33.0	13.5	33.0	13.5	26.8	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	10	26.11	0.0	19.8	13.7	33.0	13.7	33.0	13.7	26.8	0
			0.0	-	10.0	-	10.0	33.0	-	-	0
1	11	28.73	0.0	19.8	13.5	33.0	13.5	33.0	13.5	26.8	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	12	31.34	0.0	19.8	13.1	33.0	13.1	33.0	13.1	26.8	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	13	33.95	0.0	19.8	12.4	33.0	12.4	33.0	12.4	26.8	0
			0.0	-	9.1	-	9.1	33.0	-	-	0
1	14	36.56	0.0	19.8	11.5	33.0	11.5	33.0	11.5	28.3	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	15	39.17	0.0	19.8	10.3	33.0	10.3	33.0	10.3	28.3	0
			0.0	-	7.5	-	7.5	33.0	-	-	0
1	16	41.78	0.0	19.8	8.7	33.0	8.7	33.0	8.7	28.3	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	17	44.40	0.0	19.8	7.6	33.0	7.6	33.0	7.6	28.5	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	18	47.01	0.0	19.8	5.4	33.0	5.4	33.0	5.4	28.5	0
			0.0	-	5.4	-	5.4	33.0	-	-	0
1	19	49.62	0.0	19.8	2.8	33.0	2.8	33.0	2.8	28.5	0
			0.0	-	2.8	-	2.8	33.0	-	-	0
1	20	52.23	0.0	19.8	0.0	33.0	0.0	33.0	0.0	28.5	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,

the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.61	0.000	0.086	0.086	0.102	0.102	4
					0.086		0.086	3
1	2	5.22	0.000	0.163	0.163	0.194	0.194	4
					0.163		0.163	3
1	3	7.83	0.000	0.231	0.231	0.275	0.275	4
					0.231		0.231	3
1	4	10.45	0.000	0.265	0.265	0.317	0.317	4
					0.193		0.193	3
1	5	13.06	0.000	0.310	0.310	0.372	0.372	4
					0.227		0.227	3
1	6	15.67	0.000	0.348	0.348	0.417	0.417	4
					0.254		0.254	3
1	7	18.28	0.000	0.377	0.377	0.464	0.464	4
					0.276		0.276	3
1	8	20.89	0.000	0.397	0.397	0.489	0.489	4
					0.291		0.291	3
1	9	23.50	0.000	0.410	0.410	0.504	0.504	4
					0.299		0.299	3
1	10	26.11	0.000	0.414	0.414	0.509	0.509	4
					0.302		0.302	3
1	11	28.73	0.000	0.410	0.410	0.504	0.504	4
					0.299		0.299	3
1	12	31.34	0.000	0.397	0.397	0.489	0.489	4
					0.290		0.290	3
1	13	33.95	0.000	0.377	0.377	0.464	0.464	4
					0.276		0.276	3
1	14	36.56	0.000	0.348	0.348	0.406	0.406	4
					0.255		0.255	3
1	15	39.17	0.000	0.311	0.311	0.362	0.362	4
					0.227		0.227	3
1	16	41.78	0.000	0.265	0.265	0.309	0.309	4
					0.194		0.194	3
1	17	44.40	0.000	0.231	0.231	0.268	0.268	4
					0.231		0.231	3
1	18	47.01	0.000	0.163	0.163	0.189	0.189	4
					0.163		0.163	3
1	19	49.62	0.000	0.086	0.086	0.100	0.100	4
					0.086		0.086	3
1	20	52.23	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	1		2.61	0	398.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	2		5.22	0	749.4	2103.1	-	-	0.0	19.8	-	-	0--0
1	3		7.83	0	1052.7	2103.1	-	-	0.0	19.8	-	-	0--0
1	4		10.45	0	1308.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	5		13.06	0	1511.9	2803.1	-	-	0.0	19.8	-	-	0--0
1	6		15.67	0	1677.0	2803.1	-	-	0.0	19.8	-	-	0--0
1	7		18.28	0	1805.5	2803.1	-	-	0.0	19.8	-	-	0--0
1	8		20.89	0	1890.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	9		23.50	0	1920.8	2803.1	-	-	0.0	19.8	-	-	0--0
1	10		26.11	0	1930.9	2803.1	-	-	0.0	19.8	-	-	0--0
1	11		28.73	0	1920.7	2803.1	-	-	0.0	19.8	-	-	0--0
1	12		31.34	0	1890.3	2803.1	-	-	0.0	19.8	-	-	0--0
1	13		33.95	0	1805.4	2803.1	-	-	0.0	19.8	-	-	0--0
1	14		36.56	0	1677.3	2803.1	-	-	0.0	19.8	-	-	0--0
1	15		39.17	0	1512.2	2803.1	-	-	0.0	19.8	-	-	0--0
1	16		41.78	0	1308.6	2803.1	-	-	0.0	19.8	-	-	0--0
1	17		44.40	0	1052.8	2103.1	-	-	0.0	19.8	-	-	0--0
1	18		47.01	0	749.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	19		49.62	0	398.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	20		52.23	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.

1	0	0.00	0	0.000	0.000	1
1	1	2.61	0	0.189	0.189	1
1	2	5.22	0	0.356	0.356	1
1	3	7.83	0	0.501	0.501	1
1	4	10.45	0	0.467	0.467	1
1	5	13.06	0	0.539	0.539	1
1	6	15.67	0	0.598	0.598	1
1	7	18.28	0	0.644	0.644	1
1	8	20.89	0	0.674	0.674	1
1	9	23.50	0	0.685	0.685	1
1	10	26.11	0	0.689	0.689	1
1	11	28.73	0	0.685	0.685	1
1	12	31.34	0	0.674	0.674	1
1	13	33.95	0	0.644	0.644	1
1	14	36.56	0	0.598	0.598	1
1	15	39.17	0	0.539	0.539	1
1	16	41.78	0	0.467	0.467	1
1	17	44.40	0	0.501	0.501	1
1	18	47.01	0	0.356	0.356	1
1	19	49.62	0	0.190	0.190	1
1	20	52.23	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	2.61	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	5.22	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	7.83	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	10.45	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	13.06	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	15.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	18.28	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	20.89	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	23.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	26.11	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	28.73	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	31.34	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	33.95	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	36.56	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	39.17	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	41.78	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

1 17	44.40	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 18	47.01	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 19	49.62	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 20	52.23	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	191.7	<	330.8	0	0
1	1	2.61	0	177.4	<	330.8	0	0
1	2	5.22	0	163.1	<	330.8	0	0
1	3	7.83	0	149.1	<	330.8	0	0
1	4	10.45	0	135.1	<	330.8	0	0
1	5	13.06	0	121.3	<	330.8	0	0
1	6	15.67	0	107.6	<	330.8	0	0
1	7	18.28	0	93.5	<	330.8	0	0
1	8	20.89	0	80.2	<	330.8	0	0
1	9	23.50	0	67.0	<	330.8	0	0
1	10	26.11	0	54.3	<	330.8	0	0

1 11	28.73	0	67.0	<	330.8	0	0
1 12	31.34	0	80.2	<	330.8	0	0
1 13	33.95	0	93.6	<	330.8	0	0
1 14	36.56	0	107.6	<	330.8	0	0
1 15	39.17	0	121.3	<	330.8	0	0
1 16	41.78	0	135.1	<	330.8	0	0
1 17	44.40	0	149.1	<	330.8	0	0
1 18	47.01	0	163.2	<	330.8	0	0
1 19	49.62	0	177.4	<	330.8	0	0
1 20	52.23	0	191.7	<	330.8	0	0

Please read NOTE on the following page

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      NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

               For compact section  (composite sections in positive flexure)

                 w/o long. stiff.

                 D/tw <= 150 (Eq. 6.10.2.1.1-1)

                 w / long. stiff.

                 D/tw <= 300 (Eq. 6.10.2.1.2-1)

                 2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

               For compact or non-compact sections specified in Art. 6.10.8

                 (composite sections in negative flexure and
                  non-composite sections)

                 2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


           [2]: SHEAR CAPACITY ... web crippling control

               Vn = CVp .......... AASHTO Eq.6.10.9.2-1

               Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

     Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

               Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

     Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

               Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

             * For the detailed description of shear capacity,
               please refer to AASHTO Art.6.10.9.2 and .3

             * STRENGTH CATEGORY; 0 = compact section
                                  2 = braced non-compact section
                                  3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************

              YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
 SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
 NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
        (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
 ---------------------------------------------------------------------------
```

1	0	0.00	33.	191.71	330.76	0
1	1	2.61	33.	177.37	330.76	0
1	2	5.22	33.	163.15	330.76	0
1	3	7.83	33.	149.06	330.76	0
1	4	10.45	33.	135.09	330.76	0
1	5	13.06	33.	121.26	330.76	0
1	6	15.67	33.	107.55	330.76	0
1	7	18.28	33.	93.54	330.76	0
1	8	20.89	33.	80.17	330.76	0
1	9	23.50	33.	67.01	330.76	0
1	10	26.11	33.	54.27	330.76	0
1	11	28.73	33.	67.03	330.76	0
1	12	31.34	33.	80.19	330.76	0
1	13	33.95	33.	93.56	330.76	0
1	14	36.56	33.	107.57	330.76	0
1	15	39.17	33.	121.28	330.76	0
1	16	41.78	33.	135.11	330.76	0
1	17	44.40	33.	149.08	330.76	0
1	18	47.01	33.	163.17	330.76	0
1	19	49.62	33.	177.39	330.76	0
1	20	52.23	33.	191.73	330.76	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0		0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1		2.61	2.4	-	31.4	7.3	31.4	-	-	00-
1	2		5.22	4.5	-	31.4	13.7	31.4	-	-	00-
1	3		7.83	6.4	-	31.4	19.2	31.4	-	-	00-
1	4		10.45	7.9	-	31.4	16.2	31.4	-	-	00-
1	5		13.06	9.3	-	31.4	18.8	31.4	-	-	00-
1	6		15.67	10.3	-	31.4	20.8	31.4	-	-	00-
1	7		18.28	11.2	-	31.4	22.4	31.4	-	-	00-
1	8		20.89	11.8	-	31.4	23.5	31.4	-	-	00-
1	9		23.50	12.1	-	31.4	23.9	31.4	-	-	00-
1	10		26.11	12.2	-	31.4	24.1	31.4	-	-	00-
1	11		28.73	12.1	-	31.4	23.9	31.4	-	-	00-
1	12		31.34	11.8	-	31.4	23.5	31.4	-	-	00-
1	13		33.95	11.2	-	31.4	22.4	31.4	-	-	00-
1	14		36.56	10.4	-	31.4	20.8	31.4	-	-	00-
1	15		39.17	9.3	-	31.4	18.8	31.4	-	-	00-
1	16		41.78	7.9	-	31.4	16.2	31.4	-	-	00-
1	17		44.40	6.4	-	31.4	19.2	31.4	-	-	00-
1	18		47.01	4.5	-	31.4	13.7	31.4	-	-	00-
1	19		49.62	2.4	-	31.4	7.3	31.4	-	-	00-
1	20		52.23	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient

= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)

where:

Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D L FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.61	TR	-0.1	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	2	5.22	TR	-0.2	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	3	7.83	TR	-0.3	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1		9.61	SCG					
1	4	10.45	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	5	13.06	TR	-0.6	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1	6	15.67	TR	-0.6	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	7	18.28	TR	-0.6	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	8	20.89	TR	-0.7	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	9	23.50	TR	-0.7	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1	10	26.11	TR	-0.7	A B B^C^C D E E^	TR	5.2	A B B^C^C D
1	11	28.73	TR	-0.7	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1	12	31.34	TR	-0.7	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	13	33.95	TR	-0.6	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	14	36.56	TR	-0.6	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	15	39.17	TR	-0.6	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1	16	41.78	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1		42.61	SCG					
1	17	44.40	TR	-0.3	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	18	47.01	TR	-0.2	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	19	49.62	TR	-0.1	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	20	52.23	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

1

TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.61	TR	-0.1	A B B^C^C D E E^	TR	0.9	A B B^C^C D E E^
1	2	5.22	TR	-0.1	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	3	7.83	TR	-0.1	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1		9.61	SCG					
1	4	10.45	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	5	13.06	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	6	15.67	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	7	18.28	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	8	20.89	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	9	23.50	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	10	26.11	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	11	28.73	TR	-0.3	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	12	31.34	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	13	33.95	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	14	36.56	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	15	39.17	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	16	41.78	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1		42.61	SCG					
1	17	44.40	TR	-0.1	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	18	47.01	TR	-0.1	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	19	49.62	TR	-0.1	A B B^C^C D E E^	TR	0.9	A B B^C^C D E E^
1	20	52.23	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]

1	0	0.00	49.6	.29821E-01	1.5	TR	3.09	3	6.28
1	1	2.61	47.4	.29821E-01	1.4	TR	3.09	3	6.57
1	2	5.22	45.3	.29821E-01	1.4	TR	3.09	3	6.87
1	3	7.83	43.3	.29821E-01	1.3	TR	3.09	3	7.19
1	4	10.45	41.5	.27580E-01	1.1	TR	3.09	3	8.12
1	5	13.06	39.8	.27580E-01	1.1	TR	3.09	3	8.45
1	6	15.67	38.3	.27580E-01	1.1	TR	3.09	3	8.78
1	7	18.28	36.5	.27580E-01	1.0	TR	3.09	3	9.22
1	8	20.89	35.8	.27580E-01	1.0	TR	3.09	3	9.40
1	9	23.50	37.3	.27580E-01	1.0	TR	3.09	3	9.03
1	10	26.11	37.7	.27580E-01	1.0	TR	3.09	3	8.92
1	11	28.73	37.3	.27580E-01	1.0	TR	3.09	3	9.03
1	12	31.34	35.8	.27580E-01	1.0	TR	3.09	3	9.40
1	13	33.95	36.5	.27580E-01	1.0	TR	3.09	3	9.22
1	14	36.56	38.3	.27580E-01	1.1	TR	3.09	3	8.78
1	15	39.17	39.8	.27580E-01	1.1	TR	3.09	3	8.45
1	16	41.78	41.5	.27580E-01	1.1	TR	3.09	3	8.12
1	17	44.40	43.3	.29821E-01	1.3	TR	3.09	3	7.19
1	18	47.01	45.3	.29821E-01	1.4	TR	3.09	3	6.87
1	19	49.62	47.4	.29821E-01	1.4	TR	3.09	3	6.57
1	20	52.23	49.6	.29821E-01	1.5	TR	3.09	3	6.28

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	SHEAR CONN. PER TRANS. SECTION <--Input [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	24.8	.29821E-01	0.7	TR	1.36	3	5.54
1	1	2.61	23.7	.29821E-01	0.7	TR	1.36	3	5.79
1	2	5.22	22.7	.29821E-01	0.7	TR	1.36	3	6.05
1	3	7.83	21.6	.29821E-01	0.6	TR	1.36	3	6.34
1	4	10.45	20.7	.27580E-01	0.6	TR	1.36	3	7.16
1	5	13.06	19.9	.27580E-01	0.5	TR	1.36	3	7.45
1	6	15.67	19.2	.27580E-01	0.5	TR	1.36	3	7.74
1	7	18.28	18.3	.27580E-01	0.5	TR	1.36	3	8.13
1	8	20.89	17.9	.27580E-01	0.5	TR	1.36	3	8.29
1	9	23.50	18.6	.27580E-01	0.5	TR	1.36	3	7.96
1	10	26.11	18.9	.27580E-01	0.5	TR	1.36	3	7.86
1	11	28.73	18.6	.27580E-01	0.5	TR	1.36	3	7.96
1	12	31.34	17.9	.27580E-01	0.5	TR	1.36	3	8.29
1	13	33.95	18.3	.27580E-01	0.5	TR	1.36	3	8.13
1	14	36.56	19.2	.27580E-01	0.5	TR	1.36	3	7.74
1	15	39.17	19.9	.27580E-01	0.5	TR	1.36	3	7.45
1	16	41.78	20.7	.27580E-01	0.6	TR	1.36	3	7.16
1	17	44.40	21.6	.29821E-01	0.6	TR	1.36	3	6.34
1	18	47.01	22.7	.29821E-01	0.7	TR	1.36	3	6.05
1	19	49.62	23.7	.29821E-01	0.7	TR	1.36	3	5.79
1	20	52.23	24.8	.29821E-01	0.7	TR	1.36	3	5.54

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D L (ft)	FROM SUPT 1=POS. 0=NEG.	MOMENT REGION (k)	P2p,1n (k)	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) LRFD 6.10.10.4.2 FATIG. CRI.	<=>	SEE NOTE N1	STATUS N2 BLANK=OK CHECK=**
1	10	26.11	1	1379.4	1626.6			159	>	72	
1	20	52.23	1	1379.4	1626.6			159	>	72	

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange

bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	26.115	6.000
1	26.115	52.230	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	52.2	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.

```
 OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

 OK     1.2.22.4=DEPTH RATIOS

 OK     1.2.22.10=CONSTRUCBILITY CHECK

 OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

 OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

 OK     1.2.22.21=SERVICE LIMIT STATE CHECK


        MORE TABLES TO BE INSPECTED ...

        1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

        1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

        1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

        1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
               ***********************************

                            UNFACTORED   UNFACTORED L+I
                            DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT        MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
 NO NO L SUPT  CAP.                      --------------- FACTOR  LOAD  RATING  T
        (ft)  (k-ft)       D1; k-ft  D2     POS.     NEG.  [3]   TYPE
            or STRESS          ksi        MAX. STRESS
 ------------------------------------------------------------------------------

  1  0   0.00   2103.1      0.0      0.0      0.0      0.0  9.99  HL-93  9.99  0
  1  1   2.61   2103.1     56.5     29.5    163.6      0.0  9.02  HL-93  6.95  0
  1  2   5.22   2103.1    107.2     55.9    306.7      0.0  4.57  HL-93  3.52  0
  1  3   7.83   2103.1    152.0     79.1    429.2      0.0  3.11  HL-93  2.40  0
  1  4  10.45   2803.1    190.9     99.3    531.3      0.0  3.38  HL-93  2.61  0
  1  5  13.06   2803.1    223.9    116.4    610.3      0.0  2.86  HL-93  2.21  0
  1  6  15.67   2803.1    251.0    130.3    673.9      0.0  2.53  HL-93  1.95  0
  1  7  18.28   2803.1    272.1    141.2    723.6      0.0  2.32  HL-93  1.79  0
  1  8  20.89   2803.1    286.8    148.9    755.3      0.0  2.19  HL-93  1.69  0
  1  9  23.50   2803.1    295.5    153.6    762.7      0.0  2.15  HL-93  1.66  0
  1 10  26.11   2803.1    298.5    155.2    765.2      0.0  2.14  HL-93  1.65  0
  1 11  28.73   2803.1    295.5    153.6    762.7      0.0  2.15  HL-93  1.66  0
  1 12  31.34   2803.1    286.7    148.9    755.3      0.0  2.19  HL-93  1.69  0
  1 13  33.95   2803.1    272.0    141.2    723.6      0.0  2.32  HL-93  1.79  0
  1 14  36.56   2803.1    251.3    130.3    673.9      0.0  2.53  HL-93  1.95  0
  1 15  39.17   2803.1    224.1    116.4    610.3      0.0  2.86  HL-93  2.21  0
  1 16  41.78   2803.1    191.0     99.3    531.3      0.0  3.38  HL-93  2.61  0
  1 17  44.40   2103.1    152.1     79.1    429.2      0.0  3.11  HL-93  2.40  0
  1 18  47.01   2103.1    107.3     55.9    306.7      0.0  4.57  HL-93  3.52  0
  1 19  49.62   2103.1     56.6     29.5    163.6      0.0  9.02  HL-93  6.95  0
  1 20  52.23   2103.1      0.0      0.0      0.0      0.0  9.99  HL-93  9.99  0


 ------------------------------------------------------------------------------
   Please read NOTES on the following page
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                                                                   RATING
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          NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

                    ---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

               [2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION
```

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	2.61	31.4	-2.10	2.60	-0.22	3.59	0.00	0.00	8.01	HL-93	6.17
1	2	5.22	31.4	-3.99	4.94	-0.42	6.72	0.00	0.00	3.93	HL-93	3.02
1	3	7.83	31.4	-5.65	7.00	-0.58	9.41	0.00	0.00	2.59	HL-93	1.99
1	4	10.45	31.4	-6.55	5.88	-1.05	7.94	0.00	0.00	3.21	HL-93	2.47
1	5	13.06	31.4	-7.68	6.90	-1.21	9.12	0.00	0.00	2.68	HL-93	2.06
1	6	15.67	31.4	-8.61	7.73	-1.33	10.07	0.00	0.00	2.34	HL-93	1.80
1	7	18.28	31.4	-9.34	8.38	-1.43	10.82	0.00	0.00	2.12	HL-93	1.63
1	8	20.89	31.4	-9.84	8.84	-1.49	11.29	0.00	0.00	1.99	HL-93	1.53
1	9	23.50	31.4	-10.14	9.11	-1.51	11.40	0.00	0.00	1.95	HL-93	1.50
1	10	26.11	31.4	-10.24	9.20	-1.51	11.44	0.00	0.00	1.94	HL-93	1.49
1	11	28.73	31.4	-10.14	9.11	-1.51	11.40	0.00	0.00	1.95	HL-93	1.50
1	12	31.34	31.4	-9.84	8.83	-1.49	11.29	0.00	0.00	1.99	HL-93	1.53
1	13	33.95	31.4	-9.33	8.38	-1.43	10.82	0.00	0.00	2.12	HL-93	1.63
1	14	36.56	31.4	-8.62	7.74	-1.33	10.07	0.00	0.00	2.34	HL-93	1.80
1	15	39.17	31.4	-7.69	6.90	-1.21	9.12	0.00	0.00	2.68	HL-93	2.06
1	16	41.78	31.4	-6.56	5.89	-1.05	7.94	0.00	0.00	3.21	HL-93	2.47
1	17	44.40	31.4	-5.66	7.00	-0.58	9.41	0.00	0.00	2.59	HL-93	1.99
1	18	47.01	31.4	-3.99	4.94	-0.42	6.72	0.00	0.00	3.93	HL-93	3.02
1	19	49.62	31.4	-2.10	2.61	-0.22	3.59	0.00	0.00	8.01	HL-93	6.16
1	20	52.23	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

5969 Span 2-S16 Interior Fatigue and Mid M.RES

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	52.23	26.11	STRENGTH L.S.	2.14	1.65	HL-93
1	52.23	26.11	SERVICEBILITY	1.94	1.49	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.937
Inventory= 1.490

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	52.23	26.11	STRENGTH L.S.	3.06	721.49
1	52.23	26.11	SERVICEBILITY	2.63	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.634

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I SHEAR MAX. POS.	SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	22.8	11.9	83.7	0.0	2.53	HL-93	1.95
1	1	2.61	330.8	20.5	10.7	78.1	-1.9	2.75	HL-93	2.12
1	2	5.22	330.8	18.3	9.5	72.5	-3.9	3.01	HL-93	2.32
1	3	7.83	330.8	16.0	8.3	67.0	-6.8	3.30	HL-93	2.55
1	4	10.45	330.8	13.8	7.1	61.6	-10.0	3.65	HL-93	2.81
1	5	13.06	330.8	11.5	5.9	56.3	-13.3	4.05	HL-93	3.13
1	6	15.67	330.8	9.3	4.8	51.0	-16.6	4.54	HL-93	3.50
1	7	18.28	330.8	6.7	3.6	45.8	-20.0	5.14	HL-93	3.96
1	8	20.89	330.8	4.5	2.4	40.7	-23.5	5.86	HL-93	4.52

1	9	23.50	330.8	2.2	1.2	35.7	-27.0	6.76	HL-93	5.22
1	10	26.11	330.8	0.0	0.0	31.0	-31.0	7.90	HL-93	6.10
1	11	28.73	330.8	-2.3	-1.2	27.0	-35.7	6.76	HL-93	5.22
1	12	31.34	330.8	-4.5	-2.4	23.5	-40.7	5.86	HL-93	4.52
1	13	33.95	330.8	-6.8	-3.6	20.0	-45.8	5.14	HL-93	3.96
1	14	36.56	330.8	-9.3	-4.8	16.6	-51.0	4.54	HL-93	3.50
1	15	39.17	330.8	-11.5	-5.9	13.3	-56.3	4.05	HL-93	3.13
1	16	41.78	330.8	-13.8	-7.1	10.0	-61.6	3.65	HL-93	2.81
1	17	44.40	330.8	-16.0	-8.3	6.8	-67.0	3.30	HL-93	2.55
1	18	47.01	330.8	-18.3	-9.5	3.9	-72.5	3.01	HL-93	2.32
1	19	49.62	330.8	-20.5	-10.7	1.9	-78.1	2.75	HL-93	2.12
1	20	52.23	330.8	-22.8	-11.9	0.0	-83.7	2.53	HL-93	1.95

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	52.23	52.23	STRENGTH L.S.	2.53	1.95	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.527
Inventory= 1.949

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	52.23	52.23	STRENGTH L.S.	3.13	91.23

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.130

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	26.11	3	0.425	-0.26	HL-93 3.02
		4	0.433	-0.26	2.96

5969 Span 2-S16 Interior Fatigue and Mid M.RES

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span2-S17 Interior Strength Mid M	8/20

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	20	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

5969 Span 2-S17 Interior Strength Mid M.RES

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
54.45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.9375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	9.73				33.	33.	33.
2	2	2	35.00				33.	33.	33.
3	1	1	9.73				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

5969 Span 2-S17 Interior Strength Mid M.RES

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

	OPTIONAL CALCULATION OF FACTOR						OPTIONAL LOADING TYPES DISTRIBUTION FACTORS		
SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C	
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	54.45

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

5969 Span 2-S17 Interior Strength Mid M.RES

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	3.68
2	2	Diaphragm	0.000	0.00	0.00	0.29	22.52
3	2	Diaphragm	0.000	0.00	0.00	0.29	41.36
4	1	Railing	0.087	0.00	54.45	0.00	0.00
5	1	Curb	0.076	0.00	54.45	0.00	0.00
6	0	Pavement	0.292	0.00	54.45	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE			SLAB REINFORCEMENT		CONCRETE	
NO.	DIA.	CONNECTOR IN NEGAT. PER M. REGION TRAN. SEC (in) 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	DIST. FROM TOP (in**2)	(in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	3.68	33.0	33.0	3.68
3.68	22.52	33.0	33.0	18.84
22.52	41.36	33.0	33.0	18.84
41.36	54.45	33.0	33.0	13.09

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.727	0.1386	0.8656

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	54.45
1	DC2	0.087	0.000	54.450
2	DC2	0.076	0.000	54.450

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1322.1	49.0	198.3	45.3	243.59

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.711	0.711	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.72	5900.0	32.9	16.45	358.7	358.7
1	2	5.45	5900.0	32.9	16.45	358.7	358.7
1	3	8.17	5900.0	32.9	16.45	358.7	358.7
1	4	10.89	8019.6	32.9	13.79	581.6	400.0
1	5	13.61	8019.6	32.9	13.79	581.6	400.0
1	6	16.34	8019.6	32.9	13.79	581.6	400.0
1	7	19.06	8019.6	32.9	13.79	581.6	400.0
1	8	21.78	8019.6	32.9	13.79	581.6	400.0

1	9	24.50	8019.6	32.9	13.79	581.6	400.0
1	10	27.23	8019.6	32.9	13.79	581.6	400.0
1	11	29.95	8019.6	32.9	13.79	581.6	400.0
1	12	32.67	8019.6	32.9	13.79	581.6	400.0
1	13	35.39	8019.6	32.9	13.79	581.6	400.0
1	14	38.12	8019.6	32.9	13.79	581.6	400.0
1	15	40.84	8019.6	32.9	13.79	581.6	400.0
1	16	43.56	8019.6	32.9	13.79	581.6	400.0
1	17	46.28	5900.0	32.9	16.45	358.7	358.7
1	18	49.01	5900.0	32.9	16.45	358.7	358.7
1	19	51.73	5900.0	32.9	16.45	358.7	358.7
1	20	54.45	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12575.9	0.024742	496.1	1666.1	788.1
1	1	2.72	12575.9	0.024742	496.1	1666.1	788.1
1	2	5.45	12575.9	0.024742	496.1	1666.1	788.1
1	3	8.17	12575.9	0.024742	496.1	1666.1	788.1
1	4	10.89	17985.3	0.022239	789.3	1627.6	924.2
1	5	13.61	17985.3	0.022239	789.3	1627.6	924.2
1	6	16.34	17985.3	0.022239	789.3	1627.6	924.2
1	7	19.06	17985.3	0.022239	789.3	1627.6	924.2
1	8	21.78	17985.3	0.022239	789.3	1627.6	924.2
1	9	24.50	17985.3	0.022239	789.3	1627.6	924.2
1	10	27.23	17985.3	0.022239	789.3	1627.6	924.2
1	11	29.95	17985.3	0.022239	789.3	1627.6	924.2
1	12	32.67	17985.3	0.022239	789.3	1627.6	924.2
1	13	35.39	17985.3	0.022239	789.3	1627.6	924.2
1	14	38.12	17985.3	0.022239	789.3	1627.6	924.2
1	15	40.84	17985.3	0.022239	789.3	1627.6	924.2
1	16	43.56	17985.3	0.022239	789.3	1627.6	924.2
1	17	46.28	12575.9	0.024742	496.1	1666.1	788.1
1	18	49.01	12575.9	0.024742	496.1	1666.1	788.1
1	19	51.73	12575.9	0.024742	496.1	1666.1	788.1
1	20	54.45	12575.9	0.024742	496.1	1666.1	788.1

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO IN NO D FROM L SUPT (ft) | MOMENT OF INERTIA Ix (in**4) | Q/Ix Q=1ST. MOMENT OF INERTIA (1/in) | ELASTIC SECTION MODULUS,(in**3) STEEL BOT. | STEEL TOP. | CONCRETE(SLAB) TOP.

1	0	0.00	16953.0	0.029821	547.3	8815.3	1640.6
1	1	2.72	16953.0	0.029821	547.3	8815.3	1640.6
1	2	5.45	16953.0	0.029821	547.3	8815.3	1640.6
1	3	8.17	16953.0	0.029821	547.3	8815.3	1640.6
1	4	10.89	25501.1	0.027202	866.2	5800.7	1991.3
1	5	13.61	25501.1	0.027202	866.2	5800.7	1991.3
1	6	16.34	25501.1	0.027202	866.2	5800.7	1991.3
1	7	19.06	25501.1	0.027202	866.2	5800.7	1991.3
1	8	21.78	25501.1	0.027202	866.2	5800.7	1991.3
1	9	24.50	25501.1	0.027202	866.2	5800.7	1991.3
1	10	27.23	25501.1	0.027202	866.2	5800.7	1991.3
1	11	29.95	25501.1	0.027202	866.2	5800.7	1991.3
1	12	32.67	25501.1	0.027202	866.2	5800.7	1991.3
1	13	35.39	25501.1	0.027202	866.2	5800.7	1991.3
1	14	38.12	25501.1	0.027202	866.2	5800.7	1991.3
1	15	40.84	25501.1	0.027202	866.2	5800.7	1991.3
1	16	43.56	25501.1	0.027202	866.2	5800.7	1991.3
1	17	46.28	16953.0	0.029821	547.3	8815.3	1640.6
1	18	49.01	16953.0	0.029821	547.3	8815.3	1640.6
1	19	51.73	16953.0	0.029821	547.3	8815.3	1640.6
1	20	54.45	16953.0	0.029821	547.3	8815.3	1640.6

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	2.72	9.8	51.2	1.0	0.0	62.0	
1	2	5.45	18.5	97.0	1.8	0.0	117.3	

1	3	8.17	26.2	137.4	2.4	0.0	166.0
1	4	10.89	32.9	172.4	3.0	0.0	208.3
1	5	13.61	38.5	202.1	3.6	0.0	244.2
1	6	16.34	43.1	226.3	4.2	0.0	273.7
1	7	19.06	46.7	245.2	4.9	0.0	296.8
1	8	21.78	49.3	258.6	5.5	0.0	313.4
1	9	24.50	50.8	266.7	5.5	0.0	323.1
1	10	27.23	51.4	269.4	5.4	0.0	326.2
1	11	29.95	50.8	266.7	5.2	0.0	322.8
1	12	32.67	49.3	258.6	5.1	0.0	313.0
1	13	35.39	46.7	245.2	4.9	0.0	296.8
1	14	38.12	43.1	226.3	4.7	0.0	274.2
1	15	40.84	38.5	202.1	4.6	0.0	245.2
1	16	43.56	32.9	172.4	3.8	0.0	209.1
1	17	46.28	26.2	137.4	2.8	0.0	166.4
1	18	49.01	18.5	97.0	1.9	0.0	117.4
1	19	51.73	9.8	51.2	0.9	0.0	61.9
1	20	54.45	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.72	20.6	11.5	0.0	0.0	0.0	32.0
1	2	5.45	39.0	21.7	0.0	0.0	0.0	60.7
1	3	8.17	55.2	30.8	0.0	0.0	0.0	86.0
1	4	10.89	69.3	38.7	0.0	0.0	0.0	107.9
1	5	13.61	81.2	45.3	0.0	0.0	0.0	126.5
1	6	16.34	90.9	50.7	0.0	0.0	0.0	141.6
1	7	19.06	98.5	55.0	0.0	0.0	0.0	153.4
1	8	21.78	103.9	58.0	0.0	0.0	0.0	161.9
1	9	24.50	107.1	59.8	0.0	0.0	0.0	166.9
1	10	27.23	108.2	60.4	0.0	0.0	0.0	168.6
1	11	29.95	107.1	59.8	0.0	0.0	0.0	166.9
1	12	32.67	103.9	58.0	0.0	0.0	0.0	161.9
1	13	35.39	98.5	55.0	0.0	0.0	0.0	153.4
1	14	38.12	90.9	50.7	0.0	0.0	0.0	141.6
1	15	40.84	81.2	45.3	0.0	0.0	0.0	126.5
1	16	43.56	69.3	38.7	0.0	0.0	0.0	107.9
1	17	46.28	55.2	30.8	0.0	0.0	0.0	86.0
1	18	49.01	39.0	21.7	0.0	0.0	0.0	60.7
1	19	51.73	20.6	11.5	0.0	0.0	0.0	32.0
1	20	54.45	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE

1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.72	0.0	0.0	173.1	HL-93	0.0	HL-93
1	2	5.45	0.0	0.0	324.8	HL-93	0.0	HL-93
1	3	8.17	0.0	0.0	454.9	HL-93	0.0	HL-93
1	4	10.89	0.0	0.0	563.5	HL-93	0.0	HL-93
1	5	13.61	0.0	0.0	648.0	HL-93	0.0	HL-93
1	6	16.34	0.0	0.0	716.2	HL-93	0.0	HL-93
1	7	19.06	0.0	0.0	769.4	HL-93	0.0	HL-93
1	8	21.78	0.0	0.0	803.7	HL-93	0.0	HL-93
1	9	24.50	0.0	0.0	812.5	HL-93	0.0	HL-93
1	10	27.23	0.0	0.0	815.5	HL-93	0.0	HL-93
1	11	29.95	0.0	0.0	812.5	HL-93	0.0	HL-93
1	12	32.67	0.0	0.0	803.7	HL-93	0.0	HL-93
1	13	35.39	0.0	0.0	769.4	HL-93	0.0	HL-93
1	14	38.12	0.0	0.0	716.2	HL-93	0.0	HL-93
1	15	40.84	0.0	0.0	648.0	HL-93	0.0	HL-93
1	16	43.56	0.0	0.0	563.5	HL-93	0.0	HL-93
1	17	46.28	0.0	0.0	454.9	HL-93	0.0	HL-93
1	18	49.01	0.0	0.0	324.8	HL-93	0.0	HL-93
1	19	51.73	0.0	0.0	173.1	HL-93	0.0	HL-93
1	20	54.45	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.72	60.	0.	60.
1	2	5.45	110.	0.	110.
1	3	8.17	148.	0.	148.
1	4	10.89	181.	0.	181.
1	5	13.61	207.	0.	207.
1	6	16.34	226.	0.	226.
1	7	19.06	236.	0.	236.
1	8	21.78	242.	0.	242.
1	9	24.50	242.	0.	242.
1	10	27.23	242.	0.	242.
1	11	29.95	242.	0.	242.
1	12	32.67	242.	0.	242.
1	13	35.39	236.	0.	236.
1	14	38.12	226.	0.	226.
1	15	40.84	207.	0.	207.
1	16	43.56	181.	0.	181.
1	17	46.28	148.	0.	148.
1	18	49.01	110.	0.	110.
1	19	51.73	60.	0.	60.
1	20	54.45	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT	DEAD LOAD NON COMP. N=Infin.	COMP. N=27.0	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE	GOVERN LOAD	MAXIMUM NEGATIVE LOAD	GOVERN	TOTAL MAXIMUM POSITIVE	NEGATIVE

5969 Span 2-S17 Interior Strength Mid M.RES

		(ft)	(k-ft)	(k-ft)	(k-ft)	TYPE	(k-ft)	TYPE	(k-ft)	(k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.72	62.0	32.0	173.1	HL-93	0.0	HL-93	267.1	94.0
1	2	5.45	117.3	60.7	324.8	HL-93	0.0	HL-93	502.7	178.0
1	3	8.17	166.0	86.0	454.9	HL-93	0.0	HL-93	706.9	252.0
1	4	10.89	208.3	107.9	563.5	HL-93	0.0	HL-93	879.7	316.2
1	5	13.61	244.2	126.5	648.0	HL-93	0.0	HL-93	1018.7	370.7
1	6	16.34	273.7	141.6	716.2	HL-93	0.0	HL-93	1131.5	415.4
1	7	19.06	296.8	153.4	769.4	HL-93	0.0	HL-93	1219.6	450.2
1	8	21.78	313.4	161.9	803.7	HL-93	0.0	HL-93	1279.0	475.3
1	9	24.50	323.1	166.9	812.5	HL-93	0.0	HL-93	1302.6	490.1
1	10	27.23	326.2	168.6	815.5	HL-93	0.0	HL-93	1310.3	494.8
1	11	29.95	322.8	166.9	812.5	HL-93	0.0	HL-93	1302.2	489.7
1	12	32.67	313.0	161.9	803.7	HL-93	0.0	HL-93	1278.5	474.9
1	13	35.39	296.8	153.4	769.4	HL-93	0.0	HL-93	1219.6	450.3
1	14	38.12	274.2	141.6	716.2	HL-93	0.0	HL-93	1132.0	415.8
1	15	40.84	245.2	126.5	648.0	HL-93	0.0	HL-93	1019.6	371.6
1	16	43.56	209.1	107.9	563.5	HL-93	0.0	HL-93	880.5	317.0
1	17	46.28	166.4	86.0	454.9	HL-93	0.0	HL-93	707.3	252.4
1	18	49.01	117.4	60.7	324.8	HL-93	0.0	HL-93	502.8	178.1
1	19	51.73	61.9	32.0	173.1	HL-93	0.0	HL-93	267.1	93.9
1	20	54.45	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.72	267.1	319.1	425.6	356.4	141.0	44.7
1	2	5.45	502.7	600.2	800.5	670.6	266.9	82.2
1	3	8.17	706.9	843.3	1124.8	942.9	378.0	111.3
1	4	10.89	879.7	1048.8	1398.7	1173.3	474.3	135.6
1	5	13.61	1018.7	1213.1	1617.6	1358.4	556.0	155.2
1	6	16.34	1131.5	1346.4	1795.3	1508.8	623.0	169.4
1	7	19.06	1219.6	1450.4	1933.8	1626.1	675.3	177.1
1	8	21.78	1279.0	1520.1	2026.5	1705.1	713.0	181.6
1	9	24.50	1302.6	1546.3	2061.3	1736.3	735.1	181.2
1	10	27.23	1310.3	1554.9	2072.6	1746.4	742.2	181.1
1	11	29.95	1302.2	1546.0	2060.9	1735.8	734.6	181.2
1	12	32.67	1278.5	1519.6	2026.0	1704.5	712.3	181.6
1	13	35.39	1219.6	1450.4	1933.8	1626.1	675.4	177.1
1	14	38.12	1132.0	1346.9	1795.9	1509.4	623.7	169.4
1	15	40.84	1019.6	1214.0	1618.8	1359.6	557.4	155.2
1	16	43.56	880.5	1049.5	1399.7	1174.3	475.5	135.6
1	17	46.28	707.3	843.8	1125.4	943.4	378.6	111.3
1	18	49.01	502.8	600.3	800.7	670.8	267.1	82.2
1	19	51.73	267.1	319.0	425.5	356.3	140.9	44.7
1	20	54.45	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

5969 Span 2-S17 Interior Strength Mid M.RES

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL DEAD LOAD (kip)
1	0	0.00	3.8	19.8	0.4	0.0	23.9
1	1	2.72	3.4	17.8	0.4	0.0	21.6
1	2	5.45	3.0	15.8	0.2	0.0	19.1
1	3	8.17	2.6	13.9	0.2	0.0	16.7
1	4	10.89	2.3	11.9	0.2	0.0	14.4
1	5	13.61	1.9	9.9	0.2	0.0	12.0
1	6	16.34	1.5	7.9	0.2	0.0	9.7
1	7	19.06	1.1	5.9	0.2	0.0	7.3
1	8	21.78	0.8	4.0	0.2	0.0	4.9
1	9	24.50	0.4	2.0	-0.1	0.0	2.3
1	10	27.23	0.0	0.0	-0.1	0.0	-0.1
1	11	29.95	-0.4	-2.0	-0.1	0.0	-2.4
1	12	32.67	-0.8	-4.0	-0.1	0.0	-4.8
1	13	35.39	-1.1	-5.9	-0.1	0.0	-7.1
1	14	38.12	-1.5	-7.9	-0.1	0.0	-9.5
1	15	40.84	-1.9	-9.9	-0.1	0.0	-11.8
1	16	43.56	-2.3	-11.9	-0.3	0.0	-14.5
1	17	46.28	-2.6	-13.9	-0.3	0.0	-16.8
1	18	49.01	-3.0	-15.8	-0.3	0.0	-19.2
1	19	51.73	-3.4	-17.8	-0.3	0.0	-21.6
1	20	54.45	-3.8	-19.8	-0.3	0.0	-23.9

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.9	4.4	0.0	0.0	12.4	23.9	36.3
1	1	2.72	7.2	4.0	0.0	0.0	11.1	21.6	32.7
1	2	5.45	6.4	3.6	0.0	0.0	9.9	19.1	29.0
1	3	8.17	5.6	3.1	0.0	0.0	8.7	16.7	25.4
1	4	10.89	4.8	2.7	0.0	0.0	7.4	14.4	21.8
1	5	13.61	4.0	2.2	0.0	0.0	6.2	12.0	18.2
1	6	16.34	3.2	1.8	0.0	0.0	5.0	9.7	14.6
1	7	19.06	2.4	1.3	0.0	0.0	3.7	7.3	11.0
1	8	21.78	1.6	0.9	0.0	0.0	2.5	4.9	7.4
1	9	24.50	0.8	0.4	0.0	0.0	1.2	2.3	3.5
1	10	27.23	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1
1	11	29.95	-0.8	-0.4	0.0	0.0	-1.2	-2.4	-3.7
1	12	32.67	-1.6	-0.9	0.0	0.0	-2.5	-4.8	-7.2
1	13	35.39	-2.4	-1.3	0.0	0.0	-3.7	-7.1	-10.8
1	14	38.12	-3.2	-1.8	0.0	0.0	-5.0	-9.5	-14.4
1	15	40.84	-4.0	-2.2	0.0	0.0	-6.2	-11.8	-18.0
1	16	43.56	-4.8	-2.7	0.0	0.0	-7.4	-14.5	-21.9
1	17	46.28	-5.6	-3.1	0.0	0.0	-8.7	-16.8	-25.5
1	18	49.01	-6.4	-3.6	0.0	0.0	-9.9	-19.2	-29.1
1	19	51.73	-7.2	-4.0	0.0	0.0	-11.1	-21.6	-32.7
1	20	54.45	-7.9	-4.4	0.0	0.0	-12.4	-23.9	-36.3

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	84.9	HL-93	0.0	HL-93
1	1	2.72	0.0	0.0	79.3	HL-93	-1.9	HL-93
1	2	5.45	0.0	0.0	73.6	HL-93	-3.9	HL-93
1	3	8.17	0.0	0.0	68.1	HL-93	-6.9	HL-93
1	4	10.89	0.0	0.0	62.6	HL-93	-10.1	HL-93
1	5	13.61	0.0	0.0	57.2	HL-93	-13.4	HL-93
1	6	16.34	0.0	0.0	51.9	HL-93	-16.7	HL-93
1	7	19.06	0.0	0.0	46.7	HL-93	-20.2	HL-93
1	8	21.78	0.0	0.0	41.5	HL-93	-23.6	HL-93
1	9	24.50	0.0	0.0	36.4	HL-93	-27.2	HL-93
1	10	27.23	0.0	0.0	31.6	HL-93	-31.6	HL-93
1	11	29.95	0.0	0.0	27.2	HL-93	-36.4	HL-93
1	12	32.67	0.0	0.0	23.6	HL-93	-41.5	HL-93
1	13	35.39	0.0	0.0	20.2	HL-93	-46.7	HL-93
1	14	38.12	0.0	0.0	16.7	HL-93	-51.9	HL-93
1	15	40.84	0.0	0.0	13.4	HL-93	-57.2	HL-93
1	16	43.56	0.0	0.0	10.1	HL-93	-62.6	HL-93
1	17	46.28	0.0	0.0	6.9	HL-93	-68.1	HL-93
1	18	49.01	0.0	0.0	3.9	HL-93	-73.6	HL-93
1	19	51.73	0.0	0.0	1.9	HL-93	-79.3	HL-93
1	20	54.45	0.0	0.0	0.0	HL-93	-84.9	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	34.	0.	34.
1	1	2.72	31.	-1.	32.
1	2	5.45	29.	-2.	31.
1	3	8.17	26.	-3.	29.
1	4	10.89	24.	-5.	28.
1	5	13.61	21.	-6.	27.
1	6	16.34	19.	-7.	26.
1	7	19.06	17.	-8.	25.
1	8	21.78	15.	-10.	24.
1	9	24.50	14.	-11.	25.
1	10	27.23	13.	-13.	25.
1	11	29.95	11.	-14.	25.
1	12	32.67	10.	-15.	24.
1	13	35.39	8.	-17.	25.
1	14	38.12	7.	-19.	26.
1	15	40.84	6.	-21.	27.
1	16	43.56	5.	-24.	28.
1	17	46.28	3.	-26.	29.
1	18	49.01	2.	-29.	31.
1	19	51.73	1.	-31.	32.
1	20	54.45	0.	-34.	34.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	33.8	.31015E-01	0.52	0.2500*	.14206E-01	0.24	0.2500*
1	1	2.72	32.3	.31015E-01	0.50	0.2500*	.14206E-01	0.23	0.2500*
1	2	5.45	30.9	.31015E-01	0.48	0.2500*	.14206E-01	0.22	0.2500*
1	3	8.17	29.5	.31015E-01	0.46	0.2500*	.14206E-01	0.21	0.2500*
1	4	10.89	28.2	.28798E-01	0.41	0.2500*	.86429E-02	0.12	0.2500*
1	5	13.61	27.1	.28798E-01	0.39	0.2500*	.86429E-02	0.12	0.2500*
1	6	16.34	26.1	.28798E-01	0.38	0.2500*	.86429E-02	0.11	0.2500*
1	7	19.06	25.0	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	8	21.78	24.5	.28798E-01	0.35	0.2500*	.86429E-02	0.11	0.2500*
1	9	24.50	25.1	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	10	27.23	25.3	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	11	29.95	25.1	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	12	32.67	24.5	.28798E-01	0.35	0.2500*	.86429E-02	0.11	0.2500*
1	13	35.39	25.0	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	14	38.12	26.1	.28798E-01	0.38	0.2500*	.86429E-02	0.11	0.2500*
1	15	40.84	27.1	.28798E-01	0.39	0.2500*	.86429E-02	0.12	0.2500*
1	16	43.56	28.2	.28798E-01	0.41	0.2500*	.86429E-02	0.12	0.2500*
1	17	46.28	29.5	.31015E-01	0.46	0.2500*	.14206E-01	0.21	0.2500*
1	18	49.01	30.9	.31015E-01	0.48	0.2500*	.14206E-01	0.22	0.2500*
1	19	51.73	32.3	.31015E-01	0.50	0.2500*	.14206E-01	0.23	0.2500*
1	20	54.45	33.8	.31015E-01	0.52	0.2500*	.14206E-01	0.24	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	36.3	84.9	HL-93	0.0	HL-93	121.3	36.3
1	1	2.72	32.7	79.3	HL-93	-1.9	HL-93	112.0	30.8

1	2	5.45	29.0	73.6	HL-93	-3.9	HL-93	102.6	25.1
1	3	8.17	25.4	68.1	HL-93	-6.9	HL-93	93.5	18.4
1	4	10.89	21.8	62.6	HL-93	-10.1	HL-93	84.4	11.7
1	5	13.61	18.2	57.2	HL-93	-13.4	HL-93	75.4	4.8
1	6	16.34	14.6	51.9	HL-93	-16.7	HL-93	66.5	-2.1
1	7	19.06	11.0	46.7	HL-93	-20.2	HL-93	57.7	-9.1
1	8	21.78	7.4	41.5	HL-93	-23.6	HL-93	48.9	-16.2
1	9	24.50	3.5	36.4	HL-93	-27.2	HL-93	39.9	-23.7
1	10	27.23	-0.1	31.6	HL-93	-31.6	HL-93	31.5	-31.6
1	11	29.95	-3.7	27.2	HL-93	-36.4	HL-93	23.6	-40.1
1	12	32.67	-7.2	23.6	HL-93	-41.5	HL-93	16.4	-48.8
1	13	35.39	-10.8	20.2	HL-93	-46.7	HL-93	9.3	-57.5
1	14	38.12	-14.4	16.7	HL-93	-51.9	HL-93	2.3	-66.4
1	15	40.84	-18.0	13.4	HL-93	-57.2	HL-93	-4.6	-75.3
1	16	43.56	-21.9	10.1	HL-93	-62.6	HL-93	-11.8	-84.5
1	17	46.28	-25.5	6.9	HL-93	-68.1	HL-93	-18.6	-93.6
1	18	49.01	-29.1	3.9	HL-93	-73.6	HL-93	-25.2	-102.7
1	19	51.73	-32.7	1.9	HL-93	-79.3	HL-93	-30.8	-112.0
1	20	54.45	-36.3	0.0	HL-93	-84.9	HL-93	-36.3	-121.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	121.3	146.8	196.1	162.1	54.5	25.3
1	1	2.72	112.0	135.8	181.4	149.7	49.1	24.3
1	2	5.45	102.6	124.7	166.7	137.2	43.5	23.2
1	3	8.17	93.5	113.9	152.3	125.1	38.1	22.1
1	4	10.89	84.4	103.2	138.0	113.0	32.7	21.1
1	5	13.61	75.4	92.6	123.9	101.0	27.3	20.3
1	6	16.34	66.5	82.1	109.9	89.2	21.9	19.6
1	7	19.06	57.7	71.7	96.0	77.4	16.5	18.7
1	8	21.78	48.9	61.4	82.3	65.7	11.1	18.3
1	9	24.50	39.9	50.9	68.3	53.8	5.3	18.8
1	10	27.23	-31.6	-41.1	-55.3	-42.7	-0.1	19.0
1	11	29.95	-40.1	-51.0	-68.5	-53.9	-5.5	18.8
1	12	32.67	-48.8	-61.2	-82.1	-65.5	-10.9	18.3
1	13	35.39	-57.5	-71.5	-95.8	-77.2	-16.3	18.7
1	14	38.12	-66.4	-81.9	-109.7	-88.9	-21.7	19.6
1	15	40.84	-75.3	-92.4	-123.7	-100.8	-27.1	20.3
1	16	43.56	-84.5	-103.3	-138.2	-113.1	-32.9	21.1
1	17	46.28	-93.6	-114.0	-152.4	-125.2	-38.3	22.1
1	18	49.01	-102.7	-124.8	-166.8	-137.4	-43.7	23.2
1	19	51.73	-112.0	-135.7	-181.4	-149.7	-49.1	24.3
1	20	54.45	-121.2	-146.7	-196.0	-162.0	-54.4	25.3

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM

1	36.32	0.00	HL-93	84.95	HL-93	ST1	30.70	196.05
						ST2	30.70	162.07
						ST4	42.56	54.49
						SE1	36.32	121.27
						SE2	36.32	146.76
2	36.30	0.00	HL-93	84.95	HL-93	ST1	30.68	196.02
						ST2	30.68	162.04
						ST4	42.52	54.45
						SE1	36.30	121.25
						SE2	36.30	146.73

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.72	-0.0199	-0.1043	0.0000	-0.0294	-0.1262	-0.0294	-0.1556
1	2	5.45	-0.0390	-0.2047	0.0000	-0.0577	-0.2477	-0.0577	-0.3054
1	3	8.17	-0.0568	-0.2979	0.0000	-0.0839	-0.3605	-0.0839	-0.4444
1	4	10.89	-0.0727	-0.3812	0.0000	-0.1072	-0.4613	-0.1072	-0.5685
1	5	13.61	-0.0866	-0.4545	0.0000	-0.1276	-0.5501	-0.1276	-0.6777
1	6	16.34	-0.0985	-0.5167	0.0000	-0.1450	-0.6254	-0.1450	-0.7704
1	7	19.06	-0.1080	-0.5665	0.0000	-0.1589	-0.6857	-0.1589	-0.8446
1	8	21.78	-0.1149	-0.6028	0.0000	-0.1690	-0.7297	-0.1690	-0.8987
1	9	24.50	-0.1191	-0.6249	0.0000	-0.1752	-0.7564	-0.1752	-0.9316
1	10	27.23	-0.1205	-0.6323	0.0000	-0.1772	-0.7654	-0.1772	-0.9426
1	11	29.95	-0.1191	-0.6249	0.0000	-0.1752	-0.7565	-0.1752	-0.9316
1	12	32.67	-0.1149	-0.6028	0.0000	-0.1690	-0.7298	-0.1690	-0.8988
1	13	35.39	-0.1080	-0.5665	0.0000	-0.1589	-0.6859	-0.1589	-0.8447
1	14	38.12	-0.0985	-0.5167	0.0000	-0.1450	-0.6257	-0.1450	-0.7706
1	15	40.84	-0.0866	-0.4545	0.0000	-0.1276	-0.5504	-0.1276	-0.6780
1	16	43.56	-0.0727	-0.3812	0.0000	-0.1072	-0.4616	-0.1072	-0.5688
1	17	46.28	-0.0568	-0.2979	0.0000	-0.0839	-0.3608	-0.0839	-0.4447
1	18	49.01	-0.0390	-0.2047	0.0000	-0.0577	-0.2479	-0.0577	-0.3056
1	19	51.73	-0.0199	-0.1043	0.0000	-0.0294	-0.1262	-0.0294	-0.1557
1	20	54.45	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT	NONCOMPOSITE DEAD LOADS STEEL (in)	NONCOMPOSITE DEAD LOADS CAMBER (in)	NONCOMPOSITE DEAD LOADS SLAB (in)	NONCOMPOSITE DEAD LOADS CAMBER (in)	COMPOSITE DL DEFL. (in)	COMPOSITE DL CAMBER (in)	T O T A L DEFL. (in)	T O T A L CAMBER (in)

5969 Span 2-S17 Interior Strength Mid M.RES

		(ft)	DEFL.	SIZE	DEFL.	SIZE		SIZE		SIZE		SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--		
1	1	2.7	-0.020	0	1/16	-0.106	0	1/ 8	-0.029	0	1/16	-0.156	0	3/16
1	2	5.4	-0.039	0	1/16	-0.209	0	1/ 4	-0.058	0	1/16	-0.305	0	5/16
1	3	8.2	-0.057	0	1/16	-0.304	0	5/16	-0.084	0	1/ 8	-0.444	0	1/ 2
1	4	10.9	-0.073	0	1/ 8	-0.389	0	7/16	-0.107	0	1/ 8	-0.568	0	5/ 8
1	5	13.6	-0.087	0	1/ 8	-0.463	0	1/ 2	-0.128	0	3/16	-0.678	0	11/16
1	6	16.3	-0.099	0	1/ 8	-0.527	0	9/16	-0.145	0	3/16	-0.770	0	13/16
1	7	19.1	-0.108	0	1/ 8	-0.578	0	5/ 8	-0.159	0	3/16	-0.845	0	7/ 8
1	8	21.8	-0.115	0	1/ 8	-0.615	0	5/ 8	-0.169	0	3/16	-0.899	0	15/16
1	9	24.5	-0.119	0	1/ 8	-0.637	0	11/16	-0.175	0	3/16	-0.932	0	15/16
1	10	27.2	-0.121	0	1/ 8	-0.645	0	11/16	-0.177	0	3/16	-0.943	1	0/16
1	11	29.9	-0.119	0	1/ 8	-0.637	0	11/16	-0.175	0	3/16	-0.932	0	15/16
1	12	32.7	-0.115	0	1/ 8	-0.615	0	5/ 8	-0.169	0	3/16	-0.899	0	15/16
1	13	35.4	-0.108	0	1/ 8	-0.578	0	5/ 8	-0.159	0	3/16	-0.845	0	7/ 8
1	14	38.1	-0.099	0	1/ 8	-0.527	0	9/16	-0.145	0	3/16	-0.771	0	13/16
1	15	40.8	-0.087	0	1/ 8	-0.464	0	1/ 2	-0.128	0	3/16	-0.678	0	11/16
1	16	43.6	-0.073	0	1/ 8	-0.389	0	7/16	-0.107	0	1/ 8	-0.569	0	5/ 8
1	17	46.3	-0.057	0	1/16	-0.304	0	5/16	-0.084	0	1/ 8	-0.445	0	1/ 2
1	18	49.0	-0.039	0	1/16	-0.209	0	1/ 4	-0.058	0	1/16	-0.306	0	5/16
1	19	51.7	-0.020	0	1/16	-0.106	0	1/ 8	-0.029	0	1/16	-0.156	0	3/16
1	20	54.5	0.000	--	0.000	--	0.000	--	0.000	--		

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	27.23	3	0.425	-0.280 MAX HL-93	0.82	0.00251
				0.026 MIN		
				-0.075 MAX LANE		
				0.026 MIN LANE		
		4	0.433	-0.285 MAX HL-93	0.82	0.00251
				0.027 MIN		
				-0.077 MAX LANE		
				0.027 MIN LANE		
		3	0.425	-0.185 MAX FA		
				0.000 MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	-0.33	0.33	-1.75	1.75	-2.07	2.07
1	2	5.45	-0.62	0.62	-3.30	3.30	-3.92	3.92
1	3	8.17	-0.88	0.88	-4.68	4.68	-5.55	5.55
1	4	10.89	-0.99	0.68	-5.26	3.62	-6.25	4.30
1	5	13.61	-1.16	0.79	-6.17	4.24	-7.33	5.04
1	6	16.34	-1.29	0.89	-6.92	4.76	-8.21	5.65
1	7	19.06	-1.40	0.96	-7.50	5.16	-8.90	6.12
1	8	21.78	-1.48	1.02	-7.92	5.45	-9.40	6.47
1	9	24.50	-1.53	1.05	-8.17	5.62	-9.69	6.67
1	10	27.23	-1.54	1.06	-8.24	5.67	-9.78	6.73
1	11	29.95	-1.53	1.05	-8.16	5.61	-9.68	6.66
1	12	32.67	-1.48	1.02	-7.91	5.44	-9.39	6.46
1	13	35.39	-1.40	0.96	-7.50	5.16	-8.90	6.12
1	14	38.12	-1.29	0.89	-6.93	4.77	-8.23	5.66
1	15	40.84	-1.16	0.79	-6.20	4.26	-7.35	5.06
1	16	43.56	-0.99	0.68	-5.29	3.64	-6.27	4.31
1	17	46.28	-0.88	0.88	-4.69	4.69	-5.57	5.57
1	18	49.01	-0.62	0.62	-3.31	3.31	-3.93	3.93
1	19	51.73	-0.33	0.33	-1.74	1.74	-2.07	2.07
1	20	54.45	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	-0.02	-0.23	0.78	-0.02	-2.30	2.85
1	2	5.45	-0.03	-0.44	1.47	-0.03	-4.36	5.39
1	3	8.17	-0.05	-0.62	2.08	-0.05	-6.17	7.63
1	4	10.89	-0.05	-0.80	1.64	-0.05	-7.04	5.94
1	5	13.61	-0.06	-0.93	1.92	-0.06	-8.26	6.96
1	6	16.34	-0.07	-1.04	2.15	-0.07	-9.26	7.80
1	7	19.06	-0.07	-1.13	2.33	-0.07	-10.03	8.46
1	8	21.78	-0.08	-1.19	2.46	-0.08	-10.60	8.93
1	9	24.50	-0.08	-1.23	2.54	-0.08	-10.92	9.20
1	10	27.23	-0.08	-1.24	2.56	-0.08	-11.03	9.29
1	11	29.95	-0.08	-1.23	2.54	-0.08	-10.91	9.20

1 12	32.67	-0.08	-1.19	2.46	-0.08	-10.58	8.92
1 13	35.39	-0.07	-1.13	2.33	-0.07	-10.04	8.46
1 14	38.12	-0.07	-1.04	2.15	-0.07	-9.27	7.81
1 15	40.84	-0.06	-0.93	1.92	-0.06	-8.29	6.98
1 16	43.56	-0.05	-0.80	1.64	-0.05	-7.07	5.95
1 17	46.28	-0.05	-0.62	2.08	-0.05	-6.19	7.65
1 18	49.01	-0.03	-0.44	1.47	-0.03	-4.36	5.40
1 19	51.73	-0.02	-0.23	0.78	-0.02	-2.30	2.85
1 20	54.45	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	-0.14	-0.24	3.80	0.00	0.00	0.00
1	2	5.45	-0.26	-0.44	7.12	0.00	0.00	0.00
1	3	8.17	-0.37	-0.62	9.97	0.00	0.00	0.00
1	4	10.89	-0.38	-1.17	7.81	0.00	0.00	0.00
1	5	13.61	-0.43	-1.34	8.98	0.00	0.00	0.00
1	6	16.34	-0.48	-1.48	9.92	0.00	0.00	0.00
1	7	19.06	-0.52	-1.59	10.66	0.00	0.00	0.00
1	8	21.78	-0.54	-1.66	11.13	0.00	0.00	0.00
1	9	24.50	-0.54	-1.68	11.26	0.00	0.00	0.00
1	10	27.23	-0.55	-1.69	11.30	0.00	0.00	0.00
1	11	29.95	-0.54	-1.68	11.26	0.00	0.00	0.00
1	12	32.67	-0.54	-1.66	11.13	0.00	0.00	0.00
1	13	35.39	-0.52	-1.59	10.66	0.00	0.00	0.00
1	14	38.12	-0.48	-1.48	9.92	0.00	0.00	0.00
1	15	40.84	-0.43	-1.34	8.98	0.00	0.00	0.00
1	16	43.56	-0.38	-1.17	7.81	0.00	0.00	0.00
1	17	46.28	-0.37	-0.62	9.97	0.00	0.00	0.00
1	18	49.01	-0.26	-0.44	7.12	0.00	0.00	0.00
1	19	51.73	-0.14	-0.24	3.80	0.00	0.00	0.00
1	20	54.45	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	2.72	0.1	0.0	0.1	C HL	1.3	0.0	1.3	T HL
1	2	5.45	0.1	0.0	0.1	C HL	2.4	0.0	2.4	T HL
1	3	8.17	0.2	0.0	0.2	C HL	3.3	0.0	3.3	T HL
1	4	10.89	0.4	0.0	0.4	C HL	2.5	0.0	2.5	T HL
1	5	13.61	0.4	0.0	0.4	C HL	2.9	0.0	2.9	T HL
1	6	16.34	0.5	0.0	0.5	C HL	3.1	0.0	3.1	T HL
1	7	19.06	0.5	0.0	0.5	C HL	3.3	0.0	3.3	T HL
1	8	21.78	0.5	0.0	0.5	C HL	3.4	0.0	3.4	T HL

1 9	24.50	0.5	0.0	0.5 C	HL	3.3	0.0	3.3 T	HL
1 10	27.23	0.5	0.0	0.5 C	HL	3.3	0.0	3.3 T	HL
1 11	29.95	0.5	0.0	0.5 C	HL	3.3	0.0	3.3 T	HL
1 12	32.67	0.5	0.0	0.5 C	HL	3.4	0.0	3.4 T	HL
1 13	35.39	0.5	0.0	0.5 C	HL	3.3	0.0	3.3 T	HL
1 14	38.12	0.5	0.0	0.5 C	HL	3.1	0.0	3.1 T	HL
1 15	40.84	0.4	0.0	0.4 C	HL	2.9	0.0	2.9 T	HL
1 16	43.56	0.4	0.0	0.4 C	HL	2.5	0.0	2.5 T	HL
1 17	46.28	0.2	0.0	0.2 C	HL	3.3	0.0	3.3 T	HL
1 18	49.01	0.1	0.0	0.1 C	HL	2.4	0.0	2.4 T	HL
1 19	51.73	0.1	0.0	0.1 C	HL	1.3	0.0	1.3 T	HL
1 20	54.45	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	33.	33.	-0.16	-2.54	6.64	-0.02	-2.30	2.85
1	2	5.45	33.	33.	-0.30	-4.80	12.51	-0.03	-4.36	5.39
1	3	8.17	33.	33.	-0.42	-6.79	17.61	-0.05	-6.17	7.63
1	4	10.89	33.	33.	-0.43	-8.21	13.75	-0.05	-7.04	5.94
1	5	13.61	33.	33.	-0.49	-9.60	15.94	-0.06	-8.26	6.96
1	6	16.34	33.	33.	-0.55	-10.74	17.72	-0.07	-9.26	7.80
1	7	19.06	33.	33.	-0.59	-11.63	19.12	-0.07	-10.03	8.46
1	8	21.78	33.	33.	-0.62	-12.26	20.06	-0.08	-10.60	8.93
1	9	24.50	33.	33.	-0.62	-12.61	20.46	-0.08	-10.92	9.20
1	10	27.23	33.	33.	-0.63	-12.71	20.59	-0.08	-11.03	9.29
1	11	29.95	33.	33.	-0.62	-12.60	20.45	-0.08	-10.91	9.20
1	12	32.67	33.	33.	-0.62	-12.25	20.05	-0.08	-10.58	8.92
1	13	35.39	33.	33.	-0.59	-11.63	19.12	-0.07	-10.04	8.46
1	14	38.12	33.	33.	-0.55	-10.75	17.73	-0.07	-9.27	7.81
1	15	40.84	33.	33.	-0.49	-9.63	15.96	-0.06	-8.29	6.98
1	16	43.56	33.	33.	-0.43	-8.23	13.76	-0.05	-7.07	5.95
1	17	46.28	33.	33.	-0.42	-6.81	17.62	-0.05	-6.19	7.65
1	18	49.01	33.	33.	-0.30	-4.81	12.52	-0.03	-4.36	5.40
1	19	51.73	33.	33.	-0.16	-2.54	6.64	-0.02	-2.30	2.85
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

5969 Span 2-S17 Interior Strength Mid M.RES

1	1	2.72	33.	33.	-0.20	-2.61	7.78	-0.02	-2.30	2.85
1	2	5.45	33.	33.	-0.38	-4.93	14.65	-0.03	-4.36	5.39
1	3	8.17	33.	33.	-0.53	-6.98	20.60	-0.05	-6.17	7.63
1	4	10.89	33.	33.	-0.54	-8.56	16.09	-0.05	-7.04	5.94
1	5	13.61	33.	33.	-0.62	-10.00	18.63	-0.06	-8.26	6.96
1	6	16.34	33.	33.	-0.69	-11.18	20.70	-0.07	-9.26	7.80
1	7	19.06	33.	33.	-0.74	-12.10	22.31	-0.07	-10.03	8.46
1	8	21.78	33.	33.	-0.78	-12.76	23.40	-0.08	-10.60	8.93
1	9	24.50	33.	33.	-0.79	-13.11	23.84	-0.08	-10.92	9.20
1	10	27.23	33.	33.	-0.79	-13.22	23.98	-0.08	-11.03	9.29
1	11	29.95	33.	33.	-0.79	-13.10	23.83	-0.08	-10.91	9.20
1	12	32.67	33.	33.	-0.78	-12.74	23.39	-0.08	-10.58	8.92
1	13	35.39	33.	33.	-0.74	-12.10	22.31	-0.07	-10.04	8.46
1	14	38.12	33.	33.	-0.69	-11.20	20.71	-0.07	-9.27	7.81
1	15	40.84	33.	33.	-0.62	-10.03	18.65	-0.06	-8.29	6.98
1	16	43.56	33.	33.	-0.54	-8.58	16.10	-0.05	-7.07	5.95
1	17	46.28	33.	33.	-0.53	-6.99	20.62	-0.05	-6.19	7.65
1	18	49.01	33.	33.	-0.38	-4.94	14.65	-0.03	-4.36	5.40
1	19	51.73	33.	33.	-0.20	-2.61	7.78	-0.02	-2.30	2.85
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	33.	33.	-0.27	-3.33	10.33	-0.03	-2.92	3.68
1	2	5.45	33.	33.	-0.51	-6.29	19.44	-0.05	-5.52	6.97
1	3	8.17	33.	33.	-0.72	-8.90	27.33	-0.07	-7.82	9.88
1	4	10.89	33.	33.	-0.73	-10.97	21.35	-0.07	-8.93	7.69
1	5	13.61	33.	33.	-0.85	-12.82	24.72	-0.09	-10.47	9.01
1	6	16.34	33.	33.	-0.94	-14.33	27.46	-0.10	-11.74	10.10
1	7	19.06	33.	33.	-1.01	-15.51	29.60	-0.10	-12.72	10.94
1	8	21.78	33.	33.	-1.05	-16.35	31.04	-0.11	-13.44	11.56
1	9	24.50	33.	33.	-1.07	-16.79	31.61	-0.11	-13.85	11.91
1	10	27.23	33.	33.	-1.07	-16.94	31.80	-0.11	-13.98	12.03
1	11	29.95	33.	33.	-1.07	-16.78	31.60	-0.11	-13.84	11.90
1	12	32.67	33.	33.	-1.05	-16.33	31.03	-0.11	-13.42	11.54
1	13	35.39	33.	33.	-1.01	-15.51	29.60	-0.10	-12.73	10.95
1	14	38.12	33.	33.	-0.94	-14.35	27.47	-0.10	-11.75	10.11
1	15	40.84	33.	33.	-0.85	-12.85	24.75	-0.09	-10.51	9.03
1	16	43.56	33.	33.	-0.73	-11.00	21.37	-0.07	-8.96	7.71
1	17	46.28	33.	33.	-0.72	-8.92	27.35	-0.07	-7.83	9.89
1	18	49.01	33.	33.	-0.51	-6.30	19.44	-0.05	-5.53	6.98
1	19	51.73	33.	33.	-0.27	-3.33	10.33	-0.03	-2.91	3.68
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	33.	33.	-0.26	-2.45	9.08	-0.01	-2.04	2.44
1	2	5.45	33.	33.	-0.49	-4.63	17.08	-0.03	-3.85	4.62
1	3	8.17	33.	33.	-0.68	-6.54	23.99	-0.04	-5.46	6.54
1	4	10.89	33.	33.	-0.70	-8.25	18.74	-0.04	-6.21	5.08
1	5	13.61	33.	33.	-0.80	-9.63	21.67	-0.04	-7.28	5.96
1	6	16.34	33.	33.	-0.89	-10.76	24.04	-0.05	-8.16	6.68
1	7	19.06	33.	33.	-0.96	-11.64	25.89	-0.05	-8.85	7.24
1	8	21.78	33.	33.	-1.00	-12.25	27.12	-0.06	-9.35	7.64
1	9	24.50	33.	33.	-1.01	-12.58	27.58	-0.06	-9.63	7.88
1	10	27.23	33.	33.	-1.02	-12.68	27.72	-0.06	-9.73	7.95
1	11	29.95	33.	33.	-1.01	-12.57	27.57	-0.06	-9.63	7.87
1	12	32.67	33.	33.	-1.00	-12.24	27.12	-0.06	-9.33	7.63
1	13	35.39	33.	33.	-0.96	-11.64	25.89	-0.05	-8.85	7.24
1	14	38.12	33.	33.	-0.89	-10.77	24.05	-0.05	-8.18	6.68
1	15	40.84	33.	33.	-0.80	-9.65	21.69	-0.04	-7.31	5.97
1	16	43.56	33.	33.	-0.70	-8.27	18.76	-0.04	-6.23	5.10
1	17	46.28	33.	33.	-0.68	-6.55	24.00	-0.04	-5.47	6.55
1	18	49.01	33.	33.	-0.49	-4.63	17.08	-0.03	-3.86	4.62
1	19	51.73	33.	33.	-0.26	-2.45	9.08	-0.01	-2.03	2.44
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	33.	33.	-0.22	-3.23	8.81	-0.03	-2.92	3.68
1	2	5.45	33.	33.	-0.40	-6.12	16.59	-0.05	-5.52	6.97
1	3	8.17	33.	33.	-0.57	-8.65	23.34	-0.07	-7.82	9.88
1	4	10.89	33.	33.	-0.58	-10.51	18.23	-0.07	-8.93	7.69
1	5	13.61	33.	33.	-0.67	-12.28	21.13	-0.09	-10.47	9.01
1	6	16.34	33.	33.	-0.74	-13.74	23.49	-0.10	-11.74	10.10
1	7	19.06	33.	33.	-0.80	-14.87	25.33	-0.10	-12.72	10.94
1	8	21.78	33.	33.	-0.84	-15.68	26.59	-0.11	-13.44	11.56
1	9	24.50	33.	33.	-0.85	-16.12	27.11	-0.11	-13.85	11.91
1	10	27.23	33.	33.	-0.85	-16.26	27.28	-0.11	-13.98	12.03
1	11	29.95	33.	33.	-0.85	-16.11	27.10	-0.11	-13.84	11.90
1	12	32.67	33.	33.	-0.84	-15.67	26.57	-0.11	-13.42	11.54
1	13	35.39	33.	33.	-0.80	-14.87	25.34	-0.10	-12.73	10.95
1	14	38.12	33.	33.	-0.74	-13.76	23.50	-0.10	-11.75	10.11
1	15	40.84	33.	33.	-0.67	-12.32	21.15	-0.09	-10.51	9.03
1	16	43.56	33.	33.	-0.58	-10.54	18.25	-0.07	-8.96	7.71
1	17	46.28	33.	33.	-0.57	-8.67	23.36	-0.07	-7.83	9.89
1	18	49.01	33.	33.	-0.40	-6.12	16.59	-0.05	-5.53	6.98
1	19	51.73	33.	33.	-0.22	-3.23	8.81	-0.03	-2.91	3.68
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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MERLIN V 10.4
COMPOSITE
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ANALYSIS

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.72	33.	33.	-0.20	-2.35	7.56	-0.01	-2.04	2.44
1	2	5.45	33.	33.	-0.38	-4.45	14.23	-0.03	-3.85	4.62
1	3	8.17	33.	33.	-0.53	-6.29	20.00	-0.04	-5.46	6.54
1	4	10.89	33.	33.	-0.55	-7.79	15.62	-0.04	-6.21	5.08
1	5	13.61	33.	33.	-0.63	-9.09	18.08	-0.04	-7.28	5.96
1	6	16.34	33.	33.	-0.70	-10.16	20.07	-0.05	-8.16	6.68
1	7	19.06	33.	33.	-0.75	-11.00	21.63	-0.05	-8.85	7.24
1	8	21.78	33.	33.	-0.78	-11.59	22.67	-0.06	-9.35	7.64
1	9	24.50	33.	33.	-0.79	-11.90	23.07	-0.06	-9.63	7.88
1	10	27.23	33.	33.	-0.80	-12.00	23.20	-0.06	-9.73	7.95
1	11	29.95	33.	33.	-0.79	-11.89	23.07	-0.06	-9.63	7.87
1	12	32.67	33.	33.	-0.78	-11.58	22.66	-0.06	-9.33	7.63
1	13	35.39	33.	33.	-0.75	-11.00	21.63	-0.05	-8.85	7.24
1	14	38.12	33.	33.	-0.70	-10.18	20.08	-0.05	-8.18	6.68
1	15	40.84	33.	33.	-0.63	-9.12	18.09	-0.04	-7.31	5.97
1	16	43.56	33.	33.	-0.55	-7.81	15.64	-0.04	-6.23	5.10
1	17	46.28	33.	33.	-0.53	-6.31	20.02	-0.04	-5.47	6.55
1	18	49.01	33.	33.	-0.38	-4.45	14.23	-0.03	-3.86	4.62
1	19	51.73	33.	33.	-0.20	-2.35	7.56	-0.01	-2.03	2.44
1	20	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	9.73	190.	-6.4	99.	-0.7	517.	-0.7	0.	0.0	-7.78
											0.70
	LB			6.4		2.4		11.3		0.0	20.08
											11.34
	RT			-5.7		-0.7		-1.1		0.0	-7.50
											1.07
	RB			3.9		1.5		7.2		0.0	12.59
											7.16
1	LT	44.72	191.	-5.7	99.	-0.7	517.	-1.1	0.	0.0	-7.52
											1.07
	LB			3.9		1.5		7.2		0.0	12.60
											7.16
	RT			-6.4		-0.7		-0.7		0.0	-7.80
											0.70
	RB			6.4		2.4		11.3		0.0	20.10
											11.34

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK

```
  1   9.73    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

  2  35.00    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 10.00 0.9375

  3   9.73    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

 ------------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   54.5
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------

  1   9.73  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                         [3]=   32.90     0.05035      >   0.03300    OK

  2  35.00  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                         [3]=   32.90     0.05035      >   0.03300    OK

  3   9.73  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                         [3]=   32.90     0.05035      >   0.03300    OK

 ---------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                            Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1  1   2.72  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1  2   5.45  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1  3   8.17  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1  4  10.89  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
  1  5  13.61  0  1  31.42 0.550  57.1  150.0    0.00 111.46                 0
```

1	6	16.34	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	7	19.06	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	8	21.78	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	9	24.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	10	27.23	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	11	29.95	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	12	32.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	13	35.39	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	14	38.12	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	15	40.84	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	16	43.56	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	17	46.28	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	18	49.01	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	19	51.73	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0
1	20	54.45	0	1	31.42	0.550	57.1	150.0	0.00	111.46	0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.72	0	1	31.42	0.550	57.13	168.97	2
1	2	5.45	0	1	31.42	0.550	57.13	168.97	2
1	3	8.17	0	1	31.42	0.550	57.13	168.97	2
1	4	10.89	0	1	31.42	0.550	70.33	168.97	2
1	5	13.61	0	1	31.42	0.550	70.33	168.97	2
1	6	16.34	0	1	31.42	0.550	70.33	168.97	2
1	7	19.06	0	1	31.42	0.550	70.33	168.97	2
1	8	21.78	0	1	31.42	0.550	70.33	168.97	2
1	9	24.50	0	1	31.42	0.550	70.33	168.97	2
1	10	27.23	0	1	31.42	0.550	70.33	168.97	2
1	11	29.95	0	1	31.42	0.550	70.33	168.97	2
1	12	32.67	0	1	31.42	0.550	70.33	168.97	2
1	13	35.39	0	1	31.42	0.550	70.33	168.97	2
1	14	38.12	0	1	31.42	0.550	70.33	168.97	2
1	15	40.84	0	1	31.42	0.550	70.33	168.97	2
1	16	43.56	0	1	31.42	0.550	70.33	168.97	2
1	17	46.28	0	1	31.42	0.550	57.13	168.97	2
1	18	49.01	0	1	31.42	0.550	57.13	168.97	2
1	19	51.73	0	1	31.42	0.550	57.13	168.97	2
1	20	54.45	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO L (ft)	SUPT	bf/2tf	[1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 1	2.72		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 2	5.45		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 3	8.17		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 4	10.89		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 5	13.61		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 6	16.34		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 7	19.06		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 8	21.78		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 9	24.50		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 10	27.23		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 11	29.95		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 12	32.67		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 13	35.39		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 14	38.12		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 15	40.84		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 16	43.56		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 17	46.28		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 18	49.01		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 19	51.73		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1 20	54.45		7.77	12.	11.5	5.2	0.740	0.605		
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. FROM LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	3.7	0.000	4.086	2.102	0.118	1.720
			0.000	0.000	0.000	0.000	1.000
2	3.7	22.5	4.086	14.223	10.685	7.148	1.000
			0.000	0.000	0.000	0.000	1.000
3	22.5	41.4	10.720	14.223	14.203	14.183	1.001
			0.000	0.000	0.000	0.000	1.000
4	41.4	54.5	0.000	10.720	6.885	3.050	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	3.68	7.09	26.62	0	0
1	1	2.72	1	7.77	11.26	19.84	0	3.68	7.09	26.62	0	0
1	2	5.45	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	3	8.17	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	4	10.89	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	5	13.61	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	6	16.34	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	7	19.06	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	8	21.78	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	9	24.50	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	10	27.23	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	11	29.95	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	12	32.67	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	13	35.39	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	14	38.12	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	15	40.84	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	16	43.56	1	7.77	11.26	19.84	0	13.09	6.89	25.87	2	2
1	17	46.28	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	18	49.01	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	19	51.73	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	20	54.45	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange

residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.72	33.00	33.00	33.00
1	1		2.72	1	1.000	1.000	1.72	33.00	33.00	33.00
1	2		5.45	1	1.000	1.000	1.00	33.00	27.04	27.04
1	3		8.17	1	1.000	1.000	1.00	33.00	27.04	27.04
1	4		10.89	1	1.000	1.000	1.00	33.00	26.77	26.77
1	5		13.61	1	1.000	1.000	1.00	33.00	26.77	26.77
1	6		16.34	1	1.000	1.000	1.00	33.00	26.77	26.77
1	7		19.06	1	1.000	1.000	1.00	33.00	26.77	26.77
1	8		21.78	1	1.000	1.000	1.00	33.00	26.77	26.77
1	9		24.50	1	1.000	1.000	1.00	33.00	26.80	26.80
1	10		27.23	1	1.000	1.000	1.00	33.00	26.80	26.80
1	11		29.95	1	1.000	1.000	1.00	33.00	26.80	26.80
1	12		32.67	1	1.000	1.000	1.00	33.00	26.80	26.80
1	13		35.39	1	1.000	1.000	1.00	33.00	26.80	26.80
1	14		38.12	1	1.000	1.000	1.00	33.00	26.80	26.80
1	15		40.84	1	1.000	1.000	1.00	33.00	26.80	26.80
1	16		43.56	1	1.000	1.000	1.00	33.00	29.77	29.77
1	17		46.28	1	1.000	1.000	1.00	33.00	29.96	29.96
1	18		49.01	1	1.000	1.000	1.00	33.00	29.96	29.96
1	19		51.73	1	1.000	1.000	1.00	33.00	29.96	29.96
1	20		54.45	1	1.000	1.000	1.00	33.00	29.96	29.96

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	4.70	17.35
1	1		2.72	1	4.70	17.35
1	2		5.45	1	4.70	17.35
1	3		8.17	1	4.70	17.35
1	4		10.89	1	5.98	17.74
1	5		13.61	1	5.98	17.74
1	6		16.34	1	5.98	17.74
1	7		19.06	1	5.98	17.74
1	8		21.78	1	5.98	17.74
1	9		24.50	1	5.98	17.74

1 10	27.23	1	5.98	17.74
1 11	29.95	1	5.98	17.74
1 12	32.67	1	5.98	17.74
1 13	35.39	1	5.98	17.74
1 14	38.12	1	5.98	17.74
1 15	40.84	1	5.98	17.74
1 16	43.56	1	5.98	17.74
1 17	46.28	1	4.70	17.35
1 18	49.01	1	4.70	17.35
1 19	51.73	1	4.70	17.35
1 20	54.45	1	4.70	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L (ft)	FROM SUPT	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1		0	0	0		0	0
1	1	2.72	1		0	0	0		0	0
1	2	5.45	1		0	2	0		2	0
1	3	8.17	1		0	2	0		2	0
1	4	10.89	1		0	2	0		2	0
1	5	13.61	1		0	2	0		2	0
1	6	16.34	1		0	2	0		2	0
1	7	19.06	1		0	2	0		2	0
1	8	21.78	1		0	2	0		2	0
1	9	24.50	1		0	2	0		2	0
1	10	27.23	1		0	2	0		2	0
1	11	29.95	1		0	2	0		2	0
1	12	32.67	1		0	2	0		2	0
1	13	35.39	1		0	2	0		2	0
1	14	38.12	1		0	2	0		2	0
1	15	40.84	1		0	2	0		2	0
1	16	43.56	1		0	2	0		2	0
1	17	46.28	1		0	2	0		2	0
1	18	49.01	1		0	2	0		2	0
1	19	51.73	1		0	2	0		2	0
1	20	54.45	1		0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		2.72	0.0	19.8	3.1	33.0	3.1	33.0	3.1	33.0	0
				0.0	-	3.1	-	3.1	33.0	-	-	0
1	2		5.45	0.0	19.8	5.9	33.0	5.9	33.0	5.9	27.0	0
				0.0	-	5.9	-	5.9	33.0	-	-	0
1	3		8.17	0.0	19.8	8.3	33.0	8.3	33.0	8.3	27.0	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	4		10.89	0.0	19.8	9.4	33.0	9.4	33.0	9.4	26.8	0
				0.0	-	6.4	-	6.4	33.0	-	-	0
1	5		13.61	0.0	19.8	11.0	33.0	11.0	33.0	11.0	26.8	0
				0.0	-	7.6	-	7.6	33.0	-	-	0
1	6		16.34	0.0	19.8	12.3	33.0	12.3	33.0	12.3	26.8	0
				0.0	-	8.5	-	8.5	33.0	-	-	0
1	7		19.06	0.0	19.8	13.4	33.0	13.4	33.0	13.4	26.8	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	8		21.78	0.0	19.8	14.1	33.0	14.1	33.0	14.1	26.8	0
				0.0	-	9.7	-	9.7	33.0	-	-	0
1	9		24.50	0.0	19.8	14.5	33.0	14.5	33.0	14.5	26.8	0
				0.0	-	10.0	-	10.0	33.0	-	-	0
1	10		27.23	0.0	19.8	14.7	33.0	14.7	33.0	14.7	26.8	0
				0.0	-	10.1	-	10.1	33.0	-	-	0
1	11		29.95	0.0	19.8	14.5	33.0	14.5	33.0	14.5	26.8	0
				0.0	-	10.0	-	10.0	33.0	-	-	0
1	12		32.67	0.0	19.8	14.1	33.0	14.1	33.0	14.1	26.8	0
				0.0	-	9.7	-	9.7	33.0	-	-	0
1	13		35.39	0.0	19.8	13.4	33.0	13.4	33.0	13.4	26.8	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	14		38.12	0.0	19.8	12.3	33.0	12.3	33.0	12.3	26.8	0
				0.0	-	8.5	-	8.5	33.0	-	-	0
1	15		40.84	0.0	19.8	11.0	33.0	11.0	33.0	11.0	26.8	0
				0.0	-	7.6	-	7.6	33.0	-	-	0
1	16		43.56	0.0	19.8	9.4	33.0	9.4	33.0	9.4	29.8	0
				0.0	-	6.5	-	6.5	33.0	-	-	0
1	17		46.28	0.0	19.8	8.4	33.0	8.4	33.0	8.4	30.0	0
				0.0	-	8.4	-	8.4	33.0	-	-	0
1	18		49.01	0.0	19.8	5.9	33.0	5.9	33.0	5.9	30.0	0
				0.0	-	5.9	-	5.9	33.0	-	-	0
1	19		51.73	0.0	19.8	3.1	33.0	3.1	33.0	3.1	30.0	0
				0.0	-	3.1	-	3.1	33.0	-	-	0
1	20		54.45	0.0	19.8	0.0	33.0	0.0	33.0	0.0	30.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.72	0.000	0.094	0.094	0.094	0.094	2
					0.094		0.094	3
1	2	5.45	0.000	0.178	0.178	0.218	0.218	4
					0.178		0.178	3
1	3	8.17	0.000	0.252	0.252	0.308	0.308	4
					0.252		0.252	3
1	4	10.89	0.000	0.284	0.284	0.350	0.350	4
					0.195		0.195	3
1	5	13.61	0.000	0.333	0.333	0.411	0.411	4
					0.229		0.229	3
1	6	16.34	0.000	0.373	0.373	0.460	0.460	4
					0.257		0.257	3
1	7	19.06	0.000	0.405	0.405	0.499	0.499	4
					0.278		0.278	3
1	8	21.78	0.000	0.427	0.427	0.527	0.527	4
					0.294		0.294	3
1	9	24.50	0.000	0.441	0.441	0.543	0.543	4
					0.303		0.303	3
1	10	27.23	0.000	0.445	0.445	0.548	0.548	4
					0.306		0.306	3
1	11	29.95	0.000	0.440	0.440	0.542	0.542	4
					0.303		0.303	3
1	12	32.67	0.000	0.427	0.427	0.526	0.526	4
					0.294		0.294	3
1	13	35.39	0.000	0.405	0.405	0.498	0.498	4
					0.278		0.278	3
1	14	38.12	0.000	0.374	0.374	0.460	0.460	4
					0.257		0.257	3
1	15	40.84	0.000	0.334	0.334	0.412	0.412	4
					0.230		0.230	3
1	16	43.56	0.000	0.285	0.285	0.316	0.316	4
					0.196		0.196	3
1	17	46.28	0.000	0.253	0.253	0.279	0.279	4
					0.253		0.253	3
1	18	49.01	0.000	0.178	0.178	0.197	0.197	4
					0.178		0.178	3
1	19	51.73	0.000	0.094	0.094	0.104	0.104	4
					0.094		0.094	3
1	20	54.45	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	1		2.72	0	425.6	2103.1	-	-	0.0	19.8	-	-	0--0
1	2		5.45	0	800.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	3		8.17	0	1124.8	2103.1	-	-	0.0	19.8	-	-	0--0
1	4		10.89	0	1398.7	2973.8	-	-	0.0	19.8	-	-	0--0
1	5		13.61	0	1617.6	2973.8	-	-	0.0	19.8	-	-	0--0
1	6		16.34	0	1795.3	2973.8	-	-	0.0	19.8	-	-	0--0
1	7		19.06	0	1933.8	2973.8	-	-	0.0	19.8	-	-	0--0
1	8		21.78	0	2026.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	9		24.50	0	2061.3	2973.8	-	-	0.0	19.8	-	-	0--0
1	10		27.23	0	2072.6	2973.8	-	-	0.0	19.8	-	-	0--0
1	11		29.95	0	2060.9	2973.8	-	-	0.0	19.8	-	-	0--0
1	12		32.67	0	2026.0	2973.8	-	-	0.0	19.8	-	-	0--0
1	13		35.39	0	1933.8	2973.8	-	-	0.0	19.8	-	-	0--0
1	14		38.12	0	1795.9	2973.8	-	-	0.0	19.8	-	-	0--0
1	15		40.84	0	1618.8	2973.8	-	-	0.0	19.8	-	-	0--0
1	16		43.56	0	1399.7	2973.8	-	-	0.0	19.8	-	-	0--0
1	17		46.28	0	1125.4	2103.1	-	-	0.0	19.8	-	-	0--0
1	18		49.01	0	800.7	2103.1	-	-	0.0	19.8	-	-	0--0
1	19		51.73	0	425.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	20		54.45	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	2.72	0	0.202				0.202	1
1	2	5.45	0	0.381				0.381	1
1	3	8.17	0	0.535				0.535	1
1	4	10.89	0	0.470				0.470	1
1	5	13.61	0	0.544				0.544	1
1	6	16.34	0	0.604				0.604	1
1	7	19.06	0	0.650				0.650	1
1	8	21.78	0	0.681				0.681	1
1	9	24.50	0	0.693				0.693	1
1	10	27.23	0	0.697				0.697	1
1	11	29.95	0	0.693				0.693	1
1	12	32.67	0	0.681				0.681	1
1	13	35.39	0	0.650				0.650	1
1	14	38.12	0	0.604				0.604	1
1	15	40.84	0	0.544				0.544	1
1	16	43.56	0	0.471				0.471	1
1	17	46.28	0	0.535				0.535	1
1	18	49.01	0	0.381				0.381	1
1	19	51.73	0	0.202				0.202	1
1	20	54.45	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

1

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	2.72	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	5.45	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	8.17	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	10.89	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	13.61	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	16.34	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	19.06	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	21.78	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	24.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	27.23	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

1 11	29.95	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 12	32.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 13	35.39	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 14	38.12	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 15	40.84	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 16	43.56	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 17	46.28	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 18	49.01	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 19	51.73	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1 20	54.45	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0

              For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-6

              For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-7


           ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
              stiffened
              if (d0/D) < 3 without long. stiffener, or
              if (d0/D) > 3 with both transv. and long. stiffeners.
              For handling requirement, (d0/D) > (260/(D/tw))**2

                   Vu = CVp

              where D = clear, unsupported distance between flange components

                   d0 = distance between transverse stiffeners

           ** This Article (6.10.9.2) indicates that a designer cannot
              count on post-buckling shear resistance from tension-
              field action for an unstiffened girder.
```

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     TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                     ********************************

                           S T R E N G T H  C A T E G O R Y; SEE NOTE
                  ------------------------------------------------------------
     SP IN D FROM WEB STABILITY    LFD/LRFD      UNSTIFFENED            OVERALL
     NO NO L SUPT UNSTIFFENED,[1] MAX. SHEAR <=> SHEAR CAPA- CATEGORY     WEB
            (ft)  ---- D/tw -----    (kip)       CITY,   [2]
     ---------------------------------------------------------------------------

      1  0   0.00       0            196.1   <      330.8       0          0
      1  1   2.72       0            181.4   <      330.8       0          0
      1  2   5.45       0            166.7   <      330.8       0          0
      1  3   8.17       0            152.3   <      330.8       0          0
      1  4  10.89       0            138.0   <      330.8       0          0
                                                Page 37
```

1	5	13.61	0	123.9	<	330.8	0	0
1	6	16.34	0	109.9	<	330.8	0	0
1	7	19.06	0	96.0	<	330.8	0	0
1	8	21.78	0	82.3	<	330.8	0	0
1	9	24.50	0	68.3	<	330.8	0	0
1	10	27.23	0	55.3	<	330.8	0	0
1	11	29.95	0	68.5	<	330.8	0	0
1	12	32.67	0	82.1	<	330.8	0	0
1	13	35.39	0	95.8	<	330.8	0	0
1	14	38.12	0	109.7	<	330.8	0	0
1	15	40.84	0	123.7	<	330.8	0	0
1	16	43.56	0	138.2	<	330.8	0	0
1	17	46.28	0	152.4	<	330.8	0	0
1	18	49.01	0	166.8	<	330.8	0	0
1	19	51.73	0	181.4	<	330.8	0	0
1	20	54.45	0	196.0	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	196.05	330.76	0	
1	1	2.72	33.	181.39	330.76	0	
1	2	5.45	33.	166.68	330.76	0	
1	3	8.17	33.	152.29	330.76	0	
1	4	10.89	33.	138.03	330.76	0	
1	5	13.61	33.	123.91	330.76	0	
1	6	16.34	33.	109.92	330.76	0	
1	7	19.06	33.	96.03	330.76	0	
1	8	21.78	33.	82.34	330.76	0	
1	9	24.50	33.	68.34	330.76	0	
1	10	27.23	33.	55.32	330.76	0	
1	11	29.95	33.	68.48	330.76	0	
1	12	32.67	33.	82.13	330.76	0	
1	13	35.39	33.	95.82	330.76	0	
1	14	38.12	33.	109.71	330.76	0	
1	15	40.84	33.	123.70	330.76	0	
1	16	43.56	33.	138.18	330.76	0	
1	17	46.28	33.	152.44	330.76	0	
1	18	49.01	33.	166.83	330.76	0	
1	19	51.73	33.	181.36	330.76	0	
1	20	54.45	33.	196.02	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	TOP FLANGE [1] (ksi)	BOT. FLANGE ff+fl/2 (ksi)	BOT. FLANGE [2] (ksi)	COMP. FLG fc (ksi)	COMP. FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	2.72	2.6	-	31.4	7.8	31.4	-	-	00-
1	2	5.45	4.9	-	31.4	14.6	31.4	-	-	00-
1	3	8.17	7.0	-	31.4	20.6	31.4	-	-	00-
1	4	10.89	8.6	-	31.4	16.1	31.4	-	-	00-
1	5	13.61	10.0	-	31.4	18.6	31.4	-	-	00-
1	6	16.34	11.2	-	31.4	20.7	31.4	-	-	00-
1	7	19.06	12.1	-	31.4	22.3	31.4	-	-	00-
1	8	21.78	12.8	-	31.4	23.4	31.4	-	-	00-
1	9	24.50	13.1	-	31.4	23.8	31.4	-	-	00-
1	10	27.23	13.2	-	31.4	24.0	31.4	-	-	00-
1	11	29.95	13.1	-	31.4	23.8	31.4	-	-	00-
1	12	32.67	12.7	-	31.4	23.4	31.4	-	-	00-
1	13	35.39	12.1	-	31.4	22.3	31.4	-	-	00-
1	14	38.12	11.2	-	31.4	20.7	31.4	-	-	00-
1	15	40.84	10.0	-	31.4	18.7	31.4	-	-	00-
1	16	43.56	8.6	-	31.4	16.1	31.4	-	-	00-
1	17	46.28	7.0	-	31.4	20.6	31.4	-	-	00-
1	18	49.01	4.9	-	31.4	14.7	31.4	-	-	00-
1	19	51.73	2.6	-	31.4	7.8	31.4	-	-	00-
1	20	54.45	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)

```
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
            but not to exceed the smaller of RhFyc and Fyw/0.7
            k = bending-buckling coefficient
              = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
            where:
            Dc = depth of the web in compression in the elastic
                 range. For composite sections, Dc shall be
                 determined as specified in Article D6.3.1

      "-" is N.A.

      Flag check - 0 = OK; 1 = NG
      T = Top Flange; B = Bottom Flange; C = Comp. Flange

      The values of fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
               ----------------------------- -----------------------------
SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
-----------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   2.72    TR     -0.1 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1  2   5.45    TR     -0.2 A B B^C^C D E E^  TR      3.6 A B B^C^C D E
 1  3   8.17    TR     -0.3 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1      9.73  SCG
 1  4  10.89    TR     -0.6 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
 1  5  13.61    TR     -0.6 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1  6  16.34    TR     -0.7 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1  7  19.06    TR     -0.7 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1  8  21.78    TR     -0.8 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1  9  24.50    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 10  27.23    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 11  29.95    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 12  32.67    TR     -0.8 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 13  35.39    TR     -0.7 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 14  38.12    TR     -0.7 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1 15  40.84    TR     -0.6 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1 16  43.56    TR     -0.6 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
 1     44.72  SCG
 1 17  46.28    TR     -0.3 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 18  49.01    TR     -0.2 A B B^C^C D E E^  TR      3.6 A B B^C^C D E
 1 19  51.73    TR     -0.1 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 20  54.45    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


-------------------------------------------------------------------------------
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.
-------------------------------------------------------------------------------
NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
             POC = Dead load point of contraflexure
```

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.72	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	2	5.45	TR	-0.1	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	3	8.17	TR	-0.2	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1		9.73	SCG					
1	4	10.89	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	5	13.61	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	6	16.34	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	7	19.06	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	8	21.78	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	9	24.50	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	10	27.23	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	11	29.95	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	12	32.67	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	13	35.39	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	14	38.12	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	15	40.84	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	16	43.56	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1		44.72	SCG					
1	17	46.28	TR	-0.2	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	18	49.01	TR	-0.1	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	19	51.73	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	20	54.45	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

5969 Span 2-S17 Interior Strength Mid M.RES

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	50.6	.29821E-01	1.5	TR 3.09	3	6.15
1	1	2.72	48.5	.29821E-01	1.4	TR 3.09	3	6.42
1	2	5.45	46.4	.29821E-01	1.4	TR 3.09	3	6.71
1	3	8.17	44.2	.29821E-01	1.3	TR 3.09	3	7.04
1	4	10.89	42.2	.27202E-01	1.1	TR 3.09	3	8.08
1	5	13.61	40.7	.27202E-01	1.1	TR 3.09	3	8.39
1	6	16.34	39.2	.27202E-01	1.1	TR 3.09	3	8.70
1	7	19.06	37.5	.27202E-01	1.0	TR 3.09	3	9.10
1	8	21.78	36.7	.27202E-01	1.0	TR 3.09	3	9.30
1	9	24.50	37.6	.27202E-01	1.0	TR 3.09	3	9.07
1	10	27.23	37.9	.27202E-01	1.0	TR 3.09	3	9.00
1	11	29.95	37.6	.27202E-01	1.0	TR 3.09	3	9.07
1	12	32.67	36.7	.27202E-01	1.0	TR 3.09	3	9.30
1	13	35.39	37.5	.27202E-01	1.0	TR 3.09	3	9.10
1	14	38.12	39.2	.27202E-01	1.1	TR 3.09	3	8.70
1	15	40.84	40.7	.27202E-01	1.1	TR 3.09	3	8.39
1	16	43.56	42.2	.27202E-01	1.1	TR 3.09	3	8.08
1	17	46.28	44.2	.29821E-01	1.3	TR 3.09	3	7.04
1	18	49.01	46.4	.29821E-01	1.4	TR 3.09	3	6.71
1	19	51.73	48.5	.29821E-01	1.4	TR 3.09	3	6.42
1	20	54.45	50.6	.29821E-01	1.5	TR 3.09	3	6.15

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	25.3	.29821E-01	0.8	TR	1.36	3	5.42
1	1	2.72	24.3	.29821E-01	0.7	TR	1.36	3	5.66
1	2	5.45	23.2	.29821E-01	0.7	TR	1.36	3	5.91
1	3	8.17	22.1	.29821E-01	0.7	TR	1.36	3	6.21
1	4	10.89	21.1	.27202E-01	0.6	TR	1.36	3	7.12
1	5	13.61	20.3	.27202E-01	0.6	TR	1.36	3	7.40
1	6	16.34	19.6	.27202E-01	0.5	TR	1.36	3	7.67
1	7	19.06	18.7	.27202E-01	0.5	TR	1.36	3	8.03
1	8	21.78	18.3	.27202E-01	0.5	TR	1.36	3	8.20
1	9	24.50	18.8	.27202E-01	0.5	TR	1.36	3	8.00
1	10	27.23	19.0	.27202E-01	0.5	TR	1.36	3	7.93
1	11	29.95	18.8	.27202E-01	0.5	TR	1.36	3	8.00
1	12	32.67	18.3	.27202E-01	0.5	TR	1.36	3	8.20
1	13	35.39	18.7	.27202E-01	0.5	TR	1.36	3	8.03
1	14	38.12	19.6	.27202E-01	0.5	TR	1.36	3	7.67
1	15	40.84	20.3	.27202E-01	0.6	TR	1.36	3	7.40
1	16	43.56	21.1	.27202E-01	0.6	TR	1.36	3	7.12
1	17	46.28	22.1	.29821E-01	0.7	TR	1.36	3	6.21
1	18	49.01	23.2	.29821E-01	0.7	TR	1.36	3	5.91
1	19	51.73	24.3	.29821E-01	0.7	TR	1.36	3	5.66
1	20	54.45	25.3	.29821E-01	0.8	TR	1.36	3	5.42

Please read NOTE on the following page

NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D FROM L (ft)	SUPT 1=POS. 0=NEG.	MOMENT REGION P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) FATIG. CRI.	SEE NOTE <=>	N1	N2	STATUS BLANK=OK CHECK=**
1	10	27.23	1	1441.3	1626.6		165	>	75		
1	20	54.45	1	1441.3	1626.6		165	>	75		

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or
P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	27.225	6.000
1	27.225	54.450	6.000

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	TCR
1	0.0	54.5	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	UNFACTORED L+I MAX. STRESS POS.	UNFACTORED L+I MAX. STRESS NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.72	2103.1	62.0	32.0	173.1	0.0	8.47	HL-93	6.54	0
1	2	5.45	2103.1	117.3	60.7	324.8	0.0	4.27	HL-93	3.29	0
1	3	8.17	2103.1	166.0	86.0	454.9	0.0	2.89	HL-93	2.23	0
1	4	10.89	2973.8	208.3	107.9	563.5	0.0	3.37	HL-93	2.60	0
1	5	13.61	2973.8	244.2	126.5	648.0	0.0	2.85	HL-93	2.20	0
1	6	16.34	2973.8	273.7	141.6	716.2	0.0	2.52	HL-93	1.94	0
1	7	19.06	2973.8	296.8	153.4	769.4	0.0	2.30	HL-93	1.77	0
1	8	21.78	2973.8	313.4	161.9	803.7	0.0	2.17	HL-93	1.67	0
1	9	24.50	2973.8	323.1	166.9	812.5	0.0	2.13	HL-93	1.64	0
1	10	27.23	2973.8	326.2	168.6	815.5	0.0	2.11	HL-93	1.63	0
1	11	29.95	2973.8	322.8	166.9	812.5	0.0	2.13	HL-93	1.64	0
1	12	32.67	2973.8	313.0	161.9	803.7	0.0	2.17	HL-93	1.67	0
1	13	35.39	2973.8	296.8	153.4	769.4	0.0	2.30	HL-93	1.77	0
1	14	38.12	2973.8	274.2	141.6	716.2	0.0	2.51	HL-93	1.94	0
1	15	40.84	2973.8	245.2	126.5	648.0	0.0	2.85	HL-93	2.19	0
1	16	43.56	2973.8	209.1	107.9	563.5	0.0	3.37	HL-93	2.60	0
1	17	46.28	2103.1	166.4	86.0	454.9	0.0	2.89	HL-93	2.23	0
1	18	49.01	2103.1	117.4	60.7	324.8	0.0	4.27	HL-93	3.29	0
1	19	51.73	2103.1	61.9	32.0	173.1	0.0	8.47	HL-93	6.54	0
1	20	54.45	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	SUPT	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS TOP;ksi	UNFACTORED DEAD LOAD STRESS BOT;ksi	UNFACTORED L+I STRESS MAX. POS. STEEL TOP;ksi	UNFACTORED L+I STRESS MAX. POS. STEEL BOT;ksi	UNFACTORED L+I STRESS MAX. NEG. STEEL TOP;ksi	UNFACTORED L+I STRESS MAX. NEG. STEEL BOT;ksi	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	2.72	31.4	-2.30	2.85	-0.24	3.80	0.00	0.00	7.51	HL-93	5.78
1	2	5.45	31.4	-4.36	5.39	-0.44	7.12	0.00	0.00	3.65	HL-93	2.80
1	3	8.17	31.4	-6.17	7.63	-0.62	9.97	0.00	0.00	2.38	HL-93	1.83
1	4	10.89	31.4	-7.04	5.94	-1.17	7.81	0.00	0.00	3.26	HL-93	2.50
1	5	13.61	31.4	-8.26	6.96	-1.34	8.98	0.00	0.00	2.72	HL-93	2.09
1	6	16.34	31.4	-9.26	7.80	-1.48	9.92	0.00	0.00	2.37	HL-93	1.83
1	7	19.06	31.4	-10.03	8.46	-1.59	10.66	0.00	0.00	2.15	HL-93	1.65
1	8	21.78	31.4	-10.60	8.93	-1.66	11.13	0.00	0.00	2.01	HL-93	1.55
1	9	24.50	31.4	-10.92	9.20	-1.68	11.26	0.00	0.00	1.97	HL-93	1.51
1	10	27.23	31.4	-11.03	9.29	-1.69	11.30	0.00	0.00	1.95	HL-93	1.50
1	11	29.95	31.4	-10.91	9.20	-1.68	11.26	0.00	0.00	1.97	HL-93	1.51
1	12	32.67	31.4	-10.58	8.92	-1.66	11.13	0.00	0.00	2.01	HL-93	1.55
1	13	35.39	31.4	-10.04	8.46	-1.59	10.66	0.00	0.00	2.15	HL-93	1.65
1	14	38.12	31.4	-9.27	7.81	-1.48	9.92	0.00	0.00	2.37	HL-93	1.82
1	15	40.84	31.4	-8.29	6.98	-1.34	8.98	0.00	0.00	2.71	HL-93	2.09
1	16	43.56	31.4	-7.07	5.95	-1.17	7.81	0.00	0.00	3.25	HL-93	2.50
1	17	46.28	31.4	-6.19	7.65	-0.62	9.97	0.00	0.00	2.38	HL-93	1.83
1	18	49.01	31.4	-4.36	5.40	-0.44	7.12	0.00	0.00	3.64	HL-93	2.80
1	19	51.73	31.4	-2.30	2.85	-0.24	3.80	0.00	0.00	7.51	HL-93	5.78
1	20	54.45	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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RATING

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	54.45	27.23	STRENGTH L.S.	2.11	1.63	HL-93
1	54.45	27.23	SERVICEBILITY	1.95	1.50	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.952
Inventory= 1.502

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	54.45	27.23	STRENGTH L.S.	3.08	755.83
1	54.45	27.23	SERVICEBILITY	2.70	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.701

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	23.9	12.4	84.9	0.0	2.47	HL-93	1.91
1	1	2.72	330.8	21.6	11.1	79.3	-1.9	2.69	HL-93	2.08
1	2	5.45	330.8	19.1	9.9	73.6	-3.9	2.95	HL-93	2.27

1	3	8.17	330.8	16.7	8.7	68.1	-6.9	3.24	HL-93	2.50
1	4	10.89	330.8	14.4	7.4	62.6	-10.1	3.58	HL-93	2.76
1	5	13.61	330.8	12.0	6.2	57.2	-13.4	3.97	HL-93	3.07
1	6	16.34	330.8	9.7	5.0	51.9	-16.7	4.45	HL-93	3.43
1	7	19.06	330.8	7.3	3.7	46.7	-20.2	5.02	HL-93	3.87
1	8	21.78	330.8	4.9	2.5	41.5	-23.6	5.73	HL-93	4.42
1	9	24.50	330.8	2.3	1.2	36.4	-27.2	6.64	HL-93	5.12
1	10	27.23	330.8	-0.1	0.0	31.6	-31.6	7.76	HL-93	5.99
1	11	29.95	330.8	-2.4	-1.2	27.2	-36.4	6.63	HL-93	5.12
1	12	32.67	330.8	-4.8	-2.5	23.6	-41.5	5.73	HL-93	4.42
1	13	35.39	330.8	-7.1	-3.7	20.2	-46.7	5.03	HL-93	3.88
1	14	38.12	330.8	-9.5	-5.0	16.7	-51.9	4.45	HL-93	3.43
1	15	40.84	330.8	-11.8	-6.2	13.4	-57.2	3.98	HL-93	3.07
1	16	43.56	330.8	-14.5	-7.4	10.1	-62.6	3.57	HL-93	2.76
1	17	46.28	330.8	-16.8	-8.7	6.9	-68.1	3.24	HL-93	2.50
1	18	49.01	330.8	-19.2	-9.9	3.9	-73.6	2.95	HL-93	2.27
1	19	51.73	330.8	-21.6	-11.1	1.9	-79.3	2.69	HL-93	2.08
1	20	54.45	330.8	-23.9	-12.4	0.0	-84.9	2.47	HL-93	1.91

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	54.45	0.00	STRENGTH L.S.	2.47	1.91	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.471
Inventory= 1.906

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	54.45	0.00	STRENGTH L.S.	3.04	93.22

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.040

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

5969 Span 2-S17 Interior Strength Mid M.RES

1	27.23	3	0.425	-0.28	HL-93	2.92
		4	0.433	-0.29		2.86

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95NB Span2-S17 Interior Fat & CP 8/20/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Interior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       17           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
1
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
54.45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT (lb/ft)	STANDARD SECTN PLATE GIRDER DEPTH (in)	WEB THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.0000	10.00	0.9375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

1

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	9.73				33.	33.	33.
2	2	2	35.00				33.	33.	33.
3	1	1	9.73				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)	DF M-ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	OPTIONAL DISTRIBUTION FACTORS DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	POUR NO	IDENTIFICATION DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	MODULAR RATIO N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	54.45

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	3.68
2	2	Diaphragm	0.000	0.00	0.00	0.29	22.52
3	2	Diaphragm	0.000	0.00	0.00	0.29	41.36
4	1	Railing	0.087	0.00	54.45	0.00	0.00
5	1	Curb	0.076	0.00	54.45	0.00	0.00
6	0	Pavement	0.292	0.00	54.45	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

1

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied
in the negative moment region, the contribution
of rebar on the section properties in the negative moment
region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and
7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	3.68	33.0	33.0	3.68
3.68	22.52	33.0	33.0	18.84
22.52	41.36	33.0	33.0	18.84
41.36	54.45	33.0	33.0	13.09

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

---

13 GAMMA for Load DC maximum = 1.25

14 GAMMA for Load DC minimum = 0.90

15 GAMMA for Load DW minimum = 1.50

16 GAMMA for Load DW minimum = 0.65

17 GAMMA for LL Load Strength I = 1.75

18 GAMMA for LL Load Strength II = 1.35

19 GAMMA for LL Load Service I = 1.00

20 GAMMA for LL Load Service II = 1.30

21 GAMMA for LL Load Fatigue = 0.75

22 ETA for Service Limit State = 1.00

23 ETA for Strength Limit State = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.727		0.1386		0.8656

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	54.45
1	DC2	0.087	0.000	54.450
2	DC2	0.076	0.000	54.450

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1322.1	49.0	198.3	45.3	243.59

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.711	0.711	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.20	5900.0	32.9	16.45	358.7	358.7
1	2	6.41	5900.0	32.9	16.45	358.7	358.7
1	3	9.61	5900.0	32.9	16.45	358.7	358.7
1	4	12.81	8019.6	32.9	13.79	581.6	400.0
1	5	16.01	8019.6	32.9	13.79	581.6	400.0
1	6	19.22	8019.6	32.9	13.79	581.6	400.0
1	7	22.42	8019.6	32.9	13.79	581.6	400.0
1	8	25.62	8019.6	32.9	13.79	581.6	400.0

1	9	28.83	8019.6	32.9	13.79	581.6	400.0
1	10	32.03	8019.6	32.9	13.79	581.6	400.0
1	11	35.23	8019.6	32.9	13.79	581.6	400.0
1	12	38.44	8019.6	32.9	13.79	581.6	400.0
1	13	41.64	8019.6	32.9	13.79	581.6	400.0
1	14	44.84	5900.0	32.9	16.45	358.7	358.7
1	15	48.04	5900.0	32.9	16.45	358.7	358.7
1	16	51.25	5900.0	32.9	16.45	358.7	358.7
1	17	54.45	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12575.9	0.024742	496.1	1666.1	788.1
1	1	3.20	12575.9	0.024742	496.1	1666.1	788.1
1	2	6.41	12575.9	0.024742	496.1	1666.1	788.1
1	3	9.61	12575.9	0.024742	496.1	1666.1	788.1
1	4	12.81	17985.3	0.022239	789.3	1627.6	924.2
1	5	16.01	17985.3	0.022239	789.3	1627.6	924.2
1	6	19.22	17985.3	0.022239	789.3	1627.6	924.2
1	7	22.42	17985.3	0.022239	789.3	1627.6	924.2
1	8	25.62	17985.3	0.022239	789.3	1627.6	924.2
1	9	28.83	17985.3	0.022239	789.3	1627.6	924.2
1	10	32.03	17985.3	0.022239	789.3	1627.6	924.2
1	11	35.23	17985.3	0.022239	789.3	1627.6	924.2
1	12	38.44	17985.3	0.022239	789.3	1627.6	924.2
1	13	41.64	17985.3	0.022239	789.3	1627.6	924.2
1	14	44.84	12575.9	0.024742	496.1	1666.1	788.1
1	15	48.04	12575.9	0.024742	496.1	1666.1	788.1
1	16	51.25	12575.9	0.024742	496.1	1666.1	788.1
1	17	54.45	12575.9	0.024742	496.1	1666.1	788.1

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	16953.0	0.029821	547.3	8815.3	1640.6
1	1	3.20	16953.0	0.029821	547.3	8815.3	1640.6
1	2	6.41	16953.0	0.029821	547.3	8815.3	1640.6
1	3	9.61	16953.0	0.029821	547.3	8815.3	1640.6
1	4	12.81	25501.1	0.027202	866.2	5800.7	1991.3
1	5	16.01	25501.1	0.027202	866.2	5800.7	1991.3

1	6	19.22	25501.1	0.027202	866.2	5800.7	1991.3
1	7	22.42	25501.1	0.027202	866.2	5800.7	1991.3
1	8	25.62	25501.1	0.027202	866.2	5800.7	1991.3
1	9	28.83	25501.1	0.027202	866.2	5800.7	1991.3
1	10	32.03	25501.1	0.027202	866.2	5800.7	1991.3
1	11	35.23	25501.1	0.027202	866.2	5800.7	1991.3
1	12	38.44	25501.1	0.027202	866.2	5800.7	1991.3
1	13	41.64	25501.1	0.027202	866.2	5800.7	1991.3
1	14	44.84	16953.0	0.029821	547.3	8815.3	1640.6
1	15	48.04	16953.0	0.029821	547.3	8815.3	1640.6
1	16	51.25	16953.0	0.029821	547.3	8815.3	1640.6
1	17	54.45	16953.0	0.029821	547.3	8815.3	1640.6

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	3.20	11.4	59.7	1.2	0.0	72.2
1	2	6.41	21.3	111.9	2.0	0.0	135.2
1	3	9.61	29.9	156.6	2.7	0.0	189.2
1	4	12.81	37.0	193.9	3.4	0.0	234.3
1	5	16.01	42.7	223.7	4.2	0.0	270.6
1	6	19.22	46.9	246.1	4.9	0.0	297.9
1	7	22.42	49.8	261.0	5.6	0.0	316.4
1	8	25.62	51.2	268.5	5.5	0.0	325.1
1	9	28.83	51.2	268.5	5.3	0.0	325.0
1	10	32.03	49.8	261.0	5.1	0.0	315.9
1	11	35.23	46.9	246.1	4.9	0.0	297.9

1 12	38.44	42.7	223.7	4.7	0.0	271.1
1 13	41.64	37.0	193.9	4.4	0.0	235.3
1 14	44.84	29.9	156.6	3.3	0.0	189.8
1 15	48.04	21.3	111.9	2.2	0.0	135.4
1 16	51.25	11.4	59.7	1.1	0.0	72.1
1 17	54.45	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.20	24.0	13.4	0.0	0.0	0.0	37.3
1	2	6.41	44.9	25.1	0.0	0.0	0.0	70.0
1	3	9.61	62.9	35.1	0.0	0.0	0.0	98.0
1	4	12.81	77.9	43.5	0.0	0.0	0.0	121.4
1	5	16.01	89.9	50.2	0.0	0.0	0.0	140.0
1	6	19.22	98.9	55.2	0.0	0.0	0.0	154.0
1	7	22.42	104.8	58.5	0.0	0.0	0.0	163.4
1	8	25.62	107.8	60.2	0.0	0.0	0.0	168.0
1	9	28.83	107.8	60.2	0.0	0.0	0.0	168.0
1	10	32.03	104.8	58.5	0.0	0.0	0.0	163.4
1	11	35.23	98.9	55.2	0.0	0.0	0.0	154.0
1	12	38.44	89.9	50.2	0.0	0.0	0.0	140.0
1	13	41.64	77.9	43.5	0.0	0.0	0.0	121.4
1	14	44.84	62.9	35.1	0.0	0.0	0.0	98.0
1	15	48.04	44.9	25.1	0.0	0.0	0.0	70.0
1	16	51.25	24.0	13.4	0.0	0.0	0.0	37.3
1	17	54.45	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.20	0.0	0.0	201.5	HL-93	0.0	HL-93
1	2	6.41	0.0	0.0	373.1	HL-93	0.0	HL-93
1	3	9.61	0.0	0.0	515.1	HL-93	0.0	HL-93
1	4	12.81	0.0	0.0	624.8	HL-93	0.0	HL-93
1	5	16.01	0.0	0.0	709.0	HL-93	0.0	HL-93
1	6	19.22	0.0	0.0	771.9	HL-93	0.0	HL-93
1	7	22.42	0.0	0.0	806.3	HL-93	0.0	HL-93
1	8	25.62	0.0	0.0	814.5	HL-93	0.0	HL-93
1	9	28.83	0.0	0.0	814.5	HL-93	0.0	HL-93
1	10	32.03	0.0	0.0	806.3	HL-93	0.0	HL-93
1	11	35.23	0.0	0.0	771.9	HL-93	0.0	HL-93
1	12	38.44	0.0	0.0	709.0	HL-93	0.0	HL-93
1	13	41.64	0.0	0.0	624.8	HL-93	0.0	HL-93

1 14	44.84	0.0	0.0	515.1	HL-93	0.0	HL-93
1 15	48.04	0.0	0.0	373.1	HL-93	0.0	HL-93
1 16	51.25	0.0	0.0	201.5	HL-93	0.0	HL-93
1 17	54.45	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.20	69.	0.	69.
1	2	6.41	124.	0.	124.
1	3	9.61	166.	0.	166.
1	4	12.81	200.	0.	200.
1	5	16.01	224.	0.	224.
1	6	19.22	237.	0.	237.
1	7	22.42	242.	0.	242.
1	8	25.62	242.	0.	242.
1	9	28.83	242.	0.	242.
1	10	32.03	242.	0.	242.
1	11	35.23	237.	0.	237.
1	12	38.44	224.	0.	224.
1	13	41.64	200.	0.	200.
1	14	44.84	166.	0.	166.
1	15	48.04	124.	0.	124.
1	16	51.25	69.	0.	69.
1	17	54.45	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.20	72.2	37.3	201.5	HL-93	0.0	HL-93	311.0	109.6
1	2	6.41	135.2	70.0	373.1	HL-93	0.0	HL-93	578.3	205.2
1	3	9.61	189.2	98.0	515.1	HL-93	0.0	HL-93	802.3	287.2
1	4	12.81	234.3	121.4	624.8	HL-93	0.0	HL-93	980.5	355.7
1	5	16.01	270.6	140.0	709.0	HL-93	0.0	HL-93	1119.6	410.6
1	6	19.22	297.9	154.0	771.9	HL-93	0.0	HL-93	1223.9	452.0
1	7	22.42	316.4	163.4	806.3	HL-93	0.0	HL-93	1286.1	479.8
1	8	25.62	325.1	168.0	814.5	HL-93	0.0	HL-93	1307.6	493.2
1	9	28.83	325.0	168.0	814.5	HL-93	0.0	HL-93	1307.4	493.0
1	10	32.03	315.9	163.4	806.3	HL-93	0.0	HL-93	1285.5	479.3
1	11	35.23	297.9	154.0	771.9	HL-93	0.0	HL-93	1223.9	452.0
1	12	38.44	271.1	140.0	709.0	HL-93	0.0	HL-93	1120.2	411.1
1	13	41.64	235.3	121.4	624.8	HL-93	0.0	HL-93	981.5	356.7
1	14	44.84	189.8	98.0	515.1	HL-93	0.0	HL-93	802.9	287.8
1	15	48.04	135.4	70.0	373.1	HL-93	0.0	HL-93	578.6	205.4
1	16	51.25	72.1	37.3	201.5	HL-93	0.0	HL-93	311.0	109.5
1	17	54.45	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN D NO L	FROM SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.20	311.0	371.5	495.5	414.9	164.4	51.8
1	2	6.41	578.3	690.3	920.7	771.5	307.8	93.0
1	3	9.61	802.3	956.8	1276.1	1070.1	430.8	124.8
1	4	12.81	980.5	1168.0	1557.5	1307.6	533.5	150.0
1	5	16.01	1119.6	1332.3	1776.5	1492.9	615.9	168.0
1	6	19.22	1223.9	1455.5	1940.5	1631.8	678.0	177.4
1	7	22.42	1286.1	1528.0	2036.9	1714.4	719.7	181.5
1	8	25.62	1307.6	1552.0	2068.7	1743.0	739.8	181.2
1	9	28.83	1307.4	1551.8	2068.5	1742.7	739.5	181.2
1	10	32.03	1285.5	1527.4	2036.3	1713.7	718.9	181.5
1	11	35.23	1223.9	1455.5	1940.5	1631.8	678.0	177.4
1	12	38.44	1120.2	1332.9	1777.2	1493.6	616.7	168.0
1	13	41.64	981.5	1169.0	1558.8	1308.9	535.0	150.0
1	14	44.84	802.9	957.4	1276.9	1070.9	431.7	124.8
1	15	48.04	578.6	690.5	921.0	771.8	308.2	93.0
1	16	51.25	311.0	371.4	495.4	414.8	164.2	51.8
1	17	54.45	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.8	19.8	0.4	0.0	23.9
1	1	3.20	3.3	17.5	0.4	0.0	21.2
1	2	6.41	2.9	15.1	0.2	0.0	18.2
1	3	9.61	2.4	12.8	0.2	0.0	15.5
1	4	12.81	2.0	10.5	0.2	0.0	12.7
1	5	16.01	1.6	8.1	0.2	0.0	9.9
1	6	19.22	1.1	5.8	0.2	0.0	7.2
1	7	22.42	0.7	3.5	0.2	0.0	4.4
1	8	25.62	0.2	1.2	-0.1	0.0	1.3
1	9	28.83	-0.2	-1.2	-0.1	0.0	-1.4
1	10	32.03	-0.7	-3.5	-0.1	0.0	-4.2
1	11	35.23	-1.1	-5.8	-0.1	0.0	-7.0
1	12	38.44	-1.6	-8.1	-0.1	0.0	-9.8
1	13	41.64	-2.0	-10.5	-0.3	0.0	-12.8
1	14	44.84	-2.4	-12.8	-0.3	0.0	-15.6
1	15	48.04	-2.9	-15.1	-0.3	0.0	-18.4
1	16	51.25	-3.3	-17.5	-0.3	0.0	-21.1
1	17	54.45	-3.8	-19.8	-0.3	0.0	-23.9

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	DC2	CONCENTRATED LOADS DW	DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.9	4.4	0.0	0.0	12.4	23.9	36.3
1	1	3.20	7.0	3.9	0.0	0.0	10.9	21.2	32.1
1	2	6.41	6.1	3.4	0.0	0.0	9.5	18.2	27.7
1	3	9.61	5.1	2.9	0.0	0.0	8.0	15.5	23.5
1	4	12.81	4.2	2.3	0.0	0.0	6.6	12.7	19.3
1	5	16.01	3.3	1.8	0.0	0.0	5.1	9.9	15.0
1	6	19.22	2.3	1.3	0.0	0.0	3.6	7.2	10.8
1	7	22.42	1.4	0.8	0.0	0.0	2.2	4.4	6.6
1	8	25.62	0.5	0.3	0.0	0.0	0.7	1.3	2.1
1	9	28.83	-0.5	-0.3	0.0	0.0	-0.7	-1.4	-2.2
1	10	32.03	-1.4	-0.8	0.0	0.0	-2.2	-4.2	-6.4
1	11	35.23	-2.3	-1.3	0.0	0.0	-3.6	-7.0	-10.6
1	12	38.44	-3.3	-1.8	0.0	0.0	-5.1	-9.8	-14.9
1	13	41.64	-4.2	-2.3	0.0	0.0	-6.6	-12.8	-19.4
1	14	44.84	-5.1	-2.9	0.0	0.0	-8.0	-15.6	-23.6
1	15	48.04	-6.1	-3.4	0.0	0.0	-9.5	-18.4	-27.8
1	16	51.25	-7.0	-3.9	0.0	0.0	-10.9	-21.1	-32.1
1	17	54.45	-7.9	-4.4	0.0	0.0	-12.4	-23.9	-36.3

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	84.9	HL-93	0.0	HL-93
1	1	3.20	0.0	0.0	78.3	HL-93	-2.2	HL-93
1	2	6.41	0.0	0.0	71.7	HL-93	-4.9	HL-93
1	3	9.61	0.0	0.0	65.2	HL-93	-8.6	HL-93
1	4	12.81	0.0	0.0	58.8	HL-93	-12.4	HL-93
1	5	16.01	0.0	0.0	52.5	HL-93	-16.3	HL-93
1	6	19.22	0.0	0.0	46.4	HL-93	-20.4	HL-93
1	7	22.42	0.0	0.0	40.3	HL-93	-24.5	HL-93
1	8	25.62	0.0	0.0	34.4	HL-93	-28.9	HL-93
1	9	28.83	0.0	0.0	28.9	HL-93	-34.4	HL-93
1	10	32.03	0.0	0.0	24.5	HL-93	-40.3	HL-93
1	11	35.23	0.0	0.0	20.4	HL-93	-46.4	HL-93
1	12	38.44	0.0	0.0	16.3	HL-93	-52.5	HL-93
1	13	41.64	0.0	0.0	12.4	HL-93	-58.8	HL-93
1	14	44.84	0.0	0.0	8.6	HL-93	-65.2	HL-93
1	15	48.04	0.0	0.0	4.9	HL-93	-71.7	HL-93
1	16	51.25	0.0	0.0	2.2	HL-93	-78.3	HL-93
1	17	54.45	0.0	0.0	0.0	HL-93	-84.9	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	34.	0.	34.
1	1	3.20	31.	-1.	32.
1	2	6.41	28.	-3.	30.
1	3	9.61	25.	-4.	29.
1	4	12.81	22.	-5.	27.
1	5	16.01	19.	-7.	26.
1	6	19.22	16.	-8.	25.
1	7	22.42	14.	-10.	25.
1	8	25.62	13.	-12.	25.
1	9	28.83	12.	-13.	25.
1	10	32.03	10.	-14.	25.
1	11	35.23	8.	-16.	25.
1	12	38.44	7.	-19.	26.
1	13	41.64	5.	-22.	27.
1	14	44.84	4.	-25.	29.
1	15	48.04	3.	-28.	30.
1	16	51.25	1.	-31.	32.
1	17	54.45	0.	-34.	34.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN D NO L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	33.8	.31015E-01	0.52	0.2500*	.14206E-01	0.24	0.2500*
1	1	3.20	32.1	.31015E-01	0.50	0.2500*	.14206E-01	0.23	0.2500*
1	2	6.41	30.4	.31015E-01	0.47	0.2500*	.14206E-01	0.22	0.2500*
1	3	9.61	28.7	.31015E-01	0.45	0.2500*	.14206E-01	0.20	0.2500*
1	4	12.81	27.4	.28798E-01	0.39	0.2500*	.86429E-02	0.12	0.2500*
1	5	16.01	26.3	.28798E-01	0.38	0.2500*	.86429E-02	0.11	0.2500*
1	6	19.22	24.9	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	7	22.42	24.6	.28798E-01	0.35	0.2500*	.86429E-02	0.11	0.2500*
1	8	25.62	25.2	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	9	28.83	25.2	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	10	32.03	24.6	.28798E-01	0.35	0.2500*	.86429E-02	0.11	0.2500*
1	11	35.23	24.9	.28798E-01	0.36	0.2500*	.86429E-02	0.11	0.2500*
1	12	38.44	26.3	.28798E-01	0.38	0.2500*	.86429E-02	0.11	0.2500*
1	13	41.64	27.4	.28798E-01	0.39	0.2500*	.86429E-02	0.12	0.2500*
1	14	44.84	28.7	.31015E-01	0.45	0.2500*	.14206E-01	0.20	0.2500*
1	15	48.04	30.4	.31015E-01	0.47	0.2500*	.14206E-01	0.22	0.2500*
1	16	51.25	32.1	.31015E-01	0.50	0.2500*	.14206E-01	0.23	0.2500*
1	17	54.45	33.8	.31015E-01	0.52	0.2500*	.14206E-01	0.24	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange

Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	36.3	84.9	HL-93	0.0	HL-93	121.3	36.3
1	1	3.20	32.1	78.3	HL-93	-2.2	HL-93	110.3	29.8
1	2	6.41	27.7	71.7	HL-93	-4.9	HL-93	99.4	22.8
1	3	9.61	23.5	65.2	HL-93	-8.6	HL-93	88.7	14.9
1	4	12.81	19.3	58.8	HL-93	-12.4	HL-93	78.1	6.8
1	5	16.01	15.0	52.5	HL-93	-16.3	HL-93	67.6	-1.3
1	6	19.22	10.8	46.4	HL-93	-20.4	HL-93	57.2	-9.6
1	7	22.42	6.6	40.3	HL-93	-24.5	HL-93	46.9	-17.9
1	8	25.62	2.1	34.4	HL-93	-28.9	HL-93	36.4	-26.8
1	9	28.83	-2.2	28.9	HL-93	-34.4	HL-93	26.7	-36.6
1	10	32.03	-6.4	24.5	HL-93	-40.3	HL-93	18.1	-46.7
1	11	35.23	-10.6	20.4	HL-93	-46.4	HL-93	9.7	-57.0
1	12	38.44	-14.9	16.3	HL-93	-52.5	HL-93	1.5	-67.4
1	13	41.64	-19.4	12.4	HL-93	-58.8	HL-93	-7.0	-78.2
1	14	44.84	-23.6	8.6	HL-93	-65.2	HL-93	-15.0	-88.8
1	15	48.04	-27.8	4.9	HL-93	-71.7	HL-93	-22.9	-99.5
1	16	51.25	-32.1	2.2	HL-93	-78.3	HL-93	-29.8	-110.3
1	17	54.45	-36.3	0.0	HL-93	-84.9	HL-93	-36.3	-121.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	121.3	146.8	196.1	162.1	54.5	25.3
1	1	3.20	110.3	133.8	178.8	147.5	48.1	24.1
1	2	6.41	99.4	120.9	161.6	132.9	41.6	22.8
1	3	9.61	88.7	108.2	144.7	118.7	35.2	21.6
1	4	12.81	78.1	95.7	128.1	104.5	28.9	20.6
1	5	16.01	67.6	83.3	111.6	90.5	22.5	19.7

1	6	19.22	57.2	71.1	95.2	76.7	16.2	18.7
1	7	22.42	46.9	59.0	79.1	63.0	9.9	18.5
1	8	25.62	36.4	46.8	62.9	49.1	3.1	18.9
1	9	28.83	-36.6	-46.9	-63.0	-49.3	-3.3	18.9
1	10	32.03	-46.7	-58.8	-78.9	-62.8	-9.6	18.5
1	11	35.23	-57.0	-70.9	-95.0	-76.5	-16.0	18.7
1	12	38.44	-67.4	-83.2	-111.3	-90.3	-22.3	19.7
1	13	41.64	-78.2	-95.8	-128.2	-104.7	-29.1	20.6
1	14	44.84	-88.8	-108.4	-144.9	-118.8	-35.4	21.6
1	15	48.04	-99.5	-121.0	-161.7	-133.1	-41.8	22.8
1	16	51.25	-110.3	-133.8	-178.8	-147.5	-48.1	24.1
1	17	54.45	-121.2	-146.7	-196.0	-162.0	-54.4	25.3

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	36.32	0.00	HL-93	84.95	HL-93	ST1	30.70	196.05
						ST2	30.70	162.07
						ST4	42.56	54.49
						SE1	36.32	121.27
						SE2	36.32	146.76
2	36.30	0.00	HL-93	84.95	HL-93	ST1	30.68	196.02
						ST2	30.68	162.04
						ST4	42.52	54.45
						SE1	36.30	121.25
						SE2	36.30	146.73

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING
UNIVERSITY OF MARYLAND

MERLIN V 10.4
COMPOSITE
LRF -- 2010
ANALYSIS

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.20	-0.0233	-0.1224	0.0000	-0.0345	-0.1481	-0.0345	-0.1826
1	2	6.41	-0.0455	-0.2386	0.0000	-0.0672	-0.2887	-0.0672	-0.3560
1	3	9.61	-0.0654	-0.3433	0.0000	-0.0966	-0.4155	-0.0966	-0.5120
1	4	12.81	-0.0827	-0.4340	0.0000	-0.1219	-0.5253	-0.1219	-0.6472
1	5	16.01	-0.0972	-0.5100	0.0000	-0.1431	-0.6173	-0.1431	-0.7604
1	6	19.22	-0.1085	-0.5690	0.0000	-0.1596	-0.6888	-0.1596	-0.8483
1	7	22.42	-0.1161	-0.6093	0.0000	-0.1708	-0.7376	-0.1708	-0.9084
1	8	25.62	-0.1200	-0.6297	0.0000	-0.1765	-0.7623	-0.1765	-0.9388
1	9	28.83	-0.1200	-0.6297	0.0000	-0.1765	-0.7623	-0.1765	-0.9388
1	10	32.03	-0.1161	-0.6093	0.0000	-0.1708	-0.7376	-0.1708	-0.9084
1	11	35.23	-0.1085	-0.5690	0.0000	-0.1596	-0.6889	-0.1596	-0.8485
1	12	38.44	-0.0972	-0.5100	0.0000	-0.1431	-0.6176	-0.1431	-0.7607
1	13	41.64	-0.0827	-0.4340	0.0000	-0.1219	-0.5256	-0.1219	-0.6475

1	14	44.84	-0.0654	-0.3433	0.0000	-0.0966	-0.4157	-0.0966	-0.5123
1	15	48.04	-0.0455	-0.2386	0.0000	-0.0672	-0.2889	-0.0672	-0.3562
1	16	51.25	-0.0233	-0.1224	0.0000	-0.0345	-0.1482	-0.0345	-0.1827
1	17	54.45	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.2	-0.023	0 1/16	-0.125	0 1/ 8	-0.035	0 1/16	-0.183	0 3/16
1	2	6.4	-0.045	0 1/16	-0.243	0 1/ 4	-0.067	0 1/ 8	-0.356	0 3/ 8
1	3	9.6	-0.065	0 1/ 8	-0.350	0 3/ 8	-0.097	0 1/ 8	-0.512	0 9/16
1	4	12.8	-0.083	0 1/ 8	-0.443	0 1/ 2	-0.122	0 1/ 8	-0.647	0 11/16
1	5	16.0	-0.097	0 1/ 8	-0.520	0 9/16	-0.143	0 3/16	-0.760	0 13/16
1	6	19.2	-0.108	0 1/ 8	-0.580	0 5/ 8	-0.160	0 3/16	-0.848	0 7/ 8
1	7	22.4	-0.116	0 1/ 8	-0.621	0 5/ 8	-0.171	0 3/16	-0.908	0 15/16
1	8	25.6	-0.120	0 1/ 8	-0.642	0 11/16	-0.177	0 3/16	-0.939	1 0/16
1	9	28.8	-0.120	0 1/ 8	-0.642	0 11/16	-0.177	0 3/16	-0.939	1 0/16
1	10	32.0	-0.116	0 1/ 8	-0.621	0 5/ 8	-0.171	0 3/16	-0.908	0 15/16
1	11	35.2	-0.108	0 1/ 8	-0.580	0 5/ 8	-0.160	0 3/16	-0.848	0 7/ 8
1	12	38.4	-0.097	0 1/ 8	-0.520	0 9/16	-0.143	0 3/16	-0.761	0 13/16
1	13	41.6	-0.083	0 1/ 8	-0.443	0 1/ 2	-0.122	0 1/ 8	-0.647	0 11/16
1	14	44.8	-0.065	0 1/ 8	-0.350	0 3/ 8	-0.097	0 1/ 8	-0.512	0 9/16
1	15	48.0	-0.045	0 1/16	-0.243	0 1/ 4	-0.067	0 1/ 8	-0.356	0 3/ 8
1	16	51.2	-0.023	0 1/16	-0.125	0 1/ 8	-0.035	0 1/16	-0.183	0 3/16
1	17	54.5	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	27.23	3 0.425	-0.280	MAX HL-93	0.82	0.00251
			0.026	MIN		
			-0.075	MAX LANE		
			0.026	MIN LANE		
		4 0.433	-0.285	MAX HL-93	0.82	0.00251
			0.027	MIN		
			-0.077	MAX LANE		
			0.027	MIN LANE		

```
                    3    0.425     -0.185 MAX FA
                                    0.000 MIN FA

    ---------------------------------------------------------------------
    NOTE: [1] " - " indicates downward deflection

          [2] The distribution factor for LL+I deflection is defined as

              DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

              where NL= no. of traffic lanes
                    Ng= no. of girders
                    (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

          [3] This table is based upon the optional criteria specified
              in AASHTO LRFD Art. 3.6.1.3.2

          [4] The number of traffic lanes is determined according to
              AASHTO LRFD Art.3.6.1.1.1.
              The 1st line is for the most probable number of lanes
              and the 2nd line is for the next probable number of lanes.

          [5] Max rotations at left (1st line) & right (2nd line)
              supports of the span without averaging, factor and impact

          [6] If ADTT is between 100 and 1000, multi-presence
              factor of 0.95 is applied. If ADTT is below 100, factor
              is 0.9 (AASHTO LRFD C3.6.1.1.2).

          [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
              ********************************************

              STEEL DEAD LOAD    OTHER DEAD LOAD   TOTAL DEAD LOAD
 SP IN D FROM ----------------  ----------------  ----------------
 NO NO L SUPT    STEEL BEAM        STEEL BEAM        STEEL BEAM
                TOP      BOT      TOP      BOT      TOP      BOT
        (ft)       (ksi)             (ksi)             (ksi)
 ---------------------------------------------------------------------

  1  0   0.00    0.00    0.00      0.00    0.00      0.00    0.00
  1  1   3.20   -0.38    0.38     -2.04    2.04     -2.42    2.42
  1  2   6.41   -0.71    0.71     -3.81    3.81     -4.52    4.52
  1  3   9.61   -1.00    1.00     -5.33    5.33     -6.33    6.33
  1  4  12.81   -1.11    0.76     -5.92    4.07     -7.03    4.83
  1  5  16.01   -1.28    0.88     -6.84    4.70     -8.12    5.58
  1  6  19.22   -1.41    0.97     -7.53    5.18     -8.94    6.15
  1  7  22.42   -1.49    1.03     -8.00    5.50     -9.49    6.53
  1  8  25.62   -1.54    1.06     -8.22    5.65     -9.75    6.71
  1  9  28.83   -1.54    1.06     -8.21    5.65     -9.75    6.70
  1 10  32.03   -1.49    1.03     -7.98    5.49     -9.48    6.52
  1 11  35.23   -1.41    0.97     -7.53    5.18     -8.94    6.15
  1 12  38.44   -1.28    0.88     -6.85    4.71     -8.13    5.59
  1 13  41.64   -1.11    0.76     -5.95    4.09     -7.06    4.86
  1 14  44.84   -1.00    1.00     -5.35    5.35     -6.35    6.35
  1 15  48.04   -0.71    0.71     -3.82    3.82     -4.53    4.53
  1 16  51.25   -0.38    0.38     -2.03    2.03     -2.41    2.41
  1 17  54.45    0.00    0.00      0.00    0.00      0.00    0.00


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 TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
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(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	-0.02	-0.27	0.90	-0.02	-2.69	3.32
1	2	6.41	-0.04	-0.50	1.69	-0.04	-5.03	6.22
1	3	9.61	-0.06	-0.71	2.37	-0.06	-7.04	8.70
1	4	12.81	-0.06	-0.89	1.85	-0.06	-7.92	6.68
1	5	16.01	-0.07	-1.03	2.13	-0.07	-9.15	7.71
1	6	19.22	-0.07	-1.14	2.34	-0.07	-10.07	8.49
1	7	22.42	-0.08	-1.20	2.48	-0.08	-10.70	9.01
1	8	25.62	-0.08	-1.24	2.55	-0.08	-10.99	9.26
1	9	28.83	-0.08	-1.24	2.55	-0.08	-10.99	9.26
1	10	32.03	-0.08	-1.20	2.48	-0.08	-10.68	9.00
1	11	35.23	-0.07	-1.14	2.34	-0.07	-10.07	8.49
1	12	38.44	-0.07	-1.03	2.13	-0.07	-9.17	7.72
1	13	41.64	-0.06	-0.89	1.85	-0.06	-7.95	6.70
1	14	44.84	-0.06	-0.71	2.37	-0.06	-7.06	8.72
1	15	48.04	-0.04	-0.50	1.69	-0.04	-5.04	6.22
1	16	51.25	-0.02	-0.27	0.90	-0.02	-2.68	3.32
1	17	54.45	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	-0.16	-0.27	4.42	0.00	0.00	0.00
1	2	6.41	-0.30	-0.51	8.18	0.00	0.00	0.00
1	3	9.61	-0.42	-0.70	11.29	0.00	0.00	0.00
1	4	12.81	-0.42	-1.29	8.66	0.00	0.00	0.00
1	5	16.01	-0.47	-1.47	9.82	0.00	0.00	0.00
1	6	19.22	-0.52	-1.60	10.69	0.00	0.00	0.00
1	7	22.42	-0.54	-1.67	11.17	0.00	0.00	0.00
1	8	25.62	-0.55	-1.68	11.28	0.00	0.00	0.00
1	9	28.83	-0.55	-1.68	11.28	0.00	0.00	0.00
1	10	32.03	-0.54	-1.67	11.17	0.00	0.00	0.00
1	11	35.23	-0.52	-1.60	10.69	0.00	0.00	0.00
1	12	38.44	-0.47	-1.47	9.82	0.00	0.00	0.00
1	13	41.64	-0.42	-1.29	8.66	0.00	0.00	0.00
1	14	44.84	-0.42	-0.70	11.29	0.00	0.00	0.00
1	15	48.04	-0.30	-0.51	8.18	0.00	0.00	0.00
1	16	51.25	-0.16	-0.27	4.42	0.00	0.00	0.00
1	17	54.45	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.20	0.1	0.0	0.1	C	HL	1.5	0.0	1.5	T	HL
1	2	6.41	0.2	0.0	0.2	C	HL	2.7	0.0	2.7	T	HL
1	3	9.61	0.2	0.0	0.2	C	HL	3.6	0.0	3.6	T	HL
1	4	12.81	0.4	0.0	0.4	C	HL	2.8	0.0	2.8	T	HL
1	5	16.01	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	6	19.22	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	7	22.42	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	8	25.62	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	9	28.83	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	10	32.03	0.5	0.0	0.5	C	HL	3.4	0.0	3.4	T	HL
1	11	35.23	0.5	0.0	0.5	C	HL	3.3	0.0	3.3	T	HL
1	12	38.44	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	13	41.64	0.4	0.0	0.4	C	HL	2.8	0.0	2.8	T	HL
1	14	44.84	0.2	0.0	0.2	C	HL	3.6	0.0	3.6	T	HL
1	15	48.04	0.2	0.0	0.2	C	HL	2.7	0.0	2.7	T	HL
1	16	51.25	0.1	0.0	0.1	C	HL	1.5	0.0	1.5	T	HL
1	17	54.45	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.18	-2.96	7.74	-0.02	-2.69	3.32
1	2	6.41	33.	33.	-0.34	-5.54	14.40	-0.04	-5.03	6.22
1	3	9.61	33.	33.	-0.47	-7.74	19.99	-0.06	-7.04	8.70
1	4	12.81	33.	33.	-0.48	-9.22	15.34	-0.06	-7.92	6.68
1	5	16.01	33.	33.	-0.54	-10.62	17.53	-0.07	-9.15	7.71
1	6	19.22	33.	33.	-0.59	-11.67	19.18	-0.07	-10.07	8.49
1	7	22.42	33.	33.	-0.62	-12.37	20.18	-0.08	-10.70	9.01
1	8	25.62	33.	33.	-0.63	-12.68	20.55	-0.08	-10.99	9.26
1	9	28.83	33.	33.	-0.63	-12.67	20.54	-0.08	-10.99	9.26
1	10	32.03	33.	33.	-0.62	-12.35	20.17	-0.08	-10.68	9.00
1	11	35.23	33.	33.	-0.59	-11.67	19.18	-0.07	-10.07	8.49
1	12	38.44	33.	33.	-0.54	-10.63	17.55	-0.07	-9.17	7.72
1	13	41.64	33.	33.	-0.48	-9.25	15.36	-0.06	-7.95	6.70
1	14	44.84	33.	33.	-0.47	-7.76	20.02	-0.06	-7.06	8.72
1	15	48.04	33.	33.	-0.34	-5.54	14.41	-0.04	-5.04	6.22
1	16	51.25	33.	33.	-0.18	-2.96	7.73	-0.02	-2.68	3.32
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.23	-3.04	9.06	-0.02	-2.69	3.32
1	2	6.41	33.	33.	-0.43	-5.69	16.85	-0.04	-5.03	6.22
1	3	9.61	33.	33.	-0.60	-7.95	23.38	-0.06	-7.04	8.70
1	4	12.81	33.	33.	-0.60	-9.60	17.93	-0.06	-7.92	6.68
1	5	16.01	33.	33.	-0.68	-11.06	20.48	-0.07	-9.15	7.71
1	6	19.22	33.	33.	-0.75	-12.15	22.39	-0.07	-10.07	8.49
1	7	22.42	33.	33.	-0.78	-12.87	23.53	-0.08	-10.70	9.01
1	8	25.62	33.	33.	-0.79	-13.18	23.93	-0.08	-10.99	9.26
1	9	28.83	33.	33.	-0.79	-13.18	23.93	-0.08	-10.99	9.26
1	10	32.03	33.	33.	-0.78	-12.85	23.52	-0.08	-10.68	9.00
1	11	35.23	33.	33.	-0.75	-12.15	22.39	-0.07	-10.07	8.49
1	12	38.44	33.	33.	-0.68	-11.07	20.49	-0.07	-9.17	7.72
1	13	41.64	33.	33.	-0.60	-9.63	17.95	-0.06	-7.95	6.70
1	14	44.84	33.	33.	-0.60	-7.97	23.40	-0.06	-7.06	8.72
1	15	48.04	33.	33.	-0.43	-5.70	16.86	-0.04	-5.04	6.22
1	16	51.25	33.	33.	-0.23	-3.04	9.06	-0.02	-2.68	3.32
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.32	-3.88	12.03	-0.03	-3.40	4.29
1	2	6.41	33.	33.	-0.59	-7.25	22.36	-0.06	-6.37	8.04
1	3	9.61	33.	33.	-0.81	-10.14	31.02	-0.08	-8.91	11.26
1	4	12.81	33.	33.	-0.81	-12.31	23.79	-0.08	-10.05	8.65
1	5	16.01	33.	33.	-0.93	-14.17	27.17	-0.09	-11.60	9.98
1	6	19.22	33.	33.	-1.01	-15.57	29.70	-0.10	-12.77	10.99
1	7	22.42	33.	33.	-1.06	-16.48	31.21	-0.11	-13.56	11.66
1	8	25.62	33.	33.	-1.07	-16.89	31.74	-0.11	-13.94	11.99
1	9	28.83	33.	33.	-1.07	-16.88	31.73	-0.11	-13.93	11.98
1	10	32.03	33.	33.	-1.06	-16.46	31.20	-0.11	-13.54	11.65
1	11	35.23	33.	33.	-1.01	-15.57	29.70	-0.10	-12.77	10.99
1	12	38.44	33.	33.	-0.93	-14.19	27.18	-0.09	-11.62	9.99
1	13	41.64	33.	33.	-0.81	-12.35	23.82	-0.08	-10.09	8.67
1	14	44.84	33.	33.	-0.81	-10.16	31.05	-0.08	-8.93	11.28
1	15	48.04	33.	33.	-0.59	-7.26	22.37	-0.06	-6.37	8.05
1	16	51.25	33.	33.	-0.32	-3.88	12.02	-0.03	-3.40	4.29
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY

YIELD STRESS | TOTAL POSITIVE ,(ksi) | TOTAL NEGATIVE ,(ksi)

SP IN D FROM --------- CONCR. STEEL BEAM CONCR. STEEL BEAM

NO	NO	L SUPT (ft)	Fy (ksi) TOP	BOT	TOP TOP	TOP BOT	TOP TOP	TOP BOT	TOP TOP	TOP BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.30	-2.85	10.57	-0.02	-2.37	2.84
1	2	6.41	33.	33.	-0.56	-5.33	19.64	-0.03	-4.44	5.32
1	3	9.61	33.	33.	-0.77	-7.45	27.21	-0.04	-6.22	7.45
1	4	12.81	33.	33.	-0.78	-9.25	20.86	-0.04	-6.99	5.72
1	5	16.01	33.	33.	-0.88	-10.64	23.79	-0.05	-8.07	6.60
1	6	19.22	33.	33.	-0.96	-11.68	25.98	-0.05	-8.88	7.26
1	7	22.42	33.	33.	-1.00	-12.35	27.26	-0.06	-9.43	7.71
1	8	25.62	33.	33.	-1.01	-12.64	27.67	-0.06	-9.70	7.93
1	9	28.83	33.	33.	-1.01	-12.64	27.67	-0.06	-9.69	7.92
1	10	32.03	33.	33.	-1.00	-12.34	27.25	-0.06	-9.42	7.70
1	11	35.23	33.	33.	-0.96	-11.68	25.98	-0.05	-8.88	7.26
1	12	38.44	33.	33.	-0.88	-10.65	23.80	-0.05	-8.08	6.61
1	13	41.64	33.	33.	-0.78	-9.28	20.88	-0.04	-7.02	5.73
1	14	44.84	33.	33.	-0.77	-7.46	27.23	-0.04	-6.24	7.47
1	15	48.04	33.	33.	-0.56	-5.34	19.65	-0.03	-4.45	5.33
1	16	51.25	33.	33.	-0.30	-2.85	10.57	-0.02	-2.37	2.84
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.25	-3.77	10.26	-0.03	-3.40	4.29
1	2	6.41	33.	33.	-0.47	-7.05	19.09	-0.06	-6.37	8.04
1	3	9.61	33.	33.	-0.64	-9.85	26.50	-0.08	-8.91	11.26
1	4	12.81	33.	33.	-0.65	-11.79	20.33	-0.08	-10.05	8.65
1	5	16.01	33.	33.	-0.74	-13.58	23.24	-0.09	-11.60	9.98
1	6	19.22	33.	33.	-0.80	-14.93	25.42	-0.10	-12.77	10.99
1	7	22.42	33.	33.	-0.84	-15.82	26.74	-0.11	-13.56	11.66
1	8	25.62	33.	33.	-0.85	-16.21	27.22	-0.11	-13.94	11.99
1	9	28.83	33.	33.	-0.85	-16.21	27.22	-0.11	-13.93	11.98
1	10	32.03	33.	33.	-0.84	-15.80	26.73	-0.11	-13.54	11.65
1	11	35.23	33.	33.	-0.80	-14.93	25.42	-0.10	-12.77	10.99
1	12	38.44	33.	33.	-0.74	-13.60	23.26	-0.09	-11.62	9.99
1	13	41.64	33.	33.	-0.65	-11.83	20.36	-0.08	-10.09	8.67
1	14	44.84	33.	33.	-0.64	-9.88	26.53	-0.08	-8.93	11.28
1	15	48.04	33.	33.	-0.47	-7.06	19.10	-0.06	-6.37	8.05
1	16	51.25	33.	33.	-0.25	-3.77	10.25	-0.03	-3.40	4.29
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

1	1	3.20	33.	33.	-0.24	-2.74	8.81	-0.02	-2.37	2.84
1	2	6.41	33.	33.	-0.44	-5.13	16.37	-0.03	-4.44	5.32
1	3	9.61	33.	33.	-0.61	-7.17	22.70	-0.04	-6.22	7.45
1	4	12.81	33.	33.	-0.61	-8.73	17.40	-0.04	-6.99	5.72
1	5	16.01	33.	33.	-0.69	-10.05	19.86	-0.05	-8.07	6.60
1	6	19.22	33.	33.	-0.75	-11.04	21.70	-0.05	-8.88	7.26
1	7	22.42	33.	33.	-0.79	-11.69	22.79	-0.06	-9.43	7.71
1	8	25.62	33.	33.	-0.80	-11.97	23.16	-0.06	-9.70	7.93
1	9	28.83	33.	33.	-0.80	-11.96	23.16	-0.06	-9.69	7.92
1	10	32.03	33.	33.	-0.79	-11.67	22.78	-0.06	-9.42	7.70
1	11	35.23	33.	33.	-0.75	-11.04	21.70	-0.05	-8.88	7.26
1	12	38.44	33.	33.	-0.69	-10.06	19.87	-0.05	-8.08	6.61
1	13	41.64	33.	33.	-0.61	-8.76	17.42	-0.04	-7.02	5.73
1	14	44.84	33.	33.	-0.61	-7.18	22.72	-0.04	-6.24	7.47
1	15	48.04	33.	33.	-0.44	-5.14	16.38	-0.03	-4.45	5.33
1	16	51.25	33.	33.	-0.24	-2.74	8.80	-0.02	-2.37	2.84
1	17	54.45	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	9.73	191.	-6.4	99.	-0.7	519.	-0.7	0.	0.0	-7.80
											0.71
	LB			6.4		2.4		11.4		0.0	20.16
											11.38
	RT			-5.7		-0.7		-1.1		0.0	-7.53
											1.07
	RB			3.9		1.5		7.2		0.0	12.63
											7.19
1	LT	44.72	191.	-5.7	99.	-0.7	519.	-1.1	0.	0.0	-7.55
											1.07
	LB			4.0		1.5		7.2		0.0	12.64
											7.19
	RT			-6.4		-0.7		-0.7		0.0	-7.82
											0.71
	RB			6.4		2.4		11.4		0.0	20.18
											11.38

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	9.73	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	35.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.9375
3	9.73	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

```
 ------------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   54.5
                                                                LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])          1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])          1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------------

  1   9.73  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                          [3]=   32.90     0.05035      >   0.03300    OK

  2  35.00  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                          [3]=   32.90     0.05035      >   0.03300    OK

  3   9.73  6.75   32.90 [2]=   41.31     0.06322      >   0.04000    OK

                          [3]=   32.90     0.05035      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                         [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  1   3.20  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  2   6.41  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  3   9.61  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  4  12.81  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  5  16.01  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  6  19.22  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  7  22.42  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  8  25.62  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1  9  28.83  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 10  32.03  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
```

```
  1 11  35.23  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 12  38.44  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 13  41.64  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 14  44.84  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 15  48.04  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 16  51.25  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0
  1 17  54.45  0  1  31.42 0.550  57.1  150.0    0.00 111.46                  0


-------------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE  1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
                 ********************************

 SP IN D FROM  Lo Co       Web                    Web
 NO NO L SUPT  ng mp depth thick     2Dc/tw      Category
               [1]                  [2]   [3]
-------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.13 168.97    2
  1  1   3.20  0  1  31.42 0.550  57.13 168.97    2
  1  2   6.41  0  1  31.42 0.550  57.13 168.97    2
  1  3   9.61  0  1  31.42 0.550  57.13 168.97    2
  1  4  12.81  0  1  31.42 0.550  70.33 168.97    2
  1  5  16.01  0  1  31.42 0.550  70.33 168.97    2
  1  6  19.22  0  1  31.42 0.550  70.33 168.97    2
  1  7  22.42  0  1  31.42 0.550  70.33 168.97    2
  1  8  25.62  0  1  31.42 0.550  70.33 168.97    2
  1  9  28.83  0  1  31.42 0.550  70.33 168.97    2
  1 10  32.03  0  1  31.42 0.550  70.33 168.97    2
  1 11  35.23  0  1  31.42 0.550  70.33 168.97    2
  1 12  38.44  0  1  31.42 0.550  70.33 168.97    2
  1 13  41.64  0  1  31.42 0.550  70.33 168.97    2
  1 14  44.84  0  1  31.42 0.550  57.13 168.97    2
  1 15  48.04  0  1  31.42 0.550  57.13 168.97    2
  1 16  51.25  0  1  31.42 0.550  57.13 168.97    2
  1 17  54.45  0  1  31.42 0.550  57.13 168.97    2


-------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 1	3.20	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 2	6.41	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 3	9.61	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 4	12.81	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 5	16.01	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 6	19.22	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 7	22.42	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 8	25.62	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 9	28.83	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 10	32.03	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 11	35.23	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 12	38.44	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 13	41.64	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 14	44.84	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 15	48.04	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 16	51.25	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 17	54.45	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	3.7	0.000	4.096	2.082	0.069	1.732
			0.000	0.000	0.000	0.000	1.000
2	3.7	22.5	4.096	14.251	10.691	7.131	1.000
			0.000	0.000	0.000	0.000	1.000
3	22.5	41.4	10.729	14.251	14.226	14.201	1.002
			0.000	0.000	0.000	0.000	1.000
4	41.4	54.5	0.000	10.729	6.915	3.100	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	3.68	7.09	26.62	0	0
1	1	3.20	1	7.77	11.26	19.84	0	3.68	7.09	26.62	0	0
1	2	6.41	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	3	9.61	1	7.77	11.26	19.84	0	18.84	7.09	26.62	2	2
1	4	12.81	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	5	16.01	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	6	19.22	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	7	22.42	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	8	25.62	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	9	28.83	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	10	32.03	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	11	35.23	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	12	38.44	1	7.77	11.26	19.84	0	18.84	6.89	25.87	2	2
1	13	41.64	1	7.77	11.26	19.84	0	13.09	6.89	25.87	2	2
1	14	44.84	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	15	48.04	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	16	51.25	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2
1	17	54.45	1	7.77	11.26	19.84	0	13.09	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.73	33.00	33.00	33.00
1	1		3.20	1	1.000	1.000	1.73	33.00	33.00	33.00
1	2		6.41	1	1.000	1.000	1.00	33.00	27.04	27.04
1	3		9.61	1	1.000	1.000	1.00	33.00	27.04	27.04
1	4		12.81	1	1.000	1.000	1.00	33.00	26.77	26.77
1	5		16.01	1	1.000	1.000	1.00	33.00	26.77	26.77
1	6		19.22	1	1.000	1.000	1.00	33.00	26.77	26.77
1	7		22.42	1	1.000	1.000	1.00	33.00	26.77	26.77
1	8		25.62	1	1.000	1.000	1.00	33.00	26.81	26.81
1	9		28.83	1	1.000	1.000	1.00	33.00	26.81	26.81
1	10		32.03	1	1.000	1.000	1.00	33.00	26.81	26.81
1	11		35.23	1	1.000	1.000	1.00	33.00	26.81	26.81
1	12		38.44	1	1.000	1.000	1.00	33.00	26.81	26.81
1	13		41.64	1	1.000	1.000	1.00	33.00	29.77	29.77
1	14		44.84	1	1.000	1.000	1.00	33.00	29.96	29.96
1	15		48.04	1	1.000	1.000	1.00	33.00	29.96	29.96
1	16		51.25	1	1.000	1.000	1.00	33.00	29.96	29.96
1	17		54.45	1	1.000	1.000	1.47	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	4.70	17.35
1	1		3.20	1	4.70	17.35
1	2		6.41	1	4.70	17.35
1	3		9.61	1	4.70	17.35
1	4		12.81	1	5.98	17.74
1	5		16.01	1	5.98	17.74
1	6		19.22	1	5.98	17.74
1	7		22.42	1	5.98	17.74
1	8		25.62	1	5.98	17.74
1	9		28.83	1	5.98	17.74
1	10		32.03	1	5.98	17.74
1	11		35.23	1	5.98	17.74
1	12		38.44	1	5.98	17.74
1	13		41.64	1	5.98	17.74
1	14		44.84	1	4.70	17.35
1	15		48.04	1	4.70	17.35
1	16		51.25	1	4.70	17.35
1	17		54.45	1	4.70	17.35

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP	IN	D	FROM Section	S T R E N G T H C A T E G O R Y, Noncomposite	Composite	Category Non-	Comp.

NO	NO	L SUPT (ft)	Region	Web [1]	Flange [2]	Web [3]	Flange [4]	Comp.	
1	0	0.00	1	0	0	0		0	0
1	1	3.20	1	0	0	0		0	0
1	2	6.41	1	0	2	0		2	0
1	3	9.61	1	0	2	0		2	0
1	4	12.81	1	0	2	0		2	0
1	5	16.01	1	0	2	0		2	0
1	6	19.22	1	0	2	0		2	0
1	7	22.42	1	0	2	0		2	0
1	8	25.62	1	0	2	0		2	0
1	9	28.83	1	0	2	0		2	0
1	10	32.03	1	0	2	0		2	0
1	11	35.23	1	0	2	0		2	0
1	12	38.44	1	0	2	0		2	0
1	13	41.64	1	0	2	0		2	0
1	14	44.84	1	0	2	0		2	0
1	15	48.04	1	0	2	0		2	0
1	16	51.25	1	0	2	0		2	0
1	17	54.45	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.20	0.0	19.8	3.6	33.0	3.6	33.0	3.6	33.0	0
				0.0	-	3.6	-	3.6	33.0	-	-	0
1	2		6.41	0.0	19.8	6.8	33.0	6.8	33.0	6.8	27.0	0
				0.0	-	6.8	-	6.8	33.0	-	-	0

1	3	9.61	0.0	19.8	9.5	33.0	9.5	33.0	9.5	27.0	0
			0.0	-	9.5	-	9.5	33.0	-	-	0
1	4	12.81	0.0	19.8	10.5	33.0	10.5	33.0	10.5	26.8	0
			0.0	-	7.3	-	7.3	33.0	-	-	0
1	5	16.01	0.0	19.8	12.2	33.0	12.2	33.0	12.2	26.8	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	6	19.22	0.0	19.8	13.4	33.0	13.4	33.0	13.4	26.8	0
			0.0	-	9.2	-	9.2	33.0	-	-	0
1	7	22.42	0.0	19.8	14.2	33.0	14.2	33.0	14.2	26.8	0
			0.0	-	9.8	-	9.8	33.0	-	-	0
1	8	25.62	0.0	19.8	14.6	33.0	14.6	33.0	14.6	26.8	0
			0.0	-	10.1	-	10.1	33.0	-	-	0
1	9	28.83	0.0	19.8	14.6	33.0	14.6	33.0	14.6	26.8	0
			0.0	-	10.1	-	10.1	33.0	-	-	0
1	10	32.03	0.0	19.8	14.2	33.0	14.2	33.0	14.2	26.8	0
			0.0	-	9.8	-	9.8	33.0	-	-	0
1	11	35.23	0.0	19.8	13.4	33.0	13.4	33.0	13.4	26.8	0
			0.0	-	9.2	-	9.2	33.0	-	-	0
1	12	38.44	0.0	19.8	12.2	33.0	12.2	33.0	12.2	26.8	0
			0.0	-	8.4	-	8.4	33.0	-	-	0
1	13	41.64	0.0	19.8	10.6	33.0	10.6	33.0	10.6	29.8	0
			0.0	-	7.3	-	7.3	33.0	-	-	0
1	14	44.84	0.0	19.8	9.5	33.0	9.5	33.0	9.5	30.0	0
			0.0	-	9.5	-	9.5	33.0	-	-	0
1	15	48.04	0.0	19.8	6.8	33.0	6.8	33.0	6.8	30.0	0
			0.0	-	6.8	-	6.8	33.0	-	-	0
1	16	51.25	0.0	19.8	3.6	33.0	3.6	33.0	3.6	30.0	0
			0.0	-	3.6	-	3.6	33.0	-	-	0
1	17	54.45	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.20	0.000	0.110	0.110	0.110	0.110	2
					0.110		0.110	3
1	2	6.41	0.000	0.206	0.206	0.251	0.251	4
					0.206		0.206	3
1	3	9.61	0.000	0.288	0.288	0.351	0.351	4
					0.288		0.288	3
1	4	12.81	0.000	0.320	0.320	0.394	0.394	4
					0.220		0.220	3
1	5	16.01	0.000	0.369	0.369	0.455	0.455	4
					0.254		0.254	3
1	6	19.22	0.000	0.406	0.406	0.501	0.501	4

					0.279		0.279	3
1	7	22.42	0.000	0.431	0.431	0.532	0.532	4
					0.297		0.297	3
1	8	25.62	0.000	0.443	0.443	0.546	0.546	4
					0.305		0.305	3
1	9	28.83	0.000	0.443	0.443	0.545	0.545	4
					0.305		0.305	3
1	10	32.03	0.000	0.431	0.431	0.530	0.530	4
					0.296		0.296	3
1	11	35.23	0.000	0.406	0.406	0.500	0.500	4
					0.279		0.279	3
1	12	38.44	0.000	0.370	0.370	0.455	0.455	4
					0.254		0.254	3
1	13	41.64	0.000	0.321	0.321	0.356	0.356	4
					0.221		0.221	3
1	14	44.84	0.000	0.289	0.289	0.318	0.318	4
					0.289		0.289	3
1	15	48.04	0.000	0.206	0.206	0.227	0.227	4
					0.206		0.206	3
1	16	51.25	0.000	0.110	0.110	0.121	0.121	4
					0.110		0.110	3
1	17	54.45	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	1		3.20	0	495.5	2103.1	-	-	0.0	19.8	-	-	0--0
1	2		6.41	0	920.7	2103.1	-	-	0.0	19.8	-	-	0--0
1	3		9.61	0	1276.1	2103.1	-	-	0.0	19.8	-	-	0--0
1	4		12.81	0	1557.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	5		16.01	0	1776.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	6		19.22	0	1940.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	7		22.42	0	2036.9	2973.8	-	-	0.0	19.8	-	-	0--0
1	8		25.62	0	2068.7	2973.8	-	-	0.0	19.8	-	-	0--0
1	9		28.83	0	2068.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	10		32.03	0	2036.3	2973.8	-	-	0.0	19.8	-	-	0--0
1	11		35.23	0	1940.5	2973.8	-	-	0.0	19.8	-	-	0--0
1	12		38.44	0	1777.2	2973.8	-	-	0.0	19.8	-	-	0--0
1	13		41.64	0	1558.8	2973.8	-	-	0.0	19.8	-	-	0--0
1	14		44.84	0	1276.9	2103.1	-	-	0.0	19.8	-	-	0--0
1	15		48.04	0	921.0	2103.1	-	-	0.0	19.8	-	-	0--0
1	16		51.25	0	495.4	2103.1	-	-	0.0	19.8	-	-	0--0
1	17		54.45	0	0.0	2103.1	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in

```
              negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
        [3] = 0.6*Fyt for composite sections in positive flexure
              or tension flange for composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
            = 0.6*Fyc for comp. flange of composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
        [4] = (PHI)f * Fnt for non-compact tension flange
              of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
            = (PHI)f * Fnc for comp. flange of composite sections
              in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

        "-" is N.A.

        Under FLAG Column, 0 = OK; 1= NG
        M = Moment; C = Comp. Flange; T = Tension Flange
        D = Ductility

        For negative moment region or non-compact sections in
        positive moment region, the 1st line is for top
        flange and the 2nd line is for bottom flange

        The values of fbu, Mu and fl shall be determined based on
        factored loads, and shall be taken as positive in
        sign in all resistance equations (Art. 6.10.1.6)

        The value of fbu is the actual stress in this table,
        the users can use the maximum value within the
        unbraced length to do their own check
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   TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                   ***********************************************

   SP IN D FROM ID Mu+1/3flSxt/       fbu/        fl/ fbu+1/3fl/  MAX.
                          [1]        [2]        [3]        [4]  RAT. GOVN.
   ---------------------------------------------------------------------------

    1  0   0.00  0        0.000                                  0.000     1
    1  1   3.20  0        0.236                                  0.236     1
    1  2   6.41  0        0.438                                  0.438     1
    1  3   9.61  0        0.607                                  0.607     1
    1  4  12.81  0        0.524                                  0.524     1
    1  5  16.01  0        0.597                                  0.597     1
    1  6  19.22  0        0.653                                  0.653     1
    1  7  22.42  0        0.685                                  0.685     1
    1  8  25.62  0        0.696                                  0.696     1
    1  9  28.83  0        0.696                                  0.696     1
    1 10  32.03  0        0.685                                  0.685     1
    1 11  35.23  0        0.653                                  0.653     1
    1 12  38.44  0        0.598                                  0.598     1
    1 13  41.64  0        0.524                                  0.524     1
    1 14  44.84  0        0.607                                  0.607     1
    1 15  48.04  0        0.438                                  0.438     1
    1 16  51.25  0        0.236                                  0.236     1
    1 17  54.45  0        0.000                                  0.000     1


   ---------------------------------------------------------------------------

   NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
         [2] = (PHI)f * Fnc for comp. flange of composite
               sections in positive flexure (Eq. 6.10.7.2.1-1)
             = (PHI)f * Rh * Fyt for tension flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
         [3] = 0.6*Fyt for composite sections in positive flexure
               or tension flange for composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
             = 0.6*Fyc for comp. flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
         [4] = (PHI)f * Fnt for non-compact tension flange
               of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
             = (PHI)f * Fnc for comp. flange of composite sections
               in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)
```

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.20	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.41	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	9.61	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	12.81	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	16.01	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	19.22	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	22.42	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	25.62	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	28.83	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	32.03	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	35.23	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	38.44	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	41.64	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	44.84	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	48.04	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	51.25	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	17	54.45	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot
count on post-buckling shear resistance from tension-
field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	196.1	<	330.8	0	0
1	1	3.20	0	178.8	<	330.8	0	0
1	2	6.41	0	161.6	<	330.8	0	0
1	3	9.61	0	144.7	<	330.8	0	0
1	4	12.81	0	128.1	<	330.8	0	0
1	5	16.01	0	111.6	<	330.8	0	0
1	6	19.22	0	95.2	<	330.8	0	0
1	7	22.42	0	79.1	<	330.8	0	0
1	8	25.62	0	62.9	<	330.8	0	0
1	9	28.83	0	63.0	<	330.8	0	0
1	10	32.03	0	78.9	<	330.8	0	0
1	11	35.23	0	95.0	<	330.8	0	0
1	12	38.44	0	111.3	<	330.8	0	0
1	13	41.64	0	128.2	<	330.8	0	0
1	14	44.84	0	144.9	<	330.8	0	0
1	15	48.04	0	161.7	<	330.8	0	0
1	16	51.25	0	178.8	<	330.8	0	0
1	17	54.45	0	196.0	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

```
            Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

    Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

            Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

    Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

            Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

          * For the detailed description of shear capacity,
            please refer to AASHTO Art.6.10.9.2 and .3

          * STRENGTH CATEGORY; 0 = compact section
                               2 = braced non-compact section
                               3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************

             YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
 SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
 NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
        (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
 ---------------------------------------------------------------------------

  1  0   0.00   33.   196.05   330.76            0
  1  1   3.20   33.   178.82   330.76            0
  1  2   6.41   33.   161.59   330.76            0
  1  3   9.61   33.   144.73   330.76            0
  1  4  12.81   33.   128.05   330.76            0
  1  5  16.01   33.   111.56   330.76            0
  1  6  19.22   33.    95.22   330.76            0
  1  7  22.42   33.    79.09   330.76            0
  1  8  25.62   33.    62.86   330.76            0
  1  9  28.83   33.    63.01   330.76            0
  1 10  32.03   33.    78.88   330.76            0
  1 11  35.23   33.    95.01   330.76            0
  1 12  38.44   33.   111.35   330.76            0
  1 13  41.64   33.   128.20   330.76            0
  1 14  44.84   33.   144.88   330.76            0
  1 15  48.04   33.   161.74   330.76            0
  1 16  51.25   33.   178.79   330.76            0
  1 17  54.45   33.   196.02   330.76            0


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     TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
                     *************************

     SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                   ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
     NO NO L SUPT                                          [3]  TBC
             (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi) (ksi)
     ---------------------------------------------------------------

      1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
      1  1   3.20   3.0    -     31.4     9.1  31.4   -     -   00-
      1  2   6.41   5.7    -     31.4    16.9  31.4   -     -   00-
      1  3   9.61   7.9    -     31.4    23.4  31.4   -     -   00-
      1  4  12.81   9.6    -     31.4    17.9  31.4   -     -   00-
      1  5  16.01  11.1    -     31.4    20.5  31.4   -     -   00-
      1  6  19.22  12.1    -     31.4    22.4  31.4   -     -   00-
      1  7  22.42  12.9    -     31.4    23.5  31.4   -     -   00-
      1  8  25.62  13.2    -     31.4    23.9  31.4   -     -   00-
```

```
 1  9  28.83  13.2    -     31.4    23.9  31.4   -     -   00-
 1 10  32.03  12.8    -     31.4    23.5  31.4   -     -   00-
 1 11  35.23  12.1    -     31.4    22.4  31.4   -     -   00-
 1 12  38.44  11.1    -     31.4    20.5  31.4   -     -   00-
 1 13  41.64   9.6    -     31.4    18.0  31.4   -     -   00-
 1 14  44.84   8.0    -     31.4    23.4  31.4   -     -   00-
 1 15  48.04   5.7    -     31.4    16.9  31.4   -     -   00-
 1 16  51.25   3.0    -     31.4     9.1  31.4   -     -   00-
 1 17  54.45   0.0    -     31.4     0.0  31.4   -     -   00-


    ------------------------------------------------------------
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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                  ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.20    TR     -0.1 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  2   6.41    TR     -0.3 A B B^C^C D E E^  TR      4.1 A B B^C^C D E
  1  3   9.61    TR     -0.3 A B B^C^C D E E^  TR      5.5 A B B^C^C D
  1      9.73  SCG
  1  4  12.81    TR     -0.6 A B B^C^C D E E^  TR      4.2 A B B^C^C D E
  1  5  16.01    TR     -0.7 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1  6  19.22    TR     -0.7 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1  7  22.42    TR     -0.8 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1  8  25.62    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1  9  28.83    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1 10  32.03    TR     -0.8 A B B^C^C D E E^  TR      5.0 A B B^C^C D
  1 11  35.23    TR     -0.7 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1 12  38.44    TR     -0.7 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1 13  41.64    TR     -0.6 A B B^C^C D E E^  TR      4.2 A B B^C^C D E
  1     44.72  SCG
```

```
 1 14  44.84    TR     -0.3 A B B^C^C D E E^  TR      5.5 A B B^C^C D
 1 15  48.04    TR     -0.3 A B B^C^C D E E^  TR      4.1 A B B^C^C D E
 1 16  51.25    TR     -0.1 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1 17  54.45    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ------------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.20    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1  2   6.41    TR     -0.1 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1  3   9.61    TR     -0.2 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
  1      9.73  SCG
  1  4  12.81    TR     -0.3 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
  1  5  16.01    TR     -0.3 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1  6  19.22    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1  7  22.42    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1  8  25.62    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1  9  28.83    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1 10  32.03    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1 11  35.23    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1 12  38.44    TR     -0.3 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1 13  41.64    TR     -0.3 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
  1     44.72  SCG
  1 14  44.84    TR     -0.2 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
  1 15  48.04    TR     -0.1 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1 16  51.25    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1 17  54.45    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
```

I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-1 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	50.6	.29821E-01	1.5	TR	3.09	3	6.15
1	1	3.20	48.1	.29821E-01	1.4	TR	3.09	3	6.47
1	2	6.41	45.6	.29821E-01	1.4	TR	3.09	3	6.83
1	3	9.61	43.1	.29821E-01	1.3	TR	3.09	3	7.22
1	4	12.81	41.1	.27202E-01	1.1	TR	3.09	3	8.30
1	5	16.01	39.4	.27202E-01	1.1	TR	3.09	3	8.66
1	6	19.22	37.4	.27202E-01	1.0	TR	3.09	3	9.12
1	7	22.42	37.0	.27202E-01	1.0	TR	3.09	3	9.23
1	8	25.62	37.8	.27202E-01	1.0	TR	3.09	3	9.02
1	9	28.83	37.8	.27202E-01	1.0	TR	3.09	3	9.02
1	10	32.03	37.0	.27202E-01	1.0	TR	3.09	3	9.23
1	11	35.23	37.4	.27202E-01	1.0	TR	3.09	3	9.12
1	12	38.44	39.4	.27202E-01	1.1	TR	3.09	3	8.66
1	13	41.64	41.1	.27202E-01	1.1	TR	3.09	3	8.30
1	14	44.84	43.1	.29821E-01	1.3	TR	3.09	3	7.22
1	15	48.04	45.6	.29821E-01	1.4	TR	3.09	3	6.83
1	16	51.25	48.1	.29821E-01	1.4	TR	3.09	3	6.47
1	17	54.45	50.6	.29821E-01	1.5	TR	3.09	3	6.15

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-2 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	25.3	.29821E-01	0.8	TR	1.36	3	5.42
1	1	3.20	24.1	.29821E-01	0.7	TR	1.36	3	5.70
1	2	6.41	22.8	.29821E-01	0.7	TR	1.36	3	6.02
1	3	9.61	21.6	.29821E-01	0.6	TR	1.36	3	6.36
1	4	12.81	20.6	.27202E-01	0.6	TR	1.36	3	7.32
1	5	16.01	19.7	.27202E-01	0.5	TR	1.36	3	7.64
1	6	19.22	18.7	.27202E-01	0.5	TR	1.36	3	8.04
1	7	22.42	18.5	.27202E-01	0.5	TR	1.36	3	8.14
1	8	25.62	18.9	.27202E-01	0.5	TR	1.36	3	7.95
1	9	28.83	18.9	.27202E-01	0.5	TR	1.36	3	7.95
1	10	32.03	18.5	.27202E-01	0.5	TR	1.36	3	8.14
1	11	35.23	18.7	.27202E-01	0.5	TR	1.36	3	8.04
1	12	38.44	19.7	.27202E-01	0.5	TR	1.36	3	7.64
1	13	41.64	20.6	.27202E-01	0.6	TR	1.36	3	7.32
1	14	44.84	21.6	.29821E-01	0.6	TR	1.36	3	6.36
1	15	48.04	22.8	.29821E-01	0.7	TR	1.36	3	6.02
1	16	51.25	24.1	.29821E-01	0.7	TR	1.36	3	5.70
1	17	54.45	25.3	.29821E-01	0.8	TR	1.36	3	5.42

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)

```
          Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
          Ffat = radial fatigue shear range per unit length
               = Ffat2 = Frc/w for skews exceeding 20 degrees
          Frc = 25.0 kips for both exterior and interior girders
          w = the effective length of the deck
            = 48 inches except at end supports where w is taken
              as 24 inches

      [4] Zr = allowable range of horizontal shear, in Kips, on an individual
               connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

          * Default ALPHA value based on 7/8" diameter and input road type


      [5] No. of shear connectors per transverse section. This value is from
          input data type 12032.

      [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

          ** The connector spacing shown here is calculated based upon
             AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
             where ADTT Single Lane= 1140 & No. of Cycles=   31207500
         *** If ADTT Single Lane greater than or equal to 960, refer
             to Fatigue I Table.
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  25.62   1 1441.3  1626.6          156         >  75
      1 17  54.45   1 1441.3  1626.6          156         >  75


     ---------------------------------------------------------------------------
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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
             Qr = factored shear resistance of one shear connector
                = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
```

Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	25.624	6.000
1	25.624	54.450	6.000

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	54.5	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I D2 ksi	MAX. MOMENT POS. MAX. STRESS	MAX. MOMENT NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1		3.20	2103.1	72.2	37.3	201.5	0.0	7.21	HL-93	5.56	0
1	2		6.41	2103.1	135.2	70.0	373.1	0.0	3.64	HL-93	2.81	0
1	3		9.61	2103.1	189.2	98.0	515.1	0.0	2.49	HL-93	1.92	0
1	4		12.81	2973.8	234.3	121.4	624.8	0.0	2.98	HL-93	2.30	0
1	5		16.01	2973.8	270.6	140.0	709.0	0.0	2.55	HL-93	1.96	0
1	6		19.22	2973.8	297.9	154.0	771.9	0.0	2.29	HL-93	1.76	0
1	7		22.42	2973.8	316.4	163.4	806.3	0.0	2.16	HL-93	1.66	0
1	8		25.62	2973.8	325.1	168.0	814.5	0.0	2.12	HL-93	1.63	0
1	9		28.83	2973.8	325.0	168.0	814.5	0.0	2.12	HL-93	1.64	0
1	10		32.03	2973.8	315.9	163.4	806.3	0.0	2.16	HL-93	1.66	0
1	11		35.23	2973.8	297.9	154.0	771.9	0.0	2.29	HL-93	1.76	0
1	12		38.44	2973.8	271.1	140.0	709.0	0.0	2.55	HL-93	1.96	0
1	13		41.64	2973.8	235.3	121.4	624.8	0.0	2.97	HL-93	2.29	0
1	14		44.84	2103.1	189.8	98.0	515.1	0.0	2.48	HL-93	1.92	0
1	15		48.04	2103.1	135.4	70.0	373.1	0.0	3.64	HL-93	2.81	0
1	16		51.25	2103.1	72.1	37.3	201.5	0.0	7.21	HL-93	5.56	0
1	17		54.45	2103.1	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

Please read NOTES on the following page

1

NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.20	31.4	-2.69	3.32	-0.27	6.35	HL-93	4.88
1	2	6.41	31.4	-5.03	6.22	-0.51	3.07	HL-93	2.36
1	3	9.61	31.4	-7.04	8.70	-0.70	2.01	HL-93	1.54
1	4	12.81	31.4	-7.92	6.68	-1.29	2.85	HL-93	2.19
1	5	16.01	31.4	-9.15	7.71	-1.47	2.41	HL-93	1.85
1	6	19.22	31.4	-10.07	8.49	-1.60	2.14	HL-93	1.64
1	7	22.42	31.4	-10.70	9.01	-1.67	2.00	HL-93	1.54
1	8	25.62	31.4	-10.99	9.26	-1.68	1.96	HL-93	1.51
1	9	28.83	31.4	-10.99	9.26	-1.68	1.96	HL-93	1.51
1	10	32.03	31.4	-10.68	9.00	-1.67	2.00	HL-93	1.54
1	11	35.23	31.4	-10.07	8.49	-1.60	2.14	HL-93	1.64
1	12	38.44	31.4	-9.17	7.72	-1.47	2.41	HL-93	1.85
1	13	41.64	31.4	-7.95	6.70	-1.29	2.85	HL-93	2.19
1	14	44.84	31.4	-7.06	8.72	-0.70	2.00	HL-93	1.54
1	15	48.04	31.4	-5.04	6.22	-0.51	3.07	HL-93	2.36
1	16	51.25	31.4	-2.68	3.32	-0.27	6.35	HL-93	4.88
1	17	54.45	31.4	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37

5969 Span 2-S17 Interior Strength CP and Fatigue.RES
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	54.45	25.62	STRENGTH L.S.	2.12	1.63	HL-93
1	54.45	25.62	SERVICEBILITY	1.96	1.51	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.957
Inventory= 1.506

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	54.45	25.62	STRENGTH L.S.	3.08	755.99
1	54.45	44.84	SERVICEBILITY	2.54	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.542

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I D2	MAX. SHEAR POS.	MAX. SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0 0.00	330.8	23.9	12.4	84.9	0.0	2.47	HL-93	1.91

1	1	3.20	330.8	21.2	10.9	78.3	-2.2	2.73	HL-93	2.11
1	2	6.41	330.8	18.2	9.5	71.7	-4.9	3.04	HL-93	2.35
1	3	9.61	330.8	15.5	8.0	65.2	-8.6	3.41	HL-93	2.63
1	4	12.81	330.8	12.7	6.6	58.8	-12.4	3.85	HL-93	2.97
1	5	16.01	330.8	9.9	5.1	52.5	-16.3	4.39	HL-93	3.38
1	6	19.22	330.8	7.2	3.6	46.4	-20.4	5.06	HL-93	3.90
1	7	22.42	330.8	4.4	2.2	40.3	-24.5	5.92	HL-93	4.57
1	8	25.62	330.8	1.3	0.7	34.4	-28.9	7.07	HL-93	5.45
1	9	28.83	330.8	-1.4	-0.7	28.9	-34.4	7.06	HL-93	5.45
1	10	32.03	330.8	-4.2	-2.2	24.5	-40.3	5.93	HL-93	4.57
1	11	35.23	330.8	-7.0	-3.6	20.4	-46.4	5.06	HL-93	3.91
1	12	38.44	330.8	-9.8	-5.1	16.3	-52.5	4.39	HL-93	3.39
1	13	41.64	330.8	-12.8	-6.6	12.4	-58.8	3.85	HL-93	2.97
1	14	44.84	330.8	-15.6	-8.0	8.6	-65.2	3.41	HL-93	2.63
1	15	48.04	330.8	-18.4	-9.5	4.9	-71.7	3.04	HL-93	2.35
1	16	51.25	330.8	-21.1	-10.9	2.2	-78.3	2.73	HL-93	2.11
1	17	54.45	330.8	-23.9	-12.4	0.0	-84.9	2.47	HL-93	1.91

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	54.45	0.00	STRENGTH L.S.	2.47	1.91	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.471
Inventory= 1.906

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	54.45	0.00	STRENGTH L.S.	3.04	93.22

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.040

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

1	27.23	3	0.425	-0.28	HL-93	2.92
		4	0.433	-0.29		2.86

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span2-S18 Interior Strength	8/20/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	16	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
56.68	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130			0.00	0.0000	0.00	0.0000		
2	W	33	130			0.00	0.00001	0.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	10.84				33.	33.	33.
2	2	2	35.00				33.	33.	33.
3	1	1	10.84				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	56.68

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	8.76
2	2	Diaphragm	0.000	0.00	0.00	0.29	27.60
3	2	Diaphragm	0.000	0.00	0.00	0.29	46.44
4	1	Railing	0.087	0.00	56.68	0.00	0.00
5	1	Curb	0.076	0.00	56.68	0.00	0.00
6	0	Pavement	0.292	0.00	56.68	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	8.76	33.0	33.0	8.76
8.76	27.60	33.0	33.0	18.84
27.60	46.44	33.0	33.0	18.84
46.44	56.68	33.0	33.0	10.24

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION

AVERAGE DEAD LOAD INTENSITIES

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.727		0.1513		0.8783

SUPERIMPOSED DEAD LOADS

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	56.68
1	DC2	0.087	0.000	56.680
2	DC2	0.076	0.000	56.680

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1376.3	51.0	206.4	51.5	257.90

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.709	0.709	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.54	6710.0	33.1	16.55	405.6	405.6
1	2	7.09	6710.0	33.1	16.55	405.6	405.6
1	3	10.63	6710.0	33.1	16.55	405.6	405.6
1	4	14.17	9020.9	33.1	14.02	643.6	449.4
1	5	17.71	9020.9	33.1	14.02	643.6	449.4
1	6	21.25	9020.9	33.1	14.02	643.6	449.4
1	7	24.80	9020.9	33.1	14.02	643.6	449.4
1	8	28.34	9020.9	33.1	14.02	643.6	449.4

1	9	31.88	9020.9	33.1	14.02	643.6	449.4
1	10	35.42	9020.9	33.1	14.02	643.6	449.4
1	11	38.97	9020.9	33.1	14.02	643.6	449.4
1	12	42.51	9020.9	33.1	14.02	643.6	449.4
1	13	46.05	6710.0	33.1	16.55	405.6	405.6
1	14	49.59	6710.0	33.1	16.55	405.6	405.6
1	15	53.14	6710.0	33.1	16.55	405.6	405.6
1	16	56.68	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	13641.5	0.023742	547.2	1671.8	828.8
1	1	3.54	13641.5	0.023742	547.2	1671.8	828.8
1	2	7.09	13641.5	0.023742	547.2	1671.8	828.8
1	3	10.63	13641.5	0.023742	547.2	1671.8	828.8
1	4	14.17	19233.3	0.021414	856.3	1653.8	965.1
1	5	17.71	19233.3	0.021414	856.3	1653.8	965.1
1	6	21.25	19233.3	0.021414	856.3	1653.8	965.1
1	7	24.80	19233.3	0.021414	856.3	1653.8	965.1
1	8	28.34	19233.3	0.021414	856.3	1653.8	965.1
1	9	31.88	19233.3	0.021414	856.3	1653.8	965.1
1	10	35.42	19233.3	0.021414	856.3	1653.8	965.1
1	11	38.97	19233.3	0.021414	856.3	1653.8	965.1
1	12	42.51	19233.3	0.021414	856.3	1653.8	965.1
1	13	46.05	13641.5	0.023742	547.2	1671.8	828.8
1	14	49.59	13641.5	0.023742	547.2	1671.8	828.8
1	15	53.14	13641.5	0.023742	547.2	1671.8	828.8
1	16	56.68	13641.5	0.023742	547.2	1671.8	828.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	18479.7	0.029242	603.9	7418.9	1712.5
1	1	3.54	18479.7	0.029242	603.9	7418.9	1712.5
1	2	7.09	18479.7	0.029242	603.9	7418.9	1712.5
1	3	10.63	18479.7	0.029242	603.9	7418.9	1712.5
1	4	14.17	27373.9	0.026751	941.4	5460.0	2056.1
1	5	17.71	27373.9	0.026751	941.4	5460.0	2056.1
1	6	21.25	27373.9	0.026751	941.4	5460.0	2056.1
1	7	24.80	27373.9	0.026751	941.4	5460.0	2056.1

1	8	28.34	27373.9	0.026751	941.4	5460.0	2056.1
1	9	31.88	27373.9	0.026751	941.4	5460.0	2056.1
1	10	35.42	27373.9	0.026751	941.4	5460.0	2056.1
1	11	38.97	27373.9	0.026751	941.4	5460.0	2056.1
1	12	42.51	27373.9	0.026751	941.4	5460.0	2056.1
1	13	46.05	18479.7	0.029242	603.9	7418.9	1712.5
1	14	49.59	18479.7	0.029242	603.9	7418.9	1712.5
1	15	53.14	18479.7	0.029242	603.9	7418.9	1712.5
1	16	56.68	18479.7	0.029242	603.9	7418.9	1712.5

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	3.54	14.2	68.4	1.6	0.0	84.2	
1	2	7.09	26.6	127.7	3.1	0.0	157.4	
1	3	10.63	37.0	177.9	4.2	0.0	219.1	
1	4	14.17	45.6	219.0	4.7	0.0	269.2	
1	5	17.71	52.2	250.9	5.3	0.0	308.4	
1	6	21.25	57.0	273.7	5.8	0.0	336.5	
1	7	24.80	59.8	287.4	6.4	0.0	353.5	
1	8	28.34	60.8	291.9	6.7	0.0	359.4	
1	9	31.88	59.8	287.4	6.2	0.0	353.4	
1	10	35.42	57.0	273.7	5.7	0.0	336.4	
1	11	38.97	52.2	250.9	5.3	0.0	308.4	
1	12	42.51	45.6	219.0	4.8	0.0	269.3	
1	13	46.05	37.0	177.9	4.3	0.0	219.3	
1	14	49.59	26.6	127.7	3.0	0.0	157.3	

1 15	53.14	14.2	68.4	1.5	0.0	84.2
1 16	56.68	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.54	27.5	15.3	0.0	0.0	0.0	42.8
1	2	7.09	51.3	28.6	0.0	0.0	0.0	79.9
1	3	10.63	71.5	39.9	0.0	0.0	0.0	111.3
1	4	14.17	87.9	49.1	0.0	0.0	0.0	137.0
1	5	17.71	100.8	56.3	0.0	0.0	0.0	157.0
1	6	21.25	109.9	61.4	0.0	0.0	0.0	171.3
1	7	24.80	115.4	64.4	0.0	0.0	0.0	179.9
1	8	28.34	117.3	65.5	0.0	0.0	0.0	182.7
1	9	31.88	115.4	64.4	0.0	0.0	0.0	179.9
1	10	35.42	109.9	61.4	0.0	0.0	0.0	171.3
1	11	38.97	100.8	56.3	0.0	0.0	0.0	157.0
1	12	42.51	87.9	49.1	0.0	0.0	0.0	137.0
1	13	46.05	71.5	39.9	0.0	0.0	0.0	111.3
1	14	49.59	51.3	28.6	0.0	0.0	0.0	79.9
1	15	53.14	27.5	15.3	0.0	0.0	0.0	42.8
1	16	56.68	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.54	0.0	0.0	225.0	HL-93	0.0	HL-93
1	2	7.09	0.0	0.0	414.9	HL-93	0.0	HL-93
1	3	10.63	0.0	0.0	569.5	HL-93	0.0	HL-93
1	4	14.17	0.0	0.0	686.3	HL-93	0.0	HL-93
1	5	17.71	0.0	0.0	775.2	HL-93	0.0	HL-93
1	6	21.25	0.0	0.0	836.8	HL-93	0.0	HL-93
1	7	24.80	0.0	0.0	861.2	HL-93	0.0	HL-93
1	8	28.34	0.0	0.0	866.6	HL-93	0.0	HL-93
1	9	31.88	0.0	0.0	861.2	HL-93	0.0	HL-93
1	10	35.42	0.0	0.0	836.8	HL-93	0.0	HL-93
1	11	38.97	0.0	0.0	775.2	HL-93	0.0	HL-93
1	12	42.51	0.0	0.0	686.3	HL-93	0.0	HL-93
1	13	46.05	0.0	0.0	569.5	HL-93	0.0	HL-93
1	14	49.59	0.0	0.0	414.9	HL-93	0.0	HL-93
1	15	53.14	0.0	0.0	225.0	HL-93	0.0	HL-93
1	16	56.68	0.0	0.0	0.0	HL-93	0.0	HL-93

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 TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N =  9.0 (k-ft) (UNFACTORED)
                 ************************************************

 SP IN D FROM     TRUCK   ONLY
 NO NO L SUPT -------------------
        (ft)    POS    NEG  RANGE
 --------------------------------
  1  0   0.00    0.     0.     0.
  1  1   3.54   78.     0.    78.
  1  2   7.09  138.     0.   138.
  1  3  10.63  185.     0.   185.
  1  4  14.17  222.     0.   222.
  1  5  17.71  246.     0.   246.
  1  6  21.25  259.     0.   259.
  1  7  24.80  256.     0.   256.
  1  8  28.34  253.     0.   253.
  1  9  31.88  256.     0.   256.
  1 10  35.42  259.     0.   259.
  1 11  38.97  246.     0.   246.
  1 12  42.51  222.     0.   222.
  1 13  46.05  185.     0.   185.
  1 14  49.59  138.     0.   138.
  1 15  53.14   78.     0.    78.
  1 16  56.68    0.     0.     0.


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 TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
                ****************************************


                   DEAD  LOAD   LL+I:N= 9.0 LOAD TYPE= HL - 93   TOTAL MAXIMUM
               ---------------- ------------------------------ -----------------
 SP IN D FROM NON COMP. COMP.  MAXIMUM  GOVERN MAXIMUM GOVERN POSITIVE NEGATIVE
 NO NO L SUPT  N=Infin. N=27.0 POSITIVE  LOAD  NEGATIVE LOAD  -------- --------
        (ft)   (k-ft)   (k-ft)  (k-ft)   TYPE   (k-ft)  TYPE   (k-ft)   (k-ft)
 ------------------------------------------------------------------------------

  1  0   0.00     0.0     0.0     0.0  HL-93      0.0  HL-93      0.0      0.0
  1  1   3.54    84.2    42.8   225.0  HL-93      0.0  HL-93    352.1    127.1
  1  2   7.09   157.4    79.9   414.9  HL-93      0.0  HL-93    652.3    237.4
  1  3  10.63   219.1   111.3   569.5  HL-93      0.0  HL-93    899.9    330.4
  1  4  14.17   269.2   137.0   686.3  HL-93      0.0  HL-93   1092.6    406.3
  1  5  17.71   308.4   157.0   775.2  HL-93      0.0  HL-93   1240.6    465.4
  1  6  21.25   336.5   171.3   836.8  HL-93      0.0  HL-93   1344.6    507.8
  1  7  24.80   353.5   179.9   861.2  HL-93      0.0  HL-93   1394.6    533.4
  1  8  28.34   359.4   182.7   866.6  HL-93      0.0  HL-93   1408.7    542.1
  1  9  31.88   353.4   179.9   861.2  HL-93      0.0  HL-93   1394.4    533.3
  1 10  35.42   336.4   171.3   836.8  HL-93      0.0  HL-93   1344.5    507.7
  1 11  38.97   308.4   157.0   775.2  HL-93      0.0  HL-93   1240.6    465.4
  1 12  42.51   269.3   137.0   686.3  HL-93      0.0  HL-93   1092.7    406.4
  1 13  46.05   219.3   111.3   569.5  HL-93      0.0  HL-93    900.1    330.6
  1 14  49.59   157.3    79.9   414.9  HL-93      0.0  HL-93    652.1    237.2
  1 15  53.14    84.2    42.8   225.0  HL-93      0.0  HL-93    352.0    127.0
  1 16  56.68     0.0     0.0     0.0  HL-93      0.0  HL-93      0.0      0.0


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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.54	352.1	419.6	559.5	469.5	190.6	58.3
1	2	7.09	652.3	776.7	1035.6	869.6	356.1	103.8
1	3	10.63	899.9	1070.8	1427.5	1199.7	495.7	138.8
1	4	14.17	1092.6	1298.5	1730.9	1456.4	609.4	166.3
1	5	17.71	1240.6	1473.1	1963.5	1653.4	698.1	184.8
1	6	21.25	1344.6	1595.6	2126.6	1791.9	761.6	194.3
1	7	24.80	1394.6	1652.9	2202.7	1858.2	800.1	191.9
1	8	28.34	1408.7	1668.7	2223.5	1876.8	813.2	189.4
1	9	31.88	1394.4	1652.8	2202.5	1858.0	799.9	191.9
1	10	35.42	1344.5	1595.5	2126.5	1791.8	761.6	194.3
1	11	38.97	1240.6	1473.1	1963.5	1653.4	698.1	184.8
1	12	42.51	1092.7	1298.6	1731.0	1456.5	609.6	166.3
1	13	46.05	900.1	1071.0	1427.8	1200.0	495.9	138.8
1	14	49.59	652.1	776.6	1035.4	869.4	355.8	103.8
1	15	53.14	352.0	419.5	559.4	469.4	190.5	58.3
1	16	56.68	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.3	20.6	0.4	0.0	25.3
1	1	3.54	3.8	18.0	0.4	0.0	22.2
1	2	7.09	3.2	15.5	0.4	0.0	19.1
1	3	10.63	2.7	12.9	0.2	0.0	15.7
1	4	14.17	2.1	10.3	0.2	0.0	12.6
1	5	17.71	1.6	7.7	0.2	0.0	9.5
1	6	21.25	1.1	5.2	0.2	0.0	6.4
1	7	24.80	0.5	2.6	0.2	0.0	3.3
1	8	28.34	0.0	0.0	-0.1	0.0	-0.1
1	9	31.88	-0.5	-2.6	-0.1	0.0	-3.2
1	10	35.42	-1.1	-5.2	-0.1	0.0	-6.4
1	11	38.97	-1.6	-7.7	-0.1	0.0	-9.5
1	12	42.51	-2.1	-10.3	-0.1	0.0	-12.6
1	13	46.05	-2.7	-12.9	-0.1	0.0	-15.7
1	14	49.59	-3.2	-15.5	-0.4	0.0	-19.1
1	15	53.14	-3.8	-18.0	-0.4	0.0	-22.2
1	16	56.68	-4.3	-20.6	-0.4	0.0	-25.3

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.3	4.6	0.0	0.0	12.9	25.3	38.2
1	1	3.54	7.2	4.0	0.0	0.0	11.3	22.2	33.5
1	2	7.09	6.2	3.5	0.0	0.0	9.7	19.1	28.8
1	3	10.63	5.2	2.9	0.0	0.0	8.1	15.7	23.8
1	4	14.17	4.1	2.3	0.0	0.0	6.4	12.6	19.0
1	5	17.71	3.1	1.7	0.0	0.0	4.8	9.5	14.3
1	6	21.25	2.1	1.2	0.0	0.0	3.2	6.4	9.6
1	7	24.80	1.0	0.6	0.0	0.0	1.6	3.3	4.9
1	8	28.34	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1
1	9	31.88	-1.0	-0.6	0.0	0.0	-1.6	-3.2	-4.9
1	10	35.42	-2.1	-1.2	0.0	0.0	-3.2	-6.4	-9.6
1	11	38.97	-3.1	-1.7	0.0	0.0	-4.8	-9.5	-14.3
1	12	42.51	-4.1	-2.3	0.0	0.0	-6.4	-12.6	-19.0
1	13	46.05	-5.2	-2.9	0.0	0.0	-8.1	-15.7	-23.7
1	14	49.59	-6.2	-3.5	0.0	0.0	-9.7	-19.1	-28.8
1	15	53.14	-7.2	-4.0	0.0	0.0	-11.3	-22.2	-33.5
1	16	56.68	-8.3	-4.6	0.0	0.0	-12.9	-25.3	-38.2

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	86.1	HL-93	0.0	HL-93
1	1	3.54	0.0	0.0	79.0	HL-93	-2.4	HL-93
1	2	7.09	0.0	0.0	71.9	HL-93	-5.5	HL-93
1	3	10.63	0.0	0.0	65.0	HL-93	-9.4	HL-93
1	4	14.17	0.0	0.0	58.2	HL-93	-13.5	HL-93
1	5	17.71	0.0	0.0	51.5	HL-93	-17.7	HL-93
1	6	21.25	0.0	0.0	44.9	HL-93	-22.1	HL-93
1	7	24.80	0.0	0.0	38.4	HL-93	-26.5	HL-93
1	8	28.34	0.0	0.0	32.2	HL-93	-32.2	HL-93
1	9	31.88	0.0	0.0	26.5	HL-93	-38.4	HL-93
1	10	35.42	0.0	0.0	22.1	HL-93	-44.9	HL-93
1	11	38.97	0.0	0.0	17.7	HL-93	-51.5	HL-93
1	12	42.51	0.0	0.0	13.5	HL-93	-58.2	HL-93
1	13	46.05	0.0	0.0	9.4	HL-93	-65.0	HL-93
1	14	49.59	0.0	0.0	5.5	HL-93	-71.9	HL-93
1	15	53.14	0.0	0.0	2.4	HL-93	-79.0	HL-93
1	16	56.68	0.0	0.0	0.0	HL-93	-86.1	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP IN D FROM TRUCK ONLY
NO NO L SUPT ----------

		(ft)	POS	NEG	RANGE
1	0	0.00	34.	0.	34.
1	1	3.54	31.	-1.	33.
1	2	7.09	28.	-3.	31.
1	3	10.63	25.	-4.	29.
1	4	14.17	22.	-6.	28.
1	5	17.71	19.	-7.	26.
1	6	21.25	16.	-9.	25.
1	7	24.80	14.	-11.	25.
1	8	28.34	13.	-13.	25.
1	9	31.88	11.	-14.	25.
1	10	35.42	9.	-16.	25.
1	11	38.97	7.	-19.	26.
1	12	42.51	6.	-22.	28.
1	13	46.05	4.	-25.	29.
1	14	49.59	3.	-28.	31.
1	15	53.14	1.	-31.	33.
1	16	56.68	0.	-34.	34.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	34.4	.30675E-01	0.53	0.3125*	.14721E-01	0.25	0.3125*
1	1	3.54	32.7	.30675E-01	0.50	0.3125*	.14721E-01	0.24	0.3125*
1	2	7.09	30.8	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	3	10.63	29.1	.30675E-01	0.45	0.3125*	.14721E-01	0.21	0.3125*
1	4	14.17	27.7	.28592E-01	0.40	0.3125*	.90646E-02	0.13	0.3125*
1	5	17.71	26.4	.28592E-01	0.38	0.3125*	.90646E-02	0.12	0.3125*
1	6	21.25	25.2	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	7	24.80	25.2	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	8	28.34	25.4	.28592E-01	0.36	0.3125*	.90646E-02	0.12	0.3125*
1	9	31.88	25.2	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	10	35.42	25.2	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	11	38.97	26.4	.28592E-01	0.38	0.3125*	.90646E-02	0.12	0.3125*
1	12	42.51	27.7	.28592E-01	0.40	0.3125*	.90646E-02	0.13	0.3125*
1	13	46.05	29.1	.30675E-01	0.45	0.3125*	.14721E-01	0.21	0.3125*
1	14	49.59	30.8	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	15	53.14	32.7	.30675E-01	0.50	0.3125*	.14721E-01	0.24	0.3125*
1	16	56.68	34.4	.30675E-01	0.53	0.3125*	.14721E-01	0.25	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	38.2	86.1	HL-93	0.0	HL-93	124.4	38.2
1	1	3.54	33.5	79.0	HL-93	-2.4	HL-93	112.5	31.1
1	2	7.09	28.8	71.9	HL-93	-5.5	HL-93	100.7	23.3
1	3	10.63	23.8	65.0	HL-93	-9.4	HL-93	88.7	14.3
1	4	14.17	19.0	58.2	HL-93	-13.5	HL-93	77.2	5.5
1	5	17.71	14.3	51.5	HL-93	-17.7	HL-93	65.8	-3.4
1	6	21.25	9.6	44.9	HL-93	-22.1	HL-93	54.5	-12.5
1	7	24.80	4.9	38.4	HL-93	-26.5	HL-93	43.2	-21.6
1	8	28.34	-0.1	32.2	HL-93	-32.2	HL-93	32.0	-32.3
1	9	31.88	-4.9	26.5	HL-93	-38.4	HL-93	21.7	-43.2
1	10	35.42	-9.6	22.1	HL-93	-44.9	HL-93	12.5	-54.5
1	11	38.97	-14.3	17.7	HL-93	-51.5	HL-93	3.4	-65.8
1	12	42.51	-19.0	13.5	HL-93	-58.2	HL-93	-5.5	-77.2
1	13	46.05	-23.7	9.4	HL-93	-65.0	HL-93	-14.3	-88.7
1	14	49.59	-28.8	5.5	HL-93	-71.9	HL-93	-23.3	-100.7
1	15	53.14	-33.5	2.4	HL-93	-79.0	HL-93	-31.1	-112.4
1	16	56.68	-38.2	0.0	HL-93	-86.1	HL-93	-38.2	-124.3

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	124.4	150.2	200.6	166.1	57.3	25.8
1	1	3.54	112.5	136.2	181.9	150.3	50.3	24.5
1	2	7.09	100.7	122.3	163.4	134.6	43.2	23.1
1	3	10.63	88.7	108.2	144.7	118.7	35.7	21.8
1	4	14.17	77.2	94.6	126.6	103.4	28.6	20.8
1	5	17.71	65.8	81.2	108.8	88.2	21.5	19.8
1	6	21.25	54.5	68.0	91.1	73.1	14.4	18.9
1	7	24.80	43.2	54.8	73.5	58.2	7.3	18.9
1	8	28.34	-32.3	-41.9	-56.4	-43.6	-0.2	19.0
1	9	31.88	-43.2	-54.7	-73.5	-58.1	-7.3	18.9
1	10	35.42	-54.5	-68.0	-91.1	-73.1	-14.4	18.9
1	11	38.97	-65.8	-81.2	-108.7	-88.1	-21.5	19.8
1	12	42.51	-77.2	-94.6	-126.6	-103.3	-28.5	20.8
1	13	46.05	-88.7	-108.2	-144.7	-118.7	-35.6	21.8
1	14	49.59	-100.7	-122.2	-163.3	-134.6	-43.1	23.1
1	15	53.14	-112.4	-136.1	-181.8	-150.3	-50.2	24.5
1	16	56.68	-124.3	-150.2	-200.6	-166.1	-57.3	25.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	38.23	0.00	HL-93	86.14	HL-93	ST1	32.34	200.60
						ST2	32.34	166.14
						ST4	44.93	57.34
						SE1	38.23	124.37
						SE2	38.23	150.21
2	38.21	0.00	HL-93	86.14	HL-93	ST1	32.32	200.57
						ST2	32.32	166.12
						ST4	44.89	57.31
						SE1	38.21	124.35
						SE2	38.21	150.19

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D FROM NO L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1 0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 1	3.54	-0.0284	-0.1363	0.0000	-0.0405	-0.1677	-0.0405	-0.2081
1 2	7.09	-0.0552	-0.2650	0.0000	-0.0786	-0.3260	-0.0786	-0.4046
1 3	10.63	-0.0790	-0.3796	0.0000	-0.1124	-0.4670	-0.1124	-0.5793
1 4	14.17	-0.0992	-0.4768	0.0000	-0.1409	-0.5866	-0.1409	-0.7275
1 5	17.71	-0.1157	-0.5560	0.0000	-0.1642	-0.6840	-0.1642	-0.8481
1 6	21.25	-0.1279	-0.6145	0.0000	-0.1813	-0.7559	-0.1813	-0.9372
1 7	24.80	-0.1353	-0.6503	0.0000	-0.1918	-0.7999	-0.1918	-0.9918
1 8	28.34	-0.1379	-0.6624	0.0000	-0.1954	-0.8148	-0.1954	-1.0102
1 9	31.88	-0.1353	-0.6503	0.0000	-0.1918	-0.7999	-0.1918	-0.9918
1 10	35.42	-0.1279	-0.6145	0.0000	-0.1813	-0.7559	-0.1813	-0.9372
1 11	38.97	-0.1157	-0.5560	0.0000	-0.1642	-0.6840	-0.1642	-0.8481
1 12	42.51	-0.0992	-0.4768	0.0000	-0.1409	-0.5866	-0.1409	-0.7275
1 13	46.05	-0.0790	-0.3796	0.0000	-0.1124	-0.4670	-0.1124	-0.5793
1 14	49.59	-0.0552	-0.2650	0.0000	-0.0786	-0.3260	-0.0786	-0.4046
1 15	53.14	-0.0284	-0.1363	0.0000	-0.0405	-0.1676	-0.0405	-0.2081
1 16	56.68	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS STEEL DEFL. (in)	CAMBER SIZE	SLAB DEFL. (in)	CAMBER SIZE	COMPOSITE DL DEFL. (in)	CAMBER SIZE	TOTAL DEFL. (in)	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.5	-0.028	0 1/16	-0.139	0 3/16	-0.040	0 1/16	-0.208	0 1/ 4
1	2	7.1	-0.055	0 1/16	-0.271	0 5/16	-0.079	0 1/ 8	-0.405	0 7/16
1	3	10.6	-0.079	0 1/ 8	-0.388	0 7/16	-0.112	0 1/ 8	-0.579	0 5/ 8
1	4	14.2	-0.099	0 1/ 8	-0.487	0 1/ 2	-0.141	0 3/16	-0.728	0 3/ 4
1	5	17.7	-0.116	0 1/ 8	-0.568	0 5/ 8	-0.164	0 3/16	-0.848	0 7/ 8
1	6	21.3	-0.128	0 3/16	-0.628	0 11/16	-0.181	0 3/16	-0.937	0 15/16
1	7	24.8	-0.135	0 3/16	-0.665	0 11/16	-0.192	0 1/ 4	-0.992	1 0/16
1	8	28.3	-0.138	0 3/16	-0.677	0 11/16	-0.195	0 1/ 4	-1.010	1 1/16
1	9	31.9	-0.135	0 3/16	-0.665	0 11/16	-0.192	0 1/ 4	-0.992	1 0/16
1	10	35.4	-0.128	0 3/16	-0.628	0 11/16	-0.181	0 3/16	-0.937	0 15/16
1	11	39.0	-0.116	0 1/ 8	-0.568	0 5/ 8	-0.164	0 3/16	-0.848	0 7/ 8
1	12	42.5	-0.099	0 1/ 8	-0.487	0 1/ 2	-0.141	0 3/16	-0.728	0 3/ 4
1	13	46.1	-0.079	0 1/ 8	-0.388	0 7/16	-0.112	0 1/ 8	-0.579	0 5/ 8
1	14	49.6	-0.055	0 1/16	-0.271	0 5/16	-0.079	0 1/ 8	-0.405	0 7/16
1	15	53.1	-0.028	0 1/16	-0.139	0 3/16	-0.040	0 1/16	-0.208	0 1/ 4
1	16	56.7	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	28.34	3 0.425	-0.298	MAX HL-93	0.85	0.00258
			0.026	MIN		
			-0.083	MAX LANE		
			0.026	MIN LANE		
		4 0.433	-0.304	MAX HL-93	0.85	0.00258
			0.027	MIN		
			-0.084	MAX LANE		
			0.027	MIN LANE		
		3 0.425	-0.206	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	-0.42	0.42	-2.07	2.07	-2.49	2.49
1	2	7.09	-0.79	0.79	-3.87	3.87	-4.66	4.66
1	3	10.63	-1.10	1.10	-5.39	5.39	-6.48	6.48
1	4	14.17	-1.22	0.85	-5.97	4.17	-7.19	5.02
1	5	17.71	-1.39	0.97	-6.84	4.78	-8.23	5.75
1	6	21.25	-1.52	1.06	-7.46	5.21	-8.99	6.27
1	7	24.80	-1.60	1.12	-7.84	5.48	-9.44	6.59
1	8	28.34	-1.62	1.13	-7.97	5.57	-9.60	6.70
1	9	31.88	-1.60	1.12	-7.84	5.47	-9.44	6.59
1	10	35.42	-1.52	1.06	-7.46	5.21	-8.98	6.27
1	11	38.97	-1.39	0.97	-6.84	4.78	-8.24	5.75
1	12	42.51	-1.22	0.85	-5.98	4.17	-7.19	5.02
1	13	46.05	-1.10	1.10	-5.39	5.39	-6.49	6.49
1	14	49.59	-0.79	0.79	-3.87	3.87	-4.65	4.65
1	15	53.14	-0.42	0.42	-2.07	2.07	-2.49	2.49
1	16	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN D NO	FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	-0.02	-0.31	0.94	-0.02	-2.80	3.43
1	2	7.09	-0.04	-0.57	1.75	-0.04	-5.23	6.41
1	3	10.63	-0.06	-0.80	2.44	-0.06	-7.28	8.92
1	4	14.17	-0.06	-0.99	1.92	-0.06	-8.18	6.94
1	5	17.71	-0.07	-1.14	2.20	-0.07	-9.37	7.95
1	6	21.25	-0.08	-1.24	2.40	-0.08	-10.23	8.67
1	7	24.80	-0.08	-1.31	2.52	-0.08	-10.75	9.11
1	8	28.34	-0.08	-1.33	2.56	-0.08	-10.92	9.26

1	9	31.88	-0.08	-1.31	2.52	-0.08	-10.74	9.11
1	10	35.42	-0.08	-1.24	2.40	-0.08	-10.23	8.67
1	11	38.97	-0.07	-1.14	2.20	-0.07	-9.37	7.95
1	12	42.51	-0.06	-0.99	1.92	-0.06	-8.19	6.94
1	13	46.05	-0.06	-0.80	2.44	-0.06	-7.29	8.93
1	14	49.59	-0.04	-0.57	1.75	-0.04	-5.23	6.41
1	15	53.14	-0.02	-0.31	0.94	-0.02	-2.80	3.43
1	16	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	CONCR. TOP	MAXIMUM POSITIVE,(MPa) STEEL TOP	STEEL BOT	CONCR. TOP	MAXIMUM NEGATIVE,(ksi) STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	-0.18	-0.36	4.47	0.00	0.00	0.00
1	2	7.09	-0.32	-0.67	8.24	0.00	0.00	0.00
1	3	10.63	-0.44	-0.92	11.32	0.00	0.00	0.00
1	4	14.17	-0.45	-1.51	8.75	0.00	0.00	0.00
1	5	17.71	-0.50	-1.70	9.88	0.00	0.00	0.00
1	6	21.25	-0.54	-1.84	10.67	0.00	0.00	0.00
1	7	24.80	-0.56	-1.89	10.98	0.00	0.00	0.00
1	8	28.34	-0.56	-1.90	11.05	0.00	0.00	0.00
1	9	31.88	-0.56	-1.89	10.98	0.00	0.00	0.00
1	10	35.42	-0.54	-1.84	10.67	0.00	0.00	0.00
1	11	38.97	-0.50	-1.70	9.88	0.00	0.00	0.00
1	12	42.51	-0.45	-1.51	8.75	0.00	0.00	0.00
1	13	46.05	-0.44	-0.92	11.32	0.00	0.00	0.00
1	14	49.59	-0.32	-0.67	8.24	0.00	0.00	0.00
1	15	53.14	-0.18	-0.36	4.47	0.00	0.00	0.00
1	16	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	3.54	0.1	0.0	0.1	C HL	1.5	0.0	1.5	T HL
1	2	7.09	0.2	0.0	0.2	C HL	2.7	0.0	2.7	T HL
1	3	10.63	0.3	0.0	0.3	C HL	3.7	0.0	3.7	T HL
1	4	14.17	0.5	0.0	0.5	C HL	2.8	0.0	2.8	T HL
1	5	17.71	0.5	0.0	0.5	C HL	3.1	0.0	3.1	T HL
1	6	21.25	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	7	24.80	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	8	28.34	0.6	0.0	0.6	C HL	3.2	0.0	3.2	T HL
1	9	31.88	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	10	35.42	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	11	38.97	0.5	0.0	0.5	C HL	3.1	0.0	3.1	T HL
1	12	42.51	0.5	0.0	0.5	C HL	2.8	0.0	2.8	T HL
1	13	46.05	0.3	0.0	0.3	C HL	3.7	0.0	3.7	T HL

1 14	49.59	0.2	0.0	0.2	C	HL	2.7	0.0	2.7	T	HL
1 15	53.14	0.1	0.0	0.1	C	HL	1.5	0.0	1.5	T	HL
1 16	56.68	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.20	-3.16	7.90	-0.02	-2.80	3.43
1	2	7.09	33.	33.	-0.37	-5.90	14.65	-0.04	-5.23	6.41
1	3	10.63	33.	33.	-0.50	-8.20	20.24	-0.06	-7.28	8.92
1	4	14.17	33.	33.	-0.51	-9.69	15.69	-0.06	-8.18	6.94
1	5	17.71	33.	33.	-0.58	-11.08	17.83	-0.07	-9.37	7.95
1	6	21.25	33.	33.	-0.62	-12.07	19.34	-0.08	-10.23	8.67
1	7	24.80	33.	33.	-0.64	-12.64	20.09	-0.08	-10.75	9.11
1	8	28.34	33.	33.	-0.65	-12.83	20.31	-0.08	-10.92	9.26
1	9	31.88	33.	33.	-0.64	-12.64	20.09	-0.08	-10.74	9.11
1	10	35.42	33.	33.	-0.62	-12.07	19.34	-0.08	-10.23	8.67
1	11	38.97	33.	33.	-0.58	-11.08	17.83	-0.07	-9.37	7.95
1	12	42.51	33.	33.	-0.51	-9.70	15.69	-0.06	-8.19	6.94
1	13	46.05	33.	33.	-0.50	-8.21	20.25	-0.06	-7.29	8.93
1	14	49.59	33.	33.	-0.37	-5.90	14.65	-0.04	-5.23	6.41
1	15	53.14	33.	33.	-0.20	-3.16	7.90	-0.02	-2.80	3.43
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.25	-3.27	9.24	-0.02	-2.80	3.43
1	2	7.09	33.	33.	-0.46	-6.10	17.13	-0.04	-5.23	6.41
1	3	10.63	33.	33.	-0.64	-8.48	23.64	-0.06	-7.28	8.92
1	4	14.17	33.	33.	-0.64	-10.15	18.31	-0.06	-8.18	6.94
1	5	17.71	33.	33.	-0.73	-11.59	20.79	-0.07	-9.37	7.95
1	6	21.25	33.	33.	-0.78	-12.62	22.54	-0.08	-10.23	8.67
1	7	24.80	33.	33.	-0.81	-13.21	23.38	-0.08	-10.75	9.11
1	8	28.34	33.	33.	-0.81	-13.40	23.62	-0.08	-10.92	9.26
1	9	31.88	33.	33.	-0.81	-13.20	23.38	-0.08	-10.74	9.11
1	10	35.42	33.	33.	-0.78	-12.62	22.54	-0.08	-10.23	8.67
1	11	38.97	33.	33.	-0.73	-11.59	20.79	-0.07	-9.37	7.95
1	12	42.51	33.	33.	-0.64	-10.15	18.31	-0.06	-8.19	6.94
1	13	46.05	33.	33.	-0.64	-8.48	23.64	-0.06	-7.29	8.93

1	14	49.59	33.	33.	-0.46	-6.10	17.12	-0.04	-5.23	6.41
1	15	53.14	33.	33.	-0.25	-3.27	9.24	-0.02	-2.80	3.43
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.34	-4.19	12.27	-0.03	-3.55	4.44
1	2	7.09	33.	33.	-0.63	-7.81	22.72	-0.06	-6.63	8.30
1	3	10.63	33.	33.	-0.86	-10.84	31.35	-0.08	-9.23	11.55
1	4	14.17	33.	33.	-0.87	-13.03	24.29	-0.09	-10.39	8.98
1	5	17.71	33.	33.	-0.98	-14.88	27.58	-0.10	-11.90	10.29
1	6	21.25	33.	33.	-1.06	-16.20	29.89	-0.11	-12.98	11.23
1	7	24.80	33.	33.	-1.09	-16.95	31.00	-0.12	-13.64	11.79
1	8	28.34	33.	33.	-1.10	-17.20	31.32	-0.12	-13.87	11.99
1	9	31.88	33.	33.	-1.09	-16.95	31.00	-0.12	-13.64	11.79
1	10	35.42	33.	33.	-1.06	-16.20	29.89	-0.11	-12.98	11.23
1	11	38.97	33.	33.	-0.98	-14.88	27.58	-0.10	-11.90	10.29
1	12	42.51	33.	33.	-0.87	-13.03	24.29	-0.09	-10.39	8.99
1	13	46.05	33.	33.	-0.86	-10.85	31.36	-0.08	-9.24	11.55
1	14	49.59	33.	33.	-0.63	-7.80	22.72	-0.06	-6.63	8.29
1	15	53.14	33.	33.	-0.34	-4.18	12.26	-0.03	-3.55	4.44
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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ANALYSIS

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.32	-3.11	10.76	-0.02	-2.47	2.94
1	2	7.09	33.	33.	-0.60	-5.79	19.92	-0.03	-4.62	5.49
1	3	10.63	33.	33.	-0.82	-8.04	27.44	-0.04	-6.43	7.64
1	4	14.17	33.	33.	-0.83	-9.85	21.25	-0.05	-7.21	5.94
1	5	17.71	33.	33.	-0.93	-11.24	24.09	-0.05	-8.25	6.80
1	6	21.25	33.	33.	-1.01	-12.22	26.09	-0.06	-9.01	7.42
1	7	24.80	33.	33.	-1.04	-12.77	27.01	-0.06	-9.46	7.80
1	8	28.34	33.	33.	-1.05	-12.95	27.25	-0.06	-9.62	7.92
1	9	31.88	33.	33.	-1.04	-12.77	27.00	-0.06	-9.46	7.79
1	10	35.42	33.	33.	-1.01	-12.22	26.09	-0.06	-9.00	7.42
1	11	38.97	33.	33.	-0.93	-11.24	24.09	-0.05	-8.25	6.80
1	12	42.51	33.	33.	-0.83	-9.85	21.25	-0.05	-7.21	5.94
1	13	46.05	33.	33.	-0.82	-8.04	27.45	-0.04	-6.43	7.65
1	14	49.59	33.	33.	-0.60	-5.79	19.91	-0.03	-4.61	5.48
1	15	53.14	33.	33.	-0.32	-3.11	10.76	-0.02	-2.47	2.94
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.27	-4.04	10.48	-0.03	-3.55	4.44
1	2	7.09	33.	33.	-0.50	-7.54	19.42	-0.06	-6.63	8.30
1	3	10.63	33.	33.	-0.68	-10.47	26.82	-0.08	-9.23	11.55
1	4	14.17	33.	33.	-0.69	-12.43	20.79	-0.09	-10.39	8.98
1	5	17.71	33.	33.	-0.78	-14.20	23.63	-0.10	-11.90	10.29
1	6	21.25	33.	33.	-0.84	-15.47	25.63	-0.11	-12.98	11.23
1	7	24.80	33.	33.	-0.87	-16.20	26.61	-0.12	-13.64	11.79
1	8	28.34	33.	33.	-0.88	-16.44	26.90	-0.12	-13.87	11.99
1	9	31.88	33.	33.	-0.87	-16.19	26.61	-0.12	-13.64	11.79
1	10	35.42	33.	33.	-0.84	-15.47	25.62	-0.11	-12.98	11.23
1	11	38.97	33.	33.	-0.78	-14.20	23.63	-0.10	-11.90	10.29
1	12	42.51	33.	33.	-0.69	-12.43	20.80	-0.09	-10.39	8.99
1	13	46.05	33.	33.	-0.68	-10.48	26.83	-0.08	-9.24	11.55
1	14	49.59	33.	33.	-0.50	-7.53	19.42	-0.06	-6.63	8.29
1	15	53.14	33.	33.	-0.27	-4.04	10.47	-0.03	-3.55	4.44
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.54	33.	33.	-0.25	-2.96	8.97	-0.02	-2.47	2.94
1	2	7.09	33.	33.	-0.47	-5.52	16.62	-0.03	-4.62	5.49
1	3	10.63	33.	33.	-0.64	-7.67	22.92	-0.04	-6.43	7.64
1	4	14.17	33.	33.	-0.65	-9.24	17.75	-0.05	-7.21	5.94
1	5	17.71	33.	33.	-0.73	-10.55	20.14	-0.05	-8.25	6.80
1	6	21.25	33.	33.	-0.79	-11.49	21.82	-0.06	-9.01	7.42
1	7	24.80	33.	33.	-0.82	-12.02	22.61	-0.06	-9.46	7.80
1	8	28.34	33.	33.	-0.82	-12.19	22.84	-0.06	-9.62	7.92
1	9	31.88	33.	33.	-0.82	-12.01	22.61	-0.06	-9.46	7.79
1	10	35.42	33.	33.	-0.79	-11.49	21.82	-0.06	-9.00	7.42
1	11	38.97	33.	33.	-0.73	-10.55	20.14	-0.05	-8.25	6.80
1	12	42.51	33.	33.	-0.65	-9.25	17.75	-0.05	-7.21	5.94
1	13	46.05	33.	33.	-0.64	-7.67	22.92	-0.04	-6.43	7.65
1	14	49.59	33.	33.	-0.47	-5.52	16.61	-0.03	-4.61	5.48
1	15	53.14	33.	33.	-0.25	-2.96	8.97	-0.02	-2.47	2.94
1	16	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	10.84	222.	-6.6	113.	-0.8	577.	-0.9	0.	0.0	-8.31
											0.93
	LB			6.6		2.5		11.5		0.0	20.50
											11.46
	RT			-5.9		-0.8		-1.3		0.0	-8.02
											1.27
	RB			4.1		1.6		7.3		0.0	13.07
											7.35
1	LT	45.84	222.	-5.9	113.	-0.8	577.	-1.3	0.	0.0	-8.02
											1.27
	LB			4.1		1.6		7.3		0.0	13.07
											7.35
	RT			-6.6		-0.8		-0.9		0.0	-8.32
											0.93
	RB			6.6		2.5		11.5		0.0	20.51
											11.46

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	10.84	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	35.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	10.00	1.0000
3	10.84	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 56.7

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1])	<=>	LIMITATION [3]/(12*[1]) 1/25 1/30	STATUS
1	10.84	6.75	33.09	[2]= 41.39	0.06085	>	0.04000	OK
				[3]= 33.09	0.04865	>	0.03300	OK
2	35.00	6.75	33.09	[2]= 41.39	0.06085	>	0.04000	OK

```
                        [3]=   33.09     0.04865      >   0.03300    OK

  3  10.84  6.75   33.09 [2]=   41.39     0.06085      >   0.04000    OK

                        [3]=   33.09     0.04865      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                          [2]    [3]    [4]    [5]    [6]
-----------------------------------------------------------------------------

  1  0   0.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  1   3.54  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  2   7.09  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  3  10.63  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  4  14.17  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  5  17.71  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  6  21.25  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  7  24.80  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  8  28.34  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  9  31.88  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 10  35.42  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 11  38.97  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 12  42.51  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 13  46.05  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 14  49.59  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 15  53.14  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 16  56.68  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0


-----------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	3.54	0	1	31.39	0.579	54.21	168.97	2
1	2	7.09	0	1	31.39	0.579	54.21	168.97	2
1	3	10.63	0	1	31.39	0.579	54.21	168.97	2
1	4	14.17	0	1	31.39	0.579	66.55	168.97	2
1	5	17.71	0	1	31.39	0.579	66.55	168.97	2
1	6	21.25	0	1	31.39	0.579	66.55	168.97	2
1	7	24.80	0	1	31.39	0.579	66.55	168.97	2
1	8	28.34	0	1	31.39	0.579	66.55	168.97	2
1	9	31.88	0	1	31.39	0.579	66.55	168.97	2
1	10	35.42	0	1	31.39	0.579	66.55	168.97	2
1	11	38.97	0	1	31.39	0.579	66.55	168.97	2
1	12	42.51	0	1	31.39	0.579	66.55	168.97	2
1	13	46.05	0	1	31.39	0.579	54.21	168.97	2
1	14	49.59	0	1	31.39	0.579	54.21	168.97	2
1	15	53.14	0	1	31.39	0.579	54.21	168.97	2
1	16	56.68	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 1	3.54	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 2	7.09	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 3	10.63	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 4	14.17	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 5	17.71	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 6	21.25	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 7	24.80	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 8	28.34	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 9	31.88	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 10	35.42	6.76	12.	11.5	5.2	0.850	0.637	

```
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 11  38.97    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 12  42.51    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 13  46.05    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 14  49.59    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 15  53.14    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 16  56.68    6.76 12.  11.5   5.2  0.850  0.637
                 6.76 12.  11.5   5.2  0.850  0.637    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
1
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0   8.8   0.000   8.281   4.507   0.732   1.660
                       0.000   0.000   0.000   0.000   1.000
    2      8.8  27.6   8.281  14.347  12.500  10.654   1.136
                       0.000   0.000   0.000   0.000   1.000
    3     27.6  46.4   9.431  14.347  12.970  11.593   1.097
                       0.000   0.000   0.000   0.000   1.000
    4     46.4  56.7   0.000   9.431   5.180   0.929   1.649
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
1
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
         (ft)                        CAT  (ft)     (ft)     (ft)    CAT CAT
-------------------------------------------------------------------------------

  1  0   0.00  1  6.76  11.26  19.84   0     8.76     7.17    26.91   2   2
  1  1   3.54  1  6.76  11.26  19.84   0     8.76     7.17    26.91   2   2
  1  2   7.09  1  6.76  11.26  19.84   0     8.76     7.17    26.91   2   2
  1  3  10.63  1  6.76  11.26  19.84   0    18.84     7.17    26.91   2   2
  1  4  14.17  1  6.76  11.26  19.84   0    18.84     6.98    26.21   2   2
  1  5  17.71  1  6.76  11.26  19.84   0    18.84     6.98    26.21   2   2
  1  6  21.25  1  6.76  11.26  19.84   0    18.84     6.98    26.21   2   2
  1  7  24.80  1  6.76  11.26  19.84   0    18.84     6.98    26.21   2   2
```

1	8	28.34	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	9	31.88	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	10	35.42	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	11	38.97	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	12	42.51	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	13	46.05	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	14	49.59	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2
1	15	53.14	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2
1	16	56.68	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.66	33.00	33.00	33.00
1	1	3.54	1	1.000	1.000	1.66	33.00	33.00	33.00
1	2	7.09	1	1.000	1.000	1.66	33.00	33.00	33.00
1	3	10.63	1	1.000	1.000	1.14	33.00	30.83	30.83
1	4	14.17	1	1.000	1.000	1.14	33.00	30.54	30.54
1	5	17.71	1	1.000	1.000	1.14	33.00	30.54	30.54
1	6	21.25	1	1.000	1.000	1.14	33.00	30.54	30.54
1	7	24.80	1	1.000	1.000	1.14	33.00	30.54	30.54
1	8	28.34	1	1.000	1.000	1.10	33.00	29.51	29.51
1	9	31.88	1	1.000	1.000	1.10	33.00	29.51	29.51
1	10	35.42	1	1.000	1.000	1.10	33.00	29.51	29.51
1	11	38.97	1	1.000	1.000	1.10	33.00	29.51	29.51
1	12	42.51	1	1.000	1.000	1.10	33.00	29.51	29.51
1	13	46.05	1	1.000	1.000	1.10	33.00	29.79	29.79
1	14	49.59	1	1.000	1.000	1.65	33.00	33.00	33.00
1	15	53.14	1	1.000	1.000	1.65	33.00	33.00	33.00
1	16	56.68	1	1.000	1.000	1.65	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

1

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0	0.00	1	5.17	17.38
1	1	3.54	1	5.17	17.38
1	2	7.09	1	5.17	17.38
1	3	10.63	1	5.17	17.38
1	4	14.17	1	6.54	17.80
1	5	17.71	1	6.54	17.80
1	6	21.25	1	6.54	17.80
1	7	24.80	1	6.54	17.80
1	8	28.34	1	6.54	17.80
1	9	31.88	1	6.54	17.80
1	10	35.42	1	6.54	17.80
1	11	38.97	1	6.54	17.80
1	12	42.51	1	6.54	17.80
1	13	46.05	1	5.17	17.38
1	14	49.59	1	5.17	17.38
1	15	53.14	1	5.17	17.38
1	16	56.68	1	5.17	17.38

1

TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Category Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.54	1	0	2	0		2	0
1	2	7.09	1	0	2	0		2	0
1	3	10.63	1	0	2	0		2	0
1	4	14.17	1	0	2	0		2	0
1	5	17.71	1	0	2	0		2	0
1	6	21.25	1	0	2	0		2	0
1	7	24.80	1	0	2	0		2	0
1	8	28.34	1	0	2	0		2	0
1	9	31.88	1	0	2	0		2	0
1	10	35.42	1	0	2	0		2	0
1	11	38.97	1	0	2	0		2	0
1	12	42.51	1	0	2	0		2	0
1	13	46.05	1	0	2	0		2	0
1	14	49.59	1	0	2	0		2	0
1	15	53.14	1	0	2	0		2	0
1	16	56.68	1	0	2	0		2	0

Please read NOTE on the following page

1

NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to

```
            decide section category
        [3] For composite sections in positive flexure, use
            Article 6.10.6.2.2 to decide section category
            For composite sections in negative flexure, use
            Eq. 6.10.6.2.3-1
        [4] For composite FLB or LTB, use Article 6.10.8.2 to compute
            section category

        * Strength Category of Cross Section

           0 = compact section

           2 = non-compact section

           3 = slender section

        FOR N = INF. : Category No. is the maximum of [1] and [2]
        FOR N = n    : Category No. is [3] for composite sections
                       in positive flexure
                       Category No. is the maximum of [3] and [4] for
                       composite sections in negative flexure

        Non-compact for Fy of the flanges > 70 ksi (483 MPa)
1
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   TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
                  ********************************
```

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	3.54	0.0	19.8	3.7	33.0	3.7	33.0	3.7	33.0	0
			0.0	-	3.7	-	3.7	33.0	-	-	0
1	2	7.09	0.0	19.8	7.0	33.0	7.0	33.0	7.0	33.0	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	3	10.63	0.0	19.8	9.7	33.0	9.7	33.0	9.7	30.8	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	4	14.17	0.0	19.8	10.8	33.0	10.8	33.0	10.8	30.5	0
			0.0	-	7.5	-	7.5	33.0	-	-	0
1	5	17.71	0.0	19.8	12.4	33.0	12.4	33.0	12.4	30.5	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	6	21.25	0.0	19.8	13.5	33.0	13.5	33.0	13.5	30.5	0
			0.0	-	9.4	-	9.4	33.0	-	-	0
1	7	24.80	0.0	19.8	14.2	33.0	14.2	33.0	14.2	30.5	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	8	28.34	0.0	19.8	14.4	33.0	14.4	33.0	14.4	29.5	0
			0.0	-	10.1	-	10.1	33.0	-	-	0
1	9	31.88	0.0	19.8	14.2	33.0	14.2	33.0	14.2	29.5	0
			0.0	-	9.9	-	9.9	33.0	-	-	0
1	10	35.42	0.0	19.8	13.5	33.0	13.5	33.0	13.5	29.5	0
			0.0	-	9.4	-	9.4	33.0	-	-	0
1	11	38.97	0.0	19.8	12.4	33.0	12.4	33.0	12.4	29.5	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	12	42.51	0.0	19.8	10.8	33.0	10.8	33.0	10.8	29.5	0
			0.0	-	7.5	-	7.5	33.0	-	-	0
1	13	46.05	0.0	19.8	9.7	33.0	9.7	33.0	9.7	29.8	0
			0.0	-	9.7	-	9.7	33.0	-	-	0
1	14	49.59	0.0	19.8	7.0	33.0	7.0	33.0	7.0	33.0	0
			0.0	-	7.0	-	7.0	33.0	-	-	0
1	15	53.14	0.0	19.8	3.7	33.0	3.7	33.0	3.7	33.0	0
			0.0	-	3.7	-	3.7	33.0	-	-	0
1	16	56.68	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

```
   NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
         [2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
             = (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
         [3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)
```

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.54	0.000	0.113	0.113	0.113	0.113	2
					0.113		0.113	3
1	2	7.09	0.000	0.212	0.212	0.212	0.212	2
					0.212		0.212	3
1	3	10.63	0.000	0.295	0.295	0.315	0.315	4
					0.295		0.295	3
1	4	14.17	0.000	0.327	0.327	0.353	0.353	4
					0.228		0.228	3
1	5	17.71	0.000	0.374	0.374	0.404	0.404	4
					0.261		0.261	3
1	6	21.25	0.000	0.408	0.408	0.441	0.441	4
					0.285		0.285	3
1	7	24.80	0.000	0.429	0.429	0.464	0.464	4
					0.300		0.300	3
1	8	28.34	0.000	0.436	0.436	0.488	0.488	4
					0.305		0.305	3
1	9	31.88	0.000	0.429	0.429	0.480	0.480	4
					0.299		0.299	3
1	10	35.42	0.000	0.408	0.408	0.457	0.457	4
					0.285		0.285	3
1	11	38.97	0.000	0.374	0.374	0.419	0.419	4
					0.261		0.261	3
1	12	42.51	0.000	0.327	0.327	0.366	0.366	4
					0.228		0.228	3
1	13	46.05	0.000	0.295	0.295	0.327	0.327	4
					0.295		0.295	3
1	14	49.59	0.000	0.212	0.212	0.212	0.212	2
					0.212		0.212	3
1	15	53.14	0.000	0.113	0.113	0.113	0.113	2
					0.113		0.113	3
1	16	56.68	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0
1	1		3.54	0	559.5	2269.4	-	-	0.0	19.8	-	-	0--0
1	2		7.09	0	1035.6	2269.4	-	-	0.0	19.8	-	-	0--0
1	3		10.63	0	1427.5	2269.4	-	-	0.0	19.8	-	-	0--0
1	4		14.17	0	1730.9	3175.4	-	-	0.0	19.8	-	-	0--0
1	5		17.71	0	1963.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	6		21.25	0	2126.6	3175.4	-	-	0.0	19.8	-	-	0--0
1	7		24.80	0	2202.7	3175.4	-	-	0.0	19.8	-	-	0--0
1	8		28.34	0	2223.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	9		31.88	0	2202.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	10		35.42	0	2126.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	11		38.97	0	1963.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	12		42.51	0	1731.0	3175.4	-	-	0.0	19.8	-	-	0--0
1	13		46.05	0	1427.8	2269.4	-	-	0.0	19.8	-	-	0--0
1	14		49.59	0	1035.4	2269.4	-	-	0.0	19.8	-	-	0--0
1	15		53.14	0	559.4	2269.4	-	-	0.0	19.8	-	-	0--0
1	16		56.68	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

1

TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT. GOVN.

1	0	0.00	0	0.000	0.000	1
1	1	3.54	0	0.247	0.247	1
1	2	7.09	0	0.456	0.456	1
1	3	10.63	0	0.629	0.629	1
1	4	14.17	0	0.545	0.545	1
1	5	17.71	0	0.618	0.618	1
1	6	21.25	0	0.670	0.670	1
1	7	24.80	0	0.694	0.694	1
1	8	28.34	0	0.700	0.700	1
1	9	31.88	0	0.694	0.694	1
1	10	35.42	0	0.670	0.670	1
1	11	38.97	0	0.618	0.618	1
1	12	42.51	0	0.545	0.545	1
1	13	46.05	0	0.629	0.629	1
1	14	49.59	0	0.456	0.456	1
1	15	53.14	0	0.246	0.246	1
1	16	56.68	0	0.000	0.000	1

```
NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
      [2] = (PHI)f * Fnc for comp. flange of composite
            sections in positive flexure (Eq. 6.10.7.2.1-1)
          = (PHI)f * Rh * Fyt for tension flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
      [3] = 0.6*Fyt for composite sections in positive flexure
            or tension flange for composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
          = 0.6*Fyc for comp. flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
      [4] = (PHI)f * Fnt for non-compact tension flange
            of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
          = (PHI)f * Fnc for comp. flange of composite sections
            in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

      The governing number is listed as below.
        1 = (Mu+1/3flSxt) / [1]
        2 = fbu / [2]
        3 = fl / [3]
        4 = (fbu + 1/3fl) / [4]
```

1

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.54	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	7.09	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	10.63	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	14.17	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	17.71	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	21.25	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	24.80	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	28.34	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	31.88	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	35.42	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	38.97	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	42.51	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	46.05	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	14	49.59	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	15	53.14	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	16	56.68	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                = 5 for unstiffened beams and girders
                = 5 for stiffened girders when (d0/D) > 3, or
                                               (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0

              For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                          ..... AASHTO LRFD Eq.6.10.9.3.2-6

              For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                          ..... AASHTO LRFD Eq.6.10.9.3.2-7


           ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
              stiffened
              if (d0/D) < 3 without long. stiffener, or
              if (d0/D) > 3 with both transv. and long. stiffeners.
              For handling requirement, (d0/D) > (260/(D/tw))**2

                   Vu = CVp

              where D = clear, unsupported distance between flange components

                   d0 = distance between transverse stiffeners

           ** This Article (6.10.9.2) indicates that a designer cannot
              count on post-buckling shear resistance from tension-
              field action for an unstiffened girder.
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     TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                     ********************************

                          S T R E N G T H  C A T E G O R Y; SEE NOTE
                  ------------------------------------------------------------
     SP IN D FROM WEB STABILITY    LFD/LRFD      UNSTIFFENED            OVERALL
     NO NO L SUPT UNSTIFFENED,[1] MAX. SHEAR <=> SHEAR CAPA- CATEGORY     WEB
           (ft)  ---- D/tw -----    (kip)       CITY,   [2]
     ------------------------------------------------------------------------

      1  0   0.00       0            200.6   <      347.9       0          0
      1  1   3.54       0            181.9   <      347.9       0          0
      1  2   7.09       0            163.4   <      347.9       0          0
      1  3  10.63       0            144.7   <      347.9       0          0
      1  4  14.17       0            126.6   <      347.9       0          0
      1  5  17.71       0            108.8   <      347.9       0          0
      1  6  21.25       0             91.1   <      347.9       0          0
      1  7  24.80       0             73.5   <      347.9       0          0
      1  8  28.34       0             56.4   <      347.9       0          0
      1  9  31.88       0             73.5   <      347.9       0          0
      1 10  35.42       0             91.1   <      347.9       0          0
      1 11  38.97       0            108.7   <      347.9       0          0
      1 12  42.51       0            126.6   <      347.9       0          0
      1 13  46.05       0            144.7   <      347.9       0          0
      1 14  49.59       0            163.3   <      347.9       0          0
      1 15  53.14       0            181.8   <      347.9       0          0
      1 16  56.68       0            200.6   <      347.9       0          0
```

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      NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

                For compact section  (composite sections in positive flexure)

                  w/o long. stiff.

                  D/tw <= 150 (Eq. 6.10.2.1.1-1)

                  w / long. stiff.

                  D/tw <= 300 (Eq. 6.10.2.1.2-1)

                  2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

                For compact or non-compact sections specified in Art. 6.10.8

                  (composite sections in negative flexure and
                   non-composite sections)

                  2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


            [2]: SHEAR CAPACITY ... web crippling control

                Vn = CVp .......... AASHTO Eq.6.10.9.2-1

                Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

     Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

                Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

     Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

                Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

              * For the detailed description of shear capacity,
                please refer to AASHTO Art.6.10.9.2 and .3

              * STRENGTH CATEGORY; 0 = compact section
                                   2 = braced non-compact section
                                   3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************

               YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
 SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
 NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
        (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
 ---------------------------------------------------------------------------

  1  0   0.00   33.   200.60   347.87            0
  1  1   3.54   33.   181.87   347.87            0
  1  2   7.09   33.   163.35   347.87            0
  1  3  10.63   33.   144.70   347.87            0
  1  4  14.17   33.   126.62   347.87            0
  1  5  17.71   33.   108.75   347.87            0
  1  6  21.25   33.    91.11   347.87            0
  1  7  24.80   33.    73.50   347.87            0
  1  8  28.34   33.    56.44   347.87            0
  1  9  31.88   33.    73.47   347.87            0
  1 10  35.42   33.    91.08   347.87            0
                                                 Page 36
```

1 11	38.97	33.	108.72	347.87	0
1 12	42.51	33.	126.59	347.87	0
1 13	46.05	33.	144.67	347.87	0
1 14	49.59	33.	163.32	347.87	0
1 15	53.14	33.	181.84	347.87	0
1 16	56.68	33.	200.57	347.87	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	BOT. FLANGE [1] ff+fl/2 (ksi)	[2] (ksi)	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3]	TBC
1	0		0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1		3.54	3.3	-	31.4	9.2	31.4	-	-	00-
1	2		7.09	6.1	-	31.4	17.1	31.4	-	-	00-
1	3		10.63	8.5	-	31.4	23.6	31.4	-	-	00-
1	4		14.17	10.1	-	31.4	18.3	31.4	-	-	00-
1	5		17.71	11.6	-	31.4	20.8	31.4	-	-	00-
1	6		21.25	12.6	-	31.4	22.5	31.4	-	-	00-
1	7		24.80	13.2	-	31.4	23.4	31.4	-	-	00-
1	8		28.34	13.4	-	31.4	23.6	31.4	-	-	00-
1	9		31.88	13.2	-	31.4	23.4	31.4	-	-	00-
1	10		35.42	12.6	-	31.4	22.5	31.4	-	-	00-
1	11		38.97	11.6	-	31.4	20.8	31.4	-	-	00-
1	12		42.51	10.1	-	31.4	18.3	31.4	-	-	00-
1	13		46.05	8.5	-	31.4	23.6	31.4	-	-	00-
1	14		49.59	6.1	-	31.4	17.1	31.4	-	-	00-
1	15		53.14	3.3	-	31.4	9.2	31.4	-	-	00-
1	16		56.68	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.54	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	2	7.09	TR	-0.3	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	3	10.63	TR	-0.4	A B B^C^C D E E^	TR	5.5	A B B^C^C D
1		10.84	SCG					
1	4	14.17	TR	-0.7	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	5	17.71	TR	-0.8	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	6	21.25	TR	-0.9	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	7	24.80	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	8	28.34	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9	31.88	TR	-0.8	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	10	35.42	TR	-0.9	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	11	38.97	TR	-0.8	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	12	42.51	TR	-0.7	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1		45.84	SCG					
1	13	46.05	TR	-0.4	A B B^C^C D E E^	TR	5.5	A B B^C^C D
1	14	49.59	TR	-0.3	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	15	53.14	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	16	56.68	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.54	TR	-0.1	A B B^C^C D E E^	TR	1.2	A B B^C^C D E E^
1	2	7.09	TR	-0.2	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^

```
 1  3  10.63    TR     -0.2 A B B^C^C D E E^  TR      2.8 A B B^C^C D E
 1     10.84  SCG
 1  4  14.17    TR     -0.4 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1  5  17.71    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1  6  21.25    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1  7  24.80    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1  8  28.34    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1  9  31.88    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 10  35.42    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1 11  38.97    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 12  42.51    TR     -0.4 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1     45.84  SCG
 1 13  46.05    TR     -0.2 A B B^C^C D E E^  TR      2.8 A B B^C^C D E
 1 14  49.59    TR     -0.2 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1 15  53.14    TR     -0.1 A B B^C^C D E E^  TR      1.2 A B B^C^C D E E^
 1 16  56.68    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   *****************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-1     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

  1  0   0.00  51.6 .29242E-01    1.5      TR    3.09        3           6.15
  1  1   3.54  49.0 .29242E-01    1.4      TR    3.09        3           6.48
  1  2   7.09  46.2 .29242E-01    1.4      TR    3.09        3           6.86
  1  3  10.63  43.6 .29242E-01    1.3      TR    3.09        3           7.27
  1  4  14.17  41.5 .26751E-01    1.1      TR    3.09        3           8.35
  1  5  17.71  39.6 .26751E-01    1.1      TR    3.09        3           8.77
  1  6  21.25  37.8 .26751E-01    1.0      TR    3.09        3           9.17
  1  7  24.80  37.9 .26751E-01    1.0      TR    3.09        3           9.16
  1  8  28.34  38.1 .26751E-01    1.0      TR    3.09        3           9.11
  1  9  31.88  37.9 .26751E-01    1.0      TR    3.09        3           9.16
  1 10  35.42  37.8 .26751E-01    1.0      TR    3.09        3           9.17
  1 11  38.97  39.6 .26751E-01    1.1      TR    3.09        3           8.77
  1 12  42.51  41.5 .26751E-01    1.1      TR    3.09        3           8.35
  1 13  46.05  43.6 .29242E-01    1.3      TR    3.09        3           7.27
  1 14  49.59  46.2 .29242E-01    1.4      TR    3.09        3           6.86
  1 15  53.14  49.0 .29242E-01    1.4      TR    3.09        3           6.48
  1 16  56.68  51.6 .29242E-01    1.5      TR    3.09        3           6.15


 ------------------------------------------------------------------------------
 Please read NOTE on the following page

1
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```
 NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                section, the range of shear shall be taken as the difference
                in the minimum and maximum shear envelopes (excluding dead
                loads); AASHTO LRFD Eqn.6.10.10.1.2-2

       [2] Q/I : Q= statical moment about the neutral axis of the composite
                   section of the transformed compressive concrete area or
                   the area of reinforcement embedded in the concrete for
                   negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                 I= moment of inertia of the transformed composite girder in
                   positive moment regions or the moment of inertia provided
                   by the steel beam including or excluding the area of
                   reinforcement embedded in the concrete in negative moment
                   regions, in inches to the fourth power.

       [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                 of the slab and girder at the point in the span under
                 consideration.
               = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
           Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
           Ffat = radial fatigue shear range per unit length
                = Ffat2 = Frc/w for skews exceeding 20 degrees
           Frc = 25.0 kips for both exterior and interior girders
           w = the effective length of the deck
             = 48 inches except at end supports where w is taken
               as 24 inches

       [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

       [5] No. of shear connectors per transverse section. This value is from
           input data type 12032.

       [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

           ** The connector spacing shown here is calculated based upon
              AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
          *** If ADTT Single Lane is less than 960, use Fatigue II Table.
1
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                   *********************************************
```

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-2 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	25.8	.29242E-01	0.8	TR	1.36	3	5.42
1	1	3.54	24.5	.29242E-01	0.7	TR	1.36	3	5.71
1	2	7.09	23.1	.29242E-01	0.7	TR	1.36	3	6.05
1	3	10.63	21.8	.29242E-01	0.6	TR	1.36	3	6.41
1	4	14.17	20.8	.26751E-01	0.6	TR	1.36	3	7.36
1	5	17.71	19.8	.26751E-01	0.5	TR	1.36	3	7.73
1	6	21.25	18.9	.26751E-01	0.5	TR	1.36	3	8.08
1	7	24.80	18.9	.26751E-01	0.5	TR	1.36	3	8.08
1	8	28.34	19.0	.26751E-01	0.5	TR	1.36	3	8.03
1	9	31.88	18.9	.26751E-01	0.5	TR	1.36	3	8.08
1	10	35.42	18.9	.26751E-01	0.5	TR	1.36	3	8.08
1	11	38.97	19.8	.26751E-01	0.5	TR	1.36	3	7.73
1	12	42.51	20.8	.26751E-01	0.6	TR	1.36	3	7.36
1	13	46.05	21.8	.29242E-01	0.6	TR	1.36	3	6.41
1	14	49.59	23.1	.29242E-01	0.7	TR	1.36	3	6.05
1	15	53.14	24.5	.29242E-01	0.7	TR	1.36	3	5.71
1	16	56.68	25.8	.29242E-01	0.8	TR	1.36	3	5.42

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 Please read NOTE on the following page
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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
              as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

            * Default ALPHA value based on 7/8" diameter and input road type


        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
               where ADTT Single Lane= 1140 & No. of Cycles=   31207500
           *** If ADTT Single Lane greater than or equal to 960, refer
               to Fatigue I Table.
1
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ***************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  28.34   1 1574.9  1626.6          171         >  82
      1 16  56.68   1 1574.9  1626.6          171         >  82


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

1
```

Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)

```
                ----------------------------------

                    1     0.000    28.340     6.000

                    1    28.340    56.680     6.000

                ----------------------------------
1
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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                  *********************************

                 SPAN     CURRENT SPAN    T
                  NO.     FROM      TO    C
                          (ft)     (ft)   R
                ---------------------------

                    1       0.0      56.7 C

                ---------------------------

                NOTE: T: TENSION AREA, C: COMPRESSION AREA,
                R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
                STRESSES
1
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     TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
                    ************************

 STATUS                 TABLE NO.
-------------------------------------------------------------------------------

  OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

  OK     1.2.22.4=DEPTH RATIOS

  OK     1.2.22.10=CONSTRUCBILITY CHECK

  OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

  OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

  OK     1.2.22.21=SERVICE LIMIT STATE CHECK


         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
1
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
                 ***********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT       MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
```

NO	NO	L (ft)	SUPT CAP. (k-ft) or STRESS	D1; k-ft ksi	D2	POS. MAX. STRESS	NEG. MAX. STRESS	FACTOR [3]	LOAD TYPE	FACTOR	T
1	0	0.00	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.54	2269.4	84.2	42.8	225.0	0.0	6.92	HL-93	5.34	0
1	2	7.09	2269.4	157.4	79.9	414.9	0.0	3.50	HL-93	2.70	0
1	3	10.63	2269.4	219.1	111.3	569.5	0.0	2.39	HL-93	1.84	0
1	4	14.17	3175.4	269.2	137.0	686.3	0.0	2.86	HL-93	2.20	0
1	5	17.71	3175.4	308.4	157.0	775.2	0.0	2.45	HL-93	1.89	0
1	6	21.25	3175.4	336.5	171.3	836.8	0.0	2.22	HL-93	1.72	0
1	7	24.80	3175.4	353.5	179.9	861.2	0.0	2.13	HL-93	1.65	0
1	8	28.34	3175.4	359.4	182.7	866.6	0.0	2.11	HL-93	1.63	0
1	9	31.88	3175.4	353.4	179.9	861.2	0.0	2.13	HL-93	1.65	0
1	10	35.42	3175.4	336.4	171.3	836.8	0.0	2.22	HL-93	1.72	0
1	11	38.97	3175.4	308.4	157.0	775.2	0.0	2.45	HL-93	1.89	0
1	12	42.51	3175.4	269.3	137.0	686.3	0.0	2.86	HL-93	2.20	0
1	13	46.05	2269.4	219.3	111.3	569.5	0.0	2.39	HL-93	1.84	0
1	14	49.59	2269.4	157.3	79.9	414.9	0.0	3.50	HL-93	2.70	0
1	15	53.14	2269.4	84.2	42.8	225.0	0.0	6.92	HL-93	5.34	0
1	16	56.68	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	SUPT 0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS ---STEEL--- MAX. POS. TOP;ksi;BOT	--- STEEL--- MAX. NEG. TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD RATING FACTOR TYPE	
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	3.54	31.4	-2.80 3.43	-0.36 4.47	0.00 0.00	6.24	HL-93	4.80
1	2	7.09	31.4	-5.23 6.41	-0.67 8.24	0.00 0.00	3.03	HL-93	2.33
1	3	10.63	31.4	-7.28 8.92	-0.92 11.32	0.00 0.00	1.98	HL-93	1.52

1	4	14.17	31.4	-8.18	6.94	-1.51	8.75	0.00	0.00	2.79	HL-93	2.15
1	5	17.71	31.4	-9.37	7.95	-1.70	9.88	0.00	0.00	2.37	HL-93	1.82
1	6	21.25	31.4	-10.23	8.67	-1.84	10.67	0.00	0.00	2.13	HL-93	1.64
1	7	24.80	31.4	-10.75	9.11	-1.89	10.98	0.00	0.00	2.03	HL-93	1.56
1	8	28.34	31.4	-10.92	9.26	-1.90	11.05	0.00	0.00	2.00	HL-93	1.54
1	9	31.88	31.4	-10.74	9.11	-1.89	10.98	0.00	0.00	2.03	HL-93	1.56
1	10	35.42	31.4	-10.23	8.67	-1.84	10.67	0.00	0.00	2.13	HL-93	1.64
1	11	38.97	31.4	-9.37	7.95	-1.70	9.88	0.00	0.00	2.37	HL-93	1.82
1	12	42.51	31.4	-8.19	6.94	-1.51	8.75	0.00	0.00	2.79	HL-93	2.15
1	13	46.05	31.4	-7.29	8.93	-0.92	11.32	0.00	0.00	1.98	HL-93	1.52
1	14	49.59	31.4	-5.23	6.41	-0.67	8.24	0.00	0.00	3.03	HL-93	2.33
1	15	53.14	31.4	-2.80	3.43	-0.36	4.47	0.00	0.00	6.24	HL-93	4.80
1	16	56.68	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	56.68	28.34	STRENGTH L.S.	2.11	1.63	HL-93
1	56.68	46.05	SERVICEBILITY	1.98	1.52	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.981
Inventory= 1.524

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)

```
---------------------------------------------------------------

  1    56.68        24.80       STRENGTH L.S.  3.10        800.87
  1    56.68        46.05       SERVICEBILITY  2.50

-------------------------------------------------------------
    NOTE: control Rating Factor for Current Bridge:
          SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
               Legal Load Rating= 2.498
1
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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
               **********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT.
SP IN D FROM   SHEAR         SHEAR       MAX.  SHEAR    RATING  LIVE  RATING
NO NO L SUPT   CAP.                      --------------- FACTOR  LOAD  FACTOR
       (ft)  (kips)       D1; kips  D2     POS.     NEG.  [3]   TYPE
-------------------------------------------------------------------------------

 1  0   0.00    347.9     25.3     12.9     86.1      0.0  2.56  HL-93  1.98
 1  1   3.54    347.9     22.2     11.3     79.0     -2.4  2.85  HL-93  2.20
 1  2   7.09    347.9     19.1      9.7     71.9     -5.5  3.20  HL-93  2.47
 1  3  10.63    347.9     15.7      8.1     65.0     -9.4  3.61  HL-93  2.79
 1  4  14.17    347.9     12.6      6.4     58.2    -13.5  4.11  HL-93  3.17
 1  5  17.71    347.9      9.5      4.8     51.5    -17.7  4.74  HL-93  3.65
 1  6  21.25    347.9      6.4      3.2     44.9    -22.1  5.53  HL-93  4.27
 1  7  24.80    347.9      3.3      1.6     38.4    -26.5  6.59  HL-93  5.09
 1  8  28.34    347.9     -0.1      0.0     32.2    -32.2  8.01  HL-93  6.18
 1  9  31.88    347.9     -3.2     -1.6     26.5    -38.4  6.59  HL-93  5.09
 1 10  35.42    347.9     -6.4     -3.2     22.1    -44.9  5.53  HL-93  4.27
 1 11  38.97    347.9     -9.5     -4.8     17.7    -51.5  4.74  HL-93  3.66
 1 12  42.51    347.9    -12.6     -6.4     13.5    -58.2  4.11  HL-93  3.17
 1 13  46.05    347.9    -15.7     -8.1      9.4    -65.0  3.61  HL-93  2.79
 1 14  49.59    347.9    -19.1     -9.7      5.5    -71.9  3.20  HL-93  2.47
 1 15  53.14    347.9    -22.2    -11.3      2.4    -79.0  2.85  HL-93  2.20
 1 16  56.68    347.9    -25.3    -12.9      0.0    -86.1  2.56  HL-93  1.98


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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
                ************************************************

    SPAN       CRITICAL LOCATION GOVERNING                        GOVERNING
------------- ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
 NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    RATING     TYPE
       (ft)         (ft)
-----------------------------------------------------------------------------

  1    56.68         0.00       STRENGTH L.S.  2.56      1.98     HL-93

-----------------------------------------------------------------------------
    NOTE: Control Rating Factor for Current Bridge:
          STRENGTH L.S.
               Operating= 2.563
               Inventory= 1.977


TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK     FA
                ***************************************************

    SPAN       CRITICAL LOCATION GOVERNING
------------- ------------------  RATING    LEGAL LOAD   MAXIMUM
```

5969 Span 2-S18 Interior Strength.RES

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	LL SHEAR (kips)
1	56.68	0.00	STRENGTH L.S.	3.13	95.07

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.135

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	28.34	3 0.425	-0.30	HL-93	2.85
		4 0.433	-0.30		2.79

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span2-S18 Interior Fatigue	8/20/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	15	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
56.68	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	PLATE GIRDER WEB DEPTH (in)	THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130			0.00	0.0000	0.00	0.0000		
2	W	33	130			0.00	0.00001	0.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

1

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
			-->TYPE<--						
1	1	1	10.84				33.	33.	33.
2	2	2	35.00				33.	33.	33.
3	1	1	10.84				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

1

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	56.68

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	8.76
2	2	Diaphragm	0.000	0.00	0.00	0.29	27.60
3	2	Diaphragm	0.000	0.00	0.00	0.29	46.44
4	1	Railing	0.087	0.00	56.68	0.00	0.00
5	1	Curb	0.076	0.00	56.68	0.00	0.00
6	0	Pavement	0.292	0.00	56.68	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE			SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	8.76	33.0	33.0	8.76
8.76	27.60	33.0	33.0	18.84
27.60	46.44	33.0	33.0	18.84
46.44	56.68	33.0	33.0	10.24

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

```
 F A C T O R S  U S E D  B Y  L R F D
-------------------------------------------------------

 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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    TABLE 1.2.2.1=LOADING INFORMATION
                  *******************

         AVERAGE DEAD LOAD INTENSITIES
         *****************************

       SPAN     SLAB    +   STEEL   =  TOTAL
        NO.    (K/FT)       (K/FT)    (K/FT)
       ---------------------------------------

         1      0.727       0.1513     0.8783


       ---------------------------------------




    SUPERIMPOSED DEAD LOADS
    ***********************


       LOAD       INTENSITY      DIST       DIST
                                 FROM        TO
               (K or K/Ft)       (Ft)       (Ft)
       ----------------------------------------

         1   DW       0.29        0.00      56.68

         1   DC2     0.087       0.000     56.680

         2   DC2     0.076       0.000     56.680


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1376.3	51.0	206.4	51.5	257.90

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.709	0.709	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.78	6710.0	33.1	16.55	405.6	405.6
1	2	7.56	6710.0	33.1	16.55	405.6	405.6
1	3	11.34	9020.9	33.1	14.02	643.6	449.4
1	4	15.11	9020.9	33.1	14.02	643.6	449.4
1	5	18.89	9020.9	33.1	14.02	643.6	449.4
1	6	22.67	9020.9	33.1	14.02	643.6	449.4
1	7	26.45	9020.9	33.1	14.02	643.6	449.4
1	8	30.23	9020.9	33.1	14.02	643.6	449.4

1	9	34.01	9020.9	33.1	14.02	643.6	449.4
1	10	37.79	9020.9	33.1	14.02	643.6	449.4
1	11	41.57	9020.9	33.1	14.02	643.6	449.4
1	12	45.34	9020.9	33.1	14.02	643.6	449.4
1	13	49.12	6710.0	33.1	16.55	405.6	405.6
1	14	52.90	6710.0	33.1	16.55	405.6	405.6
1	15	56.68	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	13641.5	0.023742	547.2	1671.8	828.8
1	1	3.78	13641.5	0.023742	547.2	1671.8	828.8
1	2	7.56	13641.5	0.023742	547.2	1671.8	828.8
1	3	11.34	19233.3	0.021414	856.3	1653.8	965.1
1	4	15.11	19233.3	0.021414	856.3	1653.8	965.1
1	5	18.89	19233.3	0.021414	856.3	1653.8	965.1
1	6	22.67	19233.3	0.021414	856.3	1653.8	965.1
1	7	26.45	19233.3	0.021414	856.3	1653.8	965.1
1	8	30.23	19233.3	0.021414	856.3	1653.8	965.1
1	9	34.01	19233.3	0.021414	856.3	1653.8	965.1
1	10	37.79	19233.3	0.021414	856.3	1653.8	965.1
1	11	41.57	19233.3	0.021414	856.3	1653.8	965.1
1	12	45.34	19233.3	0.021414	856.3	1653.8	965.1
1	13	49.12	13641.5	0.023742	547.2	1671.8	828.8
1	14	52.90	13641.5	0.023742	547.2	1671.8	828.8
1	15	56.68	13641.5	0.023742	547.2	1671.8	828.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	18479.7	0.029242	603.9	7418.9	1712.5
1	1	3.78	18479.7	0.029242	603.9	7418.9	1712.5
1	2	7.56	18479.7	0.029242	603.9	7418.9	1712.5
1	3	11.34	27373.9	0.026751	941.4	5460.0	2056.1
1	4	15.11	27373.9	0.026751	941.4	5460.0	2056.1
1	5	18.89	27373.9	0.026751	941.4	5460.0	2056.1
1	6	22.67	27373.9	0.026751	941.4	5460.0	2056.1
1	7	26.45	27373.9	0.026751	941.4	5460.0	2056.1
1	8	30.23	27373.9	0.026751	941.4	5460.0	2056.1
1	9	34.01	27373.9	0.026751	941.4	5460.0	2056.1

1 10	37.79	27373.9	0.026751	941.4	5460.0	2056.1
1 11	41.57	27373.9	0.026751	941.4	5460.0	2056.1
1 12	45.34	27373.9	0.026751	941.4	5460.0	2056.1
1 13	49.12	18479.7	0.029242	603.9	7418.9	1712.5
1 14	52.90	18479.7	0.029242	603.9	7418.9	1712.5
1 15	56.68	18479.7	0.029242	603.9	7418.9	1712.5

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.78	15.1	72.7	1.7	0.0		89.5
1	2	7.56	28.1	134.9	3.3	0.0		166.4
1	3	11.34	38.9	186.8	4.3	0.0		230.0
1	4	15.11	47.5	228.4	4.9	0.0		280.7
1	5	18.89	54.0	259.5	5.4	0.0		319.0
1	6	22.67	58.3	280.3	6.0	0.0		344.6
1	7	26.45	60.5	290.6	6.6	0.0		357.7
1	8	30.23	60.5	290.6	6.4	0.0		357.6
1	9	34.01	58.3	280.3	5.9	0.0		344.5
1	10	37.79	54.0	259.5	5.4	0.0		319.0
1	11	41.57	47.5	228.4	4.9	0.0		280.8
1	12	45.34	38.9	186.8	4.4	0.0		230.2
1	13	49.12	28.1	134.9	3.2	0.0		166.2
1	14	52.90	15.1	72.7	1.6	0.0		89.4
1	15	56.68	0.0	0.0	0.0	0.0		0.0

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 TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
               ****************************************

                     UNIFORM                          OTHER   TOTAL (k-ft)
 SP IN D FROM      SUPERIMPOSED     CONCENTRATED    UNIFORM  --
 NO NO L SUPT       DEAD  LOAD          LOADS        LOADS    COMPOSITE
        (ft)          (k-ft)            (k-ft)                 DEAD LOAD
                    DW      DC2       DW      DC2
 ----------------------------------------------------------------------

  1  0   0.00      0.0      0.0      0.0      0.0        0.0        0.0
  1  1   3.78     29.2     16.3      0.0      0.0        0.0       45.5
  1  2   7.56     54.2     30.3      0.0      0.0        0.0       84.5
  1  3  11.34     75.0     41.9      0.0      0.0        0.0      116.9
  1  4  15.11     91.7     51.2      0.0      0.0        0.0      142.9
  1  5  18.89    104.2     58.2      0.0      0.0        0.0      162.4
  1  6  22.67    112.6     62.8      0.0      0.0        0.0      175.4
  1  7  26.45    116.7     65.2      0.0      0.0        0.0      181.9
  1  8  30.23    116.7     65.2      0.0      0.0        0.0      181.9
  1  9  34.01    112.6     62.8      0.0      0.0        0.0      175.4
  1 10  37.79    104.2     58.2      0.0      0.0        0.0      162.4
  1 11  41.57     91.7     51.2      0.0      0.0        0.0      142.9
  1 12  45.34     75.0     41.9      0.0      0.0        0.0      116.9
  1 13  49.12     54.2     30.3      0.0      0.0        0.0       84.5
  1 14  52.90     29.2     16.3      0.0      0.0        0.0       45.5
  1 15  56.68      0.0      0.0      0.0      0.0        0.0        0.0


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 TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N =  9.00 (UNFACTORED)
               ****************************************

               SIDEWALK (MAXIMUM)     LL+I ,(k-ft), LOAD TYPE= HL -93
 SP IN D FROM  ------------------  -------------------------------------
 NO NO L SUPT  POSITIVE  NEGATIVE  MAXIMUM   GOVERN.   MAXIMUM   GOVERN.
        (ft)    (k-ft)    (k-ft)   POSITIVE LOAD TYPE  NEGATIVE LOAD TYPE
 ----------------------------------------------------------------------

  1  0   0.00     0.0       0.0        0.0   HL-93        0.0    HL-93
  1  1   3.78     0.0       0.0      238.8   HL-93        0.0    HL-93
  1  2   7.56     0.0       0.0      437.5   HL-93        0.0    HL-93
  1  3  11.34     0.0       0.0      596.2   HL-93        0.0    HL-93
  1  4  15.11     0.0       0.0      712.5   HL-93        0.0    HL-93
  1  5  18.89     0.0       0.0      799.2   HL-93        0.0    HL-93
  1  6  22.67     0.0       0.0      852.7   HL-93        0.0    HL-93
  1  7  26.45     0.0       0.0      865.0   HL-93        0.0    HL-93
  1  8  30.23     0.0       0.0      865.0   HL-93        0.0    HL-93
  1  9  34.01     0.0       0.0      852.7   HL-93        0.0    HL-93
  1 10  37.79     0.0       0.0      799.2   HL-93        0.0    HL-93
  1 11  41.57     0.0       0.0      712.5   HL-93        0.0    HL-93
  1 12  45.34     0.0       0.0      596.2   HL-93        0.0    HL-93
  1 13  49.12     0.0       0.0      437.5   HL-93        0.0    HL-93
  1 14  52.90     0.0       0.0      238.8   HL-93        0.0    HL-93
  1 15  56.68     0.0       0.0        0.0   HL-93        0.0    HL-93


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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.78	82.	0.	82.
1	2	7.56	145.	0.	145.
1	3	11.34	193.	0.	193.
1	4	15.11	230.	0.	230.
1	5	18.89	252.	0.	252.
1	6	22.67	261.	0.	261.
1	7	26.45	253.	0.	253.
1	8	30.23	253.	0.	253.
1	9	34.01	261.	0.	261.
1	10	37.79	252.	0.	252.
1	11	41.57	230.	0.	230.
1	12	45.34	193.	0.	193.
1	13	49.12	145.	0.	145.
1	14	52.90	82.	0.	82.
1	15	56.68	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.78	89.5	45.5	238.8	HL-93	0.0	HL-93	373.7	134.9
1	2	7.56	166.4	84.5	437.5	HL-93	0.0	HL-93	688.3	250.8
1	3	11.34	230.0	116.9	596.2	HL-93	0.0	HL-93	943.1	346.9
1	4	15.11	280.7	142.9	712.5	HL-93	0.0	HL-93	1136.2	423.7
1	5	18.89	319.0	162.4	799.2	HL-93	0.0	HL-93	1280.5	481.4
1	6	22.67	344.6	175.4	852.7	HL-93	0.0	HL-93	1372.7	520.0
1	7	26.45	357.7	181.9	865.0	HL-93	0.0	HL-93	1404.7	539.7
1	8	30.23	357.6	181.9	865.0	HL-93	0.0	HL-93	1404.5	539.5
1	9	34.01	344.5	175.4	852.7	HL-93	0.0	HL-93	1372.7	519.9
1	10	37.79	319.0	162.4	799.2	HL-93	0.0	HL-93	1280.5	481.4
1	11	41.57	280.8	142.9	712.5	HL-93	0.0	HL-93	1136.3	423.8
1	12	45.34	230.2	116.9	596.2	HL-93	0.0	HL-93	943.3	347.1
1	13	49.12	166.2	84.5	437.5	HL-93	0.0	HL-93	688.2	250.7
1	14	52.90	89.4	45.5	238.8	HL-93	0.0	HL-93	373.6	134.8
1	15	56.68	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP IN D FROM SERVICE I SERVICE II STRENGTH I STRENGTH II STRENGTH IV FATIGUE

NO	NO	L SUPT (ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.78	373.7	445.4	593.8	498.3	202.4	61.7
1	2	7.56	688.3	819.6	1092.7	917.7	376.2	108.9
1	3	11.34	943.1	1122.0	1495.8	1257.3	520.4	144.8
1	4	15.11	1136.2	1349.9	1799.4	1514.4	635.5	172.1
1	5	18.89	1280.5	1520.3	2026.3	1706.7	722.1	188.8
1	6	22.67	1372.7	1628.6	2170.4	1829.3	780.0	195.9
1	7	26.45	1404.7	1664.2	2217.6	1871.6	809.5	190.1
1	8	30.23	1404.5	1664.0	2217.4	1871.3	809.2	190.1
1	9	34.01	1372.7	1628.5	2170.3	1829.2	779.9	195.9
1	10	37.79	1280.5	1520.3	2026.3	1706.7	722.1	188.8
1	11	41.57	1136.3	1350.0	1799.5	1514.5	635.6	172.1
1	12	45.34	943.3	1122.2	1496.0	1257.5	520.7	144.8
1	13	49.12	688.2	819.4	1092.5	917.5	376.0	108.9
1	14	52.90	373.6	445.3	593.7	498.2	202.3	61.7
1	15	56.68	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.3	20.6	0.4	0.0	25.3
1	1	3.78	3.7	17.9	0.4	0.0	22.0
1	2	7.56	3.1	15.1	0.4	0.0	18.7
1	3	11.34	2.6	12.4	0.2	0.0	15.1
1	4	15.11	2.0	9.6	0.2	0.0	11.8
1	5	18.89	1.4	6.9	0.2	0.0	8.5
1	6	22.67	0.9	4.1	0.2	0.0	5.1
1	7	26.45	0.3	1.4	0.2	0.0	1.8
1	8	30.23	-0.3	-1.4	-0.1	0.0	-1.8
1	9	34.01	-0.9	-4.1	-0.1	0.0	-5.1
1	10	37.79	-1.4	-6.9	-0.1	0.0	-8.4
1	11	41.57	-2.0	-9.6	-0.1	0.0	-11.7
1	12	45.34	-2.6	-12.4	-0.1	0.0	-15.1
1	13	49.12	-3.1	-15.1	-0.4	0.0	-18.7
1	14	52.90	-3.7	-17.9	-0.4	0.0	-22.0
1	15	56.68	-4.3	-20.6	-0.4	0.0	-25.3

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)

1	0	0.00	8.3	4.6	0.0	0.0	12.9	25.3	38.2
1	1	3.78	7.2	4.0	0.0	0.0	11.2	22.0	33.2
1	2	7.56	6.1	3.4	0.0	0.0	9.5	18.7	28.2
1	3	11.34	5.0	2.8	0.0	0.0	7.7	15.1	22.8
1	4	15.11	3.9	2.2	0.0	0.0	6.0	11.8	17.8
1	5	18.89	2.8	1.5	0.0	0.0	4.3	8.5	12.8
1	6	22.67	1.7	0.9	0.0	0.0	2.6	5.1	7.7
1	7	26.45	0.6	0.3	0.0	0.0	0.9	1.8	2.7
1	8	30.23	-0.6	-0.3	0.0	0.0	-0.9	-1.8	-2.7
1	9	34.01	-1.7	-0.9	0.0	0.0	-2.6	-5.1	-7.7
1	10	37.79	-2.8	-1.5	0.0	0.0	-4.3	-8.4	-12.7
1	11	41.57	-3.9	-2.2	0.0	0.0	-6.0	-11.7	-17.8
1	12	45.34	-5.0	-2.8	0.0	0.0	-7.7	-15.1	-22.8
1	13	49.12	-6.1	-3.4	0.0	0.0	-9.5	-18.7	-28.1
1	14	52.90	-7.2	-4.0	0.0	0.0	-11.2	-22.0	-33.2
1	15	56.68	-8.3	-4.6	0.0	0.0	-12.9	-25.3	-38.2

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	86.1	HL-93	0.0	HL-93
1	1	3.78	0.0	0.0	78.5	HL-93	-2.6	HL-93
1	2	7.56	0.0	0.0	71.0	HL-93	-6.0	HL-93
1	3	11.34	0.0	0.0	63.6	HL-93	-10.2	HL-93
1	4	15.11	0.0	0.0	56.4	HL-93	-14.6	HL-93
1	5	18.89	0.0	0.0	49.3	HL-93	-19.2	HL-93
1	6	22.67	0.0	0.0	42.3	HL-93	-23.8	HL-93
1	7	26.45	0.0	0.0	35.4	HL-93	-29.0	HL-93
1	8	30.23	0.0	0.0	29.0	HL-93	-35.4	HL-93
1	9	34.01	0.0	0.0	23.8	HL-93	-42.3	HL-93
1	10	37.79	0.0	0.0	19.2	HL-93	-49.3	HL-93
1	11	41.57	0.0	0.0	14.6	HL-93	-56.4	HL-93
1	12	45.34	0.0	0.0	10.2	HL-93	-63.6	HL-93
1	13	49.12	0.0	0.0	6.0	HL-93	-71.0	HL-93
1	14	52.90	0.0	0.0	2.6	HL-93	-78.5	HL-93
1	15	56.68	0.0	0.0	0.0	HL-93	-86.1	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	34.	0.	34.
1	1	3.78	31.	-2.	33.
1	2	7.56	28.	-3.	31.
1	3	11.34	24.	-5.	29.
1	4	15.11	21.	-6.	27.
1	5	18.89	18.	-8.	26.
1	6	22.67	15.	-10.	25.
1	7	26.45	14.	-12.	25.
1	8	30.23	12.	-14.	25.
1	9	34.01	10.	-15.	25.

1	10	37.79	8.	-18.	26.
1	11	41.57	6.	-21.	27.
1	12	45.34	5.	-24.	29.
1	13	49.12	3.	-28.	31.
1	14	52.90	2.	-31.	33.
1	15	56.68	0.	-34.	34.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	34.4	.30675E-01	0.53	0.3125*	.14721E-01	0.25	0.3125*
1	1		3.78	32.5	.30675E-01	0.50	0.3125*	.14721E-01	0.24	0.3125*
1	2		7.56	30.6	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	3		11.34	28.8	.28592E-01	0.41	0.3125*	.90646E-02	0.13	0.3125*
1	4		15.11	27.3	.28592E-01	0.39	0.3125*	.90646E-02	0.12	0.3125*
1	5		18.89	25.9	.28592E-01	0.37	0.3125*	.90646E-02	0.12	0.3125*
1	6		22.67	25.0	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	7		26.45	25.4	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	8		30.23	25.4	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	9		34.01	25.0	.28592E-01	0.36	0.3125*	.90646E-02	0.11	0.3125*
1	10		37.79	25.9	.28592E-01	0.37	0.3125*	.90646E-02	0.12	0.3125*
1	11		41.57	27.3	.28592E-01	0.39	0.3125*	.90646E-02	0.12	0.3125*
1	12		45.34	28.8	.28592E-01	0.41	0.3125*	.90646E-02	0.13	0.3125*
1	13		49.12	30.6	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	14		52.90	32.5	.30675E-01	0.50	0.3125*	.14721E-01	0.24	0.3125*
1	15		56.68	34.4	.30675E-01	0.53	0.3125*	.14721E-01	0.25	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	38.2	86.1	HL-93	0.0	HL-93	124.4	38.2
1	1	3.78	33.2	78.5	HL-93	-2.6	HL-93	111.7	30.6
1	2	7.56	28.2	71.0	HL-93	-6.0	HL-93	99.1	22.2
1	3	11.34	22.8	63.6	HL-93	-10.2	HL-93	86.4	12.6
1	4	15.11	17.8	56.4	HL-93	-14.6	HL-93	74.1	3.2
1	5	18.89	12.8	49.3	HL-93	-19.2	HL-93	62.0	-6.4
1	6	22.67	7.7	42.3	HL-93	-23.8	HL-93	50.0	-16.1
1	7	26.45	2.7	35.4	HL-93	-29.0	HL-93	38.1	-26.4
1	8	30.23	-2.7	29.0	HL-93	-35.4	HL-93	26.4	-38.1
1	9	34.01	-7.7	23.8	HL-93	-42.3	HL-93	16.1	-50.0
1	10	37.79	-12.7	19.2	HL-93	-49.3	HL-93	6.4	-62.0
1	11	41.57	-17.8	14.6	HL-93	-56.4	HL-93	-3.1	-74.1
1	12	45.34	-22.8	10.2	HL-93	-63.6	HL-93	-12.6	-86.4
1	13	49.12	-28.1	6.0	HL-93	-71.0	HL-93	-22.1	-99.1
1	14	52.90	-33.2	2.6	HL-93	-78.5	HL-93	-30.6	-111.7
1	15	56.68	-38.2	0.0	HL-93	-86.1	HL-93	-38.2	-124.3

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	124.4	150.2	200.6	166.1	57.3	25.8
1	1	3.78	111.7	135.2	180.6	149.2	49.8	24.4
1	2	7.56	99.1	120.4	160.9	132.5	42.2	22.9
1	3	11.34	86.4	105.5	141.1	115.6	34.2	21.6
1	4	15.11	74.1	91.1	121.8	99.3	26.7	20.5
1	5	18.89	62.0	76.8	102.8	83.1	19.1	19.4
1	6	22.67	50.0	62.7	84.1	67.2	11.6	18.8
1	7	26.45	38.1	48.7	65.5	51.3	4.0	19.0
1	8	30.23	-38.1	-48.7	-65.4	-51.3	-4.0	19.0
1	9	34.01	-50.0	-62.7	-84.1	-67.2	-11.5	18.8
1	10	37.79	-62.0	-76.8	-102.8	-83.1	-19.1	19.4
1	11	41.57	-74.1	-91.0	-121.8	-99.3	-26.6	20.5
1	12	45.34	-86.4	-105.5	-141.0	-115.6	-34.2	21.6
1	13	49.12	-99.1	-120.4	-160.9	-132.5	-42.2	22.9
1	14	52.90	-111.7	-135.2	-180.6	-149.2	-49.8	24.4
1	15	56.68	-124.3	-150.2	-200.6	-166.1	-57.3	25.8

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM

1	38.23	0.00	HL-93	86.14	HL-93	ST1	32.34	200.60
						ST2	32.34	166.14
						ST4	44.93	57.34
						SE1	38.23	124.37
						SE2	38.23	150.21
2	38.21	0.00	HL-93	86.14	HL-93	ST1	32.32	200.57
						ST2	32.32	166.12
						ST4	44.89	57.31
						SE1	38.21	124.35
						SE2	38.21	150.19

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.78	-0.0302	-0.1452	0.0000	-0.0431	-0.1786	-0.0431	-0.2217
1	2	7.56	-0.0585	-0.2812	0.0000	-0.0834	-0.3460	-0.0834	-0.4294
1	3	11.34	-0.0833	-0.4003	0.0000	-0.1185	-0.4925	-0.1185	-0.6109
1	4	15.11	-0.1040	-0.4999	0.0000	-0.1477	-0.6149	-0.1477	-0.7626
1	5	18.89	-0.1203	-0.5779	0.0000	-0.1706	-0.7109	-0.1706	-0.8815
1	6	22.67	-0.1315	-0.6316	0.0000	-0.1864	-0.7769	-0.1864	-0.9633
1	7	26.45	-0.1371	-0.6589	0.0000	-0.1944	-0.8106	-0.1944	-1.0049
1	8	30.23	-0.1371	-0.6589	0.0000	-0.1944	-0.8106	-0.1944	-1.0049
1	9	34.01	-0.1315	-0.6316	0.0000	-0.1864	-0.7769	-0.1864	-0.9633
1	10	37.79	-0.1203	-0.5779	0.0000	-0.1706	-0.7109	-0.1706	-0.8815
1	11	41.57	-0.1040	-0.4999	0.0000	-0.1477	-0.6149	-0.1477	-0.7626
1	12	45.34	-0.0833	-0.4003	0.0000	-0.1185	-0.4925	-0.1185	-0.6109
1	13	49.12	-0.0585	-0.2812	0.0000	-0.0834	-0.3460	-0.0834	-0.4294
1	14	52.90	-0.0302	-0.1452	0.0000	-0.0431	-0.1786	-0.0431	-0.2217
1	15	56.68	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.8	-0.030	0 1/16	-0.148	0 3/16	-0.043	0 1/16	-0.222	0 1/ 4
1	2	7.6	-0.059	0 1/16	-0.287	0 5/16	-0.083	0 1/ 8	-0.429	0 7/16
1	3	11.3	-0.083	0 1/ 8	-0.409	0 7/16	-0.118	0 1/ 8	-0.611	0 5/ 8

```
 1  4  15.1 -0.104  0   1/ 8 -0.511  0   9/16 -0.148  0   3/16 -0.763  0  13/16
 1  5  18.9 -0.120  0   1/ 8 -0.591  0   5/ 8 -0.171  0   3/16 -0.881  0  15/16
 1  6  22.7 -0.131  0   3/16 -0.645  0  11/16 -0.186  0   3/16 -0.963  1   0/16
 1  7  26.5 -0.137  0   3/16 -0.673  0  11/16 -0.194  0   1/ 4 -1.005  1   1/16
 1  8  30.2 -0.137  0   3/16 -0.673  0  11/16 -0.194  0   1/ 4 -1.005  1   1/16
 1  9  34.0 -0.131  0   3/16 -0.645  0  11/16 -0.186  0   3/16 -0.963  1   0/16
 1 10  37.8 -0.120  0   1/ 8 -0.591  0   5/ 8 -0.171  0   3/16 -0.881  0  15/16
 1 11  41.6 -0.104  0   1/ 8 -0.511  0   9/16 -0.148  0   3/16 -0.763  0  13/16
 1 12  45.3 -0.083  0   1/ 8 -0.409  0   7/16 -0.118  0   1/ 8 -0.611  0   5/ 8
 1 13  49.1 -0.059  0   1/16 -0.287  0   5/16 -0.083  0   1/ 8 -0.429  0   7/16
 1 14  52.9 -0.030  0   1/16 -0.148  0   3/16 -0.043  0   1/16 -0.222  0   1/ 4
 1 15  56.7  0.000    --      0.000    --      0.000    --      0.000    --


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 NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2
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     TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
                   ******************************************************
                    (UNFACTORED)

                  NUMBER OF LANE   LL + I.
           D FROM -----AND------ DEFLECTION  GOVERN.  1/800 OF SPAN L  ROTATION
     SPAN  L SUPT  DIST. FACTOR  ------------  LOAD   ---------------------
      NO.   (ft)   FOR LL DEFL.     (inch)     TYPE   AASHTO 2.5.2.6.2  [5] Rad.
     ---------------------------------------------------------------------

       1    28.34   3    0.425     -0.298 MAX HL-93     0.85            0.00258
                                   0.026 MIN
                                  -0.083 MAX LANE
                                   0.026 MIN LANE

                    4    0.433     -0.304 MAX HL-93     0.85            0.00258
                                   0.027 MIN
                                  -0.084 MAX LANE
                                   0.027 MIN LANE

                    3    0.425     -0.206 MAX FA
                                   0.000 MIN FA

     ----------------------------------------------------------------------
     NOTE: [1] " - " indicates downward deflection

           [2] The distribution factor for LL+I deflection is defined as

               DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

               where NL= no. of traffic lanes
                     Ng= no. of girders
                     (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

           [3] This table is based upon the optional criteria specified
               in AASHTO LRFD Art. 3.6.1.3.2

           [4] The number of traffic lanes is determined according to
               AASHTO LRFD Art.3.6.1.1.1.
               The 1st line is for the most probable number of lanes
               and the 2nd line is for the next probable number of lanes.

           [5] Max rotations at left (1st line) & right (2nd line)
               supports of the span without averaging, factor and impact

           [6] If ADTT is between 100 and 1000, multi-presence
               factor of 0.95 is applied. If ADTT is below 100, factor
               is 0.9 (AASHTO LRFD C3.6.1.1.2).

           [7] For truck rating the most probable number of lanes is assumed for avereaging.
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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	-0.45	0.45	-2.20	2.20	-2.65	2.65
1	2	7.56	-0.83	0.83	-4.09	4.09	-4.92	4.92
1	3	11.34	-1.04	0.72	-5.10	3.56	-6.14	4.29
1	4	15.11	-1.27	0.89	-6.23	4.35	-7.50	5.23
1	5	18.89	-1.44	1.01	-7.08	4.94	-8.52	5.95
1	6	22.67	-1.56	1.09	-7.65	5.34	-9.20	6.43
1	7	26.45	-1.62	1.13	-7.94	5.54	-9.55	6.67
1	8	30.23	-1.62	1.13	-7.93	5.54	-9.55	6.67
1	9	34.01	-1.56	1.09	-7.64	5.34	-9.20	6.42
1	10	37.79	-1.44	1.01	-7.08	4.94	-8.52	5.95
1	11	41.57	-1.27	0.89	-6.23	4.35	-7.50	5.24
1	12	45.34	-1.04	0.72	-5.11	3.57	-6.15	4.29
1	13	49.12	-0.83	0.83	-4.09	4.09	-4.92	4.92
1	14	52.90	-0.45	0.45	-2.20	2.20	-2.64	2.64
1	15	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	-0.02	-0.33	1.00	-0.02	-2.97	3.64
1	2	7.56	-0.05	-0.61	1.85	-0.05	-5.53	6.77
1	3	11.34	-0.05	-0.85	1.64	-0.05	-6.99	5.93
1	4	15.11	-0.07	-1.04	2.00	-0.07	-8.53	7.24
1	5	18.89	-0.07	-1.18	2.28	-0.07	-9.70	8.22
1	6	22.67	-0.08	-1.27	2.46	-0.08	-10.48	8.88
1	7	26.45	-0.08	-1.32	2.55	-0.08	-10.87	9.22
1	8	30.23	-0.08	-1.32	2.55	-0.08	-10.87	9.22
1	9	34.01	-0.08	-1.27	2.46	-0.08	-10.47	8.88
1	10	37.79	-0.07	-1.18	2.28	-0.07	-9.70	8.22
1	11	41.57	-0.07	-1.04	2.00	-0.07	-8.54	7.24
1	12	45.34	-0.05	-0.85	1.64	-0.05	-7.00	5.93
1	13	49.12	-0.05	-0.61	1.85	-0.05	-5.52	6.77
1	14	52.90	-0.02	-0.33	1.00	-0.02	-2.97	3.64
1	15	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	-0.19	-0.39	4.74	0.00	0.00	0.00
1	2	7.56	-0.34	-0.71	8.69	0.00	0.00	0.00
1	3	11.34	-0.39	-1.31	7.60	0.00	0.00	0.00
1	4	15.11	-0.46	-1.57	9.08	0.00	0.00	0.00
1	5	18.89	-0.52	-1.76	10.19	0.00	0.00	0.00
1	6	22.67	-0.55	-1.87	10.87	0.00	0.00	0.00
1	7	26.45	-0.56	-1.90	11.03	0.00	0.00	0.00
1	8	30.23	-0.56	-1.90	11.03	0.00	0.00	0.00
1	9	34.01	-0.55	-1.87	10.87	0.00	0.00	0.00
1	10	37.79	-0.52	-1.76	10.19	0.00	0.00	0.00
1	11	41.57	-0.46	-1.57	9.08	0.00	0.00	0.00
1	12	45.34	-0.39	-1.31	7.60	0.00	0.00	0.00
1	13	49.12	-0.34	-0.71	8.69	0.00	0.00	0.00
1	14	52.90	-0.19	-0.39	4.74	0.00	0.00	0.00
1	15	56.68	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK ONLY	TOP OF TOP FLANGE GOVERNING STRESS RANGE	TOP OF TOP FLANGE GOVERNING LOAD TP	TOP OF TOP FLANGE GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK ONLY	BOT OF BOT FLANGE GOVERNING STRESS RANGE	BOT OF BOT FLANGE GOVERNING LOAD TP	BOT OF BOT FLANGE GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	3.78	0.1	0.0	0.1	C HL	1.6	0.0	1.6	T HL
1	2	7.56	0.2	0.0	0.2	C HL	2.9	0.0	2.9	T HL
1	3	11.34	0.4	0.0	0.4	C HL	2.5	0.0	2.5	T HL
1	4	15.11	0.5	0.0	0.5	C HL	2.9	0.0	2.9	T HL
1	5	18.89	0.6	0.0	0.6	C HL	3.2	0.0	3.2	T HL
1	6	22.67	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	7	26.45	0.6	0.0	0.6	C HL	3.2	0.0	3.2	T HL
1	8	30.23	0.6	0.0	0.6	C HL	3.2	0.0	3.2	T HL
1	9	34.01	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	10	37.79	0.6	0.0	0.6	C HL	3.2	0.0	3.2	T HL
1	11	41.57	0.5	0.0	0.5	C HL	2.9	0.0	2.9	T HL
1	12	45.34	0.4	0.0	0.4	C HL	2.5	0.0	2.5	T HL
1	13	49.12	0.2	0.0	0.2	C HL	2.9	0.0	2.9	T HL
1	14	52.90	0.1	0.0	0.1	C HL	1.6	0.0	1.6	T HL
1	15	56.68	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY

YIELD TOTAL POSITIVE ,(ksi) TOTAL NEGATIVE ,(ksi)

SP NO	IN NO	D FROM L SUPT (ft)	STRESS Fy (ksi) TOP	STRESS Fy (ksi) BOT	CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.21	-3.36	8.39	-0.02	-2.97	3.64
1	2	7.56	33.	33.	-0.39	-6.24	15.47	-0.05	-5.53	6.77
1	3	11.34	33.	33.	-0.44	-8.30	13.53	-0.05	-6.99	5.93
1	4	15.11	33.	33.	-0.53	-10.10	16.32	-0.07	-8.53	7.24
1	5	18.89	33.	33.	-0.59	-11.45	18.41	-0.07	-9.70	8.22
1	6	22.67	33.	33.	-0.63	-12.35	19.75	-0.08	-10.48	8.88
1	7	26.45	33.	33.	-0.64	-12.77	20.24	-0.08	-10.87	9.22
1	8	30.23	33.	33.	-0.64	-12.77	20.24	-0.08	-10.87	9.22
1	9	34.01	33.	33.	-0.63	-12.35	19.75	-0.08	-10.47	8.88
1	10	37.79	33.	33.	-0.59	-11.45	18.41	-0.07	-9.70	8.22
1	11	41.57	33.	33.	-0.53	-10.10	16.32	-0.07	-8.54	7.24
1	12	45.34	33.	33.	-0.44	-8.31	13.53	-0.05	-7.00	5.93
1	13	49.12	33.	33.	-0.39	-6.23	15.46	-0.05	-5.52	6.77
1	14	52.90	33.	33.	-0.21	-3.36	8.39	-0.02	-2.97	3.64
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.27	-3.48	9.81	-0.02	-2.97	3.64
1	2	7.56	33.	33.	-0.49	-6.45	18.08	-0.05	-5.53	6.77
1	3	11.34	33.	33.	-0.56	-8.69	15.81	-0.05	-6.99	5.93
1	4	15.11	33.	33.	-0.67	-10.57	19.04	-0.07	-8.53	7.24
1	5	18.89	33.	33.	-0.75	-11.98	21.47	-0.07	-9.70	8.22
1	6	22.67	33.	33.	-0.80	-12.91	23.01	-0.08	-10.48	8.88
1	7	26.45	33.	33.	-0.81	-13.34	23.55	-0.08	-10.87	9.22
1	8	30.23	33.	33.	-0.81	-13.34	23.55	-0.08	-10.87	9.22
1	9	34.01	33.	33.	-0.80	-12.91	23.01	-0.08	-10.47	8.88
1	10	37.79	33.	33.	-0.75	-11.98	21.47	-0.07	-9.70	8.22
1	11	41.57	33.	33.	-0.67	-10.57	19.05	-0.07	-8.54	7.24
1	12	45.34	33.	33.	-0.56	-8.70	15.81	-0.05	-7.00	5.93
1	13	49.12	33.	33.	-0.49	-6.44	18.07	-0.05	-5.52	6.77
1	14	52.90	33.	33.	-0.27	-3.47	9.81	-0.02	-2.97	3.64
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.36	-4.44	13.02	-0.03	-3.77	4.72

1	2	7.56	33.	33.	-0.66	-8.25	23.98	-0.06	-7.01	8.77
1	3	11.34	33.	33.	-0.75	-11.17	20.97	-0.08	-8.87	7.67
1	4	15.11	33.	33.	-0.90	-13.57	25.26	-0.09	-10.83	9.37
1	5	18.89	33.	33.	-1.01	-15.38	28.47	-0.11	-12.31	10.64
1	6	22.67	33.	33.	-1.08	-16.58	30.52	-0.11	-13.30	11.50
1	7	26.45	33.	33.	-1.10	-17.13	31.23	-0.12	-13.80	11.93
1	8	30.23	33.	33.	-1.10	-17.12	31.22	-0.12	-13.80	11.93
1	9	34.01	33.	33.	-1.08	-16.58	30.52	-0.11	-13.30	11.50
1	10	37.79	33.	33.	-1.01	-15.38	28.47	-0.11	-12.31	10.64
1	11	41.57	33.	33.	-0.90	-13.58	25.26	-0.09	-10.84	9.37
1	12	45.34	33.	33.	-0.75	-11.17	20.97	-0.08	-8.88	7.68
1	13	49.12	33.	33.	-0.66	-8.24	23.97	-0.06	-7.00	8.76
1	14	52.90	33.	33.	-0.36	-4.44	13.02	-0.03	-3.77	4.71
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.34	-3.30	11.42	-0.02	-2.62	3.12
1	2	7.56	33.	33.	-0.63	-6.12	21.01	-0.03	-4.88	5.80
1	3	11.34	33.	33.	-0.72	-8.45	18.37	-0.04	-6.16	5.07
1	4	15.11	33.	33.	-0.86	-10.25	22.09	-0.05	-7.51	6.19
1	5	18.89	33.	33.	-0.96	-11.61	24.86	-0.06	-8.54	7.04
1	6	22.67	33.	33.	-1.03	-12.50	26.62	-0.06	-9.22	7.60
1	7	26.45	33.	33.	-1.04	-12.90	27.18	-0.06	-9.57	7.89
1	8	30.23	33.	33.	-1.04	-12.90	27.18	-0.06	-9.57	7.89
1	9	34.01	33.	33.	-1.03	-12.50	26.62	-0.06	-9.22	7.60
1	10	37.79	33.	33.	-0.96	-11.61	24.86	-0.06	-8.54	7.04
1	11	41.57	33.	33.	-0.86	-10.26	22.09	-0.05	-7.52	6.19
1	12	45.34	33.	33.	-0.72	-8.45	18.37	-0.04	-6.16	5.07
1	13	49.12	33.	33.	-0.63	-6.11	21.01	-0.03	-4.87	5.80
1	14	52.90	33.	33.	-0.34	-3.30	11.42	-0.02	-2.62	3.12
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.29	-4.29	11.12	-0.03	-3.77	4.72
1	2	7.56	33.	33.	-0.52	-7.96	20.50	-0.06	-7.01	8.77
1	3	11.34	33.	33.	-0.60	-10.64	17.93	-0.08	-8.87	7.67
1	4	15.11	33.	33.	-0.72	-12.95	21.63	-0.09	-10.83	9.37
1	5	18.89	33.	33.	-0.81	-14.68	24.40	-0.11	-12.31	10.64
1	6	22.67	33.	33.	-0.86	-15.83	26.17	-0.11	-13.30	11.50
1	7	26.45	33.	33.	-0.88	-16.37	26.82	-0.12	-13.80	11.93
1	8	30.23	33.	33.	-0.88	-16.36	26.81	-0.12	-13.80	11.93
1	9	34.01	33.	33.	-0.86	-15.83	26.17	-0.11	-13.30	11.50
1	10	37.79	33.	33.	-0.81	-14.68	24.40	-0.11	-12.31	10.64

1	11	41.57	33.	33.	-0.72	-12.95	21.63	-0.09	-10.84	9.37
1	12	45.34	33.	33.	-0.60	-10.65	17.93	-0.08	-8.88	7.68
1	13	49.12	33.	33.	-0.52	-7.96	20.50	-0.06	-7.00	8.76
1	14	52.90	33.	33.	-0.29	-4.29	11.12	-0.03	-3.77	4.71
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.78	33.	33.	-0.27	-3.14	9.53	-0.02	-2.62	3.12
1	2	7.56	33.	33.	-0.49	-5.83	17.54	-0.03	-4.88	5.80
1	3	11.34	33.	33.	-0.56	-7.92	15.33	-0.04	-6.16	5.07
1	4	15.11	33.	33.	-0.67	-9.63	18.45	-0.05	-7.51	6.19
1	5	18.89	33.	33.	-0.75	-10.91	20.79	-0.06	-8.54	7.04
1	6	22.67	33.	33.	-0.81	-11.75	22.27	-0.06	-9.22	7.60
1	7	26.45	33.	33.	-0.82	-12.14	22.77	-0.06	-9.57	7.89
1	8	30.23	33.	33.	-0.82	-12.14	22.77	-0.06	-9.57	7.89
1	9	34.01	33.	33.	-0.81	-11.75	22.27	-0.06	-9.22	7.60
1	10	37.79	33.	33.	-0.75	-10.91	20.79	-0.06	-8.54	7.04
1	11	41.57	33.	33.	-0.67	-9.63	18.45	-0.05	-7.52	6.19
1	12	45.34	33.	33.	-0.56	-7.93	15.33	-0.04	-6.16	5.07
1	13	49.12	33.	33.	-0.49	-5.83	17.53	-0.03	-4.87	5.80
1	14	52.90	33.	33.	-0.27	-3.14	9.52	-0.02	-2.62	3.12
1	15	56.68	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	10.84	222.	-6.6	113.	-0.8	575.	-0.9	0.	0.0	-8.30
											0.93
	LB			6.6		2.5		11.4		0.0	20.46
											11.43
	RT			-5.9		-0.8		-1.3		0.0	-8.00
											1.26
	RB			4.1		1.6		7.3		0.0	13.05
											7.33
1	LT	45.84	222.	-5.9	113.	-0.8	575.	-1.3	0.	0.0	-8.00
											1.26
	LB			4.1		1.6		7.3		0.0	13.05
											7.33
	RT			-6.6		-0.8		-0.9		0.0	-8.30
											0.93
	RB			6.6		2.5		11.4		0.0	20.47
											11.43

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange

RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	10.84	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	35.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	10.00	1.0000
3	10.84	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 56.7

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1]) [3]= GIRDER D. [3]/(12*[1])		<=>	LIMITATION 1/25 1/30	STATUS
1	10.84	6.75	33.09	[2]=	41.39	0.06085	>	0.04000	OK
				[3]=	33.09	0.04865	>	0.03300	OK
2	35.00	6.75	33.09	[2]=	41.39	0.06085	>	0.04000	OK
				[3]=	33.09	0.04865	>	0.03300	OK
3	10.84	6.75	33.09	[2]=	41.39	0.06085	>	0.04000	OK
				[3]=	33.09	0.04865	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	2Dcp/tw [3]	2Dc/tw [4]	2Dc/tw [5]	Web Cat [6]
1	0	0.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	1	3.78	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	2	7.56	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	3	11.34	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	4	15.11	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	5	18.89	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	6	22.67	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	7	26.45	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	8	30.23	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	9	34.01	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	10	37.79	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	11	41.57	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	12	45.34	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	13	49.12	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	14	52.90	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0
1	15	56.68	0	1	31.39	0.579	54.2	150.0	0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	3.78	0	1	31.39	0.579	54.21	168.97	2
1	2	7.56	0	1	31.39	0.579	54.21	168.97	2
1	3	11.34	0	1	31.39	0.579	66.55	168.97	2
1	4	15.11	0	1	31.39	0.579	66.55	168.97	2
1	5	18.89	0	1	31.39	0.579	66.55	168.97	2
1	6	22.67	0	1	31.39	0.579	66.55	168.97	2
1	7	26.45	0	1	31.39	0.579	66.55	168.97	2
1	8	30.23	0	1	31.39	0.579	66.55	168.97	2
1	9	34.01	0	1	31.39	0.579	66.55	168.97	2
1	10	37.79	0	1	31.39	0.579	66.55	168.97	2
1	11	41.57	0	1	31.39	0.579	66.55	168.97	2
1	12	45.34	0	1	31.39	0.579	66.55	168.97	2
1	13	49.12	0	1	31.39	0.579	54.21	168.97	2
1	14	52.90	0	1	31.39	0.579	54.21	168.97	2
1	15	56.68	0	1	31.39	0.579	54.21	168.97	2

```
-------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ------------------------------------------------------------

  1  0   0.00   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  1   3.78   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  2   7.56   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  3  11.34   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  4  15.11   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  5  18.89   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  6  22.67   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  7  26.45   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  8  30.23   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1  9  34.01   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 10  37.79   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 11  41.57   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 12  45.34   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 13  49.12   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 14  52.90   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0
  1 15  56.68   6.76 12.  11.5   5.2  0.850  0.637
                6.76 12.  11.5   5.2  0.850  0.637    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************
```

LATERAL BRACING NO	DIST. FROM LEFT SUPT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	8.8	0.000	7.966	4.513	1.061	1.615
			0.000	0.000	0.000	0.000	1.000
2	8.8	27.6	7.966	14.328	12.487	10.647	1.135
			0.000	0.000	0.000	0.000	1.000
3	27.6	46.4	8.685	14.328	12.984	11.640	1.095
			0.000	0.000	0.000	0.000	1.000
4	46.4	56.7	0.000	8.685	5.177	1.669	1.559
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	Lb (ft)	Lp (ft)	Lr (ft)	GOV CAT	CAT
1	0		0.00	1	6.76	11.26	19.84	0	8.76	7.17	26.91	2	2
1	1		3.78	1	6.76	11.26	19.84	0	8.76	7.17	26.91	2	2
1	2		7.56	1	6.76	11.26	19.84	0	8.76	7.17	26.91	2	2
1	3		11.34	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	4		15.11	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	5		18.89	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	6		22.67	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	7		26.45	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	8		30.23	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	9		34.01	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	10		37.79	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	11		41.57	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	12		45.34	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	13		49.12	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2
1	14		52.90	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2
1	15		56.68	1	6.76	11.26	19.84	0	10.24	7.17	26.91	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category

0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.62	33.00	33.00	33.00
1	1		3.78	1	1.000	1.000	1.62	33.00	33.00	33.00
1	2		7.56	1	1.000	1.000	1.62	33.00	33.00	33.00
1	3		11.34	1	1.000	1.000	1.14	33.00	30.54	30.54
1	4		15.11	1	1.000	1.000	1.14	33.00	30.54	30.54
1	5		18.89	1	1.000	1.000	1.14	33.00	30.54	30.54
1	6		22.67	1	1.000	1.000	1.14	33.00	30.54	30.54
1	7		26.45	1	1.000	1.000	1.14	33.00	30.54	30.54
1	8		30.23	1	1.000	1.000	1.09	33.00	29.45	29.45
1	9		34.01	1	1.000	1.000	1.09	33.00	29.45	29.45
1	10		37.79	1	1.000	1.000	1.09	33.00	29.45	29.45
1	11		41.57	1	1.000	1.000	1.09	33.00	29.45	29.45
1	12		45.34	1	1.000	1.000	1.09	33.00	29.45	29.45
1	13		49.12	1	1.000	1.000	1.56	33.00	33.00	33.00
1	14		52.90	1	1.000	1.000	1.56	33.00	33.00	33.00
1	15		56.68	1	1.000	1.000	1.56	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	5.17	17.38
1	1		3.78	1	5.17	17.38
1	2		7.56	1	5.17	17.38
1	3		11.34	1	6.54	17.80
1	4		15.11	1	6.54	17.80
1	5		18.89	1	6.54	17.80
1	6		22.67	1	6.54	17.80
1	7		26.45	1	6.54	17.80
1	8		30.23	1	6.54	17.80
1	9		34.01	1	6.54	17.80
1	10		37.79	1	6.54	17.80
1	11		41.57	1	6.54	17.80
1	12		45.34	1	6.54	17.80
1	13		49.12	1	5.17	17.38
1	14		52.90	1	5.17	17.38
1	15		56.68	1	5.17	17.38

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.78	1	0	2	0		2	0
1	2	7.56	1	0	2	0		2	0
1	3	11.34	1	0	2	0		2	0
1	4	15.11	1	0	2	0		2	0
1	5	18.89	1	0	2	0		2	0
1	6	22.67	1	0	2	0		2	0
1	7	26.45	1	0	2	0		2	0
1	8	30.23	1	0	2	0		2	0
1	9	34.01	1	0	2	0		2	0
1	10	37.79	1	0	2	0		2	0
1	11	41.57	1	0	2	0		2	0
1	12	45.34	1	0	2	0		2	0
1	13	49.12	1	0	2	0		2	0
1	14	52.90	1	0	2	0		2	0
1	15	56.68	1	0	2	0		2	0

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1]	fbu+fl (ksi)	[2]	fbu+1/3fl (ksi)	[3]	FLAG (ksi)

1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	3.78	0.0	19.8	4.0	33.0	4.0	33.0	4.0	33.0	0
			0.0	-	4.0	-	4.0	33.0	-	-	0
1	2	7.56	0.0	19.8	7.4	33.0	7.4	33.0	7.4	33.0	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	3	11.34	0.0	19.8	9.2	33.0	9.2	33.0	9.2	30.5	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	4	15.11	0.0	19.8	11.2	33.0	11.2	33.0	11.2	30.5	0
			0.0	-	7.9	-	7.9	33.0	-	-	0
1	5	18.89	0.0	19.8	12.8	33.0	12.8	33.0	12.8	30.5	0
			0.0	-	8.9	-	8.9	33.0	-	-	0
1	6	22.67	0.0	19.8	13.8	33.0	13.8	33.0	13.8	30.5	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	7	26.45	0.0	19.8	14.3	33.0	14.3	33.0	14.3	30.5	0
			0.0	-	10.0	-	10.0	33.0	-	-	0
1	8	30.23	0.0	19.8	14.3	33.0	14.3	33.0	14.3	29.4	0
			0.0	-	10.0	-	10.0	33.0	-	-	0
1	9	34.01	0.0	19.8	13.8	33.0	13.8	33.0	13.8	29.4	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	10	37.79	0.0	19.8	12.8	33.0	12.8	33.0	12.8	29.4	0
			0.0	-	8.9	-	8.9	33.0	-	-	0
1	11	41.57	0.0	19.8	11.2	33.0	11.2	33.0	11.2	29.4	0
			0.0	-	7.9	-	7.9	33.0	-	-	0
1	12	45.34	0.0	19.8	9.2	33.0	9.2	33.0	9.2	29.4	0
			0.0	-	6.4	-	6.4	33.0	-	-	0
1	13	49.12	0.0	19.8	7.4	33.0	7.4	33.0	7.4	33.0	0
			0.0	-	7.4	-	7.4	33.0	-	-	0
1	14	52.90	0.0	19.8	4.0	33.0	4.0	33.0	4.0	33.0	0
			0.0	-	4.0	-	4.0	33.0	-	-	0
1	15	56.68	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.78	0.000	0.120	0.120	0.120	0.120	2
					0.120		0.120	3
1	2	7.56	0.000	0.224	0.224	0.224	0.224	2
					0.224		0.224	3
1	3	11.34	0.000	0.279	0.279	0.302	0.302	4
					0.195		0.195	3
1	4	15.11	0.000	0.341	0.341	0.368	0.368	4
					0.238		0.238	3

1	5	18.89	0.000	0.387	0.387	0.418	0.418	4
					0.270		0.270	3
1	6	22.67	0.000	0.418	0.418	0.452	0.452	4
					0.292		0.292	3
1	7	26.45	0.000	0.434	0.434	0.469	0.469	4
					0.303		0.303	3
1	8	30.23	0.000	0.434	0.434	0.486	0.486	4
					0.303		0.303	3
1	9	34.01	0.000	0.418	0.418	0.469	0.469	4
					0.292		0.292	3
1	10	37.79	0.000	0.387	0.387	0.434	0.434	4
					0.270		0.270	3
1	11	41.57	0.000	0.341	0.341	0.382	0.382	4
					0.238		0.238	3
1	12	45.34	0.000	0.279	0.279	0.313	0.313	4
					0.195		0.195	3
1	13	49.12	0.000	0.224	0.224	0.224	0.224	2
					0.224		0.224	3
1	14	52.90	0.000	0.120	0.120	0.120	0.120	2
					0.120		0.120	3
1	15	56.68	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0
1	1		3.78	0	593.8	2269.4	-	-	0.0	19.8	-	-	0--0
1	2		7.56	0	1092.7	2269.4	-	-	0.0	19.8	-	-	0--0
1	3		11.34	0	1495.8	3175.4	-	-	0.0	19.8	-	-	0--0
1	4		15.11	0	1799.4	3175.4	-	-	0.0	19.8	-	-	0--0
1	5		18.89	0	2026.3	3175.4	-	-	0.0	19.8	-	-	0--0
1	6		22.67	0	2170.4	3175.4	-	-	0.0	19.8	-	-	0--0
1	7		26.45	0	2217.6	3175.4	-	-	0.0	19.8	-	-	0--0
1	8		30.23	0	2217.4	3175.4	-	-	0.0	19.8	-	-	0--0
1	9		34.01	0	2170.3	3175.4	-	-	0.0	19.8	-	-	0--0
1	10		37.79	0	2026.3	3175.4	-	-	0.0	19.8	-	-	0--0
1	11		41.57	0	1799.5	3175.4	-	-	0.0	19.8	-	-	0--0
1	12		45.34	0	1496.0	3175.4	-	-	0.0	19.8	-	-	0--0
1	13		49.12	0	1092.5	2269.4	-	-	0.0	19.8	-	-	0--0
1	14		52.90	0	593.7	2269.4	-	-	0.0	19.8	-	-	0--0
1	15		56.68	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in

```
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
            = 0.6*Fyc for comp. flange of composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
        [4] = (PHI)f * Fnt for non-compact tension flange
              of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
            = (PHI)f * Fnc for comp. flange of composite sections
              in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

        "-" is N.A.

        Under FLAG Column, 0 = OK; 1= NG
        M = Moment; C = Comp. Flange; T = Tension Flange
        D = Ductility

        For negative moment region or non-compact sections in
        positive moment region, the 1st line is for top
        flange and the 2nd line is for bottom flange

        The values of fbu, Mu and fl shall be determined based on
        factored loads, and shall be taken as positive in
        sign in all resistance equations (Art. 6.10.1.6)

        The value of fbu is the actual stress in this table,
        the users can use the maximum value within the
        unbraced length to do their own check
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   TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                   ***********************************************

   SP IN D FROM ID Mu+1/3flSxt/       fbu/        fl/ fbu+1/3fl/  MAX.
                           [1]        [2]        [3]        [4]  RAT. GOVN.
   ---------------------------------------------------------------------------

    1  0   0.00  0        0.000                                   0.000     1
    1  1   3.78  0        0.262                                   0.262     1
    1  2   7.56  0        0.482                                   0.482     1
    1  3  11.34  0        0.471                                   0.471     1
    1  4  15.11  0        0.567                                   0.567     1
    1  5  18.89  0        0.638                                   0.638     1
    1  6  22.67  0        0.684                                   0.684     1
    1  7  26.45  0        0.698                                   0.698     1
    1  8  30.23  0        0.698                                   0.698     1
    1  9  34.01  0        0.683                                   0.683     1
    1 10  37.79  0        0.638                                   0.638     1
    1 11  41.57  0        0.567                                   0.567     1
    1 12  45.34  0        0.471                                   0.471     1
    1 13  49.12  0        0.481                                   0.481     1
    1 14  52.90  0        0.262                                   0.262     1
    1 15  56.68  0        0.000                                   0.000     1


   ---------------------------------------------------------------------------

   NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
         [2] = (PHI)f * Fnc for comp. flange of composite
               sections in positive flexure (Eq. 6.10.7.2.1-1)
             = (PHI)f * Rh * Fyt for tension flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
         [3] = 0.6*Fyt for composite sections in positive flexure
               or tension flange for composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
             = 0.6*Fyc for comp. flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
         [4] = (PHI)f * Fnt for non-compact tension flange
               of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
             = (PHI)f * Fnc for comp. flange of composite sections
               in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

         The governing number is listed as below.
           1 = (Mu+1/3flSxt) / [1]
           2 = fbu / [2]
           3 = fl / [3]
           4 = (fbu + 1/3fl) / [4]
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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.78	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	7.56	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	11.34	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	15.11	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	18.89	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	22.67	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	26.45	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	30.23	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	34.01	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	37.79	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	41.57	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	45.34	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	49.12	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	14	52.90	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	15	56.68	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0

              For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-6

              For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-7


           ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
              stiffened
              if (d0/D) < 3 without long. stiffener, or
              if (d0/D) > 3 with both transv. and long. stiffeners.
              For handling requirement, (d0/D) > (260/(D/tw))**2

                   Vu = CVp

              where D = clear, unsupported distance between flange components

                   d0 = distance between transverse stiffeners

           ** This Article (6.10.9.2) indicates that a designer cannot
              count on post-buckling shear resistance from tension-
              field action for an unstiffened girder.
```

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
			S T R E N G T H C A T E G O R Y; SEE NOTE					
1	0	0.00	0	200.6	<	347.9	0	0
1	1	3.78	0	180.6	<	347.9	0	0
1	2	7.56	0	160.9	<	347.9	0	0
1	3	11.34	0	141.1	<	347.9	0	0
1	4	15.11	0	121.8	<	347.9	0	0
1	5	18.89	0	102.8	<	347.9	0	0
1	6	22.67	0	84.1	<	347.9	0	0
1	7	26.45	0	65.5	<	347.9	0	0
1	8	30.23	0	65.4	<	347.9	0	0
1	9	34.01	0	84.1	<	347.9	0	0
1	10	37.79	0	102.8	<	347.9	0	0
1	11	41.57	0	121.8	<	347.9	0	0
1	12	45.34	0	141.0	<	347.9	0	0
1	13	49.12	0	160.9	<	347.9	0	0
1	14	52.90	0	180.6	<	347.9	0	0
1	15	56.68	0	200.6	<	347.9	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
  please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
  2 = braced non-compact section
  3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN D NO	L FROM SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	200.60	347.87	0	
1	1	3.78	33.	180.63	347.87	0	
1	2	7.56	33.	160.90	347.87	0	
1	3	11.34	33.	141.06	347.87	0	
1	4	15.11	33.	121.83	347.87	0	
1	5	18.89	33.	102.85	347.87	0	
1	6	22.67	33.	84.11	347.87	0	
1	7	26.45	33.	65.45	347.87	0	
1	8	30.23	33.	65.42	347.87	0	
1	9	34.01	33.	84.08	347.87	0	
1	10	37.79	33.	102.82	347.87	0	
1	11	41.57	33.	121.80	347.87	0	
1	12	45.34	33.	141.03	347.87	0	
1	13	49.12	33.	160.87	347.87	0	
1	14	52.90	33.	180.60	347.87	0	
1	15	56.68	33.	200.57	347.87	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D NO	L FROM SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	TOP FLANGE [1] (ksi)	BOT. FLANGE ff+fl/2 (ksi)	BOT. FLANGE [2] (ksi)	COMP. FLG fc (ksi)	COMP. FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	3.78	3.5	-	31.4	9.8	31.4	-	-	00-
1	2	7.56	6.4	-	31.4	18.1	31.4	-	-	00-
1	3	11.34	8.7	-	31.4	15.8	31.4	-	-	00-
1	4	15.11	10.6	-	31.4	19.0	31.4	-	-	00-
1	5	18.89	12.0	-	31.4	21.5	31.4	-	-	00-
1	6	22.67	12.9	-	31.4	23.0	31.4	-	-	00-
1	7	26.45	13.3	-	31.4	23.6	31.4	-	-	00-
1	8	30.23	13.3	-	31.4	23.5	31.4	-	-	00-
1	9	34.01	12.9	-	31.4	23.0	31.4	-	-	00-
1	10	37.79	12.0	-	31.4	21.5	31.4	-	-	00-
1	11	41.57	10.6	-	31.4	19.0	31.4	-	-	00-
1	12	45.34	8.7	-	31.4	15.8	31.4	-	-	00-
1	13	49.12	6.4	-	31.4	18.1	31.4	-	-	00-
1	14	52.90	3.5	-	31.4	9.8	31.4	-	-	00-
1	15	56.68	0.0	-	31.4	0.0	31.4	-	-	00-

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     NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
               = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
           [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
               = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
           [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                 but not to exceed the smaller of RhFyc and Fyw/0.7
                 k = bending-buckling coefficient
                   = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                 where:
                 Dc = depth of the web in compression in the elastic
                      range. For composite sections, Dc shall be
                      determined as specified in Article D6.3.1

           "-" is N.A.

           Flag check - 0 = OK; 1 = NG
           T = Top Flange; B = Bottom Flange; C = Comp. Flange

           The values of fl shall be determined based on
           factored loads, and shall be taken as positive in
           sign in all resistance equations (Art. 6.10.1.6)
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                ---------------------------- ----------------------------
SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ki)    CATEGORY             (ki)    CATEGORY
-------------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   3.78    TR     -0.2 A B B^C^C D E E^  TR      2.5 A B B^C^C D E E^
 1  2   7.56    TR     -0.4 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1     10.84  SCG
 1  3  11.34    TR     -0.6 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1  4  15.11    TR     -0.8 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1  5  18.89    TR     -0.8 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1  6  22.67    TR     -0.9 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1  7  26.45    TR     -0.8 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1  8  30.23    TR     -0.8 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1  9  34.01    TR     -0.9 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 10  37.79    TR     -0.8 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1 11  41.57    TR     -0.8 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1 12  45.34    TR     -0.6 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1     45.84  SCG
 1 13  49.12    TR     -0.4 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1 14  52.90    TR     -0.2 A B B^C^C D E E^  TR      2.5 A B B^C^C D E E^
 1 15  56.68    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


-------------------------------------------------------------------------------
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.
-------------------------------------------------------------------------------
NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
```

POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.78	TR	-0.1	A B B^C^C D E E^	TR	1.2	A B B^C^C D E E^
1	2	7.56	TR	-0.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1		10.84	SCG					
1	3	11.34	TR	-0.3	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	4	15.11	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	5	18.89	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	6	22.67	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	7	26.45	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	8	30.23	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	9	34.01	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	10	37.79	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	11	41.57	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	12	45.34	TR	-0.3	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1		45.84	SCG					
1	13	49.12	TR	-0.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	14	52.90	TR	-0.1	A B B^C^C D E E^	TR	1.2	A B B^C^C D E E^
1	15	56.68	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT	Vf	Q/I	Vsr	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH----

		(ft)	[1]	[2]	[3]	[4],Zr <--Input	[5], input	(in),[6]
1	0	0.00	51.6	.29242E-01	1.5	TR	3.09	3	6.15
1	1	3.78	48.8	.29242E-01	1.4	TR	3.09	3	6.50
1	2	7.56	45.9	.29242E-01	1.3	TR	3.09	3	6.92
1	3	11.34	43.2	.26751E-01	1.2	TR	3.09	3	8.04
1	4	15.11	41.0	.26751E-01	1.1	TR	3.09	3	8.46
1	5	18.89	38.8	.26751E-01	1.0	TR	3.09	3	8.93
1	6	22.67	37.5	.26751E-01	1.0	TR	3.09	3	9.25
1	7	26.45	38.0	.26751E-01	1.0	TR	3.09	3	9.12
1	8	30.23	38.0	.26751E-01	1.0	TR	3.09	3	9.12
1	9	34.01	37.5	.26751E-01	1.0	TR	3.09	3	9.25
1	10	37.79	38.8	.26751E-01	1.0	TR	3.09	3	8.93
1	11	41.57	41.0	.26751E-01	1.1	TR	3.09	3	8.46
1	12	45.34	43.2	.26751E-01	1.2	TR	3.09	3	8.04
1	13	49.12	45.9	.29242E-01	1.3	TR	3.09	3	6.92
1	14	52.90	48.8	.29242E-01	1.4	TR	3.09	3	6.50
1	15	56.68	51.6	.29242E-01	1.5	TR	3.09	3	6.15

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	25.8	.29242E-01	0.8	TR 1.36	3	5.42
1	1	3.78	24.4	.29242E-01	0.7	TR 1.36	3	5.73
1	2	7.56	22.9	.29242E-01	0.7	TR 1.36	3	6.10
1	3	11.34	21.6	.26751E-01	0.6	TR 1.36	3	7.08
1	4	15.11	20.5	.26751E-01	0.5	TR 1.36	3	7.46
1	5	18.89	19.4	.26751E-01	0.5	TR 1.36	3	7.87
1	6	22.67	18.8	.26751E-01	0.5	TR 1.36	3	8.16
1	7	26.45	19.0	.26751E-01	0.5	TR 1.36	3	8.04
1	8	30.23	19.0	.26751E-01	0.5	TR 1.36	3	8.04
1	9	34.01	18.8	.26751E-01	0.5	TR 1.36	3	8.16
1	10	37.79	19.4	.26751E-01	0.5	TR 1.36	3	7.87
1	11	41.57	20.5	.26751E-01	0.5	TR 1.36	3	7.46
1	12	45.34	21.6	.26751E-01	0.6	TR 1.36	3	7.08
1	13	49.12	22.9	.29242E-01	0.7	TR 1.36	3	6.10
1	14	52.90	24.4	.29242E-01	0.7	TR 1.36	3	5.73
1	15	56.68	25.8	.29242E-01	0.8	TR 1.36	3	5.42

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2

```
            where ADTT Single Lane= 1140 & No. of Cycles=   31207500
        *** If ADTT Single Lane greater than or equal to 960, refer
            to Fatigue I Table.
1
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ***************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  7  26.45   1 1574.9  1626.6          159         >  82
      1 15  56.68   1 1574.9  1626.6          159         >  82


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
             Qr = factored shear resistance of one shear connector
                = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
             Qn = nominal shear resistance
             (Phi)sc = resistance factor for shear connectors

      [3] The total horizontal shear force between the point of
           maximum positive moment and each adjacent point of 0.0 moment
           shall be the lesser either:

                P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
           or
             P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
           Where
             f'c = specified 28-day compressive strength of the concrete
             bs = effective width of the slab
             bfc = width of compression flange
             bft = width of tension flange
             ts = slab thickness
             Fyw = specified minimum yield strength of the web
             Fyt = specified minimum yield strength of the tension flange
             Fyc = specified minimum yield strength of the compression flange
             D = web depth
             tft = thickness of tension flange
```

tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	26.451	6.000
1	26.451	56.680	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	56.7	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK

OK 1.2.22.17=TRANSVERSE STIFFENER SPACING

OK 1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.78	2269.4	89.5	45.5	238.8	0.0	6.49	HL-93	5.01	0
1	2	7.56	2269.4	166.4	84.5	437.5	0.0	3.29	HL-93	2.54	0
1	3	11.34	3175.4	230.0	116.9	596.2	0.0	3.38	HL-93	2.61	0
1	4	15.11	3175.4	280.7	142.9	712.5	0.0	2.73	HL-93	2.10	0
1	5	18.89	3175.4	319.0	162.4	799.2	0.0	2.36	HL-93	1.82	0
1	6	22.67	3175.4	344.6	175.4	852.7	0.0	2.17	HL-93	1.67	0
1	7	26.45	3175.4	357.7	181.9	865.0	0.0	2.12	HL-93	1.63	0
1	8	30.23	3175.4	357.6	181.9	865.0	0.0	2.12	HL-93	1.63	0
1	9	34.01	3175.4	344.5	175.4	852.7	0.0	2.17	HL-93	1.67	0
1	10	37.79	3175.4	319.0	162.4	799.2	0.0	2.36	HL-93	1.82	0
1	11	41.57	3175.4	280.8	142.9	712.5	0.0	2.73	HL-93	2.10	0
1	12	45.34	3175.4	230.2	116.9	596.2	0.0	3.38	HL-93	2.61	0
1	13	49.12	2269.4	166.2	84.5	437.5	0.0	3.29	HL-93	2.54	0
1	14	52.90	2269.4	89.4	45.5	238.8	0.0	6.49	HL-93	5.01	0
1	15	56.68	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L

5969 Span 2-S18 Interior Fatigue.RES
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD TYPE	RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.78	31.4	-2.97	3.64	-0.39	4.74	0.00	0.00	5.84	HL-93	4.49
1	2	7.56	31.4	-5.53	6.77	-0.71	8.69	0.00	0.00	2.83	HL-93	2.17
1	3	11.34	31.4	-6.99	5.93	-1.31	7.60	0.00	0.00	3.35	HL-93	2.57
1	4	15.11	31.4	-8.53	7.24	-1.57	9.08	0.00	0.00	2.66	HL-93	2.04
1	5	18.89	31.4	-9.70	8.22	-1.76	10.19	0.00	0.00	2.27	HL-93	1.75
1	6	22.67	31.4	-10.48	8.88	-1.87	10.87	0.00	0.00	2.07	HL-93	1.59
1	7	26.45	31.4	-10.87	9.22	-1.90	11.03	0.00	0.00	2.01	HL-93	1.54
1	8	30.23	31.4	-10.87	9.22	-1.90	11.03	0.00	0.00	2.01	HL-93	1.54
1	9	34.01	31.4	-10.47	8.88	-1.87	10.87	0.00	0.00	2.07	HL-93	1.59
1	10	37.79	31.4	-9.70	8.22	-1.76	10.19	0.00	0.00	2.27	HL-93	1.75
1	11	41.57	31.4	-8.54	7.24	-1.57	9.08	0.00	0.00	2.65	HL-93	2.04
1	12	45.34	31.4	-7.00	5.93	-1.31	7.60	0.00	0.00	3.35	HL-93	2.57
1	13	49.12	31.4	-5.52	6.77	-0.71	8.69	0.00	0.00	2.83	HL-93	2.17
1	14	52.90	31.4	-2.97	3.64	-0.39	4.74	0.00	0.00	5.84	HL-93	4.49
1	15	56.68	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	56.68	26.45	STRENGTH L.S.	2.12	1.63	HL-93
1	56.68	26.45	SERVICEBILITY	2.01	1.54	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.007
Inventory= 1.544

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	56.68	22.67	STRENGTH L.S.	3.06	817.33
1	56.68	22.67	SERVICEBILITY	2.77	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.765

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	25.3	12.9	86.1	0.0	2.56	HL-93	1.98
1	1	3.78	347.9	22.0	11.2	78.5	-2.6	2.87	HL-93	2.22
1	2	7.56	347.9	18.7	9.5	71.0	-6.0	3.25	HL-93	2.51
1	3	11.34	347.9	15.1	7.7	63.6	-10.2	3.71	HL-93	2.86
1	4	15.11	347.9	11.8	6.0	56.4	-14.6	4.27	HL-93	3.29
1	5	18.89	347.9	8.5	4.3	49.3	-19.2	4.98	HL-93	3.84
1	6	22.67	347.9	5.1	2.6	42.3	-23.8	5.91	HL-93	4.56
1	7	26.45	347.9	1.8	0.9	35.4	-29.0	7.20	HL-93	5.56
1	8	30.23	347.9	-1.8	-0.9	29.0	-35.4	7.20	HL-93	5.56
1	9	34.01	347.9	-5.1	-2.6	23.8	-42.3	5.91	HL-93	4.56
1	10	37.79	347.9	-8.4	-4.3	19.2	-49.3	4.98	HL-93	3.84
1	11	41.57	347.9	-11.7	-6.0	14.6	-56.4	4.27	HL-93	3.29
1	12	45.34	347.9	-15.1	-7.7	10.2	-63.6	3.71	HL-93	2.86
1	13	49.12	347.9	-18.7	-9.5	6.0	-71.0	3.25	HL-93	2.51
1	14	52.90	347.9	-22.0	-11.2	2.6	-78.5	2.87	HL-93	2.22
1	15	56.68	347.9	-25.3	-12.9	0.0	-86.1	2.56	HL-93	1.98

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	56.68	0.00	STRENGTH L.S.	2.56	1.98	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.563
Inventory= 1.977

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	56.68	0.00	STRENGTH L.S.	3.13	95.07

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.135

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	28.34	3 0.425	-0.30	HL-93	2.85
		4 0.433	-0.30		2.79

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

1

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span 2 - S19 Interior	8/20/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	20	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
58.91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130			0.00	0.0000	0.00	0.0000		
2	W	33	130			0.00	0.000013	.00	0.8750		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	8.95				33.	33.	33.
2	2	2	41.00				33.	33.	33.
3	1	1	8.95				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.43	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.61	0.00	0.00	0 0 0 0 0	0.88	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	58.91

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	13.83
2	2	Diaphragm	0.000	0.00	0.00	0.29	32.67
3	2	Diaphragm	0.000	0.00	0.00	0.29	51.51
4	1	Railing	0.087	0.00	58.91	0.00	0.00
5	1	Curb	0.076	0.00	58.91	0.00	0.00
6	0	Pavement	0.292	0.00	58.91	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE			SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	DIST. FROM TOP (in**2)	COMP. STRENG. AT 28 DAYS (in)	COMP. ALLOW -ABLE (ksi)	(ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	13.83	33.0	33.0	13.83
13.83	32.67	33.0	33.0	18.84
32.67	51.51	33.0	33.0	18.84
51.51	58.91	33.0	33.0	7.40

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

```
 F A C T O R S  U S E D  B Y  L R F D
-------------------------------------------------------

 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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    TABLE 1.2.2.1=LOADING INFORMATION
                  *******************

         AVERAGE DEAD LOAD INTENSITIES
         *****************************

       SPAN     SLAB    +   STEEL   =  TOTAL
        NO.    (K/FT)       (K/FT)    (K/FT)
       ---------------------------------------

         1      0.727       0.1572     0.8842


       ---------------------------------------




    SUPERIMPOSED DEAD LOADS
    ***********************


       LOAD       INTENSITY      DIST       DIST
                                 FROM        TO
               (K or K/Ft)       (Ft)       (Ft)
       ----------------------------------------

         1   DW       0.29        0.00      58.91

         1   DC2     0.087       0.000     58.910

         2   DC2     0.076       0.000     58.910


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1430.4	53.0	214.6	55.6	270.14

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.698	0.698	0.705	0.705	FOR STRENGTH POSITIVE MOMENT
	0.878				FOR STRENGTH POSITIVE SHEAR
1	0.430				FOR FATIGUE POSITIVE MOMENT
	0.613				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	2.95	6710.0	33.1	16.55	405.6	405.6
1	2	5.89	6710.0	33.1	16.55	405.6	405.6
1	3	8.84	6710.0	33.1	16.55	405.6	405.6
1	4	11.78	9247.5	33.1	13.53	683.5	452.5
1	5	14.73	9247.5	33.1	13.53	683.5	452.5
1	6	17.67	9247.5	33.1	13.53	683.5	452.5
1	7	20.62	9247.5	33.1	13.53	683.5	452.5
1	8	23.56	9247.5	33.1	13.53	683.5	452.5

1	9	26.51	9247.5	33.1	13.53	683.5	452.5
1	10	29.45	9247.5	33.1	13.53	683.5	452.5
1	11	32.40	9247.5	33.1	13.53	683.5	452.5
1	12	35.35	9247.5	33.1	13.53	683.5	452.5
1	13	38.29	9247.5	33.1	13.53	683.5	452.5
1	14	41.24	9247.5	33.1	13.53	683.5	452.5
1	15	44.18	9247.5	33.1	13.53	683.5	452.5
1	16	47.13	9247.5	33.1	13.53	683.5	452.5
1	17	50.07	6710.0	33.1	16.55	405.6	405.6
1	18	53.02	6710.0	33.1	16.55	405.6	405.6
1	19	55.96	6710.0	33.1	16.55	405.6	405.6
1	20	58.91	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	13641.5	0.023742	547.2	1671.8	828.8
1	1	2.95	13641.5	0.023742	547.2	1671.8	828.8
1	2	5.89	13641.5	0.023742	547.2	1671.8	828.8
1	3	8.84	13641.5	0.023742	547.2	1671.8	828.8
1	4	11.78	19854.6	0.021249	904.9	1651.2	976.9
1	5	14.73	19854.6	0.021249	904.9	1651.2	976.9
1	6	17.67	19854.6	0.021249	904.9	1651.2	976.9
1	7	20.62	19854.6	0.021249	904.9	1651.2	976.9
1	8	23.56	19854.6	0.021249	904.9	1651.2	976.9
1	9	26.51	19854.6	0.021249	904.9	1651.2	976.9
1	10	29.45	19854.6	0.021249	904.9	1651.2	976.9
1	11	32.40	19854.6	0.021249	904.9	1651.2	976.9
1	12	35.35	19854.6	0.021249	904.9	1651.2	976.9
1	13	38.29	19854.6	0.021249	904.9	1651.2	976.9
1	14	41.24	19854.6	0.021249	904.9	1651.2	976.9
1	15	44.18	19854.6	0.021249	904.9	1651.2	976.9
1	16	47.13	19854.6	0.021249	904.9	1651.2	976.9
1	17	50.07	13641.5	0.023742	547.2	1671.8	828.8
1	18	53.02	13641.5	0.023742	547.2	1671.8	828.8
1	19	55.96	13641.5	0.023742	547.2	1671.8	828.8
1	20	58.91	13641.5	0.023742	547.2	1671.8	828.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

MOMENT OF INERTIA Ix (in**4) | Q/Ix Q=1ST. MOMENT OF INERTIA (1/in) | ELASTIC SECTION MODULUS,(in**3) STEEL BOT. | STEEL TOP. | CONCRETE(SLAB) TOP.

SP NO | IN NO | D FROM L SUPT (ft)

1	0	0.00	18479.7	0.029242	603.9	7418.9	1712.5
1	1	2.95	18479.7	0.029242	603.9	7418.9	1712.5
1	2	5.89	18479.7	0.029242	603.9	7418.9	1712.5
1	3	8.84	18479.7	0.029242	603.9	7418.9	1712.5
1	4	11.78	28445.3	0.026571	993.1	5343.8	2088.0
1	5	14.73	28445.3	0.026571	993.1	5343.8	2088.0
1	6	17.67	28445.3	0.026571	993.1	5343.8	2088.0
1	7	20.62	28445.3	0.026571	993.1	5343.8	2088.0
1	8	23.56	28445.3	0.026571	993.1	5343.8	2088.0
1	9	26.51	28445.3	0.026571	993.1	5343.8	2088.0
1	10	29.45	28445.3	0.026571	993.1	5343.8	2088.0
1	11	32.40	28445.3	0.026571	993.1	5343.8	2088.0
1	12	35.35	28445.3	0.026571	993.1	5343.8	2088.0
1	13	38.29	28445.3	0.026571	993.1	5343.8	2088.0
1	14	41.24	28445.3	0.026571	993.1	5343.8	2088.0
1	15	44.18	28445.3	0.026571	993.1	5343.8	2088.0
1	16	47.13	28445.3	0.026571	993.1	5343.8	2088.0
1	17	50.07	18479.7	0.029242	603.9	7418.9	1712.5
1	18	53.02	18479.7	0.029242	603.9	7418.9	1712.5
1	19	55.96	18479.7	0.029242	603.9	7418.9	1712.5
1	20	58.91	18479.7	0.029242	603.9	7418.9	1712.5

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	2.95	13.0	59.9	1.1	0.0		74.0
1	2	5.89	24.6	113.5	2.3	0.0		140.3

1	3	8.84	34.8	160.8	3.4	0.0	199.0
1	4	11.78	43.7	201.8	4.5	0.0	250.0
1	5	14.73	51.2	236.5	5.4	0.0	293.1
1	6	17.67	57.3	264.9	5.7	0.0	327.9
1	7	20.62	62.1	287.0	6.0	0.0	355.0
1	8	23.56	65.5	302.8	6.2	0.0	374.5
1	9	26.51	67.5	312.2	6.5	0.0	386.3
1	10	29.45	68.2	315.4	6.8	0.0	390.4
1	11	32.40	67.5	312.2	7.1	0.0	386.8
1	12	35.35	65.5	302.8	6.6	0.0	374.8
1	13	38.29	62.1	287.0	6.1	0.0	355.1
1	14	41.24	57.3	264.9	5.5	0.0	327.7
1	15	44.18	51.2	236.5	4.9	0.0	292.6
1	16	47.13	43.7	201.8	4.4	0.0	249.9
1	17	50.07	34.8	160.8	3.8	0.0	199.4
1	18	53.02	24.6	113.5	2.8	0.0	140.9
1	19	55.96	13.0	59.9	1.4	0.0	74.3
1	20	58.91	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.95	24.1	13.4	0.0	0.0	0.0	37.5
1	2	5.89	45.6	25.5	0.0	0.0	0.0	71.1
1	3	8.84	64.6	36.1	0.0	0.0	0.0	100.7
1	4	11.78	81.1	45.3	0.0	0.0	0.0	126.3
1	5	14.73	95.0	53.0	0.0	0.0	0.0	148.0
1	6	17.67	106.4	59.4	0.0	0.0	0.0	165.8
1	7	20.62	115.3	64.3	0.0	0.0	0.0	179.6
1	8	23.56	121.6	67.9	0.0	0.0	0.0	189.5
1	9	26.51	125.4	70.0	0.0	0.0	0.0	195.4
1	10	29.45	126.7	70.7	0.0	0.0	0.0	197.4
1	11	32.40	125.4	70.0	0.0	0.0	0.0	195.4
1	12	35.35	121.6	67.9	0.0	0.0	0.0	189.5
1	13	38.29	115.3	64.3	0.0	0.0	0.0	179.6
1	14	41.24	106.4	59.4	0.0	0.0	0.0	165.8
1	15	44.18	95.0	53.0	0.0	0.0	0.0	148.0
1	16	47.13	81.1	45.3	0.0	0.0	0.0	126.3
1	17	50.07	64.6	36.1	0.0	0.0	0.0	100.7
1	18	53.02	45.6	25.5	0.0	0.0	0.0	71.1
1	19	55.96	24.1	13.4	0.0	0.0	0.0	37.5
1	20	58.91	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE

1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.95	0.0	0.0	192.7	HL-93	0.0	HL-93
1	2	5.89	0.0	0.0	361.8	HL-93	0.0	HL-93
1	3	8.84	0.0	0.0	507.3	HL-93	0.0	HL-93
1	4	11.78	0.0	0.0	629.3	HL-93	0.0	HL-93
1	5	14.73	0.0	0.0	725.1	HL-93	0.0	HL-93
1	6	17.67	0.0	0.0	802.5	HL-93	0.0	HL-93
1	7	20.62	0.0	0.0	862.9	HL-93	0.0	HL-93
1	8	23.56	0.0	0.0	902.3	HL-93	0.0	HL-93
1	9	26.51	0.0	0.0	914.3	HL-93	0.0	HL-93
1	10	29.45	0.0	0.0	918.2	HL-93	0.0	HL-93
1	11	32.40	0.0	0.0	914.3	HL-93	0.0	HL-93
1	12	35.35	0.0	0.0	902.3	HL-93	0.0	HL-93
1	13	38.29	0.0	0.0	862.9	HL-93	0.0	HL-93
1	14	41.24	0.0	0.0	802.5	HL-93	0.0	HL-93
1	15	44.18	0.0	0.0	725.1	HL-93	0.0	HL-93
1	16	47.13	0.0	0.0	629.3	HL-93	0.0	HL-93
1	17	50.07	0.0	0.0	507.3	HL-93	0.0	HL-93
1	18	53.02	0.0	0.0	361.8	HL-93	0.0	HL-93
1	19	55.96	0.0	0.0	192.7	HL-93	0.0	HL-93
1	20	58.91	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.95	67.	0.	67.
1	2	5.89	124.	0.	124.
1	3	8.84	168.	0.	168.
1	4	11.78	206.	0.	206.
1	5	14.73	237.	0.	237.
1	6	17.67	259.	0.	259.
1	7	20.62	274.	0.	274.
1	8	23.56	280.	0.	280.
1	9	26.51	268.	0.	268.
1	10	29.45	264.	0.	264.
1	11	32.40	268.	0.	268.
1	12	35.35	280.	0.	280.
1	13	38.29	274.	0.	274.
1	14	41.24	259.	0.	259.
1	15	44.18	237.	0.	237.
1	16	47.13	206.	0.	206.
1	17	50.07	168.	0.	168.
1	18	53.02	124.	0.	124.
1	19	55.96	67.	0.	67.
1	20	58.91	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT	DEAD LOAD NON COMP. N=Infin.	COMP. N=27.0	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE	GOVERN LOAD	MAXIMUM NEGATIVE	GOVERN LOAD	TOTAL MAXIMUM POSITIVE	NEGATIVE

		(ft)	(k-ft)	(k-ft)	(k-ft)	TYPE	(k-ft)	TYPE	(k-ft)	(k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.95	74.0	37.5	192.7	HL-93	0.0	HL-93	304.2	111.5
1	2	5.89	140.3	71.1	361.8	HL-93	0.0	HL-93	573.2	211.4
1	3	8.84	199.0	100.7	507.3	HL-93	0.0	HL-93	807.0	299.7
1	4	11.78	250.0	126.3	629.3	HL-93	0.0	HL-93	1005.6	376.3
1	5	14.73	293.1	148.0	725.1	HL-93	0.0	HL-93	1166.2	441.1
1	6	17.67	327.9	165.8	802.5	HL-93	0.0	HL-93	1296.2	493.7
1	7	20.62	355.0	179.6	862.9	HL-93	0.0	HL-93	1397.5	534.6
1	8	23.56	374.5	189.5	902.3	HL-93	0.0	HL-93	1466.3	564.0
1	9	26.51	386.3	195.4	914.3	HL-93	0.0	HL-93	1495.9	581.7
1	10	29.45	390.4	197.4	918.2	HL-93	0.0	HL-93	1506.0	587.8
1	11	32.40	386.8	195.4	914.3	HL-93	0.0	HL-93	1496.5	582.2
1	12	35.35	374.8	189.5	902.3	HL-93	0.0	HL-93	1466.6	564.3
1	13	38.29	355.1	179.6	862.9	HL-93	0.0	HL-93	1397.6	534.7
1	14	41.24	327.7	165.8	802.5	HL-93	0.0	HL-93	1296.0	493.5
1	15	44.18	292.6	148.0	725.1	HL-93	0.0	HL-93	1165.7	440.6
1	16	47.13	249.9	126.3	629.3	HL-93	0.0	HL-93	1005.4	376.2
1	17	50.07	199.4	100.7	507.3	HL-93	0.0	HL-93	807.4	300.1
1	18	53.02	140.9	71.1	361.8	HL-93	0.0	HL-93	573.7	212.0
1	19	55.96	74.3	37.5	192.7	HL-93	0.0	HL-93	304.5	111.8
1	20	58.91	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.95	304.2	362.0	482.6	405.5	167.3	50.4
1	2	5.89	573.2	681.7	908.7	764.0	317.1	92.9
1	3	8.84	807.0	959.2	1278.5	1075.6	449.5	126.3
1	4	11.78	1005.6	1194.4	1591.9	1340.2	564.5	154.4
1	5	14.73	1166.2	1383.7	1844.0	1554.0	661.7	177.4
1	6	17.67	1296.2	1536.9	2048.1	1727.1	740.5	194.4
1	7	20.62	1397.5	1656.4	2207.2	1862.0	801.9	205.8
1	8	23.56	1466.3	1737.0	2314.4	1953.5	845.9	210.2
1	9	26.51	1495.9	1770.2	2358.4	1992.7	872.5	200.8
1	10	29.45	1506.0	1781.5	2373.3	2006.0	881.7	197.7
1	11	32.40	1496.5	1770.8	2359.1	1993.4	873.4	200.8
1	12	35.35	1466.6	1737.3	2314.8	1953.9	846.5	210.2
1	13	38.29	1397.6	1656.5	2207.3	1862.1	802.1	205.8
1	14	41.24	1296.0	1536.8	2047.9	1726.9	740.2	194.4
1	15	44.18	1165.7	1383.2	1843.4	1553.4	661.0	177.4
1	16	47.13	1005.4	1194.2	1591.7	1340.0	564.3	154.4
1	17	50.07	807.4	959.6	1279.0	1076.1	450.1	126.3
1	18	53.02	573.7	682.2	909.4	764.7	317.9	92.9
1	19	55.96	304.5	362.3	482.9	405.8	167.7	50.4
1	20	58.91	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.6	21.4	0.4	0.0	26.4
1	1	2.95	4.2	19.3	0.4	0.0	23.8
1	2	5.89	3.7	17.1	0.4	0.0	21.2
1	3	8.84	3.2	15.0	0.4	0.0	18.6
1	4	11.78	2.8	12.8	0.4	0.0	16.0
1	5	14.73	2.3	10.7	0.1	0.0	13.1
1	6	17.67	1.9	8.6	0.1	0.0	10.5
1	7	20.62	1.4	6.4	0.1	0.0	7.9
1	8	23.56	0.9	4.3	0.1	0.0	5.3
1	9	26.51	0.5	2.1	0.1	0.0	2.7
1	10	29.45	0.0	0.0	0.1	0.0	0.1
1	11	32.40	-0.5	-2.1	0.1	0.0	-2.5
1	12	35.35	-0.9	-4.3	-0.2	0.0	-5.4
1	13	38.29	-1.4	-6.4	-0.2	0.0	-8.0
1	14	41.24	-1.9	-8.6	-0.2	0.0	-10.6
1	15	44.18	-2.3	-10.7	-0.2	0.0	-13.2
1	16	47.13	-2.8	-12.8	-0.2	0.0	-15.8
1	17	50.07	-3.2	-15.0	-0.2	0.0	-18.4
1	18	53.02	-3.7	-17.1	-0.5	0.0	-21.3
1	19	55.96	-4.2	-19.3	-0.5	0.0	-23.9
1	20	58.91	-4.6	-21.4	-0.5	0.0	-26.5

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.6	4.8	0.0	0.0	13.4	26.4	39.8
1	1	2.95	7.7	4.3	0.0	0.0	12.1	23.8	35.9
1	2	5.89	6.9	3.8	0.0	0.0	10.7	21.2	31.9
1	3	8.84	6.0	3.4	0.0	0.0	9.4	18.6	28.0
1	4	11.78	5.2	2.9	0.0	0.0	8.0	16.0	24.1
1	5	14.73	4.3	2.4	0.0	0.0	6.7	13.1	19.8
1	6	17.67	3.4	1.9	0.0	0.0	5.4	10.5	15.9
1	7	20.62	2.6	1.4	0.0	0.0	4.0	7.9	11.9
1	8	23.56	1.7	1.0	0.0	0.0	2.7	5.3	8.0
1	9	26.51	0.9	0.5	0.0	0.0	1.3	2.7	4.0
1	10	29.45	0.0	0.0	0.0	0.0	0.0	0.1	0.1
1	11	32.40	-0.9	-0.5	0.0	0.0	-1.3	-2.5	-3.8
1	12	35.35	-1.7	-1.0	0.0	0.0	-2.7	-5.4	-8.1
1	13	38.29	-2.6	-1.4	0.0	0.0	-4.0	-8.0	-12.0
1	14	41.24	-3.4	-1.9	0.0	0.0	-5.4	-10.6	-16.0
1	15	44.18	-4.3	-2.4	0.0	0.0	-6.7	-13.2	-19.9
1	16	47.13	-5.2	-2.9	0.0	0.0	-8.0	-15.8	-23.9
1	17	50.07	-6.0	-3.4	0.0	0.0	-9.4	-18.4	-27.8
1	18	53.02	-6.9	-3.8	0.0	0.0	-10.7	-21.3	-32.0
1	19	55.96	-7.7	-4.3	0.0	0.0	-12.1	-23.9	-36.0
1	20	58.91	-8.6	-4.8	0.0	0.0	-13.4	-26.5	-39.9

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	87.3	HL-93	0.0	HL-93
1	1	2.95	0.0	0.0	81.5	HL-93	-1.9	HL-93
1	2	5.89	0.0	0.0	75.7	HL-93	-4.0	HL-93
1	3	8.84	0.0	0.0	70.1	HL-93	-7.1	HL-93
1	4	11.78	0.0	0.0	64.5	HL-93	-10.3	HL-93
1	5	14.73	0.0	0.0	59.0	HL-93	-13.6	HL-93
1	6	17.67	0.0	0.0	53.6	HL-93	-17.0	HL-93
1	7	20.62	0.0	0.0	48.3	HL-93	-20.5	HL-93
1	8	23.56	0.0	0.0	43.1	HL-93	-24.0	HL-93
1	9	26.51	0.0	0.0	37.8	HL-93	-28.1	HL-93
1	10	29.45	0.0	0.0	32.8	HL-93	-32.8	HL-93
1	11	32.40	0.0	0.0	28.1	HL-93	-37.8	HL-93
1	12	35.35	0.0	0.0	24.0	HL-93	-43.1	HL-93
1	13	38.29	0.0	0.0	20.5	HL-93	-48.3	HL-93
1	14	41.24	0.0	0.0	17.0	HL-93	-53.6	HL-93
1	15	44.18	0.0	0.0	13.6	HL-93	-59.0	HL-93
1	16	47.13	0.0	0.0	10.3	HL-93	-64.5	HL-93
1	17	50.07	0.0	0.0	7.1	HL-93	-70.1	HL-93
1	18	53.02	0.0	0.0	4.0	HL-93	-75.7	HL-93
1	19	55.96	0.0	0.0	1.9	HL-93	-81.5	HL-93
1	20	58.91	0.0	0.0	0.0	HL-93	-87.3	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	35.	0.	35.
1	1	2.95	33.	-1.	34.
1	2	5.89	30.	-2.	32.
1	3	8.84	27.	-3.	31.
1	4	11.78	25.	-5.	29.
1	5	14.73	22.	-6.	28.
1	6	17.67	20.	-7.	27.
1	7	20.62	18.	-9.	26.
1	8	23.56	16.	-10.	26.
1	9	26.51	14.	-11.	26.
1	10	29.45	13.	-13.	26.
1	11	32.40	11.	-14.	26.
1	12	35.35	10.	-16.	26.
1	13	38.29	9.	-18.	26.
1	14	41.24	7.	-20.	27.
1	15	44.18	6.	-22.	28.
1	16	47.13	5.	-25.	29.
1	17	50.07	3.	-27.	31.
1	18	53.02	2.	-30.	32.
1	19	55.96	1.	-33.	34.
1	20	58.91	0.	-35.	35.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	35.0	.30675E-01	0.54	0.3125*	.14721E-01	0.26	0.3125*
1	1	2.95	33.6	.30675E-01	0.52	0.3125*	.14721E-01	0.25	0.3125*
1	2	5.89	32.2	.30675E-01	0.49	0.3125*	.14721E-01	0.24	0.3125*
1	3	8.84	30.7	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	4	11.78	29.4	.28446E-01	0.42	0.3125*	.86208E-02	0.13	0.3125*
1	5	14.73	28.2	.28446E-01	0.40	0.3125*	.86208E-02	0.12	0.3125*
1	6	17.67	27.2	.28446E-01	0.39	0.3125*	.86208E-02	0.12	0.3125*
1	7	20.62	26.1	.28446E-01	0.37	0.3125*	.86208E-02	0.11	0.3125*
1	8	23.56	25.5	.28446E-01	0.36	0.3125*	.86208E-02	0.11	0.3125*
1	9	26.51	25.5	.28446E-01	0.36	0.3125*	.86208E-02	0.11	0.3125*
1	10	29.45	25.5	.28446E-01	0.36	0.3125*	.86208E-02	0.11	0.3125*
1	11	32.40	25.5	.28446E-01	0.36	0.3125*	.86208E-02	0.11	0.3125*
1	12	35.35	25.5	.28446E-01	0.36	0.3125*	.86208E-02	0.11	0.3125*
1	13	38.29	26.1	.28446E-01	0.37	0.3125*	.86208E-02	0.11	0.3125*
1	14	41.24	27.2	.28446E-01	0.39	0.3125*	.86208E-02	0.12	0.3125*
1	15	44.18	28.2	.28446E-01	0.40	0.3125*	.86208E-02	0.12	0.3125*
1	16	47.13	29.4	.28446E-01	0.42	0.3125*	.86208E-02	0.13	0.3125*
1	17	50.07	30.7	.30675E-01	0.47	0.3125*	.14721E-01	0.23	0.3125*
1	18	53.02	32.2	.30675E-01	0.49	0.3125*	.14721E-01	0.24	0.3125*
1	19	55.96	33.6	.30675E-01	0.52	0.3125*	.14721E-01	0.25	0.3125*
1	20	58.91	35.0	.30675E-01	0.54	0.3125*	.14721E-01	0.26	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	39.8	87.3	HL-93	0.0	HL-93	127.1	39.8
1	1	2.95	35.9	81.5	HL-93	-1.9	HL-93	117.4	34.0

1	2	5.89	31.9	75.7	HL-93	-4.0	HL-93	107.7	27.9
1	3	8.84	28.0	70.1	HL-93	-7.1	HL-93	98.1	20.9
1	4	11.78	24.1	64.5	HL-93	-10.3	HL-93	88.6	13.7
1	5	14.73	19.8	59.0	HL-93	-13.6	HL-93	78.9	6.2
1	6	17.67	15.9	53.6	HL-93	-17.0	HL-93	69.5	-1.1
1	7	20.62	11.9	48.3	HL-93	-20.5	HL-93	60.2	-8.5
1	8	23.56	8.0	43.1	HL-93	-24.0	HL-93	51.1	-16.0
1	9	26.51	4.0	37.8	HL-93	-28.1	HL-93	41.9	-24.1
1	10	29.45	0.1	32.8	HL-93	-32.8	HL-93	32.9	-32.7
1	11	32.40	-3.8	28.1	HL-93	-37.8	HL-93	24.3	-41.7
1	12	35.35	-8.1	24.0	HL-93	-43.1	HL-93	15.9	-51.1
1	13	38.29	-12.0	20.5	HL-93	-48.3	HL-93	8.4	-60.3
1	14	41.24	-16.0	17.0	HL-93	-53.6	HL-93	1.0	-69.6
1	15	44.18	-19.9	13.6	HL-93	-59.0	HL-93	-6.3	-78.9
1	16	47.13	-23.9	10.3	HL-93	-64.5	HL-93	-13.5	-88.4
1	17	50.07	-27.8	7.1	HL-93	-70.1	HL-93	-20.7	-97.9
1	18	53.02	-32.0	4.0	HL-93	-75.7	HL-93	-28.0	-107.8
1	19	55.96	-36.0	1.9	HL-93	-81.5	HL-93	-34.1	-117.5
1	20	58.91	-39.9	0.0	HL-93	-87.3	HL-93	-39.9	-127.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	127.1	153.3	204.7	169.8	59.7	26.3
1	1	2.95	117.4	141.8	189.4	156.8	53.8	25.2
1	2	5.89	107.7	130.4	174.2	143.9	47.9	24.2
1	3	8.84	98.1	119.1	159.2	131.1	42.0	23.1
1	4	11.78	88.6	107.9	144.3	118.5	36.1	22.1
1	5	14.73	78.9	96.6	129.2	105.5	29.7	21.2
1	6	17.67	69.5	85.6	114.6	93.1	23.8	20.4
1	7	20.62	60.2	74.7	100.1	80.8	17.9	19.6
1	8	23.56	51.1	64.0	85.8	68.6	12.0	19.1
1	9	26.51	41.9	53.2	71.5	56.3	6.1	19.1
1	10	29.45	32.9	42.8	57.6	44.5	0.1	19.1
1	11	32.40	-41.7	-53.0	-71.2	-56.1	-5.8	19.1
1	12	35.35	-51.1	-64.1	-85.9	-68.7	-12.1	19.1
1	13	38.29	-60.3	-74.8	-100.2	-80.9	-18.0	19.6
1	14	41.24	-69.6	-85.7	-114.7	-93.2	-24.0	20.4
1	15	44.18	-78.9	-96.7	-129.3	-105.7	-29.9	21.2
1	16	47.13	-88.4	-107.7	-144.0	-118.2	-35.8	22.1
1	17	50.07	-97.9	-118.9	-158.9	-130.9	-41.7	23.1
1	18	53.02	-107.8	-130.5	-174.3	-144.0	-48.1	24.2
1	19	55.96	-117.5	-141.9	-189.5	-156.9	-54.0	25.2
1	20	58.91	-127.2	-153.4	-204.8	-169.9	-59.9	26.3

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM

1	39.83	0.00	HL-93	87.29	HL-93	ST1	33.70	204.70
						ST2	33.70	169.78
						ST4	46.84	59.75
						SE1	39.83	127.12
						SE2	39.83	153.31
2	39.92	0.00	HL-93	87.29	HL-93	ST1	33.78	204.82
						ST2	33.78	169.90
						ST4	46.99	59.89
						SE1	39.92	127.22
						SE2	39.92	153.41

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.95	-0.0264	-0.1223	0.0000	-0.0360	-0.1513	-0.0360	-0.1873
1	2	5.89	-0.0519	-0.2400	0.0000	-0.0705	-0.2970	-0.0705	-0.3676
1	3	8.84	-0.0755	-0.3490	0.0000	-0.1024	-0.4320	-0.1024	-0.5344
1	4	11.78	-0.0968	-0.4474	0.0000	-0.1311	-0.5537	-0.1311	-0.6848
1	5	14.73	-0.1156	-0.5345	0.0000	-0.1565	-0.6616	-0.1565	-0.8180
1	6	17.67	-0.1316	-0.6084	0.0000	-0.1780	-0.7530	-0.1780	-0.9310
1	7	20.62	-0.1444	-0.6675	0.0000	-0.1952	-0.8262	-0.1952	-1.0215
1	8	23.56	-0.1537	-0.7106	0.0000	-0.2078	-0.8796	-0.2078	-1.0874
1	9	26.51	-0.1594	-0.7369	0.0000	-0.2155	-0.9121	-0.2155	-1.1275
1	10	29.45	-0.1613	-0.7457	0.0000	-0.2180	-0.9230	-0.2180	-1.1410
1	11	32.40	-0.1594	-0.7369	0.0000	-0.2155	-0.9121	-0.2155	-1.1276
1	12	35.35	-0.1537	-0.7106	0.0000	-0.2078	-0.8796	-0.2078	-1.0875
1	13	38.29	-0.1444	-0.6675	0.0000	-0.1952	-0.8263	-0.1952	-1.0215
1	14	41.24	-0.1316	-0.6084	0.0000	-0.1780	-0.7531	-0.1780	-0.9311
1	15	44.18	-0.1156	-0.5345	0.0000	-0.1565	-0.6616	-0.1565	-0.8180
1	16	47.13	-0.0968	-0.4474	0.0000	-0.1311	-0.5538	-0.1311	-0.6849
1	17	50.07	-0.0755	-0.3490	0.0000	-0.1024	-0.4321	-0.1024	-0.5344
1	18	53.02	-0.0519	-0.2400	0.0000	-0.0705	-0.2971	-0.0705	-0.3677
1	19	55.96	-0.0264	-0.1223	0.0000	-0.0360	-0.1514	-0.0360	-0.1874
1	20	58.91	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT	NONCOMPOSITE DEAD LOADS STEEL (in)	NONCOMPOSITE DEAD LOADS CAMBER (in)	NONCOMPOSITE DEAD LOADS SLAB (in)	NONCOMPOSITE DEAD LOADS CAMBER (in)	COMPOSITE DL DEFL. (in)	COMPOSITE DL CAMBER (in)	T O T A L DEFL. (in)	T O T A L CAMBER (in)

		(ft)	DEFL.	SIZE	DEFL.	SIZE		SIZE		SIZE		SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--		
1	1	2.9	-0.026	0	1/16	-0.125	0	1/ 8	-0.036	0	1/16	-0.187	0	3/16
1	2	5.9	-0.052	0	1/16	-0.245	0	1/ 4	-0.071	0	1/ 8	-0.368	0	3/ 8
1	3	8.8	-0.075	0	1/ 8	-0.356	0	3/ 8	-0.102	0	1/ 8	-0.534	0	9/16
1	4	11.8	-0.097	0	1/ 8	-0.457	0	1/ 2	-0.131	0	3/16	-0.685	0	11/16
1	5	14.7	-0.116	0	1/ 8	-0.546	0	9/16	-0.156	0	3/16	-0.818	0	7/ 8
1	6	17.7	-0.132	0	3/16	-0.621	0	5/ 8	-0.178	0	3/16	-0.931	0	15/16
1	7	20.6	-0.144	0	3/16	-0.682	0	11/16	-0.195	0	1/ 4	-1.021	1	1/16
1	8	23.6	-0.154	0	3/16	-0.726	0	3/ 4	-0.208	0	1/ 4	-1.087	1	1/ 8
1	9	26.5	-0.159	0	3/16	-0.753	0	13/16	-0.215	0	1/ 4	-1.128	1	3/16
1	10	29.5	-0.161	0	3/16	-0.762	0	13/16	-0.218	0	1/ 4	-1.141	1	3/16
1	11	32.4	-0.159	0	3/16	-0.753	0	13/16	-0.215	0	1/ 4	-1.128	1	3/16
1	12	35.3	-0.154	0	3/16	-0.726	0	3/ 4	-0.208	0	1/ 4	-1.087	1	1/ 8
1	13	38.3	-0.144	0	3/16	-0.682	0	11/16	-0.195	0	1/ 4	-1.022	1	1/16
1	14	41.2	-0.132	0	3/16	-0.621	0	5/ 8	-0.178	0	3/16	-0.931	0	15/16
1	15	44.2	-0.116	0	1/ 8	-0.546	0	9/16	-0.156	0	3/16	-0.818	0	7/ 8
1	16	47.1	-0.097	0	1/ 8	-0.457	0	1/ 2	-0.131	0	3/16	-0.685	0	11/16
1	17	50.1	-0.075	0	1/ 8	-0.357	0	3/ 8	-0.102	0	1/ 8	-0.534	0	9/16
1	18	53.0	-0.052	0	1/16	-0.245	0	1/ 4	-0.071	0	1/ 8	-0.368	0	3/ 8
1	19	56.0	-0.026	0	1/16	-0.125	0	1/ 8	-0.036	0	1/16	-0.187	0	3/16
1	20	58.9	0.000	--	0.000	--	0.000	--	0.000	--		

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	29.45	3	0.425	-0.322 MAX HL-93	0.88	0.00265
				0.027 MIN		
				-0.091 MAX LANE		
				0.027 MIN LANE		
		4	0.433	-0.328 MAX HL-93	0.88	0.00265
				0.027 MIN		
				-0.093 MAX LANE		
				0.027 MIN LANE		
		3	0.425	-0.228 MAX FA		
				0.000 MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	-0.38	0.38	-1.81	1.81	-2.19	2.19
1	2	5.89	-0.73	0.73	-3.43	3.43	-4.15	4.15
1	3	8.84	-1.03	1.03	-4.86	4.86	-5.89	5.89
1	4	11.78	-1.16	0.77	-5.47	3.62	-6.63	4.39
1	5	14.73	-1.36	0.90	-6.41	4.25	-7.77	5.15
1	6	17.67	-1.52	1.01	-7.18	4.75	-8.69	5.76
1	7	20.62	-1.65	1.09	-7.77	5.14	-9.41	6.23
1	8	23.56	-1.74	1.15	-8.19	5.43	-9.93	6.57
1	9	26.51	-1.79	1.19	-8.45	5.60	-10.24	6.78
1	10	29.45	-1.81	1.20	-8.54	5.66	-10.35	6.85
1	11	32.40	-1.79	1.19	-8.47	5.61	-10.26	6.79
1	12	35.35	-1.74	1.15	-8.20	5.43	-9.94	6.58
1	13	38.29	-1.65	1.09	-7.77	5.15	-9.42	6.23
1	14	41.24	-1.52	1.01	-7.17	4.75	-8.69	5.75
1	15	44.18	-1.36	0.90	-6.40	4.24	-7.76	5.14
1	16	47.13	-1.16	0.77	-5.47	3.62	-6.63	4.39
1	17	50.07	-1.03	1.03	-4.87	4.87	-5.90	5.90
1	18	53.02	-0.73	0.73	-3.44	3.44	-4.17	4.17
1	19	55.96	-0.38	0.38	-1.81	1.81	-2.20	2.20
1	20	58.91	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	-0.02	-0.27	0.82	-0.02	-2.46	3.01
1	2	5.89	-0.04	-0.51	1.56	-0.04	-4.66	5.71
1	3	8.84	-0.05	-0.72	2.21	-0.05	-6.61	8.10
1	4	11.78	-0.06	-0.92	1.68	-0.06	-7.55	6.06
1	5	14.73	-0.07	-1.08	1.96	-0.07	-8.85	7.11
1	6	17.67	-0.08	-1.20	2.20	-0.08	-9.90	7.96
1	7	20.62	-0.08	-1.31	2.38	-0.08	-10.72	8.62
1	8	23.56	-0.09	-1.38	2.51	-0.09	-11.31	9.09
1	9	26.51	-0.09	-1.42	2.59	-0.09	-11.66	9.37
1	10	29.45	-0.09	-1.43	2.62	-0.09	-11.79	9.47
1	11	32.40	-0.09	-1.42	2.59	-0.09	-11.68	9.38

1 12	35.35	-0.09	-1.38	2.51	-0.09	-11.32	9.09
1 13	38.29	-0.08	-1.31	2.38	-0.08	-10.72	8.62
1 14	41.24	-0.08	-1.20	2.20	-0.08	-9.89	7.95
1 15	44.18	-0.07	-1.08	1.96	-0.07	-8.84	7.10
1 16	47.13	-0.06	-0.92	1.68	-0.06	-7.54	6.06
1 17	50.07	-0.05	-0.72	2.21	-0.05	-6.62	8.11
1 18	53.02	-0.04	-0.51	1.56	-0.04	-4.68	5.73
1 19	55.96	-0.02	-0.27	0.82	-0.02	-2.47	3.02
1 20	58.91	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	-0.15	-0.31	3.83	0.00	0.00	0.00
1	2	5.89	-0.28	-0.59	7.19	0.00	0.00	0.00
1	3	8.84	-0.39	-0.82	10.08	0.00	0.00	0.00
1	4	11.78	-0.40	-1.41	7.60	0.00	0.00	0.00
1	5	14.73	-0.46	-1.63	8.76	0.00	0.00	0.00
1	6	17.67	-0.51	-1.80	9.70	0.00	0.00	0.00
1	7	20.62	-0.55	-1.94	10.43	0.00	0.00	0.00
1	8	23.56	-0.58	-2.03	10.90	0.00	0.00	0.00
1	9	26.51	-0.58	-2.05	11.05	0.00	0.00	0.00
1	10	29.45	-0.59	-2.06	11.10	0.00	0.00	0.00
1	11	32.40	-0.58	-2.05	11.05	0.00	0.00	0.00
1	12	35.35	-0.58	-2.03	10.90	0.00	0.00	0.00
1	13	38.29	-0.55	-1.94	10.43	0.00	0.00	0.00
1	14	41.24	-0.51	-1.80	9.70	0.00	0.00	0.00
1	15	44.18	-0.46	-1.63	8.76	0.00	0.00	0.00
1	16	47.13	-0.40	-1.41	7.60	0.00	0.00	0.00
1	17	50.07	-0.39	-0.82	10.08	0.00	0.00	0.00
1	18	53.02	-0.28	-0.59	7.19	0.00	0.00	0.00
1	19	55.96	-0.15	-0.31	3.83	0.00	0.00	0.00
1	20	58.91	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	2.95	0.1	0.0	0.1	C HL	1.3	0.0	1.3	T HL
1	2	5.89	0.2	0.0	0.2	C HL	2.5	0.0	2.5	T HL
1	3	8.84	0.3	0.0	0.3	C HL	3.3	0.0	3.3	T HL
1	4	11.78	0.5	0.0	0.5	C HL	2.5	0.0	2.5	T HL
1	5	14.73	0.5	0.0	0.5	C HL	2.9	0.0	2.9	T HL
1	6	17.67	0.6	0.0	0.6	C HL	3.1	0.0	3.1	T HL
1	7	20.62	0.6	0.0	0.6	C HL	3.3	0.0	3.3	T HL
1	8	23.56	0.6	0.0	0.6	C HL	3.4	0.0	3.4	T HL

1 9	26.51	0.6	0.0	0.6 C	HL	3.2	0.0	3.2 T	HL
1 10	29.45	0.6	0.0	0.6 C	HL	3.2	0.0	3.2 T	HL
1 11	32.40	0.6	0.0	0.6 C	HL	3.2	0.0	3.2 T	HL
1 12	35.35	0.6	0.0	0.6 C	HL	3.4	0.0	3.4 T	HL
1 13	38.29	0.6	0.0	0.6 C	HL	3.3	0.0	3.3 T	HL
1 14	41.24	0.6	0.0	0.6 C	HL	3.1	0.0	3.1 T	HL
1 15	44.18	0.5	0.0	0.5 C	HL	2.9	0.0	2.9 T	HL
1 16	47.13	0.5	0.0	0.5 C	HL	2.5	0.0	2.5 T	HL
1 17	50.07	0.3	0.0	0.3 C	HL	3.3	0.0	3.3 T	HL
1 18	53.02	0.2	0.0	0.2 C	HL	2.5	0.0	2.5 T	HL
1 19	55.96	0.1	0.0	0.1 C	HL	1.3	0.0	1.3 T	HL
1 20	58.91	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	33.	33.	-0.17	-2.77	6.84	-0.02	-2.46	3.01
1	2	5.89	33.	33.	-0.32	-5.25	12.90	-0.04	-4.66	5.71
1	3	8.84	33.	33.	-0.45	-7.43	18.18	-0.05	-6.61	8.10
1	4	11.78	33.	33.	-0.46	-8.96	13.67	-0.06	-7.55	6.06
1	5	14.73	33.	33.	-0.53	-10.48	15.87	-0.07	-8.85	7.11
1	6	17.67	33.	33.	-0.59	-11.70	17.65	-0.08	-9.90	7.96
1	7	20.62	33.	33.	-0.63	-12.66	19.04	-0.08	-10.72	8.62
1	8	23.56	33.	33.	-0.66	-13.33	19.99	-0.09	-11.31	9.09
1	9	26.51	33.	33.	-0.67	-13.72	20.42	-0.09	-11.66	9.37
1	10	29.45	33.	33.	-0.68	-13.85	20.57	-0.09	-11.79	9.47
1	11	32.40	33.	33.	-0.67	-13.73	20.43	-0.09	-11.68	9.38
1	12	35.35	33.	33.	-0.66	-13.34	20.00	-0.09	-11.32	9.09
1	13	38.29	33.	33.	-0.63	-12.66	19.04	-0.08	-10.72	8.62
1	14	41.24	33.	33.	-0.59	-11.70	17.65	-0.08	-9.89	7.95
1	15	44.18	33.	33.	-0.53	-10.46	15.86	-0.07	-8.84	7.10
1	16	47.13	33.	33.	-0.46	-8.96	13.67	-0.06	-7.54	6.06
1	17	50.07	33.	33.	-0.45	-7.44	18.19	-0.05	-6.62	8.11
1	18	53.02	33.	33.	-0.32	-5.26	12.92	-0.04	-4.68	5.73
1	19	55.96	33.	33.	-0.17	-2.78	6.85	-0.02	-2.47	3.02
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

1	1	2.95	33.	33.	-0.22	-2.86	7.99	-0.02	-2.46	3.01
1	2	5.89	33.	33.	-0.40	-5.42	15.06	-0.04	-4.66	5.71
1	3	8.84	33.	33.	-0.57	-7.68	21.20	-0.05	-6.61	8.10
1	4	11.78	33.	33.	-0.58	-9.38	15.95	-0.06	-7.55	6.06
1	5	14.73	33.	33.	-0.67	-10.96	18.50	-0.07	-8.85	7.11
1	6	17.67	33.	33.	-0.74	-12.24	20.56	-0.08	-9.90	7.96
1	7	20.62	33.	33.	-0.80	-13.24	22.17	-0.08	-10.72	8.62
1	8	23.56	33.	33.	-0.84	-13.94	23.26	-0.09	-11.31	9.09
1	9	26.51	33.	33.	-0.85	-14.33	23.73	-0.09	-11.66	9.37
1	10	29.45	33.	33.	-0.85	-14.47	23.90	-0.09	-11.79	9.47
1	11	32.40	33.	33.	-0.85	-14.35	23.74	-0.09	-11.68	9.38
1	12	35.35	33.	33.	-0.84	-13.95	23.27	-0.09	-11.32	9.09
1	13	38.29	33.	33.	-0.80	-13.24	22.17	-0.08	-10.72	8.62
1	14	41.24	33.	33.	-0.74	-12.24	20.56	-0.08	-9.89	7.95
1	15	44.18	33.	33.	-0.67	-10.95	18.49	-0.07	-8.84	7.10
1	16	47.13	33.	33.	-0.58	-9.38	15.95	-0.06	-7.54	6.06
1	17	50.07	33.	33.	-0.57	-7.69	21.21	-0.05	-6.62	8.11
1	18	53.02	33.	33.	-0.40	-5.44	15.07	-0.04	-4.68	5.73
1	19	55.96	33.	33.	-0.22	-2.87	8.00	-0.02	-2.47	3.02
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	33.	33.	-0.29	-3.66	10.60	-0.03	-3.12	3.90
1	2	5.89	33.	33.	-0.55	-6.93	19.97	-0.05	-5.91	7.39
1	3	8.84	33.	33.	-0.77	-9.82	28.11	-0.08	-8.38	10.47
1	4	11.78	33.	33.	-0.78	-12.05	21.16	-0.08	-9.58	7.85
1	5	14.73	33.	33.	-0.91	-14.08	24.53	-0.09	-11.23	9.20
1	6	17.67	33.	33.	-1.00	-15.72	27.27	-0.11	-12.57	10.30
1	7	20.62	33.	33.	-1.08	-17.00	29.40	-0.12	-13.61	11.15
1	8	23.56	33.	33.	-1.13	-17.90	30.84	-0.12	-14.35	11.76
1	9	26.51	33.	33.	-1.15	-18.40	31.46	-0.13	-14.81	12.13
1	10	29.45	33.	33.	-1.15	-18.57	31.68	-0.13	-14.96	12.26
1	11	32.40	33.	33.	-1.15	-18.42	31.48	-0.13	-14.83	12.14
1	12	35.35	33.	33.	-1.13	-17.91	30.85	-0.12	-14.37	11.77
1	13	38.29	33.	33.	-1.08	-17.00	29.40	-0.12	-13.61	11.15
1	14	41.24	33.	33.	-1.00	-15.72	27.26	-0.11	-12.56	10.29
1	15	44.18	33.	33.	-0.91	-14.07	24.52	-0.09	-11.22	9.19
1	16	47.13	33.	33.	-0.78	-12.05	21.15	-0.08	-9.58	7.85
1	17	50.07	33.	33.	-0.77	-9.83	28.13	-0.08	-8.40	10.49
1	18	53.02	33.	33.	-0.55	-6.95	19.99	-0.05	-5.93	7.41
1	19	55.96	33.	33.	-0.29	-3.67	10.61	-0.03	-3.13	3.91
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	33.	33.	-0.28	-2.72	9.28	-0.01	-2.17	2.58
1	2	5.89	33.	33.	-0.52	-5.14	17.47	-0.03	-4.11	4.89
1	3	8.84	33.	33.	-0.73	-7.27	24.57	-0.04	-5.83	6.93
1	4	11.78	33.	33.	-0.75	-9.12	18.50	-0.04	-6.65	5.19
1	5	14.73	33.	33.	-0.86	-10.64	21.41	-0.05	-7.79	6.08
1	6	17.67	33.	33.	-0.95	-11.87	23.78	-0.06	-8.72	6.81
1	7	20.62	33.	33.	-1.02	-12.83	25.62	-0.06	-9.44	7.37
1	8	23.56	33.	33.	-1.07	-13.50	26.86	-0.06	-9.96	7.78
1	9	26.51	33.	33.	-1.09	-13.86	27.35	-0.07	-10.27	8.02
1	10	29.45	33.	33.	-1.09	-13.99	27.52	-0.07	-10.38	8.10
1	11	32.40	33.	33.	-1.09	-13.88	27.36	-0.07	-10.28	8.03
1	12	35.35	33.	33.	-1.07	-13.51	26.86	-0.06	-9.96	7.78
1	13	38.29	33.	33.	-1.02	-12.83	25.62	-0.06	-9.44	7.37
1	14	41.24	33.	33.	-0.95	-11.87	23.77	-0.06	-8.71	6.80
1	15	44.18	33.	33.	-0.86	-10.63	21.41	-0.05	-7.78	6.08
1	16	47.13	33.	33.	-0.75	-9.11	18.49	-0.04	-6.64	5.19
1	17	50.07	33.	33.	-0.73	-7.28	24.58	-0.04	-5.85	6.94
1	18	53.02	33.	33.	-0.52	-5.15	17.48	-0.03	-4.13	4.90
1	19	55.96	33.	33.	-0.28	-2.72	9.29	-0.01	-2.18	2.59
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	33.	33.	-0.23	-3.54	9.07	-0.03	-3.12	3.90
1	2	5.89	33.	33.	-0.43	-6.70	17.09	-0.05	-5.91	7.39
1	3	8.84	33.	33.	-0.61	-9.49	24.08	-0.08	-8.38	10.47
1	4	11.78	33.	33.	-0.62	-11.49	18.11	-0.08	-9.58	7.85
1	5	14.73	33.	33.	-0.72	-13.43	21.03	-0.09	-11.23	9.20
1	6	17.67	33.	33.	-0.80	-15.00	23.39	-0.11	-12.57	10.30
1	7	20.62	33.	33.	-0.86	-16.22	25.23	-0.12	-13.61	11.15
1	8	23.56	33.	33.	-0.90	-17.09	26.48	-0.12	-14.35	11.76
1	9	26.51	33.	33.	-0.91	-17.58	27.05	-0.13	-14.81	12.13
1	10	29.45	33.	33.	-0.92	-17.75	27.24	-0.13	-14.96	12.26
1	11	32.40	33.	33.	-0.91	-17.60	27.06	-0.13	-14.83	12.14
1	12	35.35	33.	33.	-0.90	-17.10	26.49	-0.12	-14.37	11.77
1	13	38.29	33.	33.	-0.86	-16.23	25.23	-0.12	-13.61	11.15
1	14	41.24	33.	33.	-0.80	-14.99	23.38	-0.11	-12.56	10.29
1	15	44.18	33.	33.	-0.72	-13.41	21.02	-0.09	-11.22	9.19
1	16	47.13	33.	33.	-0.62	-11.48	18.11	-0.08	-9.58	7.85
1	17	50.07	33.	33.	-0.61	-9.50	24.10	-0.08	-8.40	10.49
1	18	53.02	33.	33.	-0.43	-6.72	17.11	-0.05	-5.93	7.41
1	19	55.96	33.	33.	-0.23	-3.55	9.08	-0.03	-3.13	3.91
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.95	33.	33.	-0.22	-2.59	7.75	-0.01	-2.17	2.58
1	2	5.89	33.	33.	-0.41	-4.90	14.59	-0.03	-4.11	4.89
1	3	8.84	33.	33.	-0.57	-6.94	20.54	-0.04	-5.83	6.93
1	4	11.78	33.	33.	-0.58	-8.55	15.45	-0.04	-6.65	5.19
1	5	14.73	33.	33.	-0.67	-9.99	17.91	-0.05	-7.79	6.08
1	6	17.67	33.	33.	-0.75	-11.15	19.90	-0.06	-8.72	6.81
1	7	20.62	33.	33.	-0.80	-12.05	21.45	-0.06	-9.44	7.37
1	8	23.56	33.	33.	-0.84	-12.69	22.49	-0.06	-9.96	7.78
1	9	26.51	33.	33.	-0.85	-13.04	22.93	-0.07	-10.27	8.02
1	10	29.45	33.	33.	-0.86	-13.16	23.08	-0.07	-10.38	8.10
1	11	32.40	33.	33.	-0.85	-13.05	22.94	-0.07	-10.28	8.03
1	12	35.35	33.	33.	-0.84	-12.70	22.50	-0.06	-9.96	7.78
1	13	38.29	33.	33.	-0.80	-12.06	21.45	-0.06	-9.44	7.37
1	14	41.24	33.	33.	-0.75	-11.14	19.89	-0.06	-8.71	6.80
1	15	44.18	33.	33.	-0.67	-9.98	17.90	-0.05	-7.78	6.08
1	16	47.13	33.	33.	-0.58	-8.55	15.45	-0.04	-6.64	5.19
1	17	50.07	33.	33.	-0.57	-6.95	20.55	-0.04	-5.85	6.94
1	18	53.02	33.	33.	-0.41	-4.92	14.61	-0.03	-4.13	4.90
1	19	55.96	33.	33.	-0.22	-2.60	7.75	-0.01	-2.18	2.59
1	20	58.91	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	8.95	201.	-5.9	102.	-0.7	512.	-0.8	0.	0.0	-7.51
											0.83
	LB			5.9		2.2		10.2		0.0	18.36
											10.18
	RT			-5.3		-0.7		-1.2		0.0	-7.22
											1.15
	RB			3.5		1.3		6.2		0.0	11.07
											6.19
1	LT	49.96	201.	-5.3	102.	-0.7	512.	-1.2	0.	0.0	-7.23
											1.15
	LB			3.5		1.3		6.2		0.0	11.07
											6.19
	RT			-6.0		-0.7		-0.8		0.0	-7.52
											0.83
	RB			6.0		2.2		10.2		0.0	18.37
											10.18

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK

```
 1   8.95    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

 2  41.00    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 13.00 0.8750

 3   8.95    W  11.5 0.8500  31.4 0.5790 11.5 0.8500  11.50 0.0000 11.50 0.0000

 ------------------------------------------------------------------------------
 NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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 TABLE 1.2.22.4=DEPTH RATIOS
                ************

  ** [1] = Span length (ft) =   58.9
                                                          LIMITATION
   MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
 --------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
 NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
     (ft)   (in)   (in)       (in)
 ---------------------------------------------------------------------

  1   8.95  6.75   33.09 [2]=   41.39     0.05855      >   0.04000    OK

                         [3]=   33.09     0.04681      >   0.03300    OK

  2  41.00  6.75   33.09 [2]=   41.39     0.05855      >   0.04000    OK

                         [3]=   33.09     0.04681      >   0.03300    OK

  3   8.95  6.75   33.09 [2]=   41.39     0.05855      >   0.04000    OK

                         [3]=   33.09     0.04681      >   0.03300    OK

 ---------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                            Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  1   2.95  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  2   5.89  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  3   8.84  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  4  11.78  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
  1  5  14.73  0  1  31.39 0.579  54.2  150.0    0.00 111.46                 0
                                                 Page 27
```

1	6	17.67	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	7	20.62	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	8	23.56	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	9	26.51	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	10	29.45	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	11	32.40	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	12	35.35	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	13	38.29	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	14	41.24	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	15	44.18	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	16	47.13	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	17	50.07	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	18	53.02	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	19	55.96	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	20	58.91	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0		0.00	0	1	31.39	0.579	54.21	168.97	2
1	1		2.95	0	1	31.39	0.579	54.21	168.97	2
1	2		5.89	0	1	31.39	0.579	54.21	168.97	2
1	3		8.84	0	1	31.39	0.579	54.21	168.97	2
1	4		11.78	0	1	31.39	0.579	67.80	168.97	2
1	5		14.73	0	1	31.39	0.579	67.80	168.97	2
1	6		17.67	0	1	31.39	0.579	67.80	168.97	2
1	7		20.62	0	1	31.39	0.579	67.80	168.97	2
1	8		23.56	0	1	31.39	0.579	67.80	168.97	2
1	9		26.51	0	1	31.39	0.579	67.80	168.97	2
1	10		29.45	0	1	31.39	0.579	67.80	168.97	2
1	11		32.40	0	1	31.39	0.579	67.80	168.97	2
1	12		35.35	0	1	31.39	0.579	67.80	168.97	2
1	13		38.29	0	1	31.39	0.579	67.80	168.97	2
1	14		41.24	0	1	31.39	0.579	67.80	168.97	2
1	15		44.18	0	1	31.39	0.579	67.80	168.97	2
1	16		47.13	0	1	31.39	0.579	67.80	168.97	2
1	17		50.07	0	1	31.39	0.579	54.21	168.97	2
1	18		53.02	0	1	31.39	0.579	54.21	168.97	2
1	19		55.96	0	1	31.39	0.579	54.21	168.97	2
1	20		58.91	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO L (ft)	SUPT	bf/2tf	[1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	1	2.95	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	2	5.89	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	3	8.84	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	4	11.78	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	5	14.73	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	6	17.67	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	7	20.62	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	8	23.56	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	9	26.51	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	10	29.45	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	11	32.40	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	12	35.35	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	13	38.29	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	14	41.24	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	15	44.18	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	16	47.13	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	17	50.07	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	18	53.02	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	19	55.96	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1	20	58.91	6.76	12.	11.5	5.2	0.850	0.637		
			6.76	12.	11.5	5.2	0.850	0.637	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. FROM LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	13.8	0.000	11.135	7.134	3.133	1.000
			0.000	0.000	0.000	0.000	1.000
2	13.8	32.7	11.135	15.343	14.812	14.282	1.033
			0.000	0.000	0.000	0.000	1.000
3	32.7	51.5	7.584	15.343	12.630	9.917	1.197
			0.000	0.000	0.000	0.000	1.000
4	51.5	58.9	0.000	7.584	4.054	0.524	1.679
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	6.76	11.26	19.84	0	13.83	7.17	26.91	2	2
1	1	2.95	1	6.76	11.26	19.84	0	13.83	7.17	26.91	2	2
1	2	5.89	1	6.76	11.26	19.84	0	13.83	7.17	26.91	2	2
1	3	8.84	1	6.76	11.26	19.84	0	13.83	7.17	26.91	2	2
1	4	11.78	1	6.76	11.26	19.84	0	13.83	6.96	26.14	2	2
1	5	14.73	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	6	17.67	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	7	20.62	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	8	23.56	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	9	26.51	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	10	29.45	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	11	32.40	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	12	35.35	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	13	38.29	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	14	41.24	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	15	44.18	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	16	47.13	1	6.76	11.26	19.84	0	18.84	6.96	26.14	2	2
1	17	50.07	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	18	53.02	1	6.76	11.26	19.84	0	7.40	7.17	26.91	2	2
1	19	55.96	1	6.76	11.26	19.84	0	7.40	7.17	26.91	2	2
1	20	58.91	1	6.76	11.26	19.84	0	7.40	7.17	26.91	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange

residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.00	33.00	29.66	29.66
1	1		2.95	1	1.000	1.000	1.00	33.00	29.66	29.66
1	2		5.89	1	1.000	1.000	1.00	33.00	29.66	29.66
1	3		8.84	1	1.000	1.000	1.00	33.00	29.66	29.66
1	4		11.78	1	1.000	1.000	1.00	33.00	29.46	29.46
1	5		14.73	1	1.000	1.000	1.03	33.00	27.74	27.74
1	6		17.67	1	1.000	1.000	1.03	33.00	27.74	27.74
1	7		20.62	1	1.000	1.000	1.03	33.00	27.74	27.74
1	8		23.56	1	1.000	1.000	1.03	33.00	27.74	27.74
1	9		26.51	1	1.000	1.000	1.03	33.00	27.74	27.74
1	10		29.45	1	1.000	1.000	1.03	33.00	27.74	27.74
1	11		32.40	1	1.000	1.000	1.03	33.00	27.74	27.74
1	12		35.35	1	1.000	1.000	1.20	33.00	32.15	32.15
1	13		38.29	1	1.000	1.000	1.20	33.00	32.15	32.15
1	14		41.24	1	1.000	1.000	1.20	33.00	32.15	32.15
1	15		44.18	1	1.000	1.000	1.20	33.00	32.15	32.15
1	16		47.13	1	1.000	1.000	1.20	33.00	32.15	32.15
1	17		50.07	1	1.000	1.000	1.20	33.00	32.48	32.48
1	18		53.02	1	1.000	1.000	1.68	33.00	33.00	33.00
1	19		55.96	1	1.000	1.000	1.68	33.00	33.00	33.00
1	20		58.91	1	1.000	1.000	1.68	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0		0.00	1	5.17	17.38
1	1		2.95	1	5.17	17.38
1	2		5.89	1	5.17	17.38
1	3		8.84	1	5.17	17.38
1	4		11.78	1	6.72	17.75
1	5		14.73	1	6.72	17.75
1	6		17.67	1	6.72	17.75
1	7		20.62	1	6.72	17.75
1	8		23.56	1	6.72	17.75
1	9		26.51	1	6.72	17.75

1 10	29.45	1	6.72	17.75
1 11	32.40	1	6.72	17.75
1 12	35.35	1	6.72	17.75
1 13	38.29	1	6.72	17.75
1 14	41.24	1	6.72	17.75
1 15	44.18	1	6.72	17.75
1 16	47.13	1	6.72	17.75
1 17	50.07	1	5.17	17.38
1 18	53.02	1	5.17	17.38
1 19	55.96	1	5.17	17.38
1 20	58.91	1	5.17	17.38

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L (ft)	FROM SUPT	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Category Comp.
1	0	0.00	1		0	2	0		2	0
1	1	2.95	1		0	2	0		2	0
1	2	5.89	1		0	2	0		2	0
1	3	8.84	1		0	2	0		2	0
1	4	11.78	1		0	2	0		2	0
1	5	14.73	1		0	2	0		2	0
1	6	17.67	1		0	2	0		2	0
1	7	20.62	1		0	2	0		2	0
1	8	23.56	1		0	2	0		2	0
1	9	26.51	1		0	2	0		2	0
1	10	29.45	1		0	2	0		2	0
1	11	32.40	1		0	2	0		2	0
1	12	35.35	1		0	2	0		2	0
1	13	38.29	1		0	2	0		2	0
1	14	41.24	1		0	2	0		2	0
1	15	44.18	1		0	2	0		2	0
1	16	47.13	1		0	2	0		2	0
1	17	50.07	1		0	2	0		2	0
1	18	53.02	1		0	2	0		2	0
1	19	55.96	1		0	2	0		2	0
1	20	58.91	1		0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	29.7	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		2.95	0.0	19.8	3.3	33.0	3.3	33.0	3.3	29.7	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	2		5.89	0.0	19.8	6.2	33.0	6.2	33.0	6.2	29.7	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	3		8.84	0.0	19.8	8.8	33.0	8.8	33.0	8.8	29.7	0
				0.0	-	8.8	-	8.8	33.0	-	-	0
1	4		11.78	0.0	19.8	9.9	33.0	9.9	33.0	9.9	29.5	0
				0.0	-	6.6	-	6.6	33.0	-	-	0
1	5		14.73	0.0	19.8	11.7	33.0	11.7	33.0	11.7	27.7	0
				0.0	-	7.7	-	7.7	33.0	-	-	0
1	6		17.67	0.0	19.8	13.0	33.0	13.0	33.0	13.0	27.7	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	7		20.62	0.0	19.8	14.1	33.0	14.1	33.0	14.1	27.7	0
				0.0	-	9.3	-	9.3	33.0	-	-	0
1	8		23.56	0.0	19.8	14.9	33.0	14.9	33.0	14.9	27.7	0
				0.0	-	9.9	-	9.9	33.0	-	-	0
1	9		26.51	0.0	19.8	15.4	33.0	15.4	33.0	15.4	27.7	0
				0.0	-	10.2	-	10.2	33.0	-	-	0
1	10		29.45	0.0	19.8	15.5	33.0	15.5	33.0	15.5	27.7	0
				0.0	-	10.3	-	10.3	33.0	-	-	0
1	11		32.40	0.0	19.8	15.4	33.0	15.4	33.0	15.4	27.7	0
				0.0	-	10.2	-	10.2	33.0	-	-	0
1	12		35.35	0.0	19.8	14.9	33.0	14.9	33.0	14.9	32.2	0
				0.0	-	9.9	-	9.9	33.0	-	-	0
1	13		38.29	0.0	19.8	14.1	33.0	14.1	33.0	14.1	32.2	0
				0.0	-	9.4	-	9.4	33.0	-	-	0
1	14		41.24	0.0	19.8	13.0	33.0	13.0	33.0	13.0	32.2	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	15		44.18	0.0	19.8	11.6	33.0	11.6	33.0	11.6	32.2	0
				0.0	-	7.7	-	7.7	33.0	-	-	0
1	16		47.13	0.0	19.8	9.9	33.0	9.9	33.0	9.9	32.2	0
				0.0	-	6.6	-	6.6	33.0	-	-	0
1	17		50.07	0.0	19.8	8.9	33.0	8.9	33.0	8.9	32.5	0
				0.0	-	8.9	-	8.9	33.0	-	-	0
1	18		53.02	0.0	19.8	6.3	33.0	6.3	33.0	6.3	33.0	0
				0.0	-	6.3	-	6.3	33.0	-	-	0
1	19		55.96	0.0	19.8	3.3	33.0	3.3	33.0	3.3	33.0	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	20		58.91	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.95	0.000	0.100	0.100	0.111	0.111	4
					0.100		0.100	3
1	2	5.89	0.000	0.189	0.189	0.210	0.210	4
					0.189		0.189	3
1	3	8.84	0.000	0.268	0.268	0.298	0.298	4
					0.268		0.268	3
1	4	11.78	0.000	0.301	0.301	0.338	0.338	4
					0.200		0.200	3
1	5	14.73	0.000	0.353	0.353	0.420	0.420	4
					0.234		0.234	3
1	6	17.67	0.000	0.395	0.395	0.470	0.470	4
					0.262		0.262	3
1	7	20.62	0.000	0.428	0.428	0.509	0.509	4
					0.283		0.283	3
1	8	23.56	0.000	0.451	0.451	0.537	0.537	4
					0.299		0.299	3
1	9	26.51	0.000	0.466	0.466	0.554	0.554	4
					0.308		0.308	3
1	10	29.45	0.000	0.471	0.471	0.560	0.560	4
					0.312		0.312	3
1	11	32.40	0.000	0.466	0.466	0.555	0.555	4
					0.309		0.309	3
1	12	35.35	0.000	0.452	0.452	0.464	0.464	4
					0.299		0.299	3
1	13	38.29	0.000	0.428	0.428	0.439	0.439	4
					0.283		0.283	3
1	14	41.24	0.000	0.395	0.395	0.405	0.405	4
					0.262		0.262	3
1	15	44.18	0.000	0.353	0.353	0.362	0.362	4
					0.234		0.234	3
1	16	47.13	0.000	0.301	0.301	0.309	0.309	4
					0.199		0.199	3
1	17	50.07	0.000	0.268	0.268	0.272	0.272	4
					0.268		0.268	3
1	18	53.02	0.000	0.190	0.190	0.190	0.190	2
					0.190		0.190	3
1	19	55.96	0.000	0.100	0.100	0.100	0.100	2
					0.100		0.100	3
1	20	58.91	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0
1	1		2.95	0	482.6	2269.4	-	-	0.0	19.8	-	-	0--0
1	2		5.89	0	908.7	2269.4	-	-	0.0	19.8	-	-	0--0
1	3		8.84	0	1278.5	2269.4	-	-	0.0	19.8	-	-	0--0
1	4		11.78	0	1591.9	3290.0	-	-	0.0	19.8	-	-	0--0
1	5		14.73	0	1844.0	3290.0	-	-	0.0	19.8	-	-	0--0
1	6		17.67	0	2048.1	3290.0	-	-	0.0	19.8	-	-	0--0
1	7		20.62	0	2207.2	3290.0	-	-	0.0	19.8	-	-	0--0
1	8		23.56	0	2314.4	3290.0	-	-	0.0	19.8	-	-	0--0
1	9		26.51	0	2358.4	3290.0	-	-	0.0	19.8	-	-	0--0
1	10		29.45	0	2373.3	3290.0	-	-	0.0	19.8	-	-	0--0
1	11		32.40	0	2359.1	3290.0	-	-	0.0	19.8	-	-	0--0
1	12		35.35	0	2314.8	3290.0	-	-	0.0	19.8	-	-	0--0
1	13		38.29	0	2207.3	3290.0	-	-	0.0	19.8	-	-	0--0
1	14		41.24	0	2047.9	3290.0	-	-	0.0	19.8	-	-	0--0
1	15		44.18	0	1843.4	3290.0	-	-	0.0	19.8	-	-	0--0
1	16		47.13	0	1591.7	3290.0	-	-	0.0	19.8	-	-	0--0
1	17		50.07	0	1279.0	2269.4	-	-	0.0	19.8	-	-	0--0
1	18		53.02	0	909.4	2269.4	-	-	0.0	19.8	-	-	0--0
1	19		55.96	0	482.9	2269.4	-	-	0.0	19.8	-	-	0--0
1	20		58.91	0	0.0	2269.4	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	2.95	0	0.213				0.213	1
1	2	5.89	0	0.400				0.400	1
1	3	8.84	0	0.563				0.563	1
1	4	11.78	0	0.484				0.484	1
1	5	14.73	0	0.560				0.560	1
1	6	17.67	0	0.623				0.623	1
1	7	20.62	0	0.671				0.671	1
1	8	23.56	0	0.703				0.703	1
1	9	26.51	0	0.717				0.717	1
1	10	29.45	0	0.721				0.721	1
1	11	32.40	0	0.717				0.717	1
1	12	35.35	0	0.704				0.704	1
1	13	38.29	0	0.671				0.671	1
1	14	41.24	0	0.622				0.622	1
1	15	44.18	0	0.560				0.560	1
1	16	47.13	0	0.484				0.484	1
1	17	50.07	0	0.564				0.564	1
1	18	53.02	0	0.401				0.401	1
1	19	55.96	0	0.213				0.213	1
1	20	58.91	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	2.95	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	5.89	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	8.84	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	11.78	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	14.73	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	17.67	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	20.62	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	23.56	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	26.51	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	29.45	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

1 11	32.40	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 12	35.35	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 13	38.29	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 14	41.24	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 15	44.18	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 16	47.13	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 17	50.07	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 18	53.02	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 19	55.96	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1 20	58.91	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

Please read NOTE on the following page

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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0

              For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-6

              For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-7


           ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
              stiffened
              if (d0/D) < 3 without long. stiffener, or
              if (d0/D) > 3 with both transv. and long. stiffeners.
              For handling requirement, (d0/D) > (260/(D/tw))**2

                   Vu = CVp

              where D = clear, unsupported distance between flange components

                   d0 = distance between transverse stiffeners

           ** This Article (6.10.9.2) indicates that a designer cannot
              count on post-buckling shear resistance from tension-
              field action for an unstiffened girder.
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     TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                     ********************************

                           S T R E N G T H  C A T E G O R Y; SEE NOTE
                  ------------------------------------------------------------
     SP IN D FROM WEB STABILITY    LFD/LRFD      UNSTIFFENED            OVERALL
     NO NO L SUPT UNSTIFFENED,[1] MAX. SHEAR <=> SHEAR CAPA- CATEGORY     WEB
            (ft)  ---- D/tw -----    (kip)       CITY,   [2]
     ---------------------------------------------------------------------------

      1  0   0.00       0            204.7   <      347.9       0          0
      1  1   2.95       0            189.4   <      347.9       0          0
      1  2   5.89       0            174.2   <      347.9       0          0
      1  3   8.84       0            159.2   <      347.9       0          0
      1  4  11.78       0            144.3   <      347.9       0          0
                                                Page 37
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1	5	14.73	0	129.2	<	347.9	0	0
1	6	17.67	0	114.6	<	347.9	0	0
1	7	20.62	0	100.1	<	347.9	0	0
1	8	23.56	0	85.8	<	347.9	0	0
1	9	26.51	0	71.5	<	347.9	0	0
1	10	29.45	0	57.6	<	347.9	0	0
1	11	32.40	0	71.2	<	347.9	0	0
1	12	35.35	0	85.9	<	347.9	0	0
1	13	38.29	0	100.2	<	347.9	0	0
1	14	41.24	0	114.7	<	347.9	0	0
1	15	44.18	0	129.3	<	347.9	0	0
1	16	47.13	0	144.0	<	347.9	0	0
1	17	50.07	0	158.9	<	347.9	0	0
1	18	53.02	0	174.3	<	347.9	0	0
1	19	55.96	0	189.5	<	347.9	0	0
1	20	58.91	0	204.8	<	347.9	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	204.70	347.87	0	
1	1	2.95	33.	189.37	347.87	0	
1	2	5.89	33.	174.19	347.87	0	
1	3	8.84	33.	159.16	347.87	0	
1	4	11.78	33.	144.26	347.87	0	
1	5	14.73	33.	129.16	347.87	0	
1	6	17.67	33.	114.55	347.87	0	
1	7	20.62	33.	100.10	347.87	0	
1	8	23.56	33.	85.78	347.87	0	
1	9	26.51	33.	71.46	347.87	0	
1	10	29.45	33.	57.59	347.87	0	
1	11	32.40	33.	71.22	347.87	0	
1	12	35.35	33.	85.90	347.87	0	
1	13	38.29	33.	100.21	347.87	0	
1	14	41.24	33.	114.67	347.87	0	
1	15	44.18	33.	129.27	347.87	0	
1	16	47.13	33.	144.02	347.87	0	
1	17	50.07	33.	158.91	347.87	0	
1	18	53.02	33.	174.31	347.87	0	
1	19	55.96	33.	189.49	347.87	0	
1	20	58.91	33.	204.82	347.87	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	COMP. FLG fcrw (ksi)	[3]	FLAG TBC
1	0	0.00	0.0	-		31.4	0.0	31.4	-	-	00-
1	1	2.95	2.9	-		31.4	8.0	31.4	-	-	00-
1	2	5.89	5.4	-		31.4	15.1	31.4	-	-	00-
1	3	8.84	7.7	-		31.4	21.2	31.4	-	-	00-
1	4	11.78	9.4	-		31.4	15.9	31.4	-	-	00-
1	5	14.73	11.0	-		31.4	18.5	31.4	-	-	00-
1	6	17.67	12.2	-		31.4	20.6	31.4	-	-	00-
1	7	20.62	13.2	-		31.4	22.2	31.4	-	-	00-
1	8	23.56	13.9	-		31.4	23.3	31.4	-	-	00-
1	9	26.51	14.3	-		31.4	23.7	31.4	-	-	00-
1	10	29.45	14.5	-		31.4	23.9	31.4	-	-	00-
1	11	32.40	14.3	-		31.4	23.7	31.4	-	-	00-
1	12	35.35	14.0	-		31.4	23.3	31.4	-	-	00-
1	13	38.29	13.2	-		31.4	22.2	31.4	-	-	00-
1	14	41.24	12.2	-		31.4	20.6	31.4	-	-	00-
1	15	44.18	11.0	-		31.4	18.5	31.4	-	-	00-
1	16	47.13	9.4	-		31.4	15.9	31.4	-	-	00-
1	17	50.07	7.7	-		31.4	21.2	31.4	-	-	00-
1	18	53.02	5.4	-		31.4	15.1	31.4	-	-	00-
1	19	55.96	2.9	-		31.4	8.0	31.4	-	-	00-
1	20	58.91	0.0	-		31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)

```
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
            but not to exceed the smaller of RhFyc and Fyw/0.7
            k = bending-buckling coefficient
              = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
            where:
            Dc = depth of the web in compression in the elastic
                 range. For composite sections, Dc shall be
                 determined as specified in Article D6.3.1

      "-" is N.A.

      Flag check - 0 = OK; 1 = NG
      T = Top Flange; B = Bottom Flange; C = Comp. Flange

      The values of fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
               ----------------------------- -----------------------------
SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
-----------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   2.95    TR     -0.2 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1  2   5.89    TR     -0.3 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1  3   8.84    TR     -0.4 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1      8.95  SCG
 1  4  11.78    TR     -0.7 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1  5  14.73    TR     -0.8 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1  6  17.67    TR     -0.9 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1  7  20.62    TR     -0.9 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1  8  23.56    TR     -0.9 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1  9  26.51    TR     -0.9 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 10  29.45    TR     -0.9 A B B^C^C D E E^  TR      4.8 A B B^C^C D
 1 11  32.40    TR     -0.9 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 12  35.35    TR     -0.9 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1 13  38.29    TR     -0.9 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 14  41.24    TR     -0.9 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1 15  44.18    TR     -0.8 A B B^C^C D E E^  TR      4.3 A B B^C^C D E
 1 16  47.13    TR     -0.7 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1     49.96  SCG
 1 17  50.07    TR     -0.4 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 18  53.02    TR     -0.3 A B B^C^C D E E^  TR      3.7 A B B^C^C D E
 1 19  55.96    TR     -0.2 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 20  58.91    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


-------------------------------------------------------------------------------
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.
-------------------------------------------------------------------------------
NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
             POC = Dead load point of contraflexure
```

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.95	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	2	5.89	TR	-0.2	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	3	8.84	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1		8.95	SCG					
1	4	11.78	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	5	14.73	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	6	17.67	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	7	20.62	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	8	23.56	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	9	26.51	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	10	29.45	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	11	32.40	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	12	35.35	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	13	38.29	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	14	41.24	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	15	44.18	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	16	47.13	TR	-0.3	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1		49.96	SCG					
1	17	50.07	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	18	53.02	TR	-0.2	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	19	55.96	TR	-0.1	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	20	58.91	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

5969 Span 2-S19 Interior Beam.RES

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-1 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	52.6	.29242E-01	1.5	TR	3.09	3	6.04
1	1	2.95	50.4	.29242E-01	1.5	TR	3.09	3	6.29
1	2	5.89	48.3	.29242E-01	1.4	TR	3.09	3	6.57
1	3	8.84	46.1	.29242E-01	1.3	TR	3.09	3	6.88
1	4	11.78	44.1	.26571E-01	1.2	TR	3.09	3	7.92
1	5	14.73	42.4	.26571E-01	1.1	TR	3.09	3	8.25
1	6	17.67	40.8	.26571E-01	1.1	TR	3.09	3	8.56
1	7	20.62	39.2	.26571E-01	1.0	TR	3.09	3	8.91
1	8	23.56	38.3	.26571E-01	1.0	TR	3.09	3	9.13
1	9	26.51	38.3	.26571E-01	1.0	TR	3.09	3	9.13
1	10	29.45	38.3	.26571E-01	1.0	TR	3.09	3	9.13
1	11	32.40	38.3	.26571E-01	1.0	TR	3.09	3	9.13
1	12	35.35	38.3	.26571E-01	1.0	TR	3.09	3	9.13
1	13	38.29	39.2	.26571E-01	1.0	TR	3.09	3	8.91
1	14	41.24	40.8	.26571E-01	1.1	TR	3.09	3	8.56
1	15	44.18	42.4	.26571E-01	1.1	TR	3.09	3	8.25
1	16	47.13	44.1	.26571E-01	1.2	TR	3.09	3	7.92
1	17	50.07	46.1	.29242E-01	1.3	TR	3.09	3	6.88
1	18	53.02	48.3	.29242E-01	1.4	TR	3.09	3	6.57
1	19	55.96	50.4	.29242E-01	1.5	TR	3.09	3	6.29
1	20	58.91	52.6	.29242E-01	1.5	TR	3.09	3	6.04

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-2 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	26.3	.29242E-01	0.8	TR	1.36	3	5.32
1	1	2.95	25.2	.29242E-01	0.7	TR	1.36	3	5.55
1	2	5.89	24.2	.29242E-01	0.7	TR	1.36	3	5.79
1	3	8.84	23.1	.29242E-01	0.7	TR	1.36	3	6.07
1	4	11.78	22.1	.26571E-01	0.6	TR	1.36	3	6.98
1	5	14.73	21.2	.26571E-01	0.6	TR	1.36	3	7.27
1	6	17.67	20.4	.26571E-01	0.5	TR	1.36	3	7.55
1	7	20.62	19.6	.26571E-01	0.5	TR	1.36	3	7.85
1	8	23.56	19.1	.26571E-01	0.5	TR	1.36	3	8.05
1	9	26.51	19.1	.26571E-01	0.5	TR	1.36	3	8.05
1	10	29.45	19.1	.26571E-01	0.5	TR	1.36	3	8.05
1	11	32.40	19.1	.26571E-01	0.5	TR	1.36	3	8.05
1	12	35.35	19.1	.26571E-01	0.5	TR	1.36	3	8.05
1	13	38.29	19.6	.26571E-01	0.5	TR	1.36	3	7.85
1	14	41.24	20.4	.26571E-01	0.5	TR	1.36	3	7.55
1	15	44.18	21.2	.26571E-01	0.6	TR	1.36	3	7.27
1	16	47.13	22.1	.26571E-01	0.6	TR	1.36	3	6.98
1	17	50.07	23.1	.29242E-01	0.7	TR	1.36	3	6.07
1	18	53.02	24.2	.29242E-01	0.7	TR	1.36	3	5.79
1	19	55.96	25.2	.29242E-01	0.7	TR	1.36	3	5.55
1	20	58.91	26.3	.29242E-01	0.8	TR	1.36	3	5.32

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D FROM L (ft)	SUPT 1=POS. 0=NEG.	MOMENT REGION P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE) FATIG. CRI.	<=>	SEE NOTE N1	N2	STATUS BLANK=OK CHECK=**
1	10	29.45	1	1620.3	1626.6		177	>	84		
1	20	58.91	1	1620.3	1626.6		177	>	84		

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where

N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment

N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support

P = total nominal horizontal shear force

Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)

Qn = nominal shear resistance

(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or
P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	29.455	6.000
1	29.455	58.910	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	TCR
1	0.0	58.9	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft ksi	D2	UNFACTORED L+I MAX. MOMENT POS. MAX. STRESS	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.95	2269.4	74.0	37.5	192.7	0.0	8.17	HL-93	6.30	0
1	2	5.89	2269.4	140.3	71.1	361.8	0.0	4.08	HL-93	3.15	0
1	3	8.84	2269.4	199.0	100.7	507.3	0.0	2.74	HL-93	2.12	0
1	4	11.78	3290.0	250.0	126.3	629.3	0.0	3.30	HL-93	2.54	0
1	5	14.73	3290.0	293.1	148.0	725.1	0.0	2.77	HL-93	2.14	0
1	6	17.67	3290.0	327.9	165.8	802.5	0.0	2.44	HL-93	1.88	0
1	7	20.62	3290.0	355.0	179.6	862.9	0.0	2.23	HL-93	1.72	0
1	8	23.56	3290.0	374.5	189.5	902.3	0.0	2.10	HL-93	1.62	0
1	9	26.51	3290.0	386.3	195.4	914.3	0.0	2.05	HL-93	1.58	0
1	10	29.45	3290.0	390.4	197.4	918.2	0.0	2.04	HL-93	1.57	0
1	11	32.40	3290.0	386.8	195.4	914.3	0.0	2.05	HL-93	1.58	0
1	12	35.35	3290.0	374.8	189.5	902.3	0.0	2.10	HL-93	1.62	0
1	13	38.29	3290.0	355.1	179.6	862.9	0.0	2.23	HL-93	1.72	0
1	14	41.24	3290.0	327.7	165.8	802.5	0.0	2.44	HL-93	1.88	0
1	15	44.18	3290.0	292.6	148.0	725.1	0.0	2.77	HL-93	2.14	0
1	16	47.13	3290.0	249.9	126.3	629.3	0.0	3.30	HL-93	2.54	0
1	17	50.07	2269.4	199.4	100.7	507.3	0.0	2.74	HL-93	2.12	0
1	18	53.02	2269.4	140.9	71.1	361.8	0.0	4.08	HL-93	3.15	0
1	19	55.96	2269.4	74.3	37.5	192.7	0.0	8.16	HL-93	6.30	0
1	20	58.91	2269.4	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	SUPT	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	2.95	31.4	-2.46	3.01	-0.31 3.83	0.00 0.00	7.40	HL-93	5.69
1	2	5.89	31.4	-4.66	5.71	-0.59 7.19	0.00 0.00	3.57	HL-93	2.74
1	3	8.84	31.4	-6.61	8.10	-0.82 10.08	0.00 0.00	2.31	HL-93	1.77
1	4	11.78	31.4	-7.55	6.06	-1.41 7.60	0.00 0.00	3.33	HL-93	2.56
1	5	14.73	31.4	-8.85	7.11	-1.63 8.76	0.00 0.00	2.77	HL-93	2.13
1	6	17.67	31.4	-9.90	7.96	-1.80 9.70	0.00 0.00	2.41	HL-93	1.86
1	7	20.62	31.4	-10.72	8.62	-1.94 10.43	0.00 0.00	2.18	HL-93	1.68
1	8	23.56	31.4	-11.31	9.09	-2.03 10.90	0.00 0.00	2.04	HL-93	1.57
1	9	26.51	31.4	-11.66	9.37	-2.05 11.05	0.00 0.00	1.99	HL-93	1.53
1	10	29.45	31.4	-11.79	9.47	-2.06 11.10	0.00 0.00	1.97	HL-93	1.52
1	11	32.40	31.4	-11.68	9.38	-2.05 11.05	0.00 0.00	1.99	HL-93	1.53
1	12	35.35	31.4	-11.32	9.09	-2.03 10.90	0.00 0.00	2.04	HL-93	1.57
1	13	38.29	31.4	-10.72	8.62	-1.94 10.43	0.00 0.00	2.18	HL-93	1.68
1	14	41.24	31.4	-9.89	7.95	-1.80 9.70	0.00 0.00	2.41	HL-93	1.86
1	15	44.18	31.4	-8.84	7.10	-1.63 8.76	0.00 0.00	2.77	HL-93	2.13
1	16	47.13	31.4	-7.54	6.06	-1.41 7.60	0.00 0.00	3.33	HL-93	2.56
1	17	50.07	31.4	-6.62	8.11	-0.82 10.08	0.00 0.00	2.31	HL-93	1.77
1	18	53.02	31.4	-4.68	5.73	-0.59 7.19	0.00 0.00	3.56	HL-93	2.74
1	19	55.96	31.4	-2.47	3.02	-0.31 3.83	0.00 0.00	7.40	HL-93	5.69
1	20	58.91	31.4	0.00	0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	58.91	29.45	STRENGTH L.S.	2.04	1.57	HL-93
1	58.91	29.45	SERVICEBILITY	1.97	1.52	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.972
Inventory= 1.517

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	58.91	35.35	STRENGTH L.S.	2.91	876.95
1	58.91	35.35	SERVICEBILITY	2.69	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.693

1

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	26.4	13.4	87.3	0.0	2.51	HL-93	1.94
1	1	2.95	347.9	23.8	12.1	81.5	-1.9	2.74	HL-93	2.11
1	2	5.89	347.9	21.2	10.7	75.7	-4.0	2.99	HL-93	2.31

1	3	8.84	347.9	18.6	9.4	70.1	-7.1	3.29	HL-93	2.54
1	4	11.78	347.9	16.0	8.0	64.5	-10.3	3.63	HL-93	2.80
1	5	14.73	347.9	13.1	6.7	59.0	-13.6	4.04	HL-93	3.12
1	6	17.67	347.9	10.5	5.4	53.6	-17.0	4.52	HL-93	3.49
1	7	20.62	347.9	7.9	4.0	48.3	-20.5	5.10	HL-93	3.93
1	8	23.56	347.9	5.3	2.7	43.1	-24.0	5.80	HL-93	4.48
1	9	26.51	347.9	2.7	1.3	37.8	-28.1	6.71	HL-93	5.18
1	10	29.45	347.9	0.1	0.0	32.8	-32.8	7.84	HL-93	6.05
1	11	32.40	347.9	-2.5	-1.3	28.1	-37.8	6.71	HL-93	5.18
1	12	35.35	347.9	-5.4	-2.7	24.0	-43.1	5.80	HL-93	4.48
1	13	38.29	347.9	-8.0	-4.0	20.5	-48.3	5.09	HL-93	3.93
1	14	41.24	347.9	-10.6	-5.4	17.0	-53.6	4.52	HL-93	3.48
1	15	44.18	347.9	-13.2	-6.7	13.6	-59.0	4.04	HL-93	3.12
1	16	47.13	347.9	-15.8	-8.0	10.3	-64.5	3.64	HL-93	2.81
1	17	50.07	347.9	-18.4	-9.4	7.1	-70.1	3.29	HL-93	2.54
1	18	53.02	347.9	-21.3	-10.7	4.0	-75.7	2.99	HL-93	2.31
1	19	55.96	347.9	-23.9	-12.1	1.9	-81.5	2.74	HL-93	2.11
1	20	58.91	347.9	-26.5	-13.4	0.0	-87.3	2.51	HL-93	1.94

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	58.91	58.91	STRENGTH L.S.	2.51	1.94	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.510
Inventory= 1.936

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	58.91	58.91	STRENGTH L.S.	3.06	96.77

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.057

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

1	29.45	3	0.425	-0.32	HL-93	2.75
		4	0.433	-0.33		2.70

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95NB Span 2-S20 Ext. Fat & Mid M	8/19/13

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Exterior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	16	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	STANDARD SECTN PLATE GIRDER DEPTH (in)	WEIGHT (lb/ft)	WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130			0.00	0.0000	0.00	0.0000		
2	W	33	130			0.00	0.0000	10.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB END SECT LEFT	ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.07				33.	33.	33.
2	2	2	39.00				33.	33.	33.
3	1	1	11.07				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

5969 Span 2-S20 Exterior Fatigue and Mid M.RES

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1					DF M+ ST/SE	LOADING A D M G C				
1	33.00	0	0.00	0.37	0.00	15.00	1	1	1	1	1	0.56	2	2	2	2	2
1	0.00	0	0.00	0.40	0.00	0.00	0	0	0	0	0	0.56	3	3	3	3	3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	61.14

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

5969 Span 2-S20 Exterior Fatigue and Mid M.RES

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	18.84
2	2	Diaphragm	0.000	0.00	0.00	0.14	37.68
3	2	Diaphragm	0.000	0.00	0.00	0.14	56.52
4	1	Railing	0.087	0.00	61.14	0.00	0.00
5	1	Curb	0.076	0.00	61.14	0.00	0.00
6	0	Pavement	0.292	0.00	61.14	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	18.84	33.0	33.0	18.84
18.84	37.68	33.0	33.0	18.84
37.68	56.52	33.0	33.0	18.84
56.52	61.14	33.0	33.0	4.62

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.555	0.1520	0.7070

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	61.14
1	DC2	0.087	0.000	61.140
2	DC2	0.076	0.000	61.140

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1484.6	55.0	222.7	55.8	278.44

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.562	0.562	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.561				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.399				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.82	6710.0	33.1	16.55	405.6	405.6
1	2	7.64	6710.0	33.1	16.55	405.6	405.6
1	3	11.46	9020.9	33.1	14.02	643.6	449.4
1	4	15.28	9020.9	33.1	14.02	643.6	449.4
1	5	19.11	9020.9	33.1	14.02	643.6	449.4
1	6	22.93	9020.9	33.1	14.02	643.6	449.4
1	7	26.75	9020.9	33.1	14.02	643.6	449.4
1	8	30.57	9020.9	33.1	14.02	643.6	449.4

1	9	34.39	9020.9	33.1	14.02	643.6	449.4
1	10	38.21	9020.9	33.1	14.02	643.6	449.4
1	11	42.03	9020.9	33.1	14.02	643.6	449.4
1	12	45.86	9020.9	33.1	14.02	643.6	449.4
1	13	49.68	9020.9	33.1	14.02	643.6	449.4
1	14	53.50	6710.0	33.1	16.55	405.6	405.6
1	15	57.32	6710.0	33.1	16.55	405.6	405.6
1	16	61.14	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12457.3	0.021730	529.5	1302.9	697.4
1	1	3.82	12457.3	0.021730	529.5	1302.9	697.4
1	2	7.64	12457.3	0.021730	529.5	1302.9	697.4
1	3	11.46	17378.6	0.019536	829.5	1322.6	810.6
1	4	15.28	17378.6	0.019536	829.5	1322.6	810.6
1	5	19.11	17378.6	0.019536	829.5	1322.6	810.6
1	6	22.93	17378.6	0.019536	829.5	1322.6	810.6
1	7	26.75	17378.6	0.019536	829.5	1322.6	810.6
1	8	30.57	17378.6	0.019536	829.5	1322.6	810.6
1	9	34.39	17378.6	0.019536	829.5	1322.6	810.6
1	10	38.21	17378.6	0.019536	829.5	1322.6	810.6
1	11	42.03	17378.6	0.019536	829.5	1322.6	810.6
1	12	45.86	17378.6	0.019536	829.5	1322.6	810.6
1	13	49.68	17378.6	0.019536	829.5	1322.6	810.6
1	14	53.50	12457.3	0.021730	529.5	1302.9	697.4
1	15	57.32	12457.3	0.021730	529.5	1302.9	697.4
1	16	61.14	12457.3	0.021730	529.5	1302.9	697.4

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	17203.8	0.028305	589.9	4381.3	1407.1
1	1	3.82	17203.8	0.028305	589.9	4381.3	1407.1
1	2	7.64	17203.8	0.028305	589.9	4381.3	1407.1
1	3	11.46	25171.5	0.025828	920.8	3726.8	1672.1
1	4	15.28	25171.5	0.025828	920.8	3726.8	1672.1
1	5	19.11	25171.5	0.025828	920.8	3726.8	1672.1
1	6	22.93	25171.5	0.025828	920.8	3726.8	1672.1
1	7	26.75	25171.5	0.025828	920.8	3726.8	1672.1

1	8	30.57	25171.5	0.025828	920.8	3726.8	1672.1
1	9	34.39	25171.5	0.025828	920.8	3726.8	1672.1
1	10	38.21	25171.5	0.025828	920.8	3726.8	1672.1
1	11	42.03	25171.5	0.025828	920.8	3726.8	1672.1
1	12	45.86	25171.5	0.025828	920.8	3726.8	1672.1
1	13	49.68	25171.5	0.025828	920.8	3726.8	1672.1
1	14	53.50	17203.8	0.028305	589.9	4381.3	1407.1
1	15	57.32	17203.8	0.028305	589.9	4381.3	1407.1
1	16	61.14	17203.8	0.028305	589.9	4381.3	1407.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	3.82	16.6	60.8	0.6	0.0	78.1	
1	2	7.64	31.1	113.5	1.3	0.0	145.8	
1	3	11.46	43.3	158.0	1.9	0.0	203.2	
1	4	15.28	53.3	194.5	2.5	0.0	250.3	
1	5	19.11	61.0	222.9	3.1	0.0	287.0	
1	6	22.93	66.6	243.1	3.2	0.0	312.9	
1	7	26.75	69.9	255.3	3.3	0.0	328.5	
1	8	30.57	71.0	259.3	3.4	0.0	333.7	
1	9	34.39	69.9	255.3	3.5	0.0	328.7	
1	10	38.21	66.6	243.1	3.5	0.0	313.2	
1	11	42.03	61.0	222.9	3.0	0.0	286.9	
1	12	45.86	53.3	194.5	2.5	0.0	250.3	
1	13	49.68	43.3	158.0	2.1	0.0	203.4	
1	14	53.50	31.1	113.5	1.6	0.0	146.1	

1 15	57.32	16.6	60.8	1.0	0.0	78.4
1 16	61.14	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.82	32.0	17.9	0.0	0.0	0.0	49.8
1	2	7.64	59.7	33.3	0.0	0.0	0.0	93.0
1	3	11.46	83.1	46.4	0.0	0.0	0.0	129.6
1	4	15.28	102.3	57.1	0.0	0.0	0.0	159.5
1	5	19.11	117.3	65.5	0.0	0.0	0.0	182.7
1	6	22.93	127.9	71.4	0.0	0.0	0.0	199.3
1	7	26.75	134.3	75.0	0.0	0.0	0.0	209.3
1	8	30.57	136.4	76.2	0.0	0.0	0.0	212.6
1	9	34.39	134.3	75.0	0.0	0.0	0.0	209.3
1	10	38.21	127.9	71.4	0.0	0.0	0.0	199.3
1	11	42.03	117.3	65.5	0.0	0.0	0.0	182.7
1	12	45.86	102.3	57.1	0.0	0.0	0.0	159.5
1	13	49.68	83.1	46.4	0.0	0.0	0.0	129.6
1	14	53.50	59.7	33.3	0.0	0.0	0.0	93.0
1	15	57.32	32.0	17.9	0.0	0.0	0.0	49.8
1	16	61.14	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.82	0.0	0.0	200.8	HL-93	0.0	HL-93
1	2	7.64	0.0	0.0	370.6	HL-93	0.0	HL-93
1	3	11.46	0.0	0.0	509.5	HL-93	0.0	HL-93
1	4	15.28	0.0	0.0	615.3	HL-93	0.0	HL-93
1	5	19.11	0.0	0.0	696.0	HL-93	0.0	HL-93
1	6	22.93	0.0	0.0	752.2	HL-93	0.0	HL-93
1	7	26.75	0.0	0.0	775.7	HL-93	0.0	HL-93
1	8	30.57	0.0	0.0	781.4	HL-93	0.0	HL-93
1	9	34.39	0.0	0.0	775.7	HL-93	0.0	HL-93
1	10	38.21	0.0	0.0	752.2	HL-93	0.0	HL-93
1	11	42.03	0.0	0.0	696.0	HL-93	0.0	HL-93
1	12	45.86	0.0	0.0	615.3	HL-93	0.0	HL-93
1	13	49.68	0.0	0.0	509.5	HL-93	0.0	HL-93
1	14	53.50	0.0	0.0	370.6	HL-93	0.0	HL-93
1	15	57.32	0.0	0.0	200.8	HL-93	0.0	HL-93
1	16	61.14	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.82	74.	0.	74.
1	2	7.64	133.	0.	133.
1	3	11.46	178.	0.	178.
1	4	15.28	214.	0.	214.
1	5	19.11	239.	0.	239.
1	6	22.93	253.	0.	253.
1	7	26.75	244.	0.	244.
1	8	30.57	238.	0.	238.
1	9	34.39	244.	0.	244.
1	10	38.21	253.	0.	253.
1	11	42.03	239.	0.	239.
1	12	45.86	214.	0.	214.
1	13	49.68	178.	0.	178.
1	14	53.50	133.	0.	133.
1	15	57.32	74.	0.	74.
1	16	61.14	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.82	78.1	49.8	200.8	HL-93	0.0	HL-93	328.7	127.9
1	2	7.64	145.8	93.0	370.6	HL-93	0.0	HL-93	609.4	238.8
1	3	11.46	203.2	129.6	509.5	HL-93	0.0	HL-93	842.3	332.8
1	4	15.28	250.3	159.5	615.3	HL-93	0.0	HL-93	1025.1	409.8
1	5	19.11	287.0	182.7	696.0	HL-93	0.0	HL-93	1165.7	469.7
1	6	22.93	312.9	199.3	752.2	HL-93	0.0	HL-93	1264.4	512.2
1	7	26.75	328.5	209.3	775.7	HL-93	0.0	HL-93	1313.5	537.8
1	8	30.57	333.7	212.6	781.4	HL-93	0.0	HL-93	1327.7	546.3
1	9	34.39	328.7	209.3	775.7	HL-93	0.0	HL-93	1313.6	537.9
1	10	38.21	313.2	199.3	752.2	HL-93	0.0	HL-93	1264.6	512.5
1	11	42.03	286.9	182.7	696.0	HL-93	0.0	HL-93	1165.6	469.6
1	12	45.86	250.3	159.5	615.3	HL-93	0.0	HL-93	1025.1	409.8
1	13	49.68	203.4	129.6	509.5	HL-93	0.0	HL-93	842.4	332.9
1	14	53.50	146.1	93.0	370.6	HL-93	0.0	HL-93	609.8	239.1
1	15	57.32	78.4	49.8	200.8	HL-93	0.0	HL-93	329.1	128.3
1	16	61.14	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.82	328.7	388.9	519.2	438.9	191.8	55.5
1	2	7.64	609.4	720.6	962.0	813.8	358.2	99.4
1	3	11.46	842.3	995.1	1328.4	1124.6	499.1	133.6
1	4	15.28	1025.1	1209.7	1614.6	1368.5	614.6	160.5
1	5	19.11	1165.7	1374.6	1834.5	1556.1	704.6	179.5
1	6	22.93	1264.4	1490.0	1988.5	1687.7	768.4	189.9
1	7	26.75	1313.5	1546.2	2063.3	1753.0	806.7	183.2
1	8	30.57	1327.7	1562.1	2084.4	1771.9	819.5	178.2
1	9	34.39	1313.6	1546.3	2063.5	1753.2	806.9	183.2
1	10	38.21	1264.6	1490.3	1988.9	1688.0	768.7	189.9
1	11	42.03	1165.6	1374.4	1834.3	1555.9	704.4	179.5
1	12	45.86	1025.1	1209.7	1614.6	1368.5	614.6	160.5
1	13	49.68	842.4	995.3	1328.6	1124.8	499.4	133.6
1	14	53.50	609.8	721.0	962.4	814.2	358.7	99.4
1	15	57.32	329.1	389.3	519.7	439.4	192.4	55.5
1	16	61.14	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.6	17.0	0.2	0.0	21.8
1	1	3.82	4.1	14.8	0.2	0.0	19.1
1	2	7.64	3.5	12.7	0.2	0.0	16.4
1	3	11.46	2.9	10.6	0.2	0.0	13.7
1	4	15.28	2.3	8.5	0.2	0.0	11.0
1	5	19.11	1.7	6.4	0.0	0.0	8.1
1	6	22.93	1.2	4.2	0.0	0.0	5.4
1	7	26.75	0.6	2.1	0.0	0.0	2.7
1	8	30.57	0.0	0.0	0.0	0.0	0.0
1	9	34.39	-0.6	-2.1	0.0	0.0	-2.7
1	10	38.21	-1.2	-4.2	-0.1	0.0	-5.5
1	11	42.03	-1.7	-6.4	-0.1	0.0	-8.2
1	12	45.86	-2.3	-8.5	-0.1	0.0	-10.9
1	13	49.68	-2.9	-10.6	-0.1	0.0	-13.6
1	14	53.50	-3.5	-12.7	-0.1	0.0	-16.3
1	15	57.32	-4.1	-14.8	-0.3	0.0	-19.2
1	16	61.14	-4.6	-17.0	-0.3	0.0	-21.9

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	21.8	35.7
1	1	3.82	7.8	4.4	0.0	0.0	12.2	19.1	31.2
1	2	7.64	6.7	3.7	0.0	0.0	10.4	16.4	26.8
1	3	11.46	5.6	3.1	0.0	0.0	8.7	13.7	22.4
1	4	15.28	4.5	2.5	0.0	0.0	7.0	11.0	17.9
1	5	19.11	3.3	1.9	0.0	0.0	5.2	8.1	13.3
1	6	22.93	2.2	1.2	0.0	0.0	3.5	5.4	8.9
1	7	26.75	1.1	0.6	0.0	0.0	1.7	2.7	4.5
1	8	30.57	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	9	34.39	-1.1	-0.6	0.0	0.0	-1.7	-2.7	-4.4
1	10	38.21	-2.2	-1.2	0.0	0.0	-3.5	-5.5	-9.0
1	11	42.03	-3.3	-1.9	0.0	0.0	-5.2	-8.2	-13.4
1	12	45.86	-4.5	-2.5	0.0	0.0	-7.0	-10.9	-17.9
1	13	49.68	-5.6	-3.1	0.0	0.0	-8.7	-13.6	-22.3
1	14	53.50	-6.7	-3.7	0.0	0.0	-10.4	-16.3	-26.8
1	15	57.32	-7.8	-4.4	0.0	0.0	-12.2	-19.2	-31.3
1	16	61.14	-8.9	-5.0	0.0	0.0	-13.9	-21.9	-35.8

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	56.5	HL-93	0.0	HL-93
1	1	3.82	0.0	0.0	51.8	HL-93	-1.5	HL-93
1	2	7.64	0.0	0.0	47.2	HL-93	-3.6	HL-93
1	3	11.46	0.0	0.0	42.7	HL-93	-6.2	HL-93
1	4	15.28	0.0	0.0	38.3	HL-93	-8.8	HL-93
1	5	19.11	0.0	0.0	33.9	HL-93	-11.5	HL-93
1	6	22.93	0.0	0.0	29.7	HL-93	-14.3	HL-93
1	7	26.75	0.0	0.0	25.4	HL-93	-17.6	HL-93
1	8	30.57	0.0	0.0	21.4	HL-93	-21.4	HL-93
1	9	34.39	0.0	0.0	17.6	HL-93	-25.4	HL-93
1	10	38.21	0.0	0.0	14.3	HL-93	-29.7	HL-93
1	11	42.03	0.0	0.0	11.5	HL-93	-33.9	HL-93
1	12	45.86	0.0	0.0	8.8	HL-93	-38.3	HL-93
1	13	49.68	0.0	0.0	6.2	HL-93	-42.7	HL-93
1	14	53.50	0.0	0.0	3.6	HL-93	-47.2	HL-93
1	15	57.32	0.0	0.0	1.5	HL-93	-51.8	HL-93
1	16	61.14	0.0	0.0	0.0	HL-93	-56.5	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT	TRUCK ONLY

		(ft)	POS	NEG	RANGE
1	0	0.00	23.	0.	23.
1	1	3.82	21.	-1.	22.
1	2	7.64	19.	-2.	21.
1	3	11.46	17.	-3.	20.
1	4	15.28	15.	-4.	19.
1	5	19.11	13.	-5.	18.
1	6	22.93	11.	-6.	17.
1	7	26.75	9.	-7.	16.
1	8	30.57	7.	-7.	15.
1	9	34.39	7.	-9.	16.
1	10	38.21	6.	-11.	17.
1	11	42.03	5.	-13.	18.
1	12	45.86	4.	-15.	19.
1	13	49.68	3.	-17.	20.
1	14	53.50	2.	-19.	21.
1	15	57.32	1.	-21.	22.
1	16	61.14	0.	-23.	23.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	23.2	.30607E-01	0.35	0.3125*	.15051E-01	0.17	0.3125*
1	1		3.82	22.0	.30607E-01	0.34	0.3125*	.15051E-01	0.17	0.3125*
1	2		7.64	20.9	.30607E-01	0.32	0.3125*	.15051E-01	0.16	0.3125*
1	3		11.46	19.7	.28474E-01	0.28	0.3125*	.92139E-02	0.09	0.3125*
1	4		15.28	18.7	.28474E-01	0.27	0.3125*	.92139E-02	0.09	0.3125*
1	5		19.11	17.9	.28474E-01	0.26	0.3125*	.92139E-02	0.08	0.3125*
1	6		22.93	17.3	.28474E-01	0.25	0.3125*	.92139E-02	0.08	0.3125*
1	7		26.75	15.7	.28474E-01	0.22	0.3125*	.92139E-02	0.07	0.3125*
1	8		30.57	14.9	.28474E-01	0.21	0.3125*	.92139E-02	0.07	0.3125*
1	9		34.39	15.7	.28474E-01	0.22	0.3125*	.92139E-02	0.07	0.3125*
1	10		38.21	17.3	.28474E-01	0.25	0.3125*	.92139E-02	0.08	0.3125*
1	11		42.03	17.9	.28474E-01	0.26	0.3125*	.92139E-02	0.08	0.3125*
1	12		45.86	18.7	.28474E-01	0.27	0.3125*	.92139E-02	0.09	0.3125*
1	13		49.68	19.7	.28474E-01	0.28	0.3125*	.92139E-02	0.09	0.3125*
1	14		53.50	20.9	.30607E-01	0.32	0.3125*	.15051E-01	0.16	0.3125*
1	15		57.32	22.0	.30607E-01	0.34	0.3125*	.15051E-01	0.17	0.3125*
1	16		61.14	23.2	.30607E-01	0.35	0.3125*	.15051E-01	0.17	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	35.7	56.5	HL-93	0.0	HL-93	92.2	35.7
1	1	3.82	31.2	51.8	HL-93	-1.5	HL-93	83.0	29.7
1	2	7.64	26.8	47.2	HL-93	-3.6	HL-93	74.0	23.2
1	3	11.46	22.4	42.7	HL-93	-6.2	HL-93	65.1	16.2
1	4	15.28	17.9	38.3	HL-93	-8.8	HL-93	56.2	9.1
1	5	19.11	13.3	33.9	HL-93	-11.5	HL-93	47.3	1.8
1	6	22.93	8.9	29.7	HL-93	-14.3	HL-93	38.6	-5.4
1	7	26.75	4.5	25.4	HL-93	-17.6	HL-93	29.9	-13.1
1	8	30.57	0.0	21.4	HL-93	-21.4	HL-93	21.4	-21.4
1	9	34.39	-4.4	17.6	HL-93	-25.4	HL-93	13.2	-29.9
1	10	38.21	-9.0	14.3	HL-93	-29.7	HL-93	5.3	-38.7
1	11	42.03	-13.4	11.5	HL-93	-33.9	HL-93	-1.9	-47.4
1	12	45.86	-17.9	8.8	HL-93	-38.3	HL-93	-9.1	-56.1
1	13	49.68	-22.3	6.2	HL-93	-42.7	HL-93	-16.2	-65.0
1	14	53.50	-26.8	3.6	HL-93	-47.2	HL-93	-23.2	-74.0
1	15	57.32	-31.3	1.5	HL-93	-51.8	HL-93	-29.8	-83.1
1	16	61.14	-35.8	0.0	HL-93	-56.5	HL-93	-35.8	-92.3

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	92.2	109.1	145.7	123.1	53.5	17.4
1	1	3.82	83.0	98.6	131.7	110.9	46.9	16.5
1	2	7.64	74.0	88.2	117.8	98.9	40.2	15.6
1	3	11.46	65.1	77.9	104.1	87.0	33.6	14.8
1	4	15.28	56.2	67.7	90.5	75.2	26.9	14.0
1	5	19.11	47.3	57.4	76.9	63.3	20.0	13.4
1	6	22.93	38.6	47.5	63.6	51.7	13.4	12.9
1	7	26.75	29.9	37.5	50.4	40.2	6.7	11.8
1	8	30.57	21.4	27.8	37.5	28.9	0.0	11.2
1	9	34.39	-29.9	-37.5	-50.3	-40.1	-6.6	11.8
1	10	38.21	-38.7	-47.6	-63.7	-51.8	-13.5	12.9
1	11	42.03	-47.4	-57.5	-77.0	-63.4	-20.2	13.4
1	12	45.86	-56.1	-67.6	-90.4	-75.1	-26.8	14.0
1	13	49.68	-65.0	-77.8	-104.0	-86.9	-33.5	14.8
1	14	53.50	-74.0	-88.1	-117.7	-98.8	-40.1	15.6
1	15	57.32	-83.1	-98.7	-131.8	-111.1	-47.0	16.5
1	16	61.14	-92.3	-109.2	-145.8	-123.2	-53.7	17.4

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	35.69	0.00	HL-93	56.49	HL-93	ST1	29.89	145.69
						ST2	29.89	123.10
						ST4	40.14	53.53
						SE1	35.69	92.18
						SE2	35.69	109.12
2	35.79	0.00	HL-93	56.49	HL-93	ST1	29.98	145.82
						ST2	29.98	123.22
						ST4	40.29	53.68
						SE1	35.79	92.28
						SE2	35.79	109.22

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D FROM NO L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1 0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 1	3.82	-0.0383	-0.1400	0.0000	-0.0600	-0.1801	-0.0600	-0.2402
1 2	7.64	-0.0745	-0.2722	0.0000	-0.1167	-0.3503	-0.1167	-0.4669
1 3	11.46	-0.1068	-0.3898	0.0000	-0.1668	-0.5016	-0.1668	-0.6684
1 4	15.28	-0.1343	-0.4903	0.0000	-0.2096	-0.6311	-0.2096	-0.8407
1 5	19.11	-0.1567	-0.5722	0.0000	-0.2444	-0.7364	-0.2444	-0.9809
1 6	22.93	-0.1732	-0.6326	0.0000	-0.2701	-0.8142	-0.2701	-1.0843
1 7	26.75	-0.1834	-0.6696	0.0000	-0.2859	-0.8619	-0.2859	-1.1478
1 8	30.57	-0.1868	-0.6821	0.0000	-0.2912	-0.8779	-0.2912	-1.1692
1 9	34.39	-0.1834	-0.6696	0.0000	-0.2859	-0.8619	-0.2859	-1.1478
1 10	38.21	-0.1732	-0.6326	0.0000	-0.2701	-0.8143	-0.2701	-1.0844
1 11	42.03	-0.1567	-0.5722	0.0000	-0.2444	-0.7365	-0.2444	-0.9809
1 12	45.86	-0.1343	-0.4903	0.0000	-0.2096	-0.6312	-0.2096	-0.8408
1 13	49.68	-0.1068	-0.3898	0.0000	-0.1668	-0.5018	-0.1668	-0.6686
1 14	53.50	-0.0745	-0.2722	0.0000	-0.1167	-0.3504	-0.1167	-0.4670
1 15	57.32	-0.0383	-0.1400	0.0000	-0.0600	-0.1802	-0.0600	-0.2403
1 16	61.14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.8	-0.038	0 1/16	-0.142	0 3/16	-0.060	0 1/16	-0.240	0 1/ 4
1	2	7.6	-0.075	0 1/ 8	-0.276	0 5/16	-0.117	0 1/ 8	-0.467	0 1/ 2
1	3	11.5	-0.107	0 1/ 8	-0.395	0 7/16	-0.167	0 3/16	-0.668	0 11/16
1	4	15.3	-0.134	0 3/16	-0.497	0 1/ 2	-0.210	0 1/ 4	-0.841	0 7/ 8
1	5	19.1	-0.157	0 3/16	-0.580	0 5/ 8	-0.244	0 1/ 4	-0.981	1 0/16
1	6	22.9	-0.173	0 3/16	-0.641	0 11/16	-0.270	0 5/16	-1.084	1 1/ 8
1	7	26.7	-0.183	0 3/16	-0.678	0 11/16	-0.286	0 5/16	-1.148	1 3/16
1	8	30.6	-0.187	0 3/16	-0.691	0 3/ 4	-0.291	0 5/16	-1.169	1 3/16
1	9	34.4	-0.183	0 3/16	-0.679	0 11/16	-0.286	0 5/16	-1.148	1 3/16
1	10	38.2	-0.173	0 3/16	-0.641	0 11/16	-0.270	0 5/16	-1.084	1 1/ 8
1	11	42.0	-0.157	0 3/16	-0.580	0 5/ 8	-0.244	0 1/ 4	-0.981	1 0/16
1	12	45.9	-0.134	0 3/16	-0.497	0 1/ 2	-0.210	0 1/ 4	-0.841	0 7/ 8
1	13	49.7	-0.107	0 1/ 8	-0.395	0 7/16	-0.167	0 3/16	-0.669	0 11/16
1	14	53.5	-0.075	0 1/ 8	-0.276	0 5/16	-0.117	0 1/ 8	-0.467	0 1/ 2
1	15	57.3	-0.038	0 1/16	-0.142	0 3/16	-0.060	0 1/16	-0.240	0 1/ 4
1	16	61.1	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	FACTOR	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.57	3	0.425	-0.413	MAX HL-93	0.92	0.00329
				0.033	MIN		
				-0.121	MAX LANE		
				0.033	MIN LANE		
		4	0.433	-0.421	MAX HL-93	0.92	0.00329
				0.034	MIN		
				-0.123	MAX LANE		
				0.034	MIN LANE		
		3	0.425	-0.303	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	-0.49	0.49	-1.82	1.82	-2.31	2.31
1	2	7.64	-0.92	0.92	-3.39	3.39	-4.31	4.31
1	3	11.46	-1.16	0.81	-4.27	2.98	-5.43	3.79
1	4	15.28	-1.42	0.99	-5.26	3.67	-6.68	4.67
1	5	19.11	-1.63	1.14	-6.03	4.21	-7.66	5.35
1	6	22.93	-1.78	1.24	-6.58	4.59	-8.36	5.83
1	7	26.75	-1.87	1.30	-6.91	4.82	-8.77	6.12
1	8	30.57	-1.90	1.32	-7.02	4.90	-8.91	6.22
1	9	34.39	-1.87	1.30	-6.91	4.82	-8.78	6.13
1	10	38.21	-1.78	1.24	-6.59	4.60	-8.36	5.84
1	11	42.03	-1.63	1.14	-6.03	4.21	-7.66	5.35
1	12	45.86	-1.42	0.99	-5.26	3.67	-6.68	4.67
1	13	49.68	-1.16	0.81	-4.28	2.98	-5.43	3.79
1	14	53.50	-0.92	0.92	-3.40	3.40	-4.32	4.32
1	15	57.32	-0.49	0.49	-1.83	1.83	-2.32	2.32
1	16	61.14	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	-0.03	-0.46	1.13	-0.03	-2.77	3.44
1	2	7.64	-0.06	-0.86	2.11	-0.06	-5.17	6.42
1	3	11.46	-0.07	-1.18	1.87	-0.07	-6.60	5.66
1	4	15.28	-0.09	-1.45	2.31	-0.09	-8.13	6.97
1	5	19.11	-0.10	-1.66	2.64	-0.10	-9.32	7.99
1	6	22.93	-0.11	-1.81	2.88	-0.11	-10.16	8.72
1	7	26.75	-0.11	-1.90	3.03	-0.11	-10.67	9.15
1	8	30.57	-0.12	-1.93	3.08	-0.12	-10.84	9.30

1	9	34.39	-0.11	-1.90	3.03	-0.11	-10.68	9.15
1	10	38.21	-0.11	-1.81	2.88	-0.11	-10.17	8.72
1	11	42.03	-0.10	-1.66	2.64	-0.10	-9.32	7.99
1	12	45.86	-0.09	-1.45	2.31	-0.09	-8.13	6.97
1	13	49.68	-0.07	-1.18	1.87	-0.07	-6.61	5.67
1	14	53.50	-0.06	-0.86	2.11	-0.06	-5.18	6.43
1	15	57.32	-0.03	-0.46	1.13	-0.03	-2.78	3.45
1	16	61.14	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	-0.19	-0.55	4.08	0.00	0.00	0.00
1	2	7.64	-0.35	-1.02	7.54	0.00	0.00	0.00
1	3	11.46	-0.41	-1.64	6.64	0.00	0.00	0.00
1	4	15.28	-0.49	-1.98	8.02	0.00	0.00	0.00
1	5	19.11	-0.56	-2.24	9.07	0.00	0.00	0.00
1	6	22.93	-0.60	-2.42	9.80	0.00	0.00	0.00
1	7	26.75	-0.62	-2.50	10.11	0.00	0.00	0.00
1	8	30.57	-0.62	-2.52	10.18	0.00	0.00	0.00
1	9	34.39	-0.62	-2.50	10.11	0.00	0.00	0.00
1	10	38.21	-0.60	-2.42	9.80	0.00	0.00	0.00
1	11	42.03	-0.56	-2.24	9.07	0.00	0.00	0.00
1	12	45.86	-0.49	-1.98	8.02	0.00	0.00	0.00
1	13	49.68	-0.41	-1.64	6.64	0.00	0.00	0.00
1	14	53.50	-0.35	-1.02	7.54	0.00	0.00	0.00
1	15	57.32	-0.19	-0.55	4.08	0.00	0.00	0.00
1	16	61.14	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP TYPE
1	0	0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1	3.82	0.2	0.0	0.2	C HL	1.5	0.0	1.5	T HL
1	2	7.64	0.4	0.0	0.4	C HL	2.7	0.0	2.7	T HL
1	3	11.46	0.6	0.0	0.6	C HL	2.3	0.0	2.3	T HL
1	4	15.28	0.7	0.0	0.7	C HL	2.8	0.0	2.8	T HL
1	5	19.11	0.8	0.0	0.8	C HL	3.1	0.0	3.1	T HL
1	6	22.93	0.8	0.0	0.8	C HL	3.3	0.0	3.3	T HL
1	7	26.75	0.8	0.0	0.8	C HL	3.2	0.0	3.2	T HL
1	8	30.57	0.8	0.0	0.8	C HL	3.1	0.0	3.1	T HL
1	9	34.39	0.8	0.0	0.8	C HL	3.2	0.0	3.2	T HL
1	10	38.21	0.8	0.0	0.8	C HL	3.3	0.0	3.3	T HL
1	11	42.03	0.8	0.0	0.8	C HL	3.1	0.0	3.1	T HL
1	12	45.86	0.7	0.0	0.7	C HL	2.8	0.0	2.8	T HL
1	13	49.68	0.6	0.0	0.6	C HL	2.3	0.0	2.3	T HL

1 14	53.50	0.4	0.0	0.4 C	HL	2.7	0.0	2.7 T	HL
1 15	57.32	0.2	0.0	0.2 C	HL	1.5	0.0	1.5 T	HL
1 16	61.14	0.0	0.0	0.0 T	HL	0.0	0.0	0.0 T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.22	-3.32	7.52	-0.03	-2.77	3.44
1	2	7.64	33.	33.	-0.41	-6.19	13.96	-0.06	-5.17	6.42
1	3	11.46	33.	33.	-0.48	-8.24	12.30	-0.07	-6.60	5.66
1	4	15.28	33.	33.	-0.58	-10.11	14.99	-0.09	-8.13	6.97
1	5	19.11	33.	33.	-0.66	-11.56	17.06	-0.10	-9.32	7.99
1	6	22.93	33.	33.	-0.71	-12.59	18.52	-0.11	-10.16	8.72
1	7	26.75	33.	33.	-0.73	-13.17	19.26	-0.11	-10.67	9.15
1	8	30.57	33.	33.	-0.74	-13.36	19.48	-0.12	-10.84	9.30
1	9	34.39	33.	33.	-0.73	-13.17	19.26	-0.11	-10.68	9.15
1	10	38.21	33.	33.	-0.71	-12.59	18.52	-0.11	-10.17	8.72
1	11	42.03	33.	33.	-0.66	-11.56	17.06	-0.10	-9.32	7.99
1	12	45.86	33.	33.	-0.58	-10.11	14.99	-0.09	-8.13	6.97
1	13	49.68	33.	33.	-0.48	-8.25	12.31	-0.07	-6.61	5.67
1	14	53.50	33.	33.	-0.41	-6.20	13.97	-0.06	-5.18	6.43
1	15	57.32	33.	33.	-0.22	-3.33	7.53	-0.03	-2.78	3.45
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.28	-3.48	8.75	-0.03	-2.77	3.44
1	2	7.64	33.	33.	-0.52	-6.49	16.22	-0.06	-5.17	6.42
1	3	11.46	33.	33.	-0.60	-8.73	14.29	-0.07	-6.60	5.66
1	4	15.28	33.	33.	-0.73	-10.71	17.40	-0.09	-8.13	6.97
1	5	19.11	33.	33.	-0.82	-12.24	19.79	-0.10	-9.32	7.99
1	6	22.93	33.	33.	-0.89	-13.31	21.46	-0.11	-10.16	8.72
1	7	26.75	33.	33.	-0.92	-13.92	22.29	-0.11	-10.67	9.15
1	8	30.57	33.	33.	-0.93	-14.11	22.54	-0.12	-10.84	9.30
1	9	34.39	33.	33.	-0.92	-13.92	22.30	-0.11	-10.68	9.15
1	10	38.21	33.	33.	-0.89	-13.32	21.46	-0.11	-10.17	8.72
1	11	42.03	33.	33.	-0.82	-12.23	19.78	-0.10	-9.32	7.99
1	12	45.86	33.	33.	-0.73	-10.71	17.40	-0.09	-8.13	6.97
1	13	49.68	33.	33.	-0.60	-8.74	14.30	-0.07	-6.61	5.67

1	14	53.50	33.	33.	-0.52	-6.50	16.23	-0.06	-5.18	6.43
1	15	57.32	33.	33.	-0.28	-3.49	8.76	-0.03	-2.78	3.45
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.38	-4.50	11.63	-0.04	-3.53	4.48
1	2	7.64	33.	33.	-0.70	-8.38	21.56	-0.08	-6.60	8.37
1	3	11.46	33.	33.	-0.81	-11.31	19.00	-0.10	-8.44	7.38
1	4	15.28	33.	33.	-0.98	-13.86	23.12	-0.12	-10.40	9.09
1	5	19.11	33.	33.	-1.11	-15.84	26.29	-0.14	-11.92	10.42
1	6	22.93	33.	33.	-1.20	-17.23	28.51	-0.15	-13.00	11.36
1	7	26.75	33.	33.	-1.24	-18.01	29.62	-0.16	-13.64	11.93
1	8	30.57	33.	33.	-1.25	-18.26	29.94	-0.16	-13.86	12.12
1	9	34.39	33.	33.	-1.24	-18.02	29.62	-0.16	-13.65	11.93
1	10	38.21	33.	33.	-1.20	-17.24	28.52	-0.15	-13.00	11.37
1	11	42.03	33.	33.	-1.11	-15.84	26.29	-0.14	-11.91	10.41
1	12	45.86	33.	33.	-0.98	-13.86	23.12	-0.12	-10.40	9.09
1	13	49.68	33.	33.	-0.81	-11.32	19.00	-0.10	-8.45	7.38
1	14	53.50	33.	33.	-0.70	-8.39	21.57	-0.08	-6.61	8.38
1	15	57.32	33.	33.	-0.38	-4.51	11.64	-0.04	-3.55	4.49
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.36	-3.38	10.06	-0.02	-2.42	2.91
1	2	7.64	33.	33.	-0.66	-6.29	18.64	-0.04	-4.52	5.44
1	3	11.46	33.	33.	-0.76	-8.62	16.42	-0.05	-5.75	4.80
1	4	15.28	33.	33.	-0.92	-10.55	19.94	-0.06	-7.09	5.91
1	5	19.11	33.	33.	-1.05	-12.05	22.64	-0.07	-8.12	6.77
1	6	22.93	33.	33.	-1.13	-13.10	24.54	-0.08	-8.86	7.38
1	7	26.75	33.	33.	-1.17	-13.67	25.44	-0.08	-9.30	7.75
1	8	30.57	33.	33.	-1.18	-13.85	25.69	-0.09	-9.45	7.87
1	9	34.39	33.	33.	-1.17	-13.67	25.44	-0.08	-9.30	7.75
1	10	38.21	33.	33.	-1.13	-13.10	24.54	-0.08	-8.86	7.39
1	11	42.03	33.	33.	-1.05	-12.04	22.64	-0.07	-8.12	6.77
1	12	45.86	33.	33.	-0.92	-10.55	19.94	-0.06	-7.09	5.91
1	13	49.68	33.	33.	-0.76	-8.63	16.42	-0.05	-5.76	4.80
1	14	53.50	33.	33.	-0.66	-6.30	18.64	-0.04	-4.52	5.45
1	15	57.32	33.	33.	-0.36	-3.39	10.07	-0.02	-2.43	2.92
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.30	-4.28	9.99	-0.04	-3.53	4.48
1	2	7.64	33.	33.	-0.56	-7.97	18.54	-0.08	-6.60	8.37
1	3	11.46	33.	33.	-0.65	-10.66	16.34	-0.10	-8.44	7.38
1	4	15.28	33.	33.	-0.79	-13.07	19.91	-0.12	-10.40	9.09
1	5	19.11	33.	33.	-0.89	-14.94	22.66	-0.14	-11.92	10.42
1	6	22.93	33.	33.	-0.96	-16.27	24.59	-0.15	-13.00	11.36
1	7	26.75	33.	33.	-1.00	-17.01	25.57	-0.16	-13.64	11.93
1	8	30.57	33.	33.	-1.01	-17.26	25.86	-0.16	-13.86	12.12
1	9	34.39	33.	33.	-1.00	-17.02	25.58	-0.16	-13.65	11.93
1	10	38.21	33.	33.	-0.96	-16.27	24.60	-0.15	-13.00	11.37
1	11	42.03	33.	33.	-0.89	-14.94	22.66	-0.14	-11.91	10.41
1	12	45.86	33.	33.	-0.79	-13.07	19.91	-0.12	-10.40	9.09
1	13	49.68	33.	33.	-0.65	-10.66	16.35	-0.10	-8.45	7.38
1	14	53.50	33.	33.	-0.56	-7.98	18.56	-0.08	-6.61	8.38
1	15	57.32	33.	33.	-0.30	-4.29	10.01	-0.04	-3.55	4.49
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.82	33.	33.	-0.28	-3.16	8.43	-0.02	-2.42	2.91
1	2	7.64	33.	33.	-0.52	-5.89	15.62	-0.04	-4.52	5.44
1	3	11.46	33.	33.	-0.60	-7.97	13.76	-0.05	-5.75	4.80
1	4	15.28	33.	33.	-0.73	-9.76	16.73	-0.06	-7.09	5.91
1	5	19.11	33.	33.	-0.82	-11.15	19.02	-0.07	-8.12	6.77
1	6	22.93	33.	33.	-0.89	-12.13	20.62	-0.08	-8.86	7.38
1	7	26.75	33.	33.	-0.92	-12.67	21.40	-0.08	-9.30	7.75
1	8	30.57	33.	33.	-0.93	-12.84	21.62	-0.09	-9.45	7.87
1	9	34.39	33.	33.	-0.92	-12.67	21.40	-0.08	-9.30	7.75
1	10	38.21	33.	33.	-0.89	-12.13	20.62	-0.08	-8.86	7.39
1	11	42.03	33.	33.	-0.82	-11.15	19.01	-0.07	-8.12	6.77
1	12	45.86	33.	33.	-0.73	-9.76	16.73	-0.06	-7.09	5.91
1	13	49.68	33.	33.	-0.60	-7.97	13.76	-0.05	-5.76	4.80
1	14	53.50	33.	33.	-0.52	-5.90	15.63	-0.04	-4.52	5.45
1	15	57.32	33.	33.	-0.28	-3.17	8.44	-0.02	-2.43	2.92
1	16	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.07	197.	-5.8	126.	-1.2	495.	-1.4	0.	0.0	-8.35
											1.36
	LB			5.8		2.9		10.1		0.0	18.76
											10.07
	RT			-5.3		-1.1		-1.6		0.0	-8.00
											1.59
	RB			3.7		1.8		6.5		0.0	11.95
											6.45
1	LT	50.07	197.	-5.3	126.	-1.1	495.	-1.6	0.	0.0	-8.01
											1.59
	LB			3.7		1.8		6.5		0.0	11.95
											6.45
	RT			-5.8		-1.2		-1.4		0.0	-8.36
											1.36
	RB			5.8		2.9		10.1		0.0	18.77
											10.07

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.07	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	39.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	10.00	1.0000
3	11.07	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 61.1

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1]) [3]/(12*[1])	<=>	LIMITATION 1/25 1/30	STATUS
1	11.07	6.75	33.09	[2]= 41.39	0.05641	>	0.04000	OK
				[3]= 33.09	0.04510	>	0.03300	OK
2	39.00	6.75	33.09	[2]= 41.39	0.05641	>	0.04000	OK

```
                        5969 Span 2-S20 Exterior Fatigue and Mid M.RES
                       [3]=   33.09     0.04510      >   0.03300    OK

  3  11.07  6.75   33.09 [2]=   41.39     0.05641      >   0.04000    OK

                       [3]=   33.09     0.04510      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
              [1]                        [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  1   3.82  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  2   7.64  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  3  11.46  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  4  15.28  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  5  19.11  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  6  22.93  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  7  26.75  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  8  30.57  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1  9  34.39  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 10  38.21  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 11  42.03  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 12  45.86  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 13  49.68  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 14  53.50  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 15  57.32  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0
  1 16  61.14  0  1  31.39 0.579  54.2  150.0    0.00 111.46                0


-------------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.39	0.579	54.21	168.97	2
1	1	3.82	0	1	31.39	0.579	54.21	168.97	2
1	2	7.64	0	1	31.39	0.579	54.21	168.97	2
1	3	11.46	0	1	31.39	0.579	66.55	168.97	2
1	4	15.28	0	1	31.39	0.579	66.55	168.97	2
1	5	19.11	0	1	31.39	0.579	66.55	168.97	2
1	6	22.93	0	1	31.39	0.579	66.55	168.97	2
1	7	26.75	0	1	31.39	0.579	66.55	168.97	2
1	8	30.57	0	1	31.39	0.579	66.55	168.97	2
1	9	34.39	0	1	31.39	0.579	66.55	168.97	2
1	10	38.21	0	1	31.39	0.579	66.55	168.97	2
1	11	42.03	0	1	31.39	0.579	66.55	168.97	2
1	12	45.86	0	1	31.39	0.579	66.55	168.97	2
1	13	49.68	0	1	31.39	0.579	66.55	168.97	2
1	14	53.50	0	1	31.39	0.579	54.21	168.97	2
1	15	57.32	0	1	31.39	0.579	54.21	168.97	2
1	16	61.14	0	1	31.39	0.579	54.21	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 1	3.82	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 2	7.64	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 3	11.46	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 4	15.28	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 5	19.11	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 6	22.93	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 7	26.75	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 8	30.57	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 9	34.39	6.76	12.	11.5	5.2	0.850	0.637	
		6.76	12.	11.5	5.2	0.850	0.637	1.00 0
1 10	38.21	6.76	12.	11.5	5.2	0.850	0.637	

		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 11	42.03	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 12	45.86	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 13	49.68	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 14	53.50	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 15	57.32	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0
1 16	61.14	6.76	12.	11.5	5.2	0.850	0.637		
		6.76	12.	11.5	5.2	0.850	0.637	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT FROM ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	18.8	0.000	11.395	7.247	3.100	1.000
			0.000	0.000	0.000	0.000	1.000
2	18.8	37.7	11.395	12.631	13.241	13.851	1.000
			0.000	0.000	0.000	0.000	1.000
3	37.7	56.5	4.110	12.631	9.414	6.196	1.000
			0.000	0.000	0.000	0.000	1.000
4	56.5	61.1	0.000	4.110	2.105	0.100	1.725
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0		0.00	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	1		3.82	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	2		7.64	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	3		11.46	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	4		15.28	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	5		19.11	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	6		22.93	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	7		26.75	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2

1	8	30.57	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	9	34.39	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	10	38.21	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	11	42.03	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	12	45.86	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	13	49.68	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	14	53.50	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	15	57.32	1	6.76	11.26	19.84	0	4.62	7.17	26.91	0	0
1	16	61.14	1	6.76	11.26	19.84	0	4.62	7.17	26.91	0	0

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.00	33.00	27.15	27.15
1	1		3.82	1	1.000	1.000	1.00	33.00	27.15	27.15
1	2		7.64	1	1.000	1.000	1.00	33.00	27.15	27.15
1	3		11.46	1	1.000	1.000	1.00	33.00	26.89	26.89
1	4		15.28	1	1.000	1.000	1.00	33.00	26.89	26.89
1	5		19.11	1	1.000	1.000	1.00	33.00	26.89	26.89
1	6		22.93	1	1.000	1.000	1.00	33.00	26.89	26.89
1	7		26.75	1	1.000	1.000	1.00	33.00	26.89	26.89
1	8		30.57	1	1.000	1.000	1.00	33.00	26.89	26.89
1	9		34.39	1	1.000	1.000	1.00	33.00	26.89	26.89
1	10		38.21	1	1.000	1.000	1.00	33.00	26.89	26.89
1	11		42.03	1	1.000	1.000	1.00	33.00	26.89	26.89
1	12		45.86	1	1.000	1.000	1.00	33.00	26.89	26.89
1	13		49.68	1	1.000	1.000	1.00	33.00	26.89	26.89
1	14		53.50	1	1.000	1.000	1.00	33.00	27.15	27.15
1	15		57.32	1	1.000	1.000	1.72	33.00	33.00	33.00
1	16		61.14	1	1.000	1.000	1.72	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	8.35	17.38
1	1		3.82	1	8.35	17.38
1	2		7.64	1	8.35	17.38
1	3		11.46	1	8.79	17.80
1	4		15.28	1	8.79	17.80
1	5		19.11	1	8.79	17.80
1	6		22.93	1	8.79	17.80
1	7		26.75	1	8.79	17.80
1	8		30.57	1	8.79	17.80
1	9		34.39	1	8.79	17.80
1	10		38.21	1	8.79	17.80
1	11		42.03	1	8.79	17.80
1	12		45.86	1	8.79	17.80
1	13		49.68	1	8.79	17.80
1	14		53.50	1	8.35	17.38
1	15		57.32	1	8.35	17.38
1	16		61.14	1	8.35	17.38

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0		0.00	1	0	2	0		2	0
1	1		3.82	1	0	2	0		2	0
1	2		7.64	1	0	2	0		2	0
1	3		11.46	1	0	2	0		2	0
1	4		15.28	1	0	2	0		2	0
1	5		19.11	1	0	2	0		2	0
1	6		22.93	1	0	2	0		2	0
1	7		26.75	1	0	2	0		2	0
1	8		30.57	1	0	2	0		2	0
1	9		34.39	1	0	2	0		2	0
1	10		38.21	1	0	2	0		2	0
1	11		42.03	1	0	2	0		2	0
1	12		45.86	1	0	2	0		2	0
1	13		49.68	1	0	2	0		2	0
1	14		53.50	1	0	2	0		2	0
1	15		57.32	1	0	0	0		0	0
1	16		61.14	1	0	0	0		0	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to

decide section category
[3] For composite sections in positive flexure, use
Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use
Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute
section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections
in positive flexure
Category No. is the maximum of [3] and [4] for
composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.1	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.82	0.0	19.8	3.5	33.0	3.5	33.0	3.5	27.1	0
				0.0	-	3.5	-	3.5	33.0	-	-	0
1	2		7.64	0.0	19.8	6.5	33.0	6.5	33.0	6.5	27.1	0
				0.0	-	6.5	-	6.5	33.0	-	-	0
1	3		11.46	0.0	19.8	8.1	33.0	8.1	33.0	8.1	26.9	0
				0.0	-	5.7	-	5.7	33.0	-	-	0
1	4		15.28	0.0	19.8	10.0	33.0	10.0	33.0	10.0	26.9	0
				0.0	-	7.0	-	7.0	33.0	-	-	0
1	5		19.11	0.0	19.8	11.5	33.0	11.5	33.0	11.5	26.9	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	6		22.93	0.0	19.8	12.5	33.0	12.5	33.0	12.5	26.9	0
				0.0	-	8.8	-	8.8	33.0	-	-	0
1	7		26.75	0.0	19.8	13.2	33.0	13.2	33.0	13.2	26.9	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	8		30.57	0.0	19.8	13.4	33.0	13.4	33.0	13.4	26.9	0
				0.0	-	9.3	-	9.3	33.0	-	-	0
1	9		34.39	0.0	19.8	13.2	33.0	13.2	33.0	13.2	26.9	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	10		38.21	0.0	19.8	12.5	33.0	12.5	33.0	12.5	26.9	0
				0.0	-	8.8	-	8.8	33.0	-	-	0
1	11		42.03	0.0	19.8	11.5	33.0	11.5	33.0	11.5	26.9	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	12		45.86	0.0	19.8	10.0	33.0	10.0	33.0	10.0	26.9	0
				0.0	-	7.0	-	7.0	33.0	-	-	0
1	13		49.68	0.0	19.8	8.1	33.0	8.1	33.0	8.1	26.9	0
				0.0	-	5.7	-	5.7	33.0	-	-	0
1	14		53.50	0.0	19.8	6.5	33.0	6.5	33.0	6.5	27.1	0
				0.0	-	6.5	-	6.5	33.0	-	-	0
1	15		57.32	0.0	19.8	3.5	33.0	3.5	33.0	3.5	33.0	0
				0.0	-	3.5	-	3.5	33.0	-	-	0
1	16		61.14	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.82	0.000	0.105	0.105	0.128	0.128	4
					0.105		0.105	3
1	2	7.64	0.000	0.196	0.196	0.238	0.238	4
					0.196		0.196	3
1	3	11.46	0.000	0.247	0.247	0.303	0.303	4
					0.172		0.172	3
1	4	15.28	0.000	0.304	0.304	0.373	0.373	4
					0.212		0.212	3
1	5	19.11	0.000	0.348	0.348	0.427	0.427	4
					0.243		0.243	3
1	6	22.93	0.000	0.380	0.380	0.466	0.466	4
					0.265		0.265	3
1	7	26.75	0.000	0.399	0.399	0.489	0.489	4
					0.278		0.278	3
1	8	30.57	0.000	0.405	0.405	0.497	0.497	4
					0.283		0.283	3
1	9	34.39	0.000	0.399	0.399	0.489	0.489	4
					0.279		0.279	3
1	10	38.21	0.000	0.380	0.380	0.466	0.466	4
					0.265		0.265	3
1	11	42.03	0.000	0.348	0.348	0.427	0.427	4
					0.243		0.243	3
1	12	45.86	0.000	0.304	0.304	0.373	0.373	4
					0.212		0.212	3
1	13	49.68	0.000	0.247	0.247	0.303	0.303	4
					0.172		0.172	3
1	14	53.50	0.000	0.197	0.197	0.239	0.239	4
					0.197		0.197	3
1	15	57.32	0.000	0.106	0.106	0.106	0.106	2
					0.106		0.106	3
1	16	61.14	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2056.9	-	-	0.0	19.8	-	-	0--0
1	1		3.82	0	519.2	2056.9	-	-	0.0	19.8	-	-	0--0
1	2		7.64	0	962.0	2056.9	-	-	0.0	19.8	-	-	0--0
1	3		11.46	0	1328.4	2899.5	-	-	0.0	19.8	-	-	0--0
1	4		15.28	0	1614.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	5		19.11	0	1834.5	2899.5	-	-	0.0	19.8	-	-	0--0
1	6		22.93	0	1988.5	2899.5	-	-	0.0	19.8	-	-	0--0
1	7		26.75	0	2063.3	2899.5	-	-	0.0	19.8	-	-	0--0
1	8		30.57	0	2084.4	2899.5	-	-	0.0	19.8	-	-	0--0
1	9		34.39	0	2063.5	2899.5	-	-	0.0	19.8	-	-	0--0
1	10		38.21	0	1988.9	2899.5	-	-	0.0	19.8	-	-	0--0
1	11		42.03	0	1834.3	2899.5	-	-	0.0	19.8	-	-	0--0
1	12		45.86	0	1614.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	13		49.68	0	1328.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	14		53.50	0	962.4	2056.9	-	-	0.0	19.8	-	-	0--0
1	15		57.32	0	519.7	2056.9	-	-	0.0	19.8	-	-	0--0
1	16		61.14	0	0.0	2056.9	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.

1	0	0.00	0	0.000	0.000	1
1	1	3.82	0	0.252	0.252	1
1	2	7.64	0	0.468	0.468	1
1	3	11.46	0	0.458	0.458	1
1	4	15.28	0	0.557	0.557	1
1	5	19.11	0	0.633	0.633	1
1	6	22.93	0	0.686	0.686	1
1	7	26.75	0	0.712	0.712	1
1	8	30.57	0	0.719	0.719	1
1	9	34.39	0	0.712	0.712	1
1	10	38.21	0	0.686	0.686	1
1	11	42.03	0	0.633	0.633	1
1	12	45.86	0	0.557	0.557	1
1	13	49.68	0	0.458	0.458	1
1	14	53.50	0	0.468	0.468	1
1	15	57.32	0	0.253	0.253	1
1	16	61.14	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.82	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	7.64	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	11.46	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	15.28	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	19.11	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	22.93	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	26.75	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	30.57	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	34.39	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	38.21	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	42.03	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	45.86	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	49.68	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	14	53.50	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	15	57.32	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	16	61.14	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

Please read NOTE on the following page

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```
NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
            = 5 for unstiffened beams and girders
            = 5 for stiffened girders when (d0/D) > 3, or
                                           (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	CATEGORY	OVERALL WEB
			S T R E N G T H C A T E G O R Y; SEE NOTE					
1	0	0.00	0	145.7	<	347.9	0	0
1	1	3.82	0	131.7	<	347.9	0	0
1	2	7.64	0	117.8	<	347.9	0	0
1	3	11.46	0	104.1	<	347.9	0	0
1	4	15.28	0	90.5	<	347.9	0	0
1	5	19.11	0	76.9	<	347.9	0	0
1	6	22.93	0	63.6	<	347.9	0	0
1	7	26.75	0	50.4	<	347.9	0	0
1	8	30.57	0	37.5	<	347.9	0	0
1	9	34.39	0	50.3	<	347.9	0	0
1	10	38.21	0	63.7	<	347.9	0	0
1	11	42.03	0	77.0	<	347.9	0	0
1	12	45.86	0	90.4	<	347.9	0	0
1	13	49.68	0	104.0	<	347.9	0	0
1	14	53.50	0	117.7	<	347.9	0	0
1	15	57.32	0	131.8	<	347.9	0	0
1	16	61.14	0	145.8	<	347.9	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	145.69	347.87	0	
1	1	3.82	33.	131.66	347.87	0	
1	2	7.64	33.	117.78	347.87	0	
1	3	11.46	33.	104.05	347.87	0	
1	4	15.28	33.	90.47	347.87	0	
1	5	19.11	33.	76.86	347.87	0	
1	6	22.93	33.	63.58	347.87	0	
1	7	26.75	33.	50.38	347.87	0	
1	8	30.57	33.	37.47	347.87	0	
1	9	34.39	33.	50.32	347.87	0	
1	10	38.21	33.	63.70	347.87	0	

1 11	42.03	33.	76.98	347.87	0
1 12	45.86	33.	90.42	347.87	0
1 13	49.68	33.	104.00	347.87	0
1 14	53.50	33.	117.73	347.87	0
1 15	57.32	33.	131.79	347.87	0
1 16	61.14	33.	145.82	347.87	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D L	FROM (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3]	TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-		00-
1	1	3.82	3.5	-	31.4	8.7	31.4	-	-		00-
1	2	7.64	6.5	-	31.4	16.2	31.4	-	-		00-
1	3	11.46	8.7	-	31.4	14.3	31.4	-	-		00-
1	4	15.28	10.7	-	31.4	17.4	31.4	-	-		00-
1	5	19.11	12.2	-	31.4	19.8	31.4	-	-		00-
1	6	22.93	13.3	-	31.4	21.5	31.4	-	-		00-
1	7	26.75	13.9	-	31.4	22.3	31.4	-	-		00-
1	8	30.57	14.1	-	31.4	22.5	31.4	-	-		00-
1	9	34.39	13.9	-	31.4	22.3	31.4	-	-		00-
1	10	38.21	13.3	-	31.4	21.5	31.4	-	-		00-
1	11	42.03	12.2	-	31.4	19.8	31.4	-	-		00-
1	12	45.86	10.7	-	31.4	17.4	31.4	-	-		00-
1	13	49.68	8.7	-	31.4	14.3	31.4	-	-		00-
1	14	53.50	6.5	-	31.4	16.2	31.4	-	-		00-
1	15	57.32	3.5	-	31.4	8.8	31.4	-	-		00-
1	16	61.14	0.0	-	31.4	0.0	31.4	-	-		00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.82	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	2	7.64	TR	-0.5	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1		11.07	SCG					
1	3	11.46	TR	-0.9	A B B^C^C D E E^	TR	3.5	A B B^C^C D E
1	4	15.28	TR	-1.0	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	5	19.11	TR	-1.2	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	6	22.93	TR	-1.2	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	7	26.75	TR	-1.2	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	8	30.57	TR	-1.1	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	9	34.39	TR	-1.2	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	10	38.21	TR	-1.2	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	11	42.03	TR	-1.2	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	12	45.86	TR	-1.0	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	13	49.68	TR	-0.9	A B B^C^C D E E^	TR	3.5	A B B^C^C D E
1		50.07	SCG					
1	14	53.50	TR	-0.5	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	15	57.32	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	16	61.14	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.82	TR	-0.2	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2	7.64	TR	-0.3	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^

```
 1     11.07  SCG
 1  3  11.46    TR     -0.4 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1  4  15.28    TR     -0.5 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1  5  19.11    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1  6  22.93    TR     -0.6 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1  7  26.75    TR     -0.6 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1  8  30.57    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1  9  34.39    TR     -0.6 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 10  38.21    TR     -0.6 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1 11  42.03    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1 12  45.86    TR     -0.5 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1 13  49.68    TR     -0.4 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1     50.07  SCG
 1 14  53.50    TR     -0.3 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 15  57.32    TR     -0.2 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
 1 16  61.14    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   *******************************************

                 AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-1     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
       (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

 1  0   0.00  34.8 .28305E-01    1.0      TR    3.09        3           9.43
 1  1   3.82  33.1 .28305E-01    0.9      TR    3.09        3           9.92
 1  2   7.64  31.3 .28305E-01    0.9      TR    3.09        3          10.48
 1  3  11.46  29.6 .25828E-01    0.8      TR    3.09        3          12.15
 1  4  15.28  28.1 .25828E-01    0.7      TR    3.09        3          12.80
 1  5  19.11  26.9 .25828E-01    0.7      TR    3.09        3          13.37
 1  6  22.93  25.9 .25828E-01    0.7      TR    3.09        3          13.88
 1  7  26.75  23.6 .25828E-01    0.6      TR    3.09        3          15.25
 1  8  30.57  22.4 .25828E-01    0.6      TR    3.09        3          16.03
 1  9  34.39  23.6 .25828E-01    0.6      TR    3.09        3          15.25
 1 10  38.21  25.9 .25828E-01    0.7      TR    3.09        3          13.88
 1 11  42.03  26.9 .25828E-01    0.7      TR    3.09        3          13.37
 1 12  45.86  28.1 .25828E-01    0.7      TR    3.09        3          12.80
 1 13  49.68  29.6 .25828E-01    0.8      TR    3.09        3          12.15
 1 14  53.50  31.3 .28305E-01    0.9      TR    3.09        3          10.48
 1 15  57.32  33.1 .28305E-01    0.9      TR    3.09        3           9.92
 1 16  61.14  34.8 .28305E-01    1.0      TR    3.09        3           9.43


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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	AASHTO E6.10.10.1.2 Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	SHEAR CONN. PER TRANS. SECTION <--Input [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	17.4	.28305E-01	0.5	TR	1.36	3	8.31
1	1	3.82	16.5	.28305E-01	0.5	TR	1.36	3	8.75
1	2	7.64	15.6	.28305E-01	0.4	TR	1.36	3	9.24
1	3	11.46	14.8	.25828E-01	0.4	TR	1.36	3	10.71
1	4	15.28	14.0	.25828E-01	0.4	TR	1.36	3	11.29
1	5	19.11	13.4	.25828E-01	0.3	TR	1.36	3	11.79
1	6	22.93	12.9	.25828E-01	0.3	TR	1.36	3	12.24
1	7	26.75	11.8	.25828E-01	0.3	TR	1.36	3	13.44
1	8	30.57	11.2	.25828E-01	0.3	TR	1.36	3	14.13
1	9	34.39	11.8	.25828E-01	0.3	TR	1.36	3	13.44
1	10	38.21	12.9	.25828E-01	0.3	TR	1.36	3	12.24
1	11	42.03	13.4	.25828E-01	0.3	TR	1.36	3	11.79
1	12	45.86	14.0	.25828E-01	0.4	TR	1.36	3	11.29
1	13	49.68	14.8	.25828E-01	0.4	TR	1.36	3	10.71
1	14	53.50	15.6	.28305E-01	0.4	TR	1.36	3	9.24
1	15	57.32	16.5	.28305E-01	0.5	TR	1.36	3	8.75
1	16	61.14	17.4	.28305E-01	0.5	TR	1.36	3	8.31

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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

            * Default ALPHA value based on 7/8" diameter and input road type


        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
               where ADTT Single Lane= 1140 & No. of Cycles=   31207500
           *** If ADTT Single Lane greater than or equal to 960, refer
               to Fatigue I Table.
```

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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  30.57   1 1574.9  1204.9          123         >  63
      1 16  61.14   1 1574.9  1204.9          123         >  63


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page
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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)

```
                    5969 Span 2-S20 Exterior Fatigue and Mid M.RES
              ----------------------------------

                  1     0.000    30.570     9.000

                  1    30.570    61.140     9.000

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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                 **********************************

               SPAN     CURRENT SPAN    T
                NO.     FROM      TO    C
                        (ft)     (ft)   R
              ---------------------------

                  1       0.0      61.1 C

              ---------------------------

              NOTE: T: TENSION AREA, C: COMPRESSION AREA,
              R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
              STRESSES
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     TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
                   *************************

 STATUS                TABLE NO.
-------------------------------------------------------------------------------

  OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

  OK     1.2.22.4=DEPTH RATIOS

  OK     1.2.22.10=CONSTRUCBILITY CHECK

  OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

  OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

  OK     1.2.22.21=SERVICE LIMIT STATE CHECK


         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
                ***********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT       MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
                                              Page 43
```

5969 Span 2-S20 Exterior Fatigue and Mid M.RES

NO	NO	L (ft)	SUPT CAP. (k-ft) or STRESS	D1; k-ft ksi	D2	POS. MAX. STRESS	NEG.	FACTOR [3]	LOAD TYPE	FACTOR	T
1	0	0.00	2056.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.82	2056.9	78.1	49.8	200.8	0.0	6.97	HL-93	5.38	0
1	2	7.64	2056.9	145.8	93.0	370.6	0.0	3.48	HL-93	2.69	0
1	3	11.46	2899.5	203.2	129.6	509.5	0.0	3.58	HL-93	2.76	0
1	4	15.28	2899.5	250.3	159.5	615.3	0.0	2.84	HL-93	2.19	0
1	5	19.11	2899.5	287.0	182.7	696.0	0.0	2.43	HL-93	1.87	0
1	6	22.93	2899.5	312.9	199.3	752.2	0.0	2.19	HL-93	1.69	0
1	7	26.75	2899.5	328.5	209.3	775.7	0.0	2.09	HL-93	1.62	0
1	8	30.57	2899.5	333.7	212.6	781.4	0.0	2.07	HL-93	1.60	0
1	9	34.39	2899.5	328.7	209.3	775.7	0.0	2.09	HL-93	1.62	0
1	10	38.21	2899.5	313.2	199.3	752.2	0.0	2.19	HL-93	1.69	0
1	11	42.03	2899.5	286.9	182.7	696.0	0.0	2.43	HL-93	1.87	0
1	12	45.86	2899.5	250.3	159.5	615.3	0.0	2.84	HL-93	2.19	0
1	13	49.68	2899.5	203.4	129.6	509.5	0.0	3.58	HL-93	2.76	0
1	14	53.50	2056.9	146.1	93.0	370.6	0.0	3.48	HL-93	2.69	0
1	15	57.32	2056.9	78.4	49.8	200.8	0.0	6.97	HL-93	5.37	0
1	16	61.14	2056.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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MERLIN V 10.4
COMPOSITE
LRF -- 2010
RATING

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	SUPT 0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. STEEL--- TOP;ksi;BOT	MAX. NEG. STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. RATING LIVE LOAD FACTOR TYPE
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68
1	1	3.82	31.4	-2.77 3.44	-0.55 4.08	0.00 0.00	6.83	HL-93 5.26
1	2	7.64	31.4	-5.17 6.42	-1.02 7.54	0.00 0.00	3.31	HL-93 2.54
1	3	11.46	31.4	-6.60 5.66	-1.64 6.64	0.00 0.00	3.87	HL-93 2.98

1	4	15.28	31.4	-8.13	6.97	-1.98	8.02	0.00	0.00	3.04	HL-93	2.34
1	5	19.11	31.4	-9.32	7.99	-2.24	9.07	0.00	0.00	2.57	HL-93	1.98
1	6	22.93	31.4	-10.16	8.72	-2.42	9.80	0.00	0.00	2.31	HL-93	1.78
1	7	26.75	31.4	-10.67	9.15	-2.50	10.11	0.00	0.00	2.20	HL-93	1.69
1	8	30.57	31.4	-10.84	9.30	-2.52	10.18	0.00	0.00	2.17	HL-93	1.67
1	9	34.39	31.4	-10.68	9.15	-2.50	10.11	0.00	0.00	2.20	HL-93	1.69
1	10	38.21	31.4	-10.17	8.72	-2.42	9.80	0.00	0.00	2.31	HL-93	1.78
1	11	42.03	31.4	-9.32	7.99	-2.24	9.07	0.00	0.00	2.58	HL-93	1.98
1	12	45.86	31.4	-8.13	6.97	-1.98	8.02	0.00	0.00	3.04	HL-93	2.34
1	13	49.68	31.4	-6.61	5.67	-1.64	6.64	0.00	0.00	3.87	HL-93	2.98
1	14	53.50	31.4	-5.18	6.43	-1.02	7.54	0.00	0.00	3.31	HL-93	2.54
1	15	57.32	31.4	-2.78	3.45	-0.55	4.08	0.00	0.00	6.83	HL-93	5.25
1	16	61.14	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.14	30.57	STRENGTH L.S.	2.07	1.60	HL-93
1	61.14	30.57	SERVICEBILITY	2.17	1.67	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.069
Inventory= 1.596

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)

1	61.14	38.21	STRENGTH L.S.	2.97	749.66
1	61.14	38.21	SERVICEBILITY	2.97	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.970

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	347.9	21.8	13.9	56.5	0.0	3.95	HL-93	3.05
1	1	3.82	347.9	19.1	12.2	51.8	-1.5	4.39	HL-93	3.39
1	2	7.64	347.9	16.4	10.4	47.2	-3.6	4.91	HL-93	3.79
1	3	11.46	347.9	13.7	8.7	42.7	-6.2	5.53	HL-93	4.26
1	4	15.28	347.9	11.0	7.0	38.3	-8.8	6.28	HL-93	4.84
1	5	19.11	347.9	8.1	5.2	33.9	-11.5	7.22	HL-93	5.57
1	6	22.93	347.9	5.4	3.5	29.7	-14.3	8.40	HL-93	6.48
1	7	26.75	347.9	2.7	1.7	25.4	-17.6	9.96	HL-93	7.68
1	8	30.57	347.9	0.0	0.0	21.4	-21.4	9.99	HL-93	9.29
1	9	34.39	347.9	-2.7	-1.7	17.6	-25.4	9.96	HL-93	7.68
1	10	38.21	347.9	-5.5	-3.5	14.3	-29.7	8.39	HL-93	6.48
1	11	42.03	347.9	-8.2	-5.2	11.5	-33.9	7.21	HL-93	5.56
1	12	45.86	347.9	-10.9	-7.0	8.8	-38.3	6.28	HL-93	4.85
1	13	49.68	347.9	-13.6	-8.7	6.2	-42.7	5.53	HL-93	4.26
1	14	53.50	347.9	-16.3	-10.4	3.6	-47.2	4.91	HL-93	3.79
1	15	57.32	347.9	-19.2	-12.2	1.5	-51.8	4.39	HL-93	3.38
1	16	61.14	347.9	-21.9	-13.9	0.0	-56.5	3.95	HL-93	3.04

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.14	61.14	STRENGTH L.S.	3.95	3.04	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.946
Inventory= 3.044

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN	CRITICAL LOCATION	GOVERNING RATING	LEGAL LOAD	MAXIMUM

5969 Span 2-S20 Exterior Fatigue and Mid M.RES

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	LL SHEAR (kips)
1	61.14	61.14	STRENGTH L.S.	4.79	62.84

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 4.788

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	30.57	3 0.425	-0.41	HL-93	2.22
		4 0.433	-0.42		2.18

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95NB Span 2-S20 Ext. Strength CP 8/19/13


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       17           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                     1 = detailed output

 * span interval   : maximum = 20
                     default = 10

 * structural type : 1 = composite (default)
                     2 = noncomposite
                     3 = reinforced concrete
                     4 = prestressed concrete

 * type of unit    : 0 = English (default)
                     1 = Metric
                     2 = Metric input English output
                     3 = English input Metric output

 * design option   : 0 = WSD (default)
                     1 = LFD
                     2 = LRFD


 * program flow    : 0 = DL analysis only
                     1 = DL + LL analysis
                     2 = code check
                     3 = rating
                     4 = design
                     5 = design + code check
                     6 = design + recycle + code check
                     7 = DL stage only
                     8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              ******************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.40	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
61.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL DEPTH ID. (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	130		0.00	0.0000	0.00	0.0000		
2	W	33	130		0.00	0.0000	10.00	1.0000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB TYPE<-- 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.07				33.	33.	33.
2	2	2	39.00				33.	33.	33.
3	1	1	11.07				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.56	2 2 2 2 2
1	0.00	0	0.00	0.40	0.00	0.00	0 0 0 0 0	0.56	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	POUR NO	IDENTIFICATION DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	MODULAR RATIO N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.56	0.00	61.14

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	18.84
2	2	Diaphragm	0.000	0.00	0.00	0.14	37.68
3	2	Diaphragm	0.000	0.00	0.00	0.14	56.52
4	1	Railing	0.087	0.00	61.14	0.00	0.00
5	1	Curb	0.076	0.00	61.14	0.00	0.00
6	0	Pavement	0.292	0.00	61.14	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT			CONCRETE	
NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	18.84	33.0	33.0	18.84
18.84	37.68	33.0	33.0	18.84
37.68	56.52	33.0	33.0	18.84
56.52	61.14	33.0	33.0	4.62

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

13	GAMMA for Load DC maximum	= 1.25
14	GAMMA for Load DC minimum	= 0.90
15	GAMMA for Load DW minimum	= 1.50
16	GAMMA for Load DW minimum	= 0.65
17	GAMMA for LL Load Strength I	= 1.75
18	GAMMA for LL Load Strength II	= 1.35
19	GAMMA for LL Load Service I	= 1.00
20	GAMMA for LL Load Service II	= 1.30
21	GAMMA for LL Load Fatigue	= 0.75
22	ETA for Service Limit State	= 1.00
23	ETA for Strength Limit State	= 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.555	0.1520	0.7070

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.29	0.00	61.14
1	DC2	0.087	0.000	61.140
2	DC2	0.076	0.000	61.140

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1484.6	55.0	222.7	55.8	278.44

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.562	0.562	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.561				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.399				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	6710.0	33.1	16.55	405.6	405.6
1	1	3.60	6710.0	33.1	16.55	405.6	405.6
1	2	7.19	6710.0	33.1	16.55	405.6	405.6
1	3	10.79	6710.0	33.1	16.55	405.6	405.6
1	4	14.39	9020.9	33.1	14.02	643.6	449.4
1	5	17.98	9020.9	33.1	14.02	643.6	449.4
1	6	21.58	9020.9	33.1	14.02	643.6	449.4
1	7	25.18	9020.9	33.1	14.02	643.6	449.4
1	8	28.77	9020.9	33.1	14.02	643.6	449.4

1	9	32.37	9020.9	33.1	14.02	643.6	449.4
1	10	35.96	9020.9	33.1	14.02	643.6	449.4
1	11	39.56	9020.9	33.1	14.02	643.6	449.4
1	12	43.16	9020.9	33.1	14.02	643.6	449.4
1	13	46.75	9020.9	33.1	14.02	643.6	449.4
1	14	50.35	6710.0	33.1	16.55	405.6	405.6
1	15	53.95	6710.0	33.1	16.55	405.6	405.6
1	16	57.54	6710.0	33.1	16.55	405.6	405.6
1	17	61.14	6710.0	33.1	16.55	405.6	405.6

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12457.3	0.021730	529.5	1302.9	697.4
1	1	3.60	12457.3	0.021730	529.5	1302.9	697.4
1	2	7.19	12457.3	0.021730	529.5	1302.9	697.4
1	3	10.79	12457.3	0.021730	529.5	1302.9	697.4
1	4	14.39	17378.6	0.019536	829.5	1322.6	810.6
1	5	17.98	17378.6	0.019536	829.5	1322.6	810.6
1	6	21.58	17378.6	0.019536	829.5	1322.6	810.6
1	7	25.18	17378.6	0.019536	829.5	1322.6	810.6
1	8	28.77	17378.6	0.019536	829.5	1322.6	810.6
1	9	32.37	17378.6	0.019536	829.5	1322.6	810.6
1	10	35.96	17378.6	0.019536	829.5	1322.6	810.6
1	11	39.56	17378.6	0.019536	829.5	1322.6	810.6
1	12	43.16	17378.6	0.019536	829.5	1322.6	810.6
1	13	46.75	17378.6	0.019536	829.5	1322.6	810.6
1	14	50.35	12457.3	0.021730	529.5	1302.9	697.4
1	15	53.95	12457.3	0.021730	529.5	1302.9	697.4
1	16	57.54	12457.3	0.021730	529.5	1302.9	697.4
1	17	61.14	12457.3	0.021730	529.5	1302.9	697.4

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	17203.8	0.028305	589.9	4381.3	1407.1
1	1	3.60	17203.8	0.028305	589.9	4381.3	1407.1
1	2	7.19	17203.8	0.028305	589.9	4381.3	1407.1
1	3	10.79	17203.8	0.028305	589.9	4381.3	1407.1
1	4	14.39	25171.5	0.025828	920.8	3726.8	1672.1
1	5	17.98	25171.5	0.025828	920.8	3726.8	1672.1

1	6	21.58	25171.5	0.025828	920.8	3726.8	1672.1
1	7	25.18	25171.5	0.025828	920.8	3726.8	1672.1
1	8	28.77	25171.5	0.025828	920.8	3726.8	1672.1
1	9	32.37	25171.5	0.025828	920.8	3726.8	1672.1
1	10	35.96	25171.5	0.025828	920.8	3726.8	1672.1
1	11	39.56	25171.5	0.025828	920.8	3726.8	1672.1
1	12	43.16	25171.5	0.025828	920.8	3726.8	1672.1
1	13	46.75	25171.5	0.025828	920.8	3726.8	1672.1
1	14	50.35	17203.8	0.028305	589.9	4381.3	1407.1
1	15	53.95	17203.8	0.028305	589.9	4381.3	1407.1
1	16	57.54	17203.8	0.028305	589.9	4381.3	1407.1
1	17	61.14	17203.8	0.028305	589.9	4381.3	1407.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	3.60	15.7	57.4	0.6	0.0	73.8
1	2	7.19	29.5	107.7	1.2	0.0	138.4
1	3	10.79	41.3	150.8	1.8	0.0	193.8
1	4	14.39	51.1	186.6	2.4	0.0	240.1
1	5	17.98	59.0	215.4	3.0	0.0	277.3
1	6	21.58	64.9	236.9	3.2	0.0	305.0
1	7	25.18	68.8	251.3	3.3	0.0	323.3
1	8	28.77	70.8	258.4	3.3	0.0	332.5
1	9	32.37	70.8	258.4	3.4	0.0	332.6
1	10	35.96	68.8	251.3	3.5	0.0	323.6
1	11	39.56	64.9	236.9	3.3	0.0	305.1

1 12	43.16	59.0	215.4	2.9	0.0	277.2
1 13	46.75	51.1	186.6	2.4	0.0	240.2
1 14	50.35	41.3	150.8	2.0	0.0	194.0
1 15	53.95	29.5	107.7	1.5	0.0	138.7
1 16	57.54	15.7	57.4	1.0	0.0	74.1
1 17	61.14	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.60	30.2	16.9	0.0	0.0	0.0	47.1
1	2	7.19	56.7	31.6	0.0	0.0	0.0	88.3
1	3	10.79	79.3	44.3	0.0	0.0	0.0	123.6
1	4	14.39	98.2	54.8	0.0	0.0	0.0	153.0
1	5	17.98	113.3	63.3	0.0	0.0	0.0	176.6
1	6	21.58	124.6	69.6	0.0	0.0	0.0	194.2
1	7	25.18	132.2	73.8	0.0	0.0	0.0	206.0
1	8	28.77	136.0	75.9	0.0	0.0	0.0	211.9
1	9	32.37	136.0	75.9	0.0	0.0	0.0	211.9
1	10	35.96	132.2	73.8	0.0	0.0	0.0	206.0
1	11	39.56	124.6	69.6	0.0	0.0	0.0	194.2
1	12	43.16	113.3	63.3	0.0	0.0	0.0	176.6
1	13	46.75	98.2	54.8	0.0	0.0	0.0	153.0
1	14	50.35	79.3	44.3	0.0	0.0	0.0	123.6
1	15	53.95	56.7	31.6	0.0	0.0	0.0	88.3
1	16	57.54	30.2	16.9	0.0	0.0	0.0	47.1
1	17	61.14	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.60	0.0	0.0	189.8	HL-93	0.0	HL-93
1	2	7.19	0.0	0.0	352.3	HL-93	0.0	HL-93
1	3	10.79	0.0	0.0	487.2	HL-93	0.0	HL-93
1	4	14.39	0.0	0.0	592.9	HL-93	0.0	HL-93
1	5	17.98	0.0	0.0	674.5	HL-93	0.0	HL-93
1	6	21.58	0.0	0.0	735.5	HL-93	0.0	HL-93
1	7	25.18	0.0	0.0	770.1	HL-93	0.0	HL-93
1	8	28.77	0.0	0.0	780.1	HL-93	0.0	HL-93
1	9	32.37	0.0	0.0	780.1	HL-93	0.0	HL-93
1	10	35.96	0.0	0.0	770.1	HL-93	0.0	HL-93
1	11	39.56	0.0	0.0	735.5	HL-93	0.0	HL-93
1	12	43.16	0.0	0.0	674.5	HL-93	0.0	HL-93
1	13	46.75	0.0	0.0	592.9	HL-93	0.0	HL-93

1	14	50.35	0.0	0.0	487.2	HL-93	0.0	HL-93
1	15	53.95	0.0	0.0	352.3	HL-93	0.0	HL-93
1	16	57.54	0.0	0.0	189.8	HL-93	0.0	HL-93
1	17	61.14	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.60	70.	0.	70.
1	2	7.19	127.	0.	127.
1	3	10.79	171.	0.	171.
1	4	14.39	207.	0.	207.
1	5	17.98	233.	0.	233.
1	6	21.58	250.	0.	250.
1	7	25.18	251.	0.	251.
1	8	28.77	239.	0.	239.
1	9	32.37	239.	0.	239.
1	10	35.96	251.	0.	251.
1	11	39.56	250.	0.	250.
1	12	43.16	233.	0.	233.
1	13	46.75	207.	0.	207.
1	14	50.35	171.	0.	171.
1	15	53.95	127.	0.	127.
1	16	57.54	70.	0.	70.
1	17	61.14	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.60	73.8	47.1	189.8	HL-93	0.0	HL-93	310.7	120.8
1	2	7.19	138.4	88.3	352.3	HL-93	0.0	HL-93	578.9	226.6
1	3	10.79	193.8	123.6	487.2	HL-93	0.0	HL-93	804.7	317.4
1	4	14.39	240.1	153.0	592.9	HL-93	0.0	HL-93	986.1	393.2
1	5	17.98	277.3	176.6	674.5	HL-93	0.0	HL-93	1128.4	453.9
1	6	21.58	305.0	194.2	735.5	HL-93	0.0	HL-93	1234.7	499.2
1	7	25.18	323.3	206.0	770.1	HL-93	0.0	HL-93	1299.4	529.3
1	8	28.77	332.5	211.9	780.1	HL-93	0.0	HL-93	1324.5	544.4
1	9	32.37	332.6	211.9	780.1	HL-93	0.0	HL-93	1324.6	544.5
1	10	35.96	323.6	206.0	770.1	HL-93	0.0	HL-93	1299.6	529.5
1	11	39.56	305.1	194.2	735.5	HL-93	0.0	HL-93	1234.8	499.3
1	12	43.16	277.2	176.6	674.5	HL-93	0.0	HL-93	1128.3	453.8
1	13	46.75	240.2	153.0	592.9	HL-93	0.0	HL-93	986.1	393.2
1	14	50.35	194.0	123.6	487.2	HL-93	0.0	HL-93	804.9	317.6
1	15	53.95	138.7	88.3	352.3	HL-93	0.0	HL-93	579.2	227.0
1	16	57.54	74.1	47.1	189.8	HL-93	0.0	HL-93	311.0	121.2
1	17	61.14	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN D NO	FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.60	310.7	367.6	490.8	414.9	181.3	52.6
1	2	7.19	578.9	684.6	913.9	773.0	340.0	94.9
1	3	10.79	804.7	950.8	1269.3	1074.4	476.1	128.1
1	4	14.39	986.1	1163.9	1553.6	1316.4	589.7	154.9
1	5	17.98	1128.4	1330.7	1776.0	1506.2	680.8	174.6
1	6	21.58	1234.7	1455.3	1942.2	1648.0	748.8	187.5
1	7	25.18	1299.4	1530.4	2042.3	1734.3	794.0	188.2
1	8	28.77	1324.5	1558.6	2079.7	1767.7	816.6	179.3
1	9	32.37	1324.6	1558.6	2079.8	1767.8	816.7	179.3
1	10	35.96	1299.6	1530.6	2042.6	1734.6	794.3	188.2
1	11	39.56	1234.8	1455.4	1942.4	1648.2	748.9	187.5
1	12	43.16	1128.3	1330.6	1775.9	1506.1	680.6	174.6
1	13	46.75	986.1	1164.0	1553.6	1316.5	589.8	154.9
1	14	50.35	804.9	951.0	1269.5	1074.6	476.4	128.1
1	15	53.95	579.2	684.9	914.3	773.4	340.5	94.9
1	16	57.54	311.0	368.0	491.3	415.3	181.8	52.6
1	17	61.14	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	4.6	17.0	0.2	0.0	21.8
1	1	3.60	4.1	15.0	0.2	0.0	19.2
1	2	7.19	3.6	13.0	0.2	0.0	16.7
1	3	10.79	3.0	11.0	0.2	0.0	14.2
1	4	14.39	2.5	9.0	0.2	0.0	11.6
1	5	17.98	1.9	7.0	0.2	0.0	9.1
1	6	21.58	1.4	5.0	0.0	0.0	6.4
1	7	25.18	0.8	3.0	0.0	0.0	3.8
1	8	28.77	0.3	1.0	0.0	0.0	1.3
1	9	32.37	-0.3	-1.0	0.0	0.0	-1.2
1	10	35.96	-0.8	-3.0	0.0	0.0	-3.8
1	11	39.56	-1.4	-5.0	-0.1	0.0	-6.5
1	12	43.16	-1.9	-7.0	-0.1	0.0	-9.0
1	13	46.75	-2.5	-9.0	-0.1	0.0	-11.6
1	14	50.35	-3.0	-11.0	-0.1	0.0	-14.1
1	15	53.95	-3.6	-13.0	-0.1	0.0	-16.6
1	16	57.54	-4.1	-15.0	-0.3	0.0	-19.3
1	17	61.14	-4.6	-17.0	-0.3	0.0	-21.9

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	DC2	CONCENTRATED LOADS DW	DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	8.9	5.0	0.0	0.0	13.9	21.8	35.7
1	1	3.60	7.9	4.4	0.0	0.0	12.3	19.2	31.5
1	2	7.19	6.8	3.8	0.0	0.0	10.6	16.7	27.3
1	3	10.79	5.8	3.2	0.0	0.0	9.0	14.2	23.2
1	4	14.39	4.7	2.6	0.0	0.0	7.4	11.6	19.0
1	5	17.98	3.7	2.1	0.0	0.0	5.7	9.1	14.8
1	6	21.58	2.6	1.5	0.0	0.0	4.1	6.4	10.5
1	7	25.18	1.6	0.9	0.0	0.0	2.5	3.8	6.3
1	8	28.77	0.5	0.3	0.0	0.0	0.8	1.3	2.1
1	9	32.37	-0.5	-0.3	0.0	0.0	-0.8	-1.2	-2.1
1	10	35.96	-1.6	-0.9	0.0	0.0	-2.5	-3.8	-6.2
1	11	39.56	-2.6	-1.5	0.0	0.0	-4.1	-6.5	-10.6
1	12	43.16	-3.7	-2.1	0.0	0.0	-5.7	-9.0	-14.7
1	13	46.75	-4.7	-2.6	0.0	0.0	-7.4	-11.6	-18.9
1	14	50.35	-5.8	-3.2	0.0	0.0	-9.0	-14.1	-23.1
1	15	53.95	-6.8	-3.8	0.0	0.0	-10.6	-16.6	-27.3
1	16	57.54	-7.9	-4.4	0.0	0.0	-12.3	-19.3	-31.6
1	17	61.14	-8.9	-5.0	0.0	0.0	-13.9	-21.9	-35.8

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	56.5	HL-93	0.0	HL-93
1	1	3.60	0.0	0.0	52.1	HL-93	-1.4	HL-93
1	2	7.19	0.0	0.0	47.7	HL-93	-3.3	HL-93
1	3	10.79	0.0	0.0	43.5	HL-93	-5.7	HL-93
1	4	14.39	0.0	0.0	39.3	HL-93	-8.2	HL-93
1	5	17.98	0.0	0.0	35.2	HL-93	-10.7	HL-93
1	6	21.58	0.0	0.0	31.1	HL-93	-13.3	HL-93
1	7	25.18	0.0	0.0	27.2	HL-93	-16.1	HL-93
1	8	28.77	0.0	0.0	23.3	HL-93	-19.6	HL-93
1	9	32.37	0.0	0.0	19.6	HL-93	-23.3	HL-93
1	10	35.96	0.0	0.0	16.1	HL-93	-27.2	HL-93
1	11	39.56	0.0	0.0	13.3	HL-93	-31.1	HL-93
1	12	43.16	0.0	0.0	10.7	HL-93	-35.2	HL-93
1	13	46.75	0.0	0.0	8.2	HL-93	-39.3	HL-93
1	14	50.35	0.0	0.0	5.7	HL-93	-43.5	HL-93
1	15	53.95	0.0	0.0	3.3	HL-93	-47.7	HL-93
1	16	57.54	0.0	0.0	1.4	HL-93	-52.1	HL-93
1	17	61.14	0.0	0.0	0.0	HL-93	-56.5	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	23.	0.	23.
1	1	3.60	21.	-1.	22.
1	2	7.19	19.	-2.	21.
1	3	10.79	17.	-3.	20.
1	4	14.39	15.	-4.	19.
1	5	17.98	14.	-5.	18.
1	6	21.58	12.	-6.	18.
1	7	25.18	10.	-7.	16.
1	8	28.77	8.	-7.	15.
1	9	32.37	7.	-8.	15.
1	10	35.96	7.	-10.	16.
1	11	39.56	6.	-12.	18.
1	12	43.16	5.	-14.	18.
1	13	46.75	4.	-15.	19.
1	14	50.35	3.	-17.	20.
1	15	53.95	2.	-19.	21.
1	16	57.54	1.	-21.	22.
1	17	61.14	0.	-23.	23.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN D NO L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	23.2	.30607E-01	0.35	0.3125*	.15051E-01	0.17	0.3125*
1	1	3.60	22.1	.30607E-01	0.34	0.3125*	.15051E-01	0.17	0.3125*
1	2	7.19	21.0	.30607E-01	0.32	0.3125*	.15051E-01	0.16	0.3125*
1	3	10.79	19.9	.30607E-01	0.30	0.3125*	.15051E-01	0.15	0.3125*
1	4	14.39	18.9	.28474E-01	0.27	0.3125*	.92139E-02	0.09	0.3125*
1	5	17.98	18.1	.28474E-01	0.26	0.3125*	.92139E-02	0.08	0.3125*
1	6	21.58	17.5	.28474E-01	0.25	0.3125*	.92139E-02	0.08	0.3125*
1	7	25.18	16.5	.28474E-01	0.23	0.3125*	.92139E-02	0.08	0.3125*
1	8	28.77	15.1	.28474E-01	0.22	0.3125*	.92139E-02	0.07	0.3125*
1	9	32.37	15.1	.28474E-01	0.22	0.3125*	.92139E-02	0.07	0.3125*
1	10	35.96	16.5	.28474E-01	0.23	0.3125*	.92139E-02	0.08	0.3125*
1	11	39.56	17.5	.28474E-01	0.25	0.3125*	.92139E-02	0.08	0.3125*
1	12	43.16	18.1	.28474E-01	0.26	0.3125*	.92139E-02	0.08	0.3125*
1	13	46.75	18.9	.28474E-01	0.27	0.3125*	.92139E-02	0.09	0.3125*
1	14	50.35	19.9	.30607E-01	0.30	0.3125*	.15051E-01	0.15	0.3125*
1	15	53.95	21.0	.30607E-01	0.32	0.3125*	.15051E-01	0.16	0.3125*
1	16	57.54	22.1	.30607E-01	0.34	0.3125*	.15051E-01	0.17	0.3125*
1	17	61.14	23.2	.30607E-01	0.35	0.3125*	.15051E-01	0.17	0.3125*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange

Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	35.7	56.5	HL-93	0.0	HL-93	92.2	35.7
1	1	3.60	31.5	52.1	HL-93	-1.4	HL-93	83.6	30.1
1	2	7.19	27.3	47.7	HL-93	-3.3	HL-93	75.1	24.0
1	3	10.79	23.2	43.5	HL-93	-5.7	HL-93	66.6	17.5
1	4	14.39	19.0	39.3	HL-93	-8.2	HL-93	58.3	10.8
1	5	17.98	14.8	35.2	HL-93	-10.7	HL-93	50.0	4.1
1	6	21.58	10.5	31.1	HL-93	-13.3	HL-93	41.6	-2.8
1	7	25.18	6.3	27.2	HL-93	-16.1	HL-93	33.5	-9.8
1	8	28.77	2.1	23.3	HL-93	-19.6	HL-93	25.4	-17.5
1	9	32.37	-2.1	19.6	HL-93	-23.3	HL-93	17.5	-25.3
1	10	35.96	-6.2	16.1	HL-93	-27.2	HL-93	9.8	-33.4
1	11	39.56	-10.6	13.3	HL-93	-31.1	HL-93	2.7	-41.7
1	12	43.16	-14.7	10.7	HL-93	-35.2	HL-93	-4.1	-49.9
1	13	46.75	-18.9	8.2	HL-93	-39.3	HL-93	-10.8	-58.2
1	14	50.35	-23.1	5.7	HL-93	-43.5	HL-93	-17.4	-66.6
1	15	53.95	-27.3	3.3	HL-93	-47.7	HL-93	-24.0	-75.0
1	16	57.54	-31.6	1.4	HL-93	-52.1	HL-93	-30.2	-83.7
1	17	61.14	-35.8	0.0	HL-93	-56.5	HL-93	-35.8	-92.3

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	92.2	109.1	145.7	123.1	53.5	17.4
1	1	3.60	83.6	99.2	132.5	111.7	47.3	16.6
1	2	7.19	75.1	89.4	119.4	100.3	41.0	15.7
1	3	10.79	66.6	79.7	106.5	89.1	34.7	14.9
1	4	14.39	58.3	70.0	93.7	77.9	28.5	14.2
1	5	17.98	50.0	60.5	81.0	66.9	22.2	13.6

1	6	21.58	41.6	51.0	68.2	55.8	15.7	13.1
1	7	25.18	33.5	41.6	55.8	44.9	9.4	12.4
1	8	28.77	25.4	32.4	43.5	34.2	3.2	11.3
1	9	32.37	-25.3	-32.3	-43.4	-34.1	-3.1	11.3
1	10	35.96	-33.4	-41.6	-55.8	-44.9	-9.4	12.4
1	11	39.56	-41.7	-51.1	-68.4	-55.9	-15.9	13.1
1	12	43.16	-49.9	-60.5	-80.9	-66.8	-22.1	13.6
1	13	46.75	-58.2	-70.0	-93.6	-77.9	-28.4	14.2
1	14	50.35	-66.6	-79.6	-106.4	-89.0	-34.7	14.9
1	15	53.95	-75.0	-89.3	-119.4	-100.3	-40.9	15.7
1	16	57.54	-83.7	-99.3	-132.6	-111.8	-47.4	16.6
1	17	61.14	-92.3	-109.2	-145.8	-123.2	-53.7	17.4

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	35.69	0.00	HL-93	56.49	HL-93	ST1	29.89	145.69
						ST2	29.89	123.10
						ST4	40.14	53.53
						SE1	35.69	92.18
						SE2	35.69	109.12
2	35.79	0.00	HL-93	56.49	HL-93	ST1	29.98	145.82
						ST2	29.98	123.22
						ST4	40.29	53.68
						SE1	35.79	92.28
						SE2	35.79	109.22

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.60	-0.0361	-0.1319	0.0000	-0.0566	-0.1697	-0.0566	-0.2263
1	2	7.19	-0.0705	-0.2573	0.0000	-0.1103	-0.3311	-0.1103	-0.4413
1	3	10.79	-0.1014	-0.3703	0.0000	-0.1585	-0.4766	-0.1585	-0.6351
1	4	14.39	-0.1282	-0.4682	0.0000	-0.2002	-0.6026	-0.2002	-0.8028
1	5	17.98	-0.1507	-0.5502	0.0000	-0.2351	-0.7082	-0.2351	-0.9432
1	6	21.58	-0.1681	-0.6138	0.0000	-0.2622	-0.7901	-0.2622	-1.0522
1	7	25.18	-0.1800	-0.6573	0.0000	-0.2807	-0.8460	-0.2807	-1.1267
1	8	28.77	-0.1860	-0.6793	0.0000	-0.2900	-0.8744	-0.2900	-1.1644
1	9	32.37	-0.1860	-0.6793	0.0000	-0.2900	-0.8744	-0.2900	-1.1644
1	10	35.96	-0.1800	-0.6573	0.0000	-0.2807	-0.8461	-0.2807	-1.1267
1	11	39.56	-0.1681	-0.6138	0.0000	-0.2622	-0.7902	-0.2622	-1.0523
1	12	43.16	-0.1507	-0.5502	0.0000	-0.2351	-0.7083	-0.2351	-0.9433
1	13	46.75	-0.1282	-0.4682	0.0000	-0.2002	-0.6027	-0.2002	-0.8029

1	14	50.35	-0.1014	-0.3703	0.0000	-0.1585	-0.4767	-0.1585	-0.6353
1	15	53.95	-0.0705	-0.2573	0.0000	-0.1103	-0.3312	-0.1103	-0.4415
1	16	57.54	-0.0361	-0.1319	0.0000	-0.0566	-0.1698	-0.0566	-0.2264
1	17	61.14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.6	-0.036	0 1/16	-0.134	0 3/16	-0.057	0 1/16	-0.226	0 1/ 4
1	2	7.2	-0.070	0 1/ 8	-0.261	0 5/16	-0.110	0 1/ 8	-0.441	0 1/ 2
1	3	10.8	-0.101	0 1/ 8	-0.375	0 7/16	-0.159	0 3/16	-0.635	0 11/16
1	4	14.4	-0.128	0 3/16	-0.474	0 1/ 2	-0.200	0 1/ 4	-0.803	0 13/16
1	5	18.0	-0.151	0 3/16	-0.557	0 9/16	-0.235	0 1/ 4	-0.943	1 0/16
1	6	21.6	-0.168	0 3/16	-0.622	0 5/ 8	-0.262	0 5/16	-1.052	1 1/16
1	7	25.2	-0.180	0 3/16	-0.666	0 11/16	-0.281	0 5/16	-1.127	1 3/16
1	8	28.8	-0.186	0 3/16	-0.688	0 3/ 4	-0.290	0 5/16	-1.164	1 3/16
1	9	32.4	-0.186	0 3/16	-0.688	0 3/ 4	-0.290	0 5/16	-1.164	1 3/16
1	10	36.0	-0.180	0 3/16	-0.666	0 11/16	-0.281	0 5/16	-1.127	1 3/16
1	11	39.6	-0.168	0 3/16	-0.622	0 5/ 8	-0.262	0 5/16	-1.052	1 1/16
1	12	43.2	-0.151	0 3/16	-0.558	0 9/16	-0.235	0 1/ 4	-0.943	1 0/16
1	13	46.8	-0.128	0 3/16	-0.474	0 1/ 2	-0.200	0 1/ 4	-0.803	0 13/16
1	14	50.4	-0.101	0 1/ 8	-0.375	0 7/16	-0.159	0 3/16	-0.635	0 11/16
1	15	53.9	-0.070	0 1/ 8	-0.261	0 5/16	-0.110	0 1/ 8	-0.441	0 1/ 2
1	16	57.5	-0.036	0 1/16	-0.134	0 3/16	-0.057	0 1/16	-0.226	0 1/ 4
1	17	61.1	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	30.57	3 0.425	-0.413	MAX HL-93	0.92	0.00329
			0.033	MIN		
			-0.121	MAX LANE		
			0.033	MIN LANE		
		4 0.433	-0.421	MAX HL-93	0.92	0.00329
			0.034	MIN		
			-0.123	MAX LANE		
			0.034	MIN LANE		

```
                    3    0.425     -0.303 MAX FA
                                    0.000 MIN FA

    ---------------------------------------------------------------------
    NOTE: [1] " - " indicates downward deflection

          [2] The distribution factor for LL+I deflection is defined as

              DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

              where NL= no. of traffic lanes
                    Ng= no. of girders
                    (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

          [3] This table is based upon the optional criteria specified
              in AASHTO LRFD Art. 3.6.1.3.2

          [4] The number of traffic lanes is determined according to
              AASHTO LRFD Art.3.6.1.1.1.
              The 1st line is for the most probable number of lanes
              and the 2nd line is for the next probable number of lanes.

          [5] Max rotations at left (1st line) & right (2nd line)
              supports of the span without averaging, factor and impact

          [6] If ADTT is between 100 and 1000, multi-presence
              factor of 0.95 is applied. If ADTT is below 100, factor
              is 0.9 (AASHTO LRFD C3.6.1.1.2).

          [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
              ********************************************

              STEEL DEAD LOAD    OTHER DEAD LOAD   TOTAL DEAD LOAD
 SP IN D FROM ----------------  ----------------  ----------------
 NO NO L SUPT    STEEL BEAM        STEEL BEAM        STEEL BEAM
                TOP      BOT      TOP      BOT      TOP      BOT
        (ft)       (ksi)             (ksi)             (ksi)
 ---------------------------------------------------------------------

  1  0   0.00    0.00    0.00      0.00    0.00      0.00    0.00
  1  1   3.60   -0.47    0.47     -1.72    1.72     -2.18    2.18
  1  2   7.19   -0.87    0.87     -3.22    3.22     -4.09    4.09
  1  3  10.79   -1.22    1.22     -4.51    4.51     -5.74    5.74
  1  4  14.39   -1.37    0.95     -5.05    3.52     -6.41    4.48
  1  5  17.98   -1.58    1.10     -5.83    4.07     -7.41    5.17
  1  6  21.58   -1.73    1.21     -6.41    4.48     -8.14    5.69
  1  7  25.18   -1.84    1.28     -6.80    4.75     -8.63    6.03
  1  8  28.77   -1.89    1.32     -6.99    4.88     -8.88    6.20
  1  9  32.37   -1.89    1.32     -6.99    4.88     -8.88    6.20
  1 10  35.96   -1.84    1.28     -6.80    4.75     -8.64    6.03
  1 11  39.56   -1.73    1.21     -6.41    4.48     -8.15    5.69
  1 12  43.16   -1.58    1.10     -5.83    4.07     -7.40    5.17
  1 13  46.75   -1.37    0.95     -5.05    3.52     -6.41    4.48
  1 14  50.35   -1.22    1.22     -4.52    4.52     -5.74    5.74
  1 15  53.95   -0.87    0.87     -3.23    3.23     -4.10    4.10
  1 16  57.54   -0.47    0.47     -1.73    1.73     -2.19    2.19
  1 17  61.14    0.00    0.00      0.00    0.00      0.00    0.00


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 TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
```

5969 Span 2-S20 Exterior Strength CP.RES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	-0.03	-0.43	1.07	-0.03	-2.62	3.25
1	2	7.19	-0.06	-0.81	2.00	-0.06	-4.91	6.09
1	3	10.79	-0.08	-1.14	2.80	-0.08	-6.87	8.54
1	4	14.39	-0.08	-1.39	2.21	-0.08	-7.80	6.69
1	5	17.98	-0.10	-1.60	2.55	-0.10	-9.01	7.72
1	6	21.58	-0.11	-1.76	2.81	-0.11	-9.91	8.50
1	7	25.18	-0.11	-1.87	2.98	-0.11	-10.50	9.01
1	8	28.77	-0.12	-1.92	3.07	-0.12	-10.80	9.26
1	9	32.37	-0.12	-1.92	3.07	-0.12	-10.80	9.27
1	10	35.96	-0.11	-1.87	2.98	-0.11	-10.51	9.01
1	11	39.56	-0.11	-1.76	2.81	-0.11	-9.91	8.50
1	12	43.16	-0.10	-1.60	2.55	-0.10	-9.00	7.72
1	13	46.75	-0.08	-1.39	2.21	-0.08	-7.80	6.69
1	14	50.35	-0.08	-1.14	2.80	-0.08	-6.88	8.54
1	15	53.95	-0.06	-0.81	2.00	-0.06	-4.92	6.11
1	16	57.54	-0.03	-0.43	1.07	-0.03	-2.63	3.26
1	17	61.14	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	-0.18	-0.52	3.86	0.00	0.00	0.00
1	2	7.19	-0.33	-0.96	7.17	0.00	0.00	0.00
1	3	10.79	-0.46	-1.33	9.91	0.00	0.00	0.00
1	4	14.39	-0.47	-1.91	7.73	0.00	0.00	0.00
1	5	17.98	-0.54	-2.17	8.79	0.00	0.00	0.00
1	6	21.58	-0.59	-2.37	9.58	0.00	0.00	0.00
1	7	25.18	-0.61	-2.48	10.04	0.00	0.00	0.00
1	8	28.77	-0.62	-2.51	10.17	0.00	0.00	0.00
1	9	32.37	-0.62	-2.51	10.17	0.00	0.00	0.00
1	10	35.96	-0.61	-2.48	10.04	0.00	0.00	0.00
1	11	39.56	-0.59	-2.37	9.58	0.00	0.00	0.00
1	12	43.16	-0.54	-2.17	8.79	0.00	0.00	0.00
1	13	46.75	-0.47	-1.91	7.73	0.00	0.00	0.00
1	14	50.35	-0.46	-1.33	9.91	0.00	0.00	0.00
1	15	53.95	-0.33	-0.96	7.17	0.00	0.00	0.00
1	16	57.54	-0.18	-0.52	3.86	0.00	0.00	0.00
1	17	61.14	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING STRESS TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING STRESS TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.60	0.2	0.0	0.2	C	HL	1.4	0.0	1.4	T	HL
1	2	7.19	0.3	0.0	0.3	C	HL	2.6	0.0	2.6	T	HL
1	3	10.79	0.5	0.0	0.5	C	HL	3.5	0.0	3.5	T	HL
1	4	14.39	0.7	0.0	0.7	C	HL	2.7	0.0	2.7	T	HL
1	5	17.98	0.7	0.0	0.7	C	HL	3.0	0.0	3.0	T	HL
1	6	21.58	0.8	0.0	0.8	C	HL	3.3	0.0	3.3	T	HL
1	7	25.18	0.8	0.0	0.8	C	HL	3.3	0.0	3.3	T	HL
1	8	28.77	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	9	32.37	0.8	0.0	0.8	C	HL	3.1	0.0	3.1	T	HL
1	10	35.96	0.8	0.0	0.8	C	HL	3.3	0.0	3.3	T	HL
1	11	39.56	0.8	0.0	0.8	C	HL	3.3	0.0	3.3	T	HL
1	12	43.16	0.7	0.0	0.7	C	HL	3.0	0.0	3.0	T	HL
1	13	46.75	0.7	0.0	0.7	C	HL	2.7	0.0	2.7	T	HL
1	14	50.35	0.5	0.0	0.5	C	HL	3.5	0.0	3.5	T	HL
1	15	53.95	0.3	0.0	0.3	C	HL	2.6	0.0	2.6	T	HL
1	16	57.54	0.2	0.0	0.2	C	HL	1.4	0.0	1.4	T	HL
1	17	61.14	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	33.	33.	-0.21	-3.14	7.11	-0.03	-2.62	3.25
1	2	7.19	33.	33.	-0.39	-5.87	13.26	-0.06	-4.91	6.09
1	3	10.79	33.	33.	-0.54	-8.21	18.45	-0.08	-6.87	8.54
1	4	14.39	33.	33.	-0.56	-9.71	14.42	-0.08	-7.80	6.69
1	5	17.98	33.	33.	-0.63	-11.18	16.51	-0.10	-9.01	7.72
1	6	21.58	33.	33.	-0.69	-12.27	18.08	-0.11	-9.91	8.50
1	7	25.18	33.	33.	-0.73	-12.98	19.04	-0.11	-10.50	9.01
1	8	28.77	33.	33.	-0.74	-13.31	19.43	-0.12	-10.80	9.26
1	9	32.37	33.	33.	-0.74	-13.32	19.43	-0.12	-10.80	9.27
1	10	35.96	33.	33.	-0.73	-12.99	19.05	-0.11	-10.51	9.01
1	11	39.56	33.	33.	-0.69	-12.28	18.08	-0.11	-9.91	8.50
1	12	43.16	33.	33.	-0.63	-11.18	16.51	-0.10	-9.00	7.72
1	13	46.75	33.	33.	-0.56	-9.71	14.42	-0.08	-7.80	6.69
1	14	50.35	33.	33.	-0.54	-8.21	18.45	-0.08	-6.88	8.54
1	15	53.95	33.	33.	-0.39	-5.88	13.27	-0.06	-4.92	6.11
1	16	57.54	33.	33.	-0.21	-3.15	7.12	-0.03	-2.63	3.26
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

5969 Span 2-S20 Exterior Strength CP.RES
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	33.	33.	-0.26	-3.29	8.27	-0.03	-2.62	3.25
1	2	7.19	33.	33.	-0.49	-6.16	15.41	-0.06	-4.91	6.09
1	3	10.79	33.	33.	-0.68	-8.61	21.42	-0.08	-6.87	8.54
1	4	14.39	33.	33.	-0.70	-10.28	16.74	-0.08	-7.80	6.69
1	5	17.98	33.	33.	-0.80	-11.83	19.15	-0.10	-9.01	7.72
1	6	21.58	33.	33.	-0.87	-12.98	20.96	-0.11	-9.91	8.50
1	7	25.18	33.	33.	-0.91	-13.73	22.05	-0.11	-10.50	9.01
1	8	28.77	33.	33.	-0.92	-14.07	22.48	-0.12	-10.80	9.26
1	9	32.37	33.	33.	-0.92	-14.07	22.48	-0.12	-10.80	9.27
1	10	35.96	33.	33.	-0.91	-13.73	22.06	-0.11	-10.51	9.01
1	11	39.56	33.	33.	-0.87	-12.99	20.96	-0.11	-9.91	8.50
1	12	43.16	33.	33.	-0.80	-11.83	19.15	-0.10	-9.00	7.72
1	13	46.75	33.	33.	-0.70	-10.28	16.74	-0.08	-7.80	6.69
1	14	50.35	33.	33.	-0.68	-8.61	21.43	-0.08	-6.88	8.54
1	15	53.95	33.	33.	-0.49	-6.17	15.42	-0.06	-4.92	6.11
1	16	57.54	33.	33.	-0.26	-3.30	8.28	-0.03	-2.63	3.26
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	33.	33.	-0.36	-4.25	10.99	-0.04	-3.34	4.23
1	2	7.19	33.	33.	-0.66	-7.95	20.48	-0.08	-6.26	7.94
1	3	10.79	33.	33.	-0.92	-11.11	28.46	-0.11	-8.77	11.12
1	4	14.39	33.	33.	-0.95	-13.32	22.24	-0.12	-9.97	8.72
1	5	17.98	33.	33.	-1.08	-15.32	25.45	-0.14	-11.52	10.07
1	6	21.58	33.	33.	-1.18	-16.81	27.84	-0.15	-12.66	11.07
1	7	25.18	33.	33.	-1.23	-17.77	29.30	-0.16	-13.43	11.74
1	8	28.77	33.	33.	-1.25	-18.21	29.86	-0.16	-13.81	12.07
1	9	32.37	33.	33.	-1.25	-18.21	29.87	-0.16	-13.81	12.07
1	10	35.96	33.	33.	-1.23	-17.78	29.31	-0.16	-13.44	11.74
1	11	39.56	33.	33.	-1.18	-16.81	27.85	-0.15	-12.67	11.07
1	12	43.16	33.	33.	-1.08	-15.31	25.44	-0.14	-11.51	10.06
1	13	46.75	33.	33.	-0.95	-13.32	22.24	-0.12	-9.98	8.72
1	14	50.35	33.	33.	-0.92	-11.12	28.47	-0.11	-8.78	11.13
1	15	53.95	33.	33.	-0.66	-7.97	20.49	-0.08	-6.28	7.95
1	16	57.54	33.	33.	-0.36	-4.26	11.00	-0.04	-3.35	4.25
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

YIELD STRESS | TOTAL POSITIVE ,(ksi) | TOTAL NEGATIVE ,(ksi)

SP IN D FROM --------- CONCR. STEEL BEAM CONCR. STEEL BEAM

NO	NO	L SUPT (ft)	Fy (ksi) TOP	Fy (ksi) BOT	TOP TOP	TOP BOT	TOP BOT	TOP TOP	TOP BOT	TOP BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	33.	33.	-0.34	-3.19	9.51	-0.02	-2.28	2.75
1	2	7.19	33.	33.	-0.63	-5.97	17.70	-0.04	-4.29	5.16
1	3	10.79	33.	33.	-0.87	-8.34	24.58	-0.06	-6.00	7.23
1	4	14.39	33.	33.	-0.89	-10.14	19.19	-0.06	-6.80	5.67
1	5	17.98	33.	33.	-1.01	-11.65	21.92	-0.07	-7.85	6.54
1	6	21.58	33.	33.	-1.11	-12.78	23.97	-0.08	-8.63	7.19
1	7	25.18	33.	33.	-1.16	-13.49	25.19	-0.08	-9.15	7.63
1	8	28.77	33.	33.	-1.17	-13.81	25.64	-0.09	-9.41	7.85
1	9	32.37	33.	33.	-1.17	-13.81	25.64	-0.09	-9.42	7.85
1	10	35.96	33.	33.	-1.16	-13.50	25.19	-0.08	-9.16	7.63
1	11	39.56	33.	33.	-1.11	-12.78	23.97	-0.08	-8.64	7.20
1	12	43.16	33.	33.	-1.01	-11.65	21.92	-0.07	-7.85	6.54
1	13	46.75	33.	33.	-0.89	-10.14	19.19	-0.06	-6.80	5.67
1	14	50.35	33.	33.	-0.87	-8.34	24.58	-0.06	-6.01	7.24
1	15	53.95	33.	33.	-0.63	-5.98	17.71	-0.04	-4.30	5.17
1	16	57.54	33.	33.	-0.34	-3.20	9.52	-0.02	-2.29	2.76
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.60	33.	33.	-0.29	-4.04	9.45	-0.04	-3.34	4.23
1	2	7.19	33.	33.	-0.53	-7.57	17.61	-0.08	-6.26	7.94
1	3	10.79	33.	33.	-0.73	-10.58	24.50	-0.11	-8.77	11.12
1	4	14.39	33.	33.	-0.76	-12.55	19.15	-0.12	-9.97	8.72
1	5	17.98	33.	33.	-0.86	-14.45	21.93	-0.14	-11.52	10.07
1	6	21.58	33.	33.	-0.94	-15.86	24.01	-0.15	-12.66	11.07
1	7	25.18	33.	33.	-0.99	-16.78	25.29	-0.16	-13.43	11.74
1	8	28.77	33.	33.	-1.00	-17.20	25.80	-0.16	-13.81	12.07
1	9	32.37	33.	33.	-1.00	-17.21	25.80	-0.16	-13.81	12.07
1	10	35.96	33.	33.	-0.99	-16.78	25.29	-0.16	-13.44	11.74
1	11	39.56	33.	33.	-0.94	-15.87	24.01	-0.15	-12.67	11.07
1	12	43.16	33.	33.	-0.86	-14.44	21.93	-0.14	-11.51	10.06
1	13	46.75	33.	33.	-0.76	-12.55	19.15	-0.12	-9.98	8.72
1	14	50.35	33.	33.	-0.73	-10.58	24.51	-0.11	-8.78	11.13
1	15	53.95	33.	33.	-0.53	-7.58	17.63	-0.08	-6.28	7.95
1	16	57.54	33.	33.	-0.29	-4.05	9.46	-0.04	-3.35	4.25
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

1	1	3.60	33.	33.	-0.27	-2.99	7.97	-0.02	-2.28	2.75
1	2	7.19	33.	33.	-0.49	-5.59	14.84	-0.04	-4.29	5.16
1	3	10.79	33.	33.	-0.68	-7.81	20.61	-0.06	-6.00	7.23
1	4	14.39	33.	33.	-0.70	-9.38	16.10	-0.06	-6.80	5.67
1	5	17.98	33.	33.	-0.80	-10.78	18.41	-0.07	-7.85	6.54
1	6	21.58	33.	33.	-0.87	-11.83	20.13	-0.08	-8.63	7.19
1	7	25.18	33.	33.	-0.91	-12.50	21.18	-0.08	-9.15	7.63
1	8	28.77	33.	33.	-0.93	-12.81	21.57	-0.09	-9.41	7.85
1	9	32.37	33.	33.	-0.93	-12.81	21.57	-0.09	-9.42	7.85
1	10	35.96	33.	33.	-0.91	-12.51	21.18	-0.08	-9.16	7.63
1	11	39.56	33.	33.	-0.87	-11.83	20.14	-0.08	-8.64	7.20
1	12	43.16	33.	33.	-0.80	-10.78	18.41	-0.07	-7.85	6.54
1	13	46.75	33.	33.	-0.70	-9.38	16.10	-0.06	-6.80	5.67
1	14	50.35	33.	33.	-0.68	-7.81	20.62	-0.06	-6.01	7.24
1	15	53.95	33.	33.	-0.49	-5.60	14.85	-0.04	-4.30	5.17
1	16	57.54	33.	33.	-0.27	-3.00	7.98	-0.02	-2.29	2.76
1	17	61.14	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.07	197.	-5.8	126.	-1.2	495.	-1.4	0.	0.0	-8.36
											1.36
	LB			5.8		2.9		10.1		0.0	18.77
											10.08
	RT			-5.3		-1.1		-1.6		0.0	-8.01
											1.60
	RB			3.7		1.8		6.5		0.0	11.96
											6.46
1	LT	50.07	198.	-5.3	126.	-1.1	495.	-1.6	0.	0.0	-8.02
											1.60
	LB			3.7		1.8		6.5		0.0	11.96
											6.46
	RT			-5.8		-1.2		-1.4		0.0	-8.36
											1.36
	RB			5.8		2.9		10.1		0.0	18.78
											10.08

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.07	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000
2	39.00	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	10.00	1.0000
3	11.07	W	11.5	0.8500	31.4	0.5790	11.5	0.8500	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 61.1

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. (in)	[2]/(12*[1])	<=>	LIMITATION 1/25	STATUS
				[3]= GIRDER D. (in)	[3]/(12*[1])		1/30	
1	11.07	6.75	33.09	[2]= 41.39	0.05641	>	0.04000	OK
				[3]= 33.09	0.04510	>	0.03300	OK
2	39.00	6.75	33.09	[2]= 41.39	0.05641	>	0.04000	OK
				[3]= 33.09	0.04510	>	0.03300	OK
3	11.07	6.75	33.09	[2]= 41.39	0.05641	>	0.04000	OK
				[3]= 33.09	0.04510	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN D NO	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	[2]	2Dcp/tw [3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0	0.00	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	1	3.60	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	2	7.19	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	3	10.79	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	4	14.39	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	5	17.98	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	6	21.58	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	7	25.18	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	8	28.77	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	9	32.37	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0
1	10	35.96	0	1	31.39	0.579	54.2	150.0	0.00	111.46			0

1	11	39.56	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	12	43.16	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	13	46.75	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	14	50.35	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	15	53.95	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	16	57.54	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0
1	17	61.14	0	1	31.39	0.579	54.2	150.0	0.00	111.46	0

NOTE:

[1] 0 - No long. stiffeners
1 - long. stiffeners

[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)

[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0

[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)

[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw

[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0		0.00	0	1	31.39	0.579	54.21	168.97	2
1	1		3.60	0	1	31.39	0.579	54.21	168.97	2
1	2		7.19	0	1	31.39	0.579	54.21	168.97	2
1	3		10.79	0	1	31.39	0.579	54.21	168.97	2
1	4		14.39	0	1	31.39	0.579	66.55	168.97	2
1	5		17.98	0	1	31.39	0.579	66.55	168.97	2
1	6		21.58	0	1	31.39	0.579	66.55	168.97	2
1	7		25.18	0	1	31.39	0.579	66.55	168.97	2
1	8		28.77	0	1	31.39	0.579	66.55	168.97	2
1	9		32.37	0	1	31.39	0.579	66.55	168.97	2
1	10		35.96	0	1	31.39	0.579	66.55	168.97	2
1	11		39.56	0	1	31.39	0.579	66.55	168.97	2
1	12		43.16	0	1	31.39	0.579	66.55	168.97	2
1	13		46.75	0	1	31.39	0.579	66.55	168.97	2
1	14		50.35	0	1	31.39	0.579	54.21	168.97	2
1	15		53.95	0	1	31.39	0.579	54.21	168.97	2
1	16		57.54	0	1	31.39	0.579	54.21	168.97	2
1	17		61.14	0	1	31.39	0.579	54.21	168.97	2

NOTE:

[1] 0 - No long. stiffeners
1 - long. stiffeners

[2] For non-composite sections, calculate Dc and 2*Dc/tw

[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM NO SUPT L (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 1	3.60	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 2	7.19	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 3	10.79	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 4	14.39	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 5	17.98	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 6	21.58	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 7	25.18	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 8	28.77	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 9	32.37	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 10	35.96	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 11	39.56	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 12	43.16	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 13	46.75	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 14	50.35	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 15	53.95	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 16	57.54	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0
1 17	61.14	6.76 12.	11.5	5.2	0.850	0.637		
		6.76 12.	11.5	5.2	0.850	0.637	1.00	0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	18.8	0.000	11.373	7.665	3.958	1.000
			0.000	0.000	0.000	0.000	1.000
2	18.8	37.7	11.373	12.608	13.268	13.929	1.000
			0.000	0.000	0.000	0.000	1.000
3	37.7	56.5	4.105	12.608	9.524	6.440	1.000
			0.000	0.000	0.000	0.000	1.000
4	56.5	61.1	0.000	4.105	2.113	0.120	1.720
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	1	3.60	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	2	7.19	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	3	10.79	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	4	14.39	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	5	17.98	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	6	21.58	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	7	25.18	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	8	28.77	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	9	32.37	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	10	35.96	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	11	39.56	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	12	43.16	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	13	46.75	1	6.76	11.26	19.84	0	18.84	6.98	26.21	2	2
1	14	50.35	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	15	53.95	1	6.76	11.26	19.84	0	18.84	7.17	26.91	2	2
1	16	57.54	1	6.76	11.26	19.84	0	4.62	7.17	26.91	0	0
1	17	61.14	1	6.76	11.26	19.84	0	4.62	7.17	26.91	0	0

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.00	33.00	27.15	27.15
1	1		3.60	1	1.000	1.000	1.00	33.00	27.15	27.15
1	2		7.19	1	1.000	1.000	1.00	33.00	27.15	27.15
1	3		10.79	1	1.000	1.000	1.00	33.00	27.15	27.15
1	4		14.39	1	1.000	1.000	1.00	33.00	26.89	26.89
1	5		17.98	1	1.000	1.000	1.00	33.00	26.89	26.89
1	6		21.58	1	1.000	1.000	1.00	33.00	26.89	26.89
1	7		25.18	1	1.000	1.000	1.00	33.00	26.89	26.89
1	8		28.77	1	1.000	1.000	1.00	33.00	26.89	26.89
1	9		32.37	1	1.000	1.000	1.00	33.00	26.89	26.89
1	10		35.96	1	1.000	1.000	1.00	33.00	26.89	26.89
1	11		39.56	1	1.000	1.000	1.00	33.00	26.89	26.89
1	12		43.16	1	1.000	1.000	1.00	33.00	26.89	26.89
1	13		46.75	1	1.000	1.000	1.00	33.00	26.89	26.89
1	14		50.35	1	1.000	1.000	1.00	33.00	27.15	27.15
1	15		53.95	1	1.000	1.000	1.00	33.00	27.15	27.15
1	16		57.54	1	1.000	1.000	1.72	33.00	33.00	33.00
1	17		61.14	1	1.000	1.000	1.72	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	8.35	17.38
1	1		3.60	1	8.35	17.38
1	2		7.19	1	8.35	17.38
1	3		10.79	1	8.35	17.38
1	4		14.39	1	8.79	17.80
1	5		17.98	1	8.79	17.80
1	6		21.58	1	8.79	17.80
1	7		25.18	1	8.79	17.80
1	8		28.77	1	8.79	17.80
1	9		32.37	1	8.79	17.80
1	10		35.96	1	8.79	17.80
1	11		39.56	1	8.79	17.80
1	12		43.16	1	8.79	17.80
1	13		46.75	1	8.79	17.80
1	14		50.35	1	8.35	17.38
1	15		53.95	1	8.35	17.38
1	16		57.54	1	8.35	17.38
1	17		61.14	1	8.35	17.38

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP	IN	D	FROM	Section	STRENGTH CATEGORY, Noncomposite	Composite	Non-	Comp.

5969 Span 2-S20 Exterior Strength CP.RES

NO	NO	L SUPT (ft)	Region	Web [1]	Flange [2]	Web [3]	Flange [4]	Comp.	
1	0	0.00	1	0	2	0		2	0
1	1	3.60	1	0	2	0		2	0
1	2	7.19	1	0	2	0		2	0
1	3	10.79	1	0	2	0		2	0
1	4	14.39	1	0	2	0		2	0
1	5	17.98	1	0	2	0		2	0
1	6	21.58	1	0	2	0		2	0
1	7	25.18	1	0	2	0		2	0
1	8	28.77	1	0	2	0		2	0
1	9	32.37	1	0	2	0		2	0
1	10	35.96	1	0	2	0		2	0
1	11	39.56	1	0	2	0		2	0
1	12	43.16	1	0	2	0		2	0
1	13	46.75	1	0	2	0		2	0
1	14	50.35	1	0	2	0		2	0
1	15	53.95	1	0	2	0		2	0
1	16	57.54	1	0	0	0		0	0
1	17	61.14	1	0	0	0		0	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	27.1	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.60	0.0	19.8	3.3	33.0	3.3	33.0	3.3	27.1	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	2		7.19	0.0	19.8	6.1	33.0	6.1	33.0	6.1	27.1	0
				0.0	-	6.1	-	6.1	33.0	-	-	0

1	3	10.79	0.0	19.8	8.6	33.0	8.6	33.0	8.6	27.1	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	4	14.39	0.0	19.8	9.6	33.0	9.6	33.0	9.6	26.9	0
			0.0	-	6.7	-	6.7	33.0	-	-	0
1	5	17.98	0.0	19.8	11.1	33.0	11.1	33.0	11.1	26.9	0
			0.0	-	7.8	-	7.8	33.0	-	-	0
1	6	21.58	0.0	19.8	12.2	33.0	12.2	33.0	12.2	26.9	0
			0.0	-	8.5	-	8.5	33.0	-	-	0
1	7	25.18	0.0	19.8	13.0	33.0	13.0	33.0	13.0	26.9	0
			0.0	-	9.0	-	9.0	33.0	-	-	0
1	8	28.77	0.0	19.8	13.3	33.0	13.3	33.0	13.3	26.9	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	9	32.37	0.0	19.8	13.3	33.0	13.3	33.0	13.3	26.9	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	10	35.96	0.0	19.8	13.0	33.0	13.0	33.0	13.0	26.9	0
			0.0	-	9.0	-	9.0	33.0	-	-	0
1	11	39.56	0.0	19.8	12.2	33.0	12.2	33.0	12.2	26.9	0
			0.0	-	8.5	-	8.5	33.0	-	-	0
1	12	43.16	0.0	19.8	11.1	33.0	11.1	33.0	11.1	26.9	0
			0.0	-	7.8	-	7.8	33.0	-	-	0
1	13	46.75	0.0	19.8	9.6	33.0	9.6	33.0	9.6	26.9	0
			0.0	-	6.7	-	6.7	33.0	-	-	0
1	14	50.35	0.0	19.8	8.6	33.0	8.6	33.0	8.6	27.1	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	15	53.95	0.0	19.8	6.2	33.0	6.2	33.0	6.2	27.1	0
			0.0	-	6.2	-	6.2	33.0	-	-	0
1	16	57.54	0.0	19.8	3.3	33.0	3.3	33.0	3.3	33.0	0
			0.0	-	3.3	-	3.3	33.0	-	-	0
1	17	61.14	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.60	0.000	0.099	0.099	0.121	0.121	4
					0.099		0.099	3
1	2	7.19	0.000	0.186	0.186	0.226	0.226	4
					0.186		0.186	3
1	3	10.79	0.000	0.261	0.261	0.317	0.317	4
					0.261		0.261	3
1	4	14.39	0.000	0.291	0.291	0.358	0.358	4
					0.204		0.204	3
1	5	17.98	0.000	0.337	0.337	0.413	0.413	4
					0.235		0.235	3
1	6	21.58	0.000	0.370	0.370	0.454	0.454	4

					0.258		0.258	3
1	7	25.18	0.000	0.392	0.392	0.482	0.482	4
					0.274		0.274	3
1	8	28.77	0.000	0.404	0.404	0.495	0.495	4
					0.282		0.282	3
1	9	32.37	0.000	0.404	0.404	0.495	0.495	4
					0.282		0.282	3
1	10	35.96	0.000	0.393	0.393	0.482	0.482	4
					0.274		0.274	3
1	11	39.56	0.000	0.370	0.370	0.454	0.454	4
					0.259		0.259	3
1	12	43.16	0.000	0.336	0.336	0.413	0.413	4
					0.235		0.235	3
1	13	46.75	0.000	0.292	0.292	0.358	0.358	4
					0.204		0.204	3
1	14	50.35	0.000	0.261	0.261	0.317	0.317	4
					0.261		0.261	3
1	15	53.95	0.000	0.187	0.187	0.227	0.227	4
					0.187		0.187	3
1	16	57.54	0.000	0.100	0.100	0.100	0.100	2
					0.100		0.100	3
1	17	61.14	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2056.9	-	-	0.0	19.8	-	-	0--0
1	1		3.60	0	490.8	2056.9	-	-	0.0	19.8	-	-	0--0
1	2		7.19	0	913.9	2056.9	-	-	0.0	19.8	-	-	0--0
1	3		10.79	0	1269.3	2056.9	-	-	0.0	19.8	-	-	0--0
1	4		14.39	0	1553.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	5		17.98	0	1776.0	2899.5	-	-	0.0	19.8	-	-	0--0
1	6		21.58	0	1942.2	2899.5	-	-	0.0	19.8	-	-	0--0
1	7		25.18	0	2042.3	2899.5	-	-	0.0	19.8	-	-	0--0
1	8		28.77	0	2079.7	2899.5	-	-	0.0	19.8	-	-	0--0
1	9		32.37	0	2079.8	2899.5	-	-	0.0	19.8	-	-	0--0
1	10		35.96	0	2042.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	11		39.56	0	1942.4	2899.5	-	-	0.0	19.8	-	-	0--0
1	12		43.16	0	1775.9	2899.5	-	-	0.0	19.8	-	-	0--0
1	13		46.75	0	1553.6	2899.5	-	-	0.0	19.8	-	-	0--0
1	14		50.35	0	1269.5	2056.9	-	-	0.0	19.8	-	-	0--0
1	15		53.95	0	914.3	2056.9	-	-	0.0	19.8	-	-	0--0
1	16		57.54	0	491.3	2056.9	-	-	0.0	19.8	-	-	0--0
1	17		61.14	0	0.0	2056.9	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in

```
              negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
        [3] = 0.6*Fyt for composite sections in positive flexure
              or tension flange for composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
            = 0.6*Fyc for comp. flange of composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
        [4] = (PHI)f * Fnt for non-compact tension flange
              of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
            = (PHI)f * Fnc for comp. flange of composite sections
              in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

        "-" is N.A.

        Under FLAG Column, 0 = OK; 1= NG
        M = Moment; C = Comp. Flange; T = Tension Flange
        D = Ductility

        For negative moment region or non-compact sections in
        positive moment region, the 1st line is for top
        flange and the 2nd line is for bottom flange

        The values of fbu, Mu and fl shall be determined based on
        factored loads, and shall be taken as positive in
        sign in all resistance equations (Art. 6.10.1.6)

        The value of fbu is the actual stress in this table,
        the users can use the maximum value within the
        unbraced length to do their own check
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   TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                   ***********************************************

   SP IN D FROM ID Mu+1/3flSxt/       fbu/        fl/ fbu+1/3fl/  MAX.
                          [1]        [2]        [3]        [4]  RAT. GOVN.
   ---------------------------------------------------------------------------

    1  0   0.00  0        0.000                                  0.000     1
    1  1   3.60  0        0.239                                  0.239     1
    1  2   7.19  0        0.444                                  0.444     1
    1  3  10.79  0        0.617                                  0.617     1
    1  4  14.39  0        0.536                                  0.536     1
    1  5  17.98  0        0.613                                  0.613     1
    1  6  21.58  0        0.670                                  0.670     1
    1  7  25.18  0        0.704                                  0.704     1
    1  8  28.77  0        0.717                                  0.717     1
    1  9  32.37  0        0.717                                  0.717     1
    1 10  35.96  0        0.704                                  0.704     1
    1 11  39.56  0        0.670                                  0.670     1
    1 12  43.16  0        0.612                                  0.612     1
    1 13  46.75  0        0.536                                  0.536     1
    1 14  50.35  0        0.617                                  0.617     1
    1 15  53.95  0        0.445                                  0.445     1
    1 16  57.54  0        0.239                                  0.239     1
    1 17  61.14  0        0.000                                  0.000     1


   ---------------------------------------------------------------------------

   NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
         [2] = (PHI)f * Fnc for comp. flange of composite
               sections in positive flexure (Eq. 6.10.7.2.1-1)
             = (PHI)f * Rh * Fyt for tension flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
         [3] = 0.6*Fyt for composite sections in positive flexure
               or tension flange for composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
             = 0.6*Fyc for comp. flange of composite sections in
               negative flexure and non-composite sections (Eq. 6.10.1.6-1)
         [4] = (PHI)f * Fnt for non-compact tension flange
               of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
             = (PHI)f * Fnc for comp. flange of composite sections
               in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)
```

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	1	3.60	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	2	7.19	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	3	10.79	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	4	14.39	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	5	17.98	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	6	21.58	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	7	25.18	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	8	28.77	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	9	32.37	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	10	35.96	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	11	39.56	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	12	43.16	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	13	46.75	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	14	50.35	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	15	53.95	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	16	57.54	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9
1	17	61.14	33.0	5.0	74.2	92.8	54.2	1.000	347.9	347.9

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot
count on post-buckling shear resistance from tension-
field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	145.7	<	347.9	0	0
1	1	3.60	0	132.5	<	347.9	0	0
1	2	7.19	0	119.4	<	347.9	0	0
1	3	10.79	0	106.5	<	347.9	0	0
1	4	14.39	0	93.7	<	347.9	0	0
1	5	17.98	0	81.0	<	347.9	0	0
1	6	21.58	0	68.2	<	347.9	0	0
1	7	25.18	0	55.8	<	347.9	0	0
1	8	28.77	0	43.5	<	347.9	0	0
1	9	32.37	0	43.4	<	347.9	0	0
1	10	35.96	0	55.8	<	347.9	0	0
1	11	39.56	0	68.4	<	347.9	0	0
1	12	43.16	0	80.9	<	347.9	0	0
1	13	46.75	0	93.6	<	347.9	0	0
1	14	50.35	0	106.4	<	347.9	0	0
1	15	53.95	0	119.4	<	347.9	0	0
1	16	57.54	0	132.6	<	347.9	0	0
1	17	61.14	0	145.8	<	347.9	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

```
            Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

    Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

            Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

    Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

            Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

          * For the detailed description of shear capacity,
            please refer to AASHTO Art.6.10.9.2 and .3

          * STRENGTH CATEGORY; 0 = compact section
                               2 = braced non-compact section
                               3 = slender section
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 TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                 ****************************

             YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
 SP IN D FROM STRESS MAXIMUM    SHEAR   ---------------- TRANS. STIFFENERS
 NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
        (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
 ---------------------------------------------------------------------------

  1  0   0.00   33.   145.69   347.87            0
  1  1   3.60   33.   132.48   347.87            0
  1  2   7.19   33.   119.41   347.87            0
  1  3  10.79   33.   106.46   347.87            0
  1  4  14.39   33.    93.65   347.87            0
  1  5  17.98   33.    80.97   347.87            0
  1  6  21.58   33.    68.25   347.87            0
  1  7  25.18   33.    55.81   347.87            0
  1  8  28.77   33.    43.49   347.87            0
  1  9  32.37   33.    43.44   347.87            0
  1 10  35.96   33.    55.75   347.87            0
  1 11  39.56   33.    68.37   347.87            0
  1 12  43.16   33.    80.92   347.87            0
  1 13  46.75   33.    93.60   347.87            0
  1 14  50.35   33.   106.41   347.87            0
  1 15  53.95   33.   119.35   347.87            0
  1 16  57.54   33.   132.61   347.87            0
  1 17  61.14   33.   145.82   347.87            0


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     TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
                     *************************

     SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                   ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
     NO NO L SUPT                                          [3]  TBC
             (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi) (ksi)
     ---------------------------------------------------------------

      1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
      1  1   3.60   3.3    -     31.4     8.3  31.4   -     -   00-
      1  2   7.19   6.2    -     31.4    15.4  31.4   -     -   00-
      1  3  10.79   8.6    -     31.4    21.4  31.4   -     -   00-
      1  4  14.39  10.3    -     31.4    16.7  31.4   -     -   00-
      1  5  17.98  11.8    -     31.4    19.2  31.4   -     -   00-
      1  6  21.58  13.0    -     31.4    21.0  31.4   -     -   00-
      1  7  25.18  13.7    -     31.4    22.1  31.4   -     -   00-
      1  8  28.77  14.1    -     31.4    22.5  31.4   -     -   00-
```

```
 1  9  32.37  14.1    -     31.4    22.5  31.4   -     -   00-
 1 10  35.96  13.7    -     31.4    22.1  31.4   -     -   00-
 1 11  39.56  13.0    -     31.4    21.0  31.4   -     -   00-
 1 12  43.16  11.8    -     31.4    19.1  31.4   -     -   00-
 1 13  46.75  10.3    -     31.4    16.7  31.4   -     -   00-
 1 14  50.35   8.6    -     31.4    21.4  31.4   -     -   00-
 1 15  53.95   6.2    -     31.4    15.4  31.4   -     -   00-
 1 16  57.54   3.3    -     31.4     8.3  31.4   -     -   00-
 1 17  61.14   0.0    -     31.4     0.0  31.4   -     -   00-


    ------------------------------------------------------------
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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                  ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.60    TR     -0.3 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
  1  2   7.19    TR     -0.5 A B B^C^C D E E^  TR      3.9 A B B^C^C D E
  1  3  10.79    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
  1     11.07  SCG
  1  4  14.39    TR     -1.0 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
  1  5  17.98    TR     -1.1 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  6  21.58    TR     -1.2 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1  7  25.18    TR     -1.2 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1  8  28.77    TR     -1.2 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1  9  32.37    TR     -1.2 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1 10  35.96    TR     -1.2 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1 11  39.56    TR     -1.2 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1 12  43.16    TR     -1.1 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1 13  46.75    TR     -1.0 A B B^C^C D E E^  TR      4.0 A B B^C^C D E
  1     50.07  SCG
```

```
 1 14  50.35    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
 1 15  53.95    TR     -0.5 A B B^C^C D E E^  TR      3.9 A B B^C^C D E
 1 16  57.54    TR     -0.3 A B B^C^C D E E^  TR      2.1 A B B^C^C D E E^
 1 17  61.14    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ------------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.60    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1  2   7.19    TR     -0.3 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
  1  3  10.79    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
  1     11.07  SCG
  1  4  14.39    TR     -0.5 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1  5  17.98    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1  6  21.58    TR     -0.6 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1  7  25.18    TR     -0.6 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1  8  28.77    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1  9  32.37    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
  1 10  35.96    TR     -0.6 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
  1 11  39.56    TR     -0.6 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1 12  43.16    TR     -0.6 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
  1 13  46.75    TR     -0.5 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
  1     50.07  SCG
  1 14  50.35    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
  1 15  53.95    TR     -0.3 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
  1 16  57.54    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1 17  61.14    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
```

I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0 0.00	34.8	.28305E-01	1.0	TR	3.09	3	9.43
1	1 3.60	33.2	.28305E-01	0.9	TR	3.09	3	9.89
1	2 7.19	31.5	.28305E-01	0.9	TR	3.09	3	10.41
1	3 10.79	29.9	.28305E-01	0.8	TR	3.09	3	10.98
1	4 14.39	28.4	.25828E-01	0.7	TR	3.09	3	12.65
1	5 17.98	27.1	.25828E-01	0.7	TR	3.09	3	13.25
1	6 21.58	26.3	.25828E-01	0.7	TR	3.09	3	13.66
1	7 25.18	24.7	.25828E-01	0.6	TR	3.09	3	14.54
1	8 28.77	22.7	.25828E-01	0.6	TR	3.09	3	15.85
1	9 32.37	22.7	.25828E-01	0.6	TR	3.09	3	15.85
1	10 35.96	24.7	.25828E-01	0.6	TR	3.09	3	14.54
1	11 39.56	26.3	.25828E-01	0.7	TR	3.09	3	13.66
1	12 43.16	27.1	.25828E-01	0.7	TR	3.09	3	13.25
1	13 46.75	28.4	.25828E-01	0.7	TR	3.09	3	12.65
1	14 50.35	29.9	.28305E-01	0.8	TR	3.09	3	10.98
1	15 53.95	31.5	.28305E-01	0.9	TR	3.09	3	10.41
1	16 57.54	33.2	.28305E-01	0.9	TR	3.09	3	9.89
1	17 61.14	34.8	.28305E-01	1.0	TR	3.09	3	9.43

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 [4],Zr	AASHTO LRFD Eqn. 6.10.10.2-2 <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	17.4	.28305E-01	0.5	TR	1.36	3	8.31
1	1	3.60	16.6	.28305E-01	0.5	TR	1.36	3	8.72
1	2	7.19	15.7	.28305E-01	0.4	TR	1.36	3	9.18
1	3	10.79	14.9	.28305E-01	0.4	TR	1.36	3	9.68
1	4	14.39	14.2	.25828E-01	0.4	TR	1.36	3	11.15
1	5	17.98	13.6	.25828E-01	0.4	TR	1.36	3	11.68
1	6	21.58	13.1	.25828E-01	0.3	TR	1.36	3	12.05
1	7	25.18	12.4	.25828E-01	0.3	TR	1.36	3	12.82
1	8	28.77	11.3	.25828E-01	0.3	TR	1.36	3	13.97
1	9	32.37	11.3	.25828E-01	0.3	TR	1.36	3	13.97
1	10	35.96	12.4	.25828E-01	0.3	TR	1.36	3	12.82
1	11	39.56	13.1	.25828E-01	0.3	TR	1.36	3	12.05
1	12	43.16	13.6	.25828E-01	0.4	TR	1.36	3	11.68
1	13	46.75	14.2	.25828E-01	0.4	TR	1.36	3	11.15
1	14	50.35	14.9	.28305E-01	0.4	TR	1.36	3	9.68
1	15	53.95	15.7	.28305E-01	0.4	TR	1.36	3	9.18
1	16	57.54	16.6	.28305E-01	0.5	TR	1.36	3	8.72
1	17	61.14	17.4	.28305E-01	0.5	TR	1.36	3	8.31

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)

```
          Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
          Ffat = radial fatigue shear range per unit length
               = Ffat2 = Frc/w for skews exceeding 20 degrees
          Frc = 25.0 kips for both exterior and interior girders
          w = the effective length of the deck
            = 48 inches except at end supports where w is taken
              as 24 inches

      [4] Zr = allowable range of horizontal shear, in Kips, on an individual
               connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

          * Default ALPHA value based on 7/8" diameter and input road type


      [5] No. of shear connectors per transverse section. This value is from
          input data type 12032.

      [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

          ** The connector spacing shown here is calculated based upon
             AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
             where ADTT Single Lane= 1140 & No. of Cycles=   31207500
         *** If ADTT Single Lane greater than or equal to 960, refer
             to Fatigue I Table.
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                 ***************************************

                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  9  32.37   1 1574.9  1204.9          132         >  63
      1 17  61.14   1 1574.9  1204.9          132         >  63


     ---------------------------------------------------------------------------
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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
             Qr = factored shear resistance of one shear connector
                = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
```

Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	32.368	9.000
1	32.368	61.140	9.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	61.1	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE

STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MOMENT D2 ksi	UNFACTORED L+I MAX. MOMENT POS. MAX. STRESS	UNFACTORED L+I MAX. MOMENT NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2056.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.60	2056.9	73.8	47.1	189.8	0.0	7.41	HL-93	5.71	0
1	2	7.19	2056.9	138.4	88.3	352.3	0.0	3.70	HL-93	2.85	0
1	3	10.79	2056.9	193.8	123.6	487.2	0.0	2.49	HL-93	1.92	0
1	4	14.39	2899.5	240.1	153.0	592.9	0.0	2.98	HL-93	2.30	0
1	5	17.98	2899.5	277.3	176.6	674.5	0.0	2.53	HL-93	1.95	0
1	6	21.58	2899.5	305.0	194.2	735.5	0.0	2.26	HL-93	1.74	0
1	7	25.18	2899.5	323.3	206.0	770.1	0.0	2.12	HL-93	1.64	0
1	8	28.77	2899.5	332.5	211.9	780.1	0.0	2.07	HL-93	1.60	0
1	9	32.37	2899.5	332.6	211.9	780.1	0.0	2.07	HL-93	1.60	0
1	10	35.96	2899.5	323.6	206.0	770.1	0.0	2.12	HL-93	1.64	0
1	11	39.56	2899.5	305.1	194.2	735.5	0.0	2.26	HL-93	1.74	0
1	12	43.16	2899.5	277.2	176.6	674.5	0.0	2.53	HL-93	1.95	0
1	13	46.75	2899.5	240.2	153.0	592.9	0.0	2.98	HL-93	2.30	0
1	14	50.35	2056.9	194.0	123.6	487.2	0.0	2.49	HL-93	1.92	0
1	15	53.95	2056.9	138.7	88.3	352.3	0.0	3.70	HL-93	2.85	0
1	16	57.54	2056.9	74.1	47.1	189.8	0.0	7.41	HL-93	5.71	0
1	17	61.14	2056.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

Please read NOTES on the following page

1

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

1

TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.60	31.4	-2.62	3.25	-0.52	7.28	HL-93	5.60
1	2	7.19	31.4	-4.91	6.09	-0.96	3.52	HL-93	2.71
1	3	10.79	31.4	-6.87	8.54	-1.33	2.30	HL-93	1.77
1	4	14.39	31.4	-7.80	6.69	-1.91	3.19	HL-93	2.45
1	5	17.98	31.4	-9.01	7.72	-2.17	2.69	HL-93	2.07
1	6	21.58	31.4	-9.91	8.50	-2.37	2.38	HL-93	1.83
1	7	25.18	31.4	-10.50	9.01	-2.48	2.23	HL-93	1.71
1	8	28.77	31.4	-10.80	9.26	-2.51	2.17	HL-93	1.67
1	9	32.37	31.4	-10.80	9.27	-2.51	2.17	HL-93	1.67
1	10	35.96	31.4	-10.51	9.01	-2.48	2.23	HL-93	1.71
1	11	39.56	31.4	-9.91	8.50	-2.37	2.38	HL-93	1.83
1	12	43.16	31.4	-9.00	7.72	-2.17	2.69	HL-93	2.07
1	13	46.75	31.4	-7.80	6.69	-1.91	3.19	HL-93	2.45
1	14	50.35	31.4	-6.88	8.54	-1.33	2.30	HL-93	1.77
1	15	53.95	31.4	-4.92	6.11	-0.96	3.52	HL-93	2.71
1	16	57.54	31.4	-2.63	3.26	-0.52	3.86	HL-93	5.60
1	17	61.14	31.4	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37

```
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA


[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]


[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
     SECTION IS NOT GOVERNING AT ALL.

     RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
     IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
        THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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 TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
                 ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                       GOVERNING
 ------------- ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    RATING     TYPE
        (ft)         (ft)
 ---------------------------------------------------------------------------

   1    61.14        32.37       STRENGTH L.S.  2.07      1.60     HL-93
   1    61.14        32.37       SERVICEBILITY  2.17      1.67     HL-93

 ---------------------------------------------------------------------------
     NOTE: Control Rating Factor for Current Bridge:
           STRENGTH L.S.
                Operating= 2.075
                Inventory= 1.600


 TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK     FA
                 ******************************************************

     SPAN       CRITICAL LOCATION GOVERNING
 ------------- ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    LL MOMENT
        (ft)         (ft)                                  (k-ft)
 ---------------------------------------------------------------
   1    61.14        35.96       STRENGTH L.S.  2.97        742.83
   1    61.14        50.35       SERVICEBILITY  2.84

 ---------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
                Legal Load Rating= 2.844
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 TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
                ********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT.
 SP IN D FROM   SHEAR         SHEAR       MAX.  SHEAR    RATING  LIVE  RATING
 NO NO L SUPT   CAP.                      --------------- FACTOR  LOAD  FACTOR
        (ft)  (kips)       D1; kips  D2     POS.     NEG.  [3]   TYPE
 ------------------------------------------------------------------------------

  1  0   0.00    347.9     21.8     13.9     56.5      0.0  3.95  HL-93  3.05
```

1	1	3.60	347.9	19.2	12.3	52.1	-1.4	4.36	HL-93	3.36
1	2	7.19	347.9	16.7	10.6	47.7	-3.3	4.84	HL-93	3.73
1	3	10.79	347.9	14.2	9.0	43.5	-5.7	5.41	HL-93	4.17
1	4	14.39	347.9	11.6	7.4	39.3	-8.2	6.09	HL-93	4.70
1	5	17.98	347.9	9.1	5.7	35.2	-10.7	6.92	HL-93	5.34
1	6	21.58	347.9	6.4	4.1	31.1	-13.3	7.95	HL-93	6.13
1	7	25.18	347.9	3.8	2.5	27.2	-16.1	9.26	HL-93	7.14
1	8	28.77	347.9	1.3	0.8	23.3	-19.6	9.99	HL-93	8.47
1	9	32.37	347.9	-1.2	-0.8	19.6	-23.3	9.99	HL-93	8.48
1	10	35.96	347.9	-3.8	-2.5	16.1	-27.2	9.26	HL-93	7.14
1	11	39.56	347.9	-6.5	-4.1	13.3	-31.1	7.94	HL-93	6.13
1	12	43.16	347.9	-9.0	-5.7	10.7	-35.2	6.92	HL-93	5.34
1	13	46.75	347.9	-11.6	-7.4	8.2	-39.3	6.09	HL-93	4.70
1	14	50.35	347.9	-14.1	-9.0	5.7	-43.5	5.41	HL-93	4.17
1	15	53.95	347.9	-16.6	-10.6	3.3	-47.7	4.84	HL-93	3.74
1	16	57.54	347.9	-19.3	-12.3	1.4	-52.1	4.36	HL-93	3.36
1	17	61.14	347.9	-21.9	-13.9	0.0	-56.5	3.95	HL-93	3.04

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	61.14	61.14	STRENGTH L.S.	3.95	3.04	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.946
Inventory= 3.044

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	61.14	61.14	STRENGTH L.S.	4.79	62.84

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 4.788

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

1	30.57	3	0.425	-0.41	HL-93	2.22
		4	0.433	-0.42		2.18

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95 NB Span 3 Interior Strength	8/19/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	18	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.15	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
48.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	WEB THICK.	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	TOP PLATE THICK.	BOT. PLATE WIDTH	BOT. PLATE THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.4375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.50				33.	33.	33.
2	2	2	25.00				33.	33.	33.
3	1	1	11.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.44	0.00	15.00	1 1 1 1 1	0.71	2 2 2 2 2
1	0.00	0	0.00	0.60	0.00	0.00	0 0 0 0 0	0.86	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	48.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	15.10
2	2	Diaphragm	0.000	0.00	0.00	0.29	35.85
3	1	Railing	0.087	0.00	48.00	0.00	0.00
4	1	Curb	0.076	0.00	48.00	0.00	0.00
5	0	Pavement	0.292	0.00	48.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	15.10	33.0	33.0	15.10
15.10	35.85	33.0	33.0	20.75
35.85	48.00	33.0	33.0	12.15

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

FACTORS USED BY LRFD

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.726       0.1258     0.8518


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      48.00

     1   DC2     0.087       0.000     48.000

     2   DC2     0.076       0.000     48.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1165.5	43.2	174.8	36.2	211.07

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.710	0.710	0.719	0.719	FOR STRENGTH POSITIVE MOMENT
	0.860				FOR STRENGTH POSITIVE SHEAR
1	0.443				FOR FATIGUE POSITIVE MOMENT
	0.601				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.67	5900.0	32.9	16.45	358.7	358.7
1	2	5.33	5900.0	32.9	16.45	358.7	358.7
1	3	8.00	5900.0	32.9	16.45	358.7	358.7
1	4	10.67	5900.0	32.9	16.45	358.7	358.7
1	5	13.33	6981.8	32.9	15.02	464.8	381.2
1	6	16.00	6981.8	32.9	15.02	464.8	381.2
1	7	18.67	6981.8	32.9	15.02	464.8	381.2
1	8	21.33	6981.8	32.9	15.02	464.8	381.2
1	9	24.00	6981.8	32.9	15.02	464.8	381.2
1	10	26.67	6981.8	32.9	15.02	464.8	381.2
1	11	29.33	6981.8	32.9	15.02	464.8	381.2
1	12	32.00	6981.8	32.9	15.02	464.8	381.2

1 13	34.67	6981.8	32.9	15.02	464.8	381.2
1 14	37.33	5900.0	32.9	16.45	358.7	358.7
1 15	40.00	5900.0	32.9	16.45	358.7	358.7
1 16	42.67	5900.0	32.9	16.45	358.7	358.7
1 17	45.33	5900.0	32.9	16.45	358.7	358.7
1 18	48.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	CONCRETE(SLAB) TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12412.8	0.024716	492.1	1617.3	783.9
1	1	2.67	12412.8	0.024716	492.1	1617.3	783.9
1	2	5.33	12412.8	0.024716	492.1	1617.3	783.9
1	3	8.00	12412.8	0.024716	492.1	1617.3	783.9
1	4	10.67	12412.8	0.024716	492.1	1617.3	783.9
1	5	13.33	15053.7	0.023292	629.3	1598.8	856.5
1	6	16.00	15053.7	0.023292	629.3	1598.8	856.5
1	7	18.67	15053.7	0.023292	629.3	1598.8	856.5
1	8	21.33	15053.7	0.023292	629.3	1598.8	856.5
1	9	24.00	15053.7	0.023292	629.3	1598.8	856.5
1	10	26.67	15053.7	0.023292	629.3	1598.8	856.5
1	11	29.33	15053.7	0.023292	629.3	1598.8	856.5
1	12	32.00	15053.7	0.023292	629.3	1598.8	856.5
1	13	34.67	15053.7	0.023292	629.3	1598.8	856.5
1	14	37.33	12412.8	0.024716	492.1	1617.3	783.9
1	15	40.00	12412.8	0.024716	492.1	1617.3	783.9
1	16	42.67	12412.8	0.024716	492.1	1617.3	783.9
1	17	45.33	12412.8	0.024716	492.1	1617.3	783.9
1	18	48.00	12412.8	0.024716	492.1	1617.3	783.9

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	CONCRETE(SLAB) TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	16700.9	0.029909	542.3	7943.7	1627.4
1	1	2.67	16700.9	0.029909	542.3	7943.7	1627.4
1	2	5.33	16700.9	0.029909	542.3	7943.7	1627.4
1	3	8.00	16700.9	0.029909	542.3	7943.7	1627.4
1	4	10.67	16700.9	0.029909	542.3	7943.7	1627.4
1	5	13.33	20750.0	0.028418	690.7	6295.0	1811.2
1	6	16.00	20750.0	0.028418	690.7	6295.0	1811.2
1	7	18.67	20750.0	0.028418	690.7	6295.0	1811.2

1	8	21.33	20750.0	0.028418	690.7	6295.0	1811.2
1	9	24.00	20750.0	0.028418	690.7	6295.0	1811.2
1	10	26.67	20750.0	0.028418	690.7	6295.0	1811.2
1	11	29.33	20750.0	0.028418	690.7	6295.0	1811.2
1	12	32.00	20750.0	0.028418	690.7	6295.0	1811.2
1	13	34.67	20750.0	0.028418	690.7	6295.0	1811.2
1	14	37.33	16700.9	0.029909	542.3	7943.7	1627.4
1	15	40.00	16700.9	0.029909	542.3	7943.7	1627.4
1	16	42.67	16700.9	0.029909	542.3	7943.7	1627.4
1	17	45.33	16700.9	0.029909	542.3	7943.7	1627.4
1	18	48.00	16700.9	0.029909	542.3	7943.7	1627.4

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	2.67	7.6	43.9	0.7	0.0	52.2	
1	2	5.33	14.3	82.6	1.4	0.0	98.4	
1	3	8.00	20.1	116.2	2.2	0.0	138.4	
1	4	10.67	25.1	144.6	2.9	0.0	172.5	
1	5	13.33	29.1	167.8	3.6	0.0	200.5	
1	6	16.00	32.2	185.9	4.1	0.0	222.1	
1	7	18.67	34.4	198.8	4.0	0.0	237.2	
1	8	21.33	35.8	206.5	4.0	0.0	246.3	
1	9	24.00	36.2	209.1	3.9	0.0	249.2	
1	10	26.67	35.8	206.5	3.9	0.0	246.2	
1	11	29.33	34.4	198.8	3.8	0.0	237.0	
1	12	32.00	32.2	185.9	3.8	0.0	221.8	

1 13	34.67	29.1	167.8	3.7	0.0	200.6
1 14	37.33	25.1	144.6	3.2	0.0	172.9
1 15	40.00	20.1	116.2	2.4	0.0	138.7
1 16	42.67	14.3	82.6	1.6	0.0	98.5
1 17	45.33	7.6	43.9	0.8	0.0	52.3
1 18	48.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.67	17.6	9.9	0.0	0.0	0.0	27.5
1	2	5.33	33.2	18.5	0.0	0.0	0.0	51.8
1	3	8.00	46.7	26.1	0.0	0.0	0.0	72.8
1	4	10.67	58.1	32.5	0.0	0.0	0.0	90.6
1	5	13.33	67.5	37.7	0.0	0.0	0.0	105.2
1	6	16.00	74.8	41.7	0.0	0.0	0.0	116.5
1	7	18.67	79.9	44.6	0.0	0.0	0.0	124.6
1	8	21.33	83.1	46.4	0.0	0.0	0.0	129.4
1	9	24.00	84.1	46.9	0.0	0.0	0.0	131.0
1	10	26.67	83.1	46.4	0.0	0.0	0.0	129.4
1	11	29.33	79.9	44.6	0.0	0.0	0.0	124.6
1	12	32.00	74.8	41.7	0.0	0.0	0.0	116.5
1	13	34.67	67.5	37.7	0.0	0.0	0.0	105.2
1	14	37.33	58.1	32.5	0.0	0.0	0.0	90.6
1	15	40.00	46.7	26.1	0.0	0.0	0.0	72.8
1	16	42.67	33.2	18.5	0.0	0.0	0.0	51.8
1	17	45.33	17.6	9.9	0.0	0.0	0.0	27.5
1	18	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.67	0.0	0.0	163.4	HL-93	0.0	HL-93
1	2	5.33	0.0	0.0	303.5	HL-93	0.0	HL-93
1	3	8.00	0.0	0.0	420.2	HL-93	0.0	HL-93
1	4	10.67	0.0	0.0	511.7	HL-93	0.0	HL-93
1	5	13.33	0.0	0.0	581.6	HL-93	0.0	HL-93
1	6	16.00	0.0	0.0	635.9	HL-93	0.0	HL-93
1	7	18.67	0.0	0.0	671.5	HL-93	0.0	HL-93
1	8	21.33	0.0	0.0	680.4	HL-93	0.0	HL-93
1	9	24.00	0.0	0.0	682.3	HL-93	0.0	HL-93
1	10	26.67	0.0	0.0	680.4	HL-93	0.0	HL-93
1	11	29.33	0.0	0.0	671.5	HL-93	0.0	HL-93
1	12	32.00	0.0	0.0	635.9	HL-93	0.0	HL-93

1 13	34.67	0.0	0.0	581.6	HL-93	0.0	HL-93
1 14	37.33	0.0	0.0	511.7	HL-93	0.0	HL-93
1 15	40.00	0.0	0.0	420.2	HL-93	0.0	HL-93
1 16	42.67	0.0	0.0	303.5	HL-93	0.0	HL-93
1 17	45.33	0.0	0.0	163.4	HL-93	0.0	HL-93
1 18	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.67	55.	0.	55.
1	2	5.33	100.	0.	100.
1	3	8.00	136.	0.	136.
1	4	10.67	160.	0.	160.
1	5	13.33	176.	0.	176.
1	6	16.00	195.	0.	195.
1	7	18.67	210.	0.	210.
1	8	21.33	215.	0.	215.
1	9	24.00	216.	0.	216.
1	10	26.67	215.	0.	215.
1	11	29.33	210.	0.	210.
1	12	32.00	195.	0.	195.
1	13	34.67	176.	0.	176.
1	14	37.33	160.	0.	160.
1	15	40.00	136.	0.	136.
1	16	42.67	100.	0.	100.
1	17	45.33	55.	0.	55.
1	18	48.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.67	52.2	27.5	163.4	HL-93	0.0	HL-93	243.2	79.7
1	2	5.33	98.4	51.8	303.5	HL-93	0.0	HL-93	453.6	150.1
1	3	8.00	138.4	72.8	420.2	HL-93	0.0	HL-93	631.5	211.2
1	4	10.67	172.5	90.6	511.7	HL-93	0.0	HL-93	774.8	263.1
1	5	13.33	200.5	105.2	581.6	HL-93	0.0	HL-93	887.2	305.6
1	6	16.00	222.1	116.5	635.9	HL-93	0.0	HL-93	974.5	338.6
1	7	18.67	237.2	124.6	671.5	HL-93	0.0	HL-93	1033.3	361.8
1	8	21.33	246.3	129.4	680.4	HL-93	0.0	HL-93	1056.1	375.7
1	9	24.00	249.2	131.0	682.3	HL-93	0.0	HL-93	1062.6	380.3
1	10	26.67	246.2	129.4	680.4	HL-93	0.0	HL-93	1056.0	375.6
1	11	29.33	237.0	124.6	671.5	HL-93	0.0	HL-93	1033.1	361.6
1	12	32.00	221.8	116.5	635.9	HL-93	0.0	HL-93	974.2	338.3
1	13	34.67	200.6	105.2	581.6	HL-93	0.0	HL-93	887.4	305.7
1	14	37.33	172.9	90.6	511.7	HL-93	0.0	HL-93	775.1	263.5

1	15	40.00	138.7	72.8	420.2	HL-93	0.0	HL-93	631.7	211.5
1	16	42.67	98.5	51.8	303.5	HL-93	0.0	HL-93	453.8	150.3
1	17	45.33	52.3	27.5	163.4	HL-93	0.0	HL-93	243.2	79.8
1	18	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.67	243.2	292.2	390.1	324.7	119.6	41.2
1	2	5.33	453.6	544.7	727.1	605.7	225.2	75.1
1	3	8.00	631.5	757.5	1011.1	843.0	316.9	101.8
1	4	10.67	774.8	928.3	1238.8	1034.2	394.6	119.6
1	5	13.33	887.2	1061.7	1416.7	1184.1	458.4	132.3
1	6	16.00	974.5	1165.3	1554.8	1300.4	507.9	146.6
1	7	18.67	1033.3	1234.7	1647.3	1378.7	542.7	157.4
1	8	21.33	1056.1	1260.2	1681.1	1408.9	563.5	161.0
1	9	24.00	1062.6	1267.3	1690.4	1417.5	570.4	162.0
1	10	26.67	1056.0	1260.1	1681.0	1408.8	563.4	161.0
1	11	29.33	1033.1	1234.5	1647.1	1378.5	542.4	157.4
1	12	32.00	974.2	1165.0	1554.4	1300.0	507.5	146.6
1	13	34.67	887.4	1061.9	1416.9	1184.3	458.6	132.3
1	14	37.33	775.1	928.7	1239.3	1034.6	395.2	119.6
1	15	40.00	631.7	757.8	1011.4	843.4	317.3	101.8
1	16	42.67	453.8	544.9	727.3	605.9	225.5	75.1
1	17	45.33	243.2	292.3	390.2	324.8	119.7	41.2
1	18	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.0	17.4	0.3	0.0	20.7
1	1	2.67	2.7	15.5	0.3	0.0	18.4
1	2	5.33	2.3	13.6	0.3	0.0	16.2
1	3	8.00	2.0	11.6	0.3	0.0	13.9
1	4	10.67	1.7	9.7	0.3	0.0	11.6
1	5	13.33	1.3	7.7	0.3	0.0	9.4
1	6	16.00	1.0	5.8	0.0	0.0	6.8
1	7	18.67	0.7	3.9	0.0	0.0	4.5
1	8	21.33	0.3	1.9	0.0	0.0	2.3
1	9	24.00	0.0	0.0	0.0	0.0	0.0
1	10	26.67	-0.3	-1.9	0.0	0.0	-2.3
1	11	29.33	-0.7	-3.9	0.0	0.0	-4.6
1	12	32.00	-1.0	-5.8	0.0	0.0	-6.8
1	13	34.67	-1.3	-7.7	0.0	0.0	-9.1
1	14	37.33	-1.7	-9.7	-0.3	0.0	-11.7

1	15	40.00	-2.0	-11.6	-0.3	0.0	-13.9
1	16	42.67	-2.3	-13.6	-0.3	0.0	-16.2
1	17	45.33	-2.7	-15.5	-0.3	0.0	-18.5
1	18	48.00	-3.0	-17.4	-0.3	0.0	-20.7

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.0	3.9	0.0	0.0	10.9	20.7	31.6
1	1	2.67	6.2	3.5	0.0	0.0	9.7	18.4	28.1
1	2	5.33	5.5	3.0	0.0	0.0	8.5	16.2	24.7
1	3	8.00	4.7	2.6	0.0	0.0	7.3	13.9	21.2
1	4	10.67	3.9	2.2	0.0	0.0	6.1	11.6	17.7
1	5	13.33	3.1	1.7	0.0	0.0	4.9	9.4	14.2
1	6	16.00	2.3	1.3	0.0	0.0	3.6	6.8	10.4
1	7	18.67	1.6	0.9	0.0	0.0	2.4	4.5	7.0
1	8	21.33	0.8	0.4	0.0	0.0	1.2	2.3	3.5
1	9	24.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	10	26.67	-0.8	-0.4	0.0	0.0	-1.2	-2.3	-3.5
1	11	29.33	-1.6	-0.9	0.0	0.0	-2.4	-4.6	-7.0
1	12	32.00	-2.3	-1.3	0.0	0.0	-3.6	-6.8	-10.5
1	13	34.67	-3.1	-1.7	0.0	0.0	-4.9	-9.1	-14.0
1	14	37.33	-3.9	-2.2	0.0	0.0	-6.1	-11.7	-17.7
1	15	40.00	-4.7	-2.6	0.0	0.0	-7.3	-13.9	-21.2
1	16	42.67	-5.5	-3.0	0.0	0.0	-8.5	-16.2	-24.7
1	17	45.33	-6.2	-3.5	0.0	0.0	-9.7	-18.5	-28.2
1	18	48.00	-7.0	-3.9	0.0	0.0	-10.9	-20.7	-31.7

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	79.5	HL-93	0.0	HL-93
1	1	2.67	0.0	0.0	73.5	HL-93	-2.1	HL-93
1	2	5.33	0.0	0.0	67.6	HL-93	-4.2	HL-93
1	3	8.00	0.0	0.0	61.8	HL-93	-7.5	HL-93
1	4	10.67	0.0	0.0	56.0	HL-93	-11.0	HL-93
1	5	13.33	0.0	0.0	50.3	HL-93	-14.5	HL-93
1	6	16.00	0.0	0.0	44.7	HL-93	-18.1	HL-93
1	7	18.67	0.0	0.0	39.2	HL-93	-21.8	HL-93
1	8	21.33	0.0	0.0	34.1	HL-93	-25.6	HL-93
1	9	24.00	0.0	0.0	29.5	HL-93	-29.5	HL-93
1	10	26.67	0.0	0.0	25.6	HL-93	-34.1	HL-93
1	11	29.33	0.0	0.0	21.8	HL-93	-39.2	HL-93
1	12	32.00	0.0	0.0	18.1	HL-93	-44.7	HL-93
1	13	34.67	0.0	0.0	14.5	HL-93	-50.3	HL-93

1	14	37.33	0.0	0.0	11.0	HL-93	-56.0	HL-93
1	15	40.00	0.0	0.0	7.5	HL-93	-61.8	HL-93
1	16	42.67	0.0	0.0	4.2	HL-93	-67.6	HL-93
1	17	45.33	0.0	0.0	2.1	HL-93	-73.5	HL-93
1	18	48.00	0.0	0.0	0.0	HL-93	-79.5	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	31.	0.	31.
1	1	2.67	28.	-1.	29.
1	2	5.33	25.	-2.	28.
1	3	8.00	23.	-4.	27.
1	4	10.67	20.	-5.	25.
1	5	13.33	18.	-6.	24.
1	6	16.00	16.	-8.	24.
1	7	18.67	15.	-9.	24.
1	8	21.33	14.	-11.	24.
1	9	24.00	12.	-12.	24.
1	10	26.67	11.	-14.	24.
1	11	29.33	9.	-15.	24.
1	12	32.00	8.	-16.	24.
1	13	34.67	6.	-18.	24.
1	14	37.33	5.	-20.	25.
1	15	40.00	4.	-23.	27.
1	16	42.67	2.	-25.	28.
1	17	45.33	1.	-28.	29.
1	18	48.00	0.	-31.	31.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	30.9	.31121E-01	0.48	0.2500*	.14421E-01	0.22	0.2500*
1	1	2.67	29.3	.31121E-01	0.46	0.2500*	.14421E-01	0.21	0.2500*
1	2	5.33	27.9	.31121E-01	0.43	0.2500*	.14421E-01	0.20	0.2500*
1	3	8.00	26.7	.31121E-01	0.42	0.2500*	.14421E-01	0.19	0.2500*
1	4	10.67	25.3	.31121E-01	0.39	0.2500*	.14421E-01	0.18	0.2500*
1	5	13.33	24.0	.29870E-01	0.36	0.2500*	.11130E-01	0.13	0.2500*
1	6	16.00	24.1	.29870E-01	0.36	0.2500*	.11130E-01	0.13	0.2500*
1	7	18.67	24.4	.29870E-01	0.36	0.2500*	.11130E-01	0.14	0.2500*
1	8	21.33	24.4	.29870E-01	0.36	0.2500*	.11130E-01	0.14	0.2500*
1	9	24.00	24.4	.29870E-01	0.36	0.2500*	.11130E-01	0.14	0.2500*
1	10	26.67	24.4	.29870E-01	0.36	0.2500*	.11130E-01	0.14	0.2500*
1	11	29.33	24.4	.29870E-01	0.36	0.2500*	.11130E-01	0.14	0.2500*
1	12	32.00	24.1	.29870E-01	0.36	0.2500*	.11130E-01	0.13	0.2500*
1	13	34.67	24.0	.29870E-01	0.36	0.2500*	.11130E-01	0.13	0.2500*
1	14	37.33	25.3	.31121E-01	0.39	0.2500*	.14421E-01	0.18	0.2500*
1	15	40.00	26.7	.31121E-01	0.42	0.2500*	.14421E-01	0.19	0.2500*
1	16	42.67	27.9	.31121E-01	0.43	0.2500*	.14421E-01	0.20	0.2500*
1	17	45.33	29.3	.31121E-01	0.46	0.2500*	.14421E-01	0.21	0.2500*
1	18	48.00	30.9	.31121E-01	0.48	0.2500*	.14421E-01	0.22	0.2500*

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-----------------------------------------------------------------
  NOTE:  [1] Vr = range of shear due to live loads and impact

         [2] For non-composite construction:

             Q/I = (At * Dt) / Inc -- top flange
             Inc = moment of inertia of non-composite section
             At = area of top flange
             Dt = distance between the center of top flange and neutral axis
             Q/I = (Ab * Db) / Inc -- bottom flange
             Ab = area of bottom flange
             Db = distance between the center of bottom flange and neutral axis

             For composite construction:

             Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
             Q = statical moment about the neutral axis of the
                 composite section of the transformed compressive
                 concrete area or the area of reinforcement embedded
                 in the concrete for negative moment
             Ic = moment of inertia for composite section
             Q/I = (Ab * Db) / Ic -- bottom flange

         [3] shear flow = (Vr * Q) / (2 * I)

         * -- minimum weld size governs
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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN D L SUPT	FROM (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	31.6	79.5	HL-93	0.0	HL-93	111.2	31.6
1	1	2.67	28.1	73.5	HL-93	-2.1	HL-93	101.7	26.1
1	2	5.33	24.7	67.6	HL-93	-4.2	HL-93	92.3	20.5
1	3	8.00	21.2	61.8	HL-93	-7.5	HL-93	82.9	13.7
1	4	10.67	17.7	56.0	HL-93	-11.0	HL-93	73.7	6.7
1	5	13.33	14.2	50.3	HL-93	-14.5	HL-93	64.5	-0.3
1	6	16.00	10.4	44.7	HL-93	-18.1	HL-93	55.1	-7.7
1	7	18.67	7.0	39.2	HL-93	-21.8	HL-93	46.1	-14.9
1	8	21.33	3.5	34.1	HL-93	-25.6	HL-93	37.6	-22.2
1	9	24.00	0.0	29.5	HL-93	-29.5	HL-93	29.5	-29.5
1	10	26.67	-3.5	25.6	HL-93	-34.1	HL-93	22.1	-37.6
1	11	29.33	-7.0	21.8	HL-93	-39.2	HL-93	14.9	-46.2
1	12	32.00	-10.5	18.1	HL-93	-44.7	HL-93	7.7	-55.1
1	13	34.67	-14.0	14.5	HL-93	-50.3	HL-93	0.6	-64.3
1	14	37.33	-17.7	11.0	HL-93	-56.0	HL-93	-6.8	-73.7
1	15	40.00	-21.2	7.5	HL-93	-61.8	HL-93	-13.7	-83.0
1	16	42.67	-24.7	4.2	HL-93	-67.6	HL-93	-20.5	-92.3
1	17	45.33	-28.2	2.1	HL-93	-73.5	HL-93	-26.1	-101.7
1	18	48.00	-31.7	0.0	HL-93	-79.5	HL-93	-31.7	-111.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	111.2	135.0	180.5	148.7	47.4	23.1
1	1	2.67	101.7	123.7	165.4	136.0	42.2	22.0
1	2	5.33	92.3	112.6	150.5	123.5	37.0	21.0
1	3	8.00	82.9	101.5	135.7	111.0	31.8	20.0
1	4	10.67	73.7	90.5	121.1	98.7	26.5	18.9
1	5	13.33	64.5	79.6	106.6	86.5	21.3	18.0
1	6	16.00	55.1	68.5	91.8	73.9	15.7	18.1
1	7	18.67	46.1	57.9	77.7	62.0	10.4	18.3
1	8	21.33	37.6	47.8	64.2	50.6	5.2	18.3
1	9	24.00	-29.5	-38.4	-51.7	-39.9	0.0	18.3
1	10	26.67	-37.6	-47.9	-64.3	-50.6	-5.3	18.3
1	11	29.33	-46.2	-57.9	-77.7	-62.0	-10.5	18.3
1	12	32.00	-55.1	-68.5	-91.8	-74.0	-15.7	18.1
1	13	34.67	-64.3	-79.4	-106.3	-86.2	-20.9	18.0
1	14	37.33	-73.7	-90.5	-121.2	-98.8	-26.6	18.9
1	15	40.00	-83.0	-101.5	-135.8	-111.1	-31.8	20.0
1	16	42.67	-92.3	-112.6	-150.6	-123.5	-37.0	21.0
1	17	45.33	-101.7	-123.8	-165.5	-136.1	-42.3	22.0
1	18	48.00	-111.2	-135.1	-180.5	-148.7	-47.5	23.1

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	31.63	0.00	HL-93	79.53	HL-93	ST1	26.72	180.47
						ST2	26.72	148.66
						ST4	36.94	47.45
						SE1	31.63	111.16
						SE2	31.63	135.02
2	31.67	0.00	HL-93	79.53	HL-93	ST1	26.75	180.52
						ST2	26.75	148.71
						ST4	36.99	47.50
						SE1	31.67	111.20
						SE2	31.67	135.06

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.67	-0.0138	-0.0796	0.0000	-0.0233	-0.0950	-0.0233	-0.1183

1	2	5.33	-0.0271	-0.1562	0.0000	-0.0457	-0.1863	-0.0457	-0.2320
1	3	8.00	-0.0393	-0.2268	0.0000	-0.0664	-0.2706	-0.0664	-0.3370
1	4	10.67	-0.0501	-0.2891	0.0000	-0.0845	-0.3449	-0.0845	-0.4295
1	5	13.33	-0.0592	-0.3414	0.0000	-0.0998	-0.4075	-0.0998	-0.5072
1	6	16.00	-0.0665	-0.3836	0.0000	-0.1120	-0.4577	-0.1120	-0.5698
1	7	18.67	-0.0718	-0.4144	0.0000	-0.1210	-0.4946	-0.1210	-0.6156
1	8	21.33	-0.0751	-0.4333	0.0000	-0.1265	-0.5170	-0.1265	-0.6435
1	9	24.00	-0.0762	-0.4396	0.0000	-0.1283	-0.5246	-0.1283	-0.6529
1	10	26.67	-0.0751	-0.4333	0.0000	-0.1265	-0.5170	-0.1265	-0.6435
1	11	29.33	-0.0718	-0.4144	0.0000	-0.1210	-0.4946	-0.1210	-0.6156
1	12	32.00	-0.0665	-0.3836	0.0000	-0.1120	-0.4578	-0.1120	-0.5698
1	13	34.67	-0.0592	-0.3414	0.0000	-0.0998	-0.4075	-0.0998	-0.5073
1	14	37.33	-0.0501	-0.2891	0.0000	-0.0845	-0.3450	-0.0845	-0.4296
1	15	40.00	-0.0393	-0.2268	0.0000	-0.0664	-0.2707	-0.0664	-0.3371
1	16	42.67	-0.0271	-0.1562	0.0000	-0.0457	-0.1864	-0.0457	-0.2321
1	17	45.33	-0.0138	-0.0796	0.0000	-0.0233	-0.0951	-0.0233	-0.1184
1	18	48.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.7	-0.014	0 1/16	-0.081	0 1/ 8	-0.023	0 1/16	-0.118	0 1/ 8
1	2	5.3	-0.027	0 1/16	-0.159	0 3/16	-0.046	0 1/16	-0.232	0 1/ 4
1	3	8.0	-0.039	0 1/16	-0.231	0 1/ 4	-0.066	0 1/ 8	-0.337	0 3/ 8
1	4	10.7	-0.050	0 1/16	-0.295	0 5/16	-0.085	0 1/ 8	-0.429	0 7/16
1	5	13.3	-0.059	0 1/16	-0.348	0 3/ 8	-0.100	0 1/ 8	-0.507	0 9/16
1	6	16.0	-0.066	0 1/ 8	-0.391	0 7/16	-0.112	0 1/ 8	-0.570	0 5/ 8
1	7	18.7	-0.072	0 1/ 8	-0.423	0 7/16	-0.121	0 1/ 8	-0.616	0 5/ 8
1	8	21.3	-0.075	0 1/ 8	-0.442	0 1/ 2	-0.126	0 3/16	-0.643	0 11/16
1	9	24.0	-0.076	0 1/ 8	-0.448	0 1/ 2	-0.128	0 3/16	-0.653	0 11/16
1	10	26.7	-0.075	0 1/ 8	-0.442	0 1/ 2	-0.126	0 3/16	-0.643	0 11/16
1	11	29.3	-0.072	0 1/ 8	-0.423	0 7/16	-0.121	0 1/ 8	-0.616	0 5/ 8
1	12	32.0	-0.066	0 1/ 8	-0.391	0 7/16	-0.112	0 1/ 8	-0.570	0 5/ 8
1	13	34.7	-0.059	0 1/16	-0.348	0 3/ 8	-0.100	0 1/ 8	-0.507	0 9/16
1	14	37.3	-0.050	0 1/16	-0.295	0 5/16	-0.085	0 1/ 8	-0.430	0 7/16
1	15	40.0	-0.039	0 1/16	-0.231	0 1/ 4	-0.066	0 1/ 8	-0.337	0 3/ 8
1	16	42.7	-0.027	0 1/16	-0.159	0 3/16	-0.046	0 1/16	-0.232	0 1/ 4
1	17	45.3	-0.014	0 1/16	-0.081	0 1/ 8	-0.023	0 1/16	-0.118	0 1/ 8
1	18	48.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

NUMBER OF LANE LL + I.

SPAN NO.	D FROM L SUPT (ft)	AND DIST. FOR LL DEFL.	FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	24.00	3	0.425	-0.226	MAX HL-93	0.72	0.00227
				0.025	MIN		
				-0.056	MAX LANE		
				0.025	MIN LANE		
		4	0.433	-0.231	MAX HL-93	0.72	0.00227
				0.025	MIN		
				-0.057	MAX LANE		
				0.025	MIN LANE		
		3	0.425	-0.141	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	-0.25	0.25	-1.49	1.49	-1.75	1.75
1	2	5.33	-0.48	0.48	-2.81	2.81	-3.29	3.29
1	3	8.00	-0.67	0.67	-3.96	3.96	-4.63	4.63
1	4	10.67	-0.84	0.84	-4.93	4.93	-5.77	5.77
1	5	13.33	-0.92	0.75	-5.40	4.42	-6.31	5.18
1	6	16.00	-1.01	0.83	-5.98	4.90	-6.99	5.73
1	7	18.67	-1.08	0.89	-6.38	5.24	-7.47	6.12
1	8	21.33	-1.13	0.92	-6.63	5.43	-7.75	6.36
1	9	24.00	-1.14	0.94	-6.71	5.50	-7.85	6.43
1	10	26.67	-1.13	0.92	-6.62	5.43	-7.75	6.36
1	11	29.33	-1.08	0.89	-6.38	5.23	-7.46	6.12
1	12	32.00	-1.01	0.83	-5.97	4.90	-6.98	5.73
1	13	34.67	-0.92	0.75	-5.40	4.43	-6.31	5.18
1	14	37.33	-0.84	0.84	-4.95	4.95	-5.78	5.78
1	15	40.00	-0.67	0.67	-3.97	3.97	-4.64	4.64

1	16	42.67	-0.48	0.48	-2.82	2.82	-3.30	3.30
1	17	45.33	-0.25	0.25	-1.50	1.50	-1.75	1.75
1	18	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	-0.02	-0.20	0.67	-0.02	-1.95	2.42
1	2	5.33	-0.03	-0.38	1.26	-0.03	-3.67	4.55
1	3	8.00	-0.04	-0.54	1.78	-0.04	-5.17	6.41
1	4	10.67	-0.05	-0.67	2.21	-0.05	-6.44	7.98
1	5	13.33	-0.05	-0.79	2.01	-0.05	-7.10	7.18
1	6	16.00	-0.06	-0.87	2.22	-0.06	-7.87	7.96
1	7	18.67	-0.06	-0.93	2.38	-0.06	-8.40	8.50
1	8	21.33	-0.07	-0.97	2.47	-0.07	-8.72	8.83
1	9	24.00	-0.07	-0.98	2.50	-0.07	-8.83	8.93
1	10	26.67	-0.07	-0.97	2.47	-0.07	-8.72	8.82
1	11	29.33	-0.06	-0.93	2.38	-0.06	-8.40	8.49
1	12	32.00	-0.06	-0.87	2.22	-0.06	-7.86	7.95
1	13	34.67	-0.05	-0.79	2.01	-0.05	-7.10	7.18
1	14	37.33	-0.05	-0.67	2.21	-0.05	-6.46	7.99
1	15	40.00	-0.04	-0.54	1.78	-0.04	-5.18	6.42
1	16	42.67	-0.03	-0.38	1.26	-0.03	-3.68	4.56
1	17	45.33	-0.02	-0.20	0.67	-0.02	-1.95	2.42
1	18	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	-0.13	-0.25	3.62	0.00	0.00	0.00
1	2	5.33	-0.25	-0.46	6.72	0.00	0.00	0.00
1	3	8.00	-0.34	-0.63	9.30	0.00	0.00	0.00
1	4	10.67	-0.42	-0.77	11.32	0.00	0.00	0.00
1	5	13.33	-0.43	-1.11	10.10	0.00	0.00	0.00
1	6	16.00	-0.47	-1.21	11.05	0.00	0.00	0.00
1	7	18.67	-0.49	-1.28	11.67	0.00	0.00	0.00
1	8	21.33	-0.50	-1.30	11.82	0.00	0.00	0.00
1	9	24.00	-0.50	-1.30	11.85	0.00	0.00	0.00
1	10	26.67	-0.50	-1.30	11.82	0.00	0.00	0.00
1	11	29.33	-0.49	-1.28	11.67	0.00	0.00	0.00
1	12	32.00	-0.47	-1.21	11.05	0.00	0.00	0.00
1	13	34.67	-0.43	-1.11	10.10	0.00	0.00	0.00

1 14	37.33	-0.42	-0.77	11.32	0.00	0.00	0.00
1 15	40.00	-0.34	-0.63	9.30	0.00	0.00	0.00
1 16	42.67	-0.25	-0.46	6.72	0.00	0.00	0.00
1 17	45.33	-0.13	-0.25	3.62	0.00	0.00	0.00
1 18	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	2.67	0.1	0.0	0.1	C	HL	1.2	0.0	1.2	T	HL
1	2	5.33	0.2	0.0	0.2	C	HL	2.2	0.0	2.2	T	HL
1	3	8.00	0.2	0.0	0.2	C	HL	3.0	0.0	3.0	T	HL
1	4	10.67	0.2	0.0	0.2	C	HL	3.5	0.0	3.5	T	HL
1	5	13.33	0.3	0.0	0.3	C	HL	3.1	0.0	3.1	T	HL
1	6	16.00	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	7	18.67	0.4	0.0	0.4	C	HL	3.6	0.0	3.6	T	HL
1	8	21.33	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	9	24.00	0.4	0.0	0.4	C	HL	3.8	0.0	3.8	T	HL
1	10	26.67	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	11	29.33	0.4	0.0	0.4	C	HL	3.6	0.0	3.6	T	HL
1	12	32.00	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	13	34.67	0.3	0.0	0.3	C	HL	3.1	0.0	3.1	T	HL
1	14	37.33	0.2	0.0	0.2	C	HL	3.5	0.0	3.5	T	HL
1	15	40.00	0.2	0.0	0.2	C	HL	3.0	0.0	3.0	T	HL
1	16	42.67	0.2	0.0	0.2	C	HL	2.2	0.0	2.2	T	HL
1	17	45.33	0.1	0.0	0.1	C	HL	1.2	0.0	1.2	T	HL
1	18	48.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.15	-2.20	6.03	-0.02	-1.95	2.42
1	2	5.33	33.	33.	-0.28	-4.13	11.27	-0.03	-3.67	4.55
1	3	8.00	33.	33.	-0.39	-5.81	15.71	-0.04	-5.17	6.41
1	4	10.67	33.	33.	-0.47	-7.22	19.30	-0.05	-6.44	7.98
1	5	13.33	33.	33.	-0.48	-8.21	17.29	-0.05	-7.10	7.18
1	6	16.00	33.	33.	-0.53	-9.08	19.00	-0.06	-7.87	7.96
1	7	18.67	33.	33.	-0.56	-9.68	20.17	-0.06	-8.40	8.50
1	8	21.33	33.	33.	-0.57	-10.02	20.65	-0.07	-8.72	8.83
1	9	24.00	33.	33.	-0.57	-10.13	20.79	-0.07	-8.83	8.93
1	10	26.67	33.	33.	-0.57	-10.02	20.64	-0.07	-8.72	8.82

1 11	29.33	33.	33.	-0.56	-9.68	20.16	-0.06	-8.40	8.49
1 12	32.00	33.	33.	-0.53	-9.07	19.00	-0.06	-7.86	7.95
1 13	34.67	33.	33.	-0.48	-8.21	17.29	-0.05	-7.10	7.18
1 14	37.33	33.	33.	-0.47	-7.23	19.32	-0.05	-6.46	7.99
1 15	40.00	33.	33.	-0.39	-5.82	15.72	-0.04	-5.18	6.42
1 16	42.67	33.	33.	-0.28	-4.14	11.28	-0.03	-3.68	4.56
1 17	45.33	33.	33.	-0.15	-2.20	6.04	-0.02	-1.95	2.42
1 18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.19	-2.27	7.12	-0.02	-1.95	2.42
1	2	5.33	33.	33.	-0.35	-4.27	13.28	-0.03	-3.67	4.55
1	3	8.00	33.	33.	-0.49	-6.00	18.50	-0.04	-5.17	6.41
1	4	10.67	33.	33.	-0.60	-7.45	22.70	-0.05	-6.44	7.98
1	5	13.33	33.	33.	-0.61	-8.54	20.32	-0.05	-7.10	7.18
1	6	16.00	33.	33.	-0.67	-9.44	22.32	-0.06	-7.87	7.96
1	7	18.67	33.	33.	-0.71	-10.07	23.67	-0.06	-8.40	8.50
1	8	21.33	33.	33.	-0.72	-10.41	24.19	-0.07	-8.72	8.83
1	9	24.00	33.	33.	-0.72	-10.52	24.34	-0.07	-8.83	8.93
1	10	26.67	33.	33.	-0.72	-10.41	24.19	-0.07	-8.72	8.82
1	11	29.33	33.	33.	-0.71	-10.06	23.66	-0.06	-8.40	8.49
1	12	32.00	33.	33.	-0.67	-9.43	22.31	-0.06	-7.86	7.95
1	13	34.67	33.	33.	-0.61	-8.55	20.32	-0.05	-7.10	7.18
1	14	37.33	33.	33.	-0.60	-7.46	22.71	-0.05	-6.46	7.99
1	15	40.00	33.	33.	-0.49	-6.01	18.51	-0.04	-5.18	6.42
1	16	42.67	33.	33.	-0.35	-4.28	13.29	-0.03	-3.68	4.56
1	17	45.33	33.	33.	-0.19	-2.27	7.12	-0.02	-1.95	2.42
1	18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.26	-2.90	9.46	-0.02	-2.47	3.13
1	2	5.33	33.	33.	-0.48	-5.46	17.65	-0.04	-4.66	5.89
1	3	8.00	33.	33.	-0.66	-7.66	24.57	-0.06	-6.55	8.29
1	4	10.67	33.	33.	-0.81	-9.51	30.15	-0.07	-8.16	10.33
1	5	13.33	33.	33.	-0.83	-10.94	26.98	-0.08	-9.00	9.30
1	6	16.00	33.	33.	-0.90	-12.10	29.63	-0.09	-9.97	10.30
1	7	18.67	33.	33.	-0.96	-12.89	31.42	-0.09	-10.65	11.01
1	8	21.33	33.	33.	-0.97	-13.33	32.12	-0.09	-11.06	11.43
1	9	24.00	33.	33.	-0.97	-13.47	32.31	-0.10	-11.20	11.57
1	10	26.67	33.	33.	-0.97	-13.33	32.11	-0.09	-11.06	11.43
1	11	29.33	33.	33.	-0.96	-12.89	31.42	-0.09	-10.65	11.00
1	12	32.00	33.	33.	-0.90	-12.08	29.63	-0.09	-9.96	10.29

1 13	34.67	33.	33.	-0.83	-10.95	26.99	-0.08	-9.01	9.30
1 14	37.33	33.	33.	-0.81	-9.53	30.16	-0.07	-8.18	10.35
1 15	40.00	33.	33.	-0.66	-7.67	24.58	-0.06	-6.56	8.31
1 16	42.67	33.	33.	-0.48	-5.47	17.66	-0.04	-4.66	5.90
1 17	45.33	33.	33.	-0.26	-2.91	9.46	-0.02	-2.48	3.13
1 18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.25	-2.16	8.40	-0.01	-1.72	2.07
1	2	5.33	33.	33.	-0.46	-4.05	15.65	-0.02	-3.25	3.90
1	3	8.00	33.	33.	-0.63	-5.68	21.75	-0.03	-4.57	5.48
1	4	10.67	33.	33.	-0.77	-7.04	26.64	-0.04	-5.69	6.83
1	5	13.33	33.	33.	-0.79	-8.20	23.82	-0.04	-6.26	6.14
1	6	16.00	33.	33.	-0.86	-9.06	26.14	-0.04	-6.94	6.80
1	7	18.67	33.	33.	-0.91	-9.65	27.68	-0.05	-7.41	7.27
1	8	21.33	33.	33.	-0.93	-9.97	28.23	-0.05	-7.70	7.55
1	9	24.00	33.	33.	-0.93	-10.07	28.38	-0.05	-7.79	7.64
1	10	26.67	33.	33.	-0.93	-9.96	28.23	-0.05	-7.69	7.55
1	11	29.33	33.	33.	-0.91	-9.65	27.68	-0.05	-7.41	7.26
1	12	32.00	33.	33.	-0.86	-9.05	26.13	-0.04	-6.93	6.80
1	13	34.67	33.	33.	-0.79	-8.21	23.83	-0.04	-6.27	6.14
1	14	37.33	33.	33.	-0.77	-7.05	26.65	-0.04	-5.70	6.84
1	15	40.00	33.	33.	-0.63	-5.69	21.76	-0.03	-4.58	5.49
1	16	42.67	33.	33.	-0.46	-4.05	15.65	-0.02	-3.25	3.90
1	17	45.33	33.	33.	-0.25	-2.16	8.40	-0.01	-1.73	2.07
1	18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.20	-2.80	8.01	-0.02	-2.47	3.13
1	2	5.33	33.	33.	-0.38	-5.27	14.96	-0.04	-4.66	5.89
1	3	8.00	33.	33.	-0.52	-7.41	20.85	-0.06	-6.55	8.29
1	4	10.67	33.	33.	-0.64	-9.21	25.62	-0.07	-8.16	10.33
1	5	13.33	33.	33.	-0.65	-10.50	22.94	-0.08	-9.00	9.30
1	6	16.00	33.	33.	-0.72	-11.61	25.22	-0.09	-9.97	10.30
1	7	18.67	33.	33.	-0.76	-12.38	26.75	-0.09	-10.65	11.01
1	8	21.33	33.	33.	-0.77	-12.81	27.39	-0.09	-11.06	11.43
1	9	24.00	33.	33.	-0.77	-12.95	27.57	-0.10	-11.20	11.57
1	10	26.67	33.	33.	-0.77	-12.81	27.38	-0.09	-11.06	11.43
1	11	29.33	33.	33.	-0.76	-12.37	26.75	-0.09	-10.65	11.00
1	12	32.00	33.	33.	-0.72	-11.60	25.21	-0.09	-9.96	10.29
1	13	34.67	33.	33.	-0.65	-10.50	22.94	-0.08	-9.01	9.30
1	14	37.33	33.	33.	-0.64	-9.22	25.63	-0.07	-8.18	10.35
1	15	40.00	33.	33.	-0.52	-7.42	20.86	-0.06	-6.56	8.31

1 16	42.67	33.	33.	-0.38	-5.28	14.97	-0.04	-4.66	5.90
1 17	45.33	33.	33.	-0.20	-2.81	8.02	-0.02	-2.48	3.13
1 18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.67	33.	33.	-0.19	-2.06	6.95	-0.01	-1.72	2.07
1	2	5.33	33.	33.	-0.36	-3.86	12.96	-0.02	-3.25	3.90
1	3	8.00	33.	33.	-0.50	-5.43	18.03	-0.03	-4.57	5.48
1	4	10.67	33.	33.	-0.60	-6.73	22.11	-0.04	-5.69	6.83
1	5	13.33	33.	33.	-0.62	-7.76	19.78	-0.04	-6.26	6.14
1	6	16.00	33.	33.	-0.68	-8.58	21.72	-0.04	-6.94	6.80
1	7	18.67	33.	33.	-0.72	-9.14	23.02	-0.05	-7.41	7.27
1	8	21.33	33.	33.	-0.73	-9.45	23.51	-0.05	-7.70	7.55
1	9	24.00	33.	33.	-0.73	-9.55	23.64	-0.05	-7.79	7.64
1	10	26.67	33.	33.	-0.73	-9.44	23.50	-0.05	-7.69	7.55
1	11	29.33	33.	33.	-0.72	-9.14	23.01	-0.05	-7.41	7.26
1	12	32.00	33.	33.	-0.68	-8.57	21.71	-0.04	-6.93	6.80
1	13	34.67	33.	33.	-0.62	-7.76	19.79	-0.04	-6.27	6.14
1	14	37.33	33.	33.	-0.60	-6.75	22.13	-0.04	-5.70	6.84
1	15	40.00	33.	33.	-0.50	-5.43	18.04	-0.03	-4.58	5.49
1	16	42.67	33.	33.	-0.36	-3.87	12.97	-0.02	-3.25	3.90
1	17	45.33	33.	33.	-0.19	-2.06	6.95	-0.01	-1.73	2.07
1	18	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.50	181.	-6.1	95.	-0.7	534.	-0.8	0.	0.0	-7.58
											0.81
	LB			6.1		2.3		11.8		0.0	20.19
											11.81
	RT			-5.7		-0.7		-1.0		0.0	-7.44
											1.02
	RB			4.7		1.8		9.3		0.0	15.76
											9.27
1	LT	36.50	182.	-5.7	95.	-0.7	534.	-1.0	0.	0.0	-7.45
											1.02
	LB			4.7		1.8		9.3		0.0	15.77
											9.27
	RT			-6.1		-0.7		-0.8		0.0	-7.59
											0.81
	RB			6.1		2.3		11.8		0.0	20.20
											11.81

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange

RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK
1	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	25.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.4375
3	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 48.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1]) [3]/(12*[1])	<=>	LIMITATION 1/25 1/30	STATUS
1	11.50	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK
2	25.00	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK
3	11.50	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	1	2.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	2	5.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	3	8.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	4	10.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	5	13.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	6	16.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	7	18.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	8	21.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	9	24.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	10	26.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	11	29.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	12	32.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	13	34.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	14	37.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	15	40.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	16	42.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	17	45.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	18	48.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.67	0	1	31.42	0.550	57.13	168.97	2
1	2	5.33	0	1	31.42	0.550	57.13	168.97	2
1	3	8.00	0	1	31.42	0.550	57.13	168.97	2
1	4	10.67	0	1	31.42	0.550	57.13	168.97	2
1	5	13.33	0	1	31.42	0.550	63.98	168.97	2
1	6	16.00	0	1	31.42	0.550	63.98	168.97	2
1	7	18.67	0	1	31.42	0.550	63.98	168.97	2
1	8	21.33	0	1	31.42	0.550	63.98	168.97	2
1	9	24.00	0	1	31.42	0.550	63.98	168.97	2
1	10	26.67	0	1	31.42	0.550	63.98	168.97	2
1	11	29.33	0	1	31.42	0.550	63.98	168.97	2
1	12	32.00	0	1	31.42	0.550	63.98	168.97	2

1	13	34.67	0	1	31.42	0.550	63.98	168.97	2
1	14	37.33	0	1	31.42	0.550	57.13	168.97	2
1	15	40.00	0	1	31.42	0.550	57.13	168.97	2
1	16	42.67	0	1	31.42	0.550	57.13	168.97	2
1	17	45.33	0	1	31.42	0.550	57.13	168.97	2
1	18	48.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	1	2.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	2	5.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	3	8.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	4	10.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	5	13.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	6	16.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	7	18.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	8	21.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	9	24.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	10	26.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	11	29.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	12	32.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	13	34.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	14	37.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	15	40.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	16	42.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	17	45.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	18	48.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)

[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT FROM ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	15.1	0.000	10.144	6.609	3.074	1.000
			0.000	0.000	0.000	0.000	1.000
2	15.1	35.8	9.119	10.144	11.689	13.235	1.000
			0.000	0.000	0.000	0.000	1.000
3	35.8	48.0	0.000	9.119	5.506	1.894	1.545
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	1	2.67	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	2	5.33	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	3	8.00	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	4	10.67	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	5	13.33	1	7.77	11.26	19.84	0	15.10	6.98	26.22	2	2
1	6	16.00	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	7	18.67	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	8	21.33	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	9	24.00	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	10	26.67	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	11	29.33	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	12	32.00	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	13	34.67	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	14	37.33	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	15	40.00	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	16	42.67	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	17	45.33	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	18	48.00	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)

```
       Lb = unbraced length
       Lp = limiting unbraced length to achieve the nominal flexural
            of RbRhFyc under uniform bending
          = 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
       Lr = limiting unbraced length to achieve the onset of
            nominal yielding in either flange under uniform
            bending with consideration of comp. flange
            residual stress effects
          = pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

       In negative moment region, the fist line is for non-composite sections
       and the second line is for composite sections

       Flange Category
        0 = compact section
        2 = non-compact section
        3 = slender section

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 TABLE  1.2.22.7B=FLB AND LTB RESISTANCE
                 **********************

 SP IN D FROM  Co                        FLB       LTB       GOV
 NO NO L SUPT  mp  Rh    Rb    Cb        Fnc       Fnc       Fnc
         (ft)                           (Ksi)     (Ksi)     (Ksi)
-----------------------------------------------------------------

  1  0   0.00  1 1.000 1.000  1.00      33.00     28.94     28.94
  1  1   2.67  1 1.000 1.000  1.00      33.00     28.94     28.94
  1  2   5.33  1 1.000 1.000  1.00      33.00     28.94     28.94
  1  3   8.00  1 1.000 1.000  1.00      33.00     28.94     28.94
  1  4  10.67  1 1.000 1.000  1.00      33.00     28.94     28.94
  1  5  13.33  1 1.000 1.000  1.00      33.00     28.82     28.82
  1  6  16.00  1 1.000 1.000  1.00      33.00     25.92     25.92
  1  7  18.67  1 1.000 1.000  1.00      33.00     25.92     25.92
  1  8  21.33  1 1.000 1.000  1.00      33.00     25.92     25.92
  1  9  24.00  1 1.000 1.000  1.00      33.00     25.92     25.92
  1 10  26.67  1 1.000 1.000  1.00      33.00     25.92     25.92
  1 11  29.33  1 1.000 1.000  1.00      33.00     25.92     25.92
  1 12  32.00  1 1.000 1.000  1.00      33.00     25.92     25.92
  1 13  34.67  1 1.000 1.000  1.00      33.00     25.92     25.92
  1 14  37.33  1 1.000 1.000  1.54      33.00     33.00     33.00
  1 15  40.00  1 1.000 1.000  1.54      33.00     33.00     33.00
  1 16  42.67  1 1.000 1.000  1.54      33.00     33.00     33.00
  1 17  45.33  1 1.000 1.000  1.54      33.00     33.00     33.00
  1 18  48.00  1 1.000 1.000  1.54      33.00     33.00     33.00


-----------------------------------------------------------------

 Note: In the positive moment region, the result is for DL case
       In the negative moment region, the 1st line is for DL case
       and the 2nd line is for LL case
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 TABLE  1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
                 ******************************

 SP IN D FROM  Co
 NO NO L SUPT  mp     Dp       0.42Dt
         (ft)        (in)       (in)
-------------------------------------

  1  0   0.00  1       4.70      17.25
  1  1   2.67  1       4.70      17.25
  1  2   5.33  1       4.70      17.25
  1  3   8.00  1       4.70      17.25
  1  4  10.67  1       4.70      17.25
```

1	5	13.33	1	5.30	17.43
1	6	16.00	1	5.30	17.43
1	7	18.67	1	5.30	17.43
1	8	21.33	1	5.30	17.43
1	9	24.00	1	5.30	17.43
1	10	26.67	1	5.30	17.43
1	11	29.33	1	5.30	17.43
1	12	32.00	1	5.30	17.43
1	13	34.67	1	5.30	17.43
1	14	37.33	1	4.70	17.25
1	15	40.00	1	4.70	17.25
1	16	42.67	1	4.70	17.25
1	17	45.33	1	4.70	17.25
1	18	48.00	1	4.70	17.25

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	2.67	1	0	2	0		2	0
1	2	5.33	1	0	2	0		2	0
1	3	8.00	1	0	2	0		2	0
1	4	10.67	1	0	2	0		2	0
1	5	13.33	1	0	2	0		2	0
1	6	16.00	1	0	2	0		2	0
1	7	18.67	1	0	2	0		2	0
1	8	21.33	1	0	2	0		2	0
1	9	24.00	1	0	2	0		2	0
1	10	26.67	1	0	2	0		2	0
1	11	29.33	1	0	2	0		2	0
1	12	32.00	1	0	2	0		2	0
1	13	34.67	1	0	2	0		2	0
1	14	37.33	1	0	2	0		2	0
1	15	40.00	1	0	2	0		2	0
1	16	42.67	1	0	2	0		2	0
1	17	45.33	1	0	2	0		2	0
1	18	48.00	1	0	2	0		2	0

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

1

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	28.9	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		2.67	0.0	19.8	2.6	33.0	2.6	33.0	2.6	28.9	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	2		5.33	0.0	19.8	4.9	33.0	4.9	33.0	4.9	28.9	0
				0.0	-	4.9	-	4.9	33.0	-	-	0
1	3		8.00	0.0	19.8	6.9	33.0	6.9	33.0	6.9	28.9	0
				0.0	-	6.9	-	6.9	33.0	-	-	0
1	4		10.67	0.0	19.8	8.7	33.0	8.7	33.0	8.7	28.9	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	5		13.33	0.0	19.8	9.5	33.0	9.5	33.0	9.5	28.8	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	6		16.00	0.0	19.8	10.5	33.0	10.5	33.0	10.5	25.9	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	7		18.67	0.0	19.8	11.2	33.0	11.2	33.0	11.2	25.9	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	8		21.33	0.0	19.8	11.6	33.0	11.6	33.0	11.6	25.9	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	9		24.00	0.0	19.8	11.8	33.0	11.8	33.0	11.8	25.9	0
				0.0	-	9.7	-	9.7	33.0	-	-	0
1	10		26.67	0.0	19.8	11.6	33.0	11.6	33.0	11.6	25.9	0
				0.0	-	9.5	-	9.5	33.0	-	-	0
1	11		29.33	0.0	19.8	11.2	33.0	11.2	33.0	11.2	25.9	0
				0.0	-	9.2	-	9.2	33.0	-	-	0
1	12		32.00	0.0	19.8	10.5	33.0	10.5	33.0	10.5	25.9	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	13		34.67	0.0	19.8	9.5	33.0	9.5	33.0	9.5	25.9	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	14		37.33	0.0	19.8	8.7	33.0	8.7	33.0	8.7	33.0	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	15		40.00	0.0	19.8	7.0	33.0	7.0	33.0	7.0	33.0	0
				0.0	-	7.0	-	7.0	33.0	-	-	0
1	16		42.67	0.0	19.8	4.9	33.0	4.9	33.0	4.9	33.0	0
				0.0	-	4.9	-	4.9	33.0	-	-	0
1	17		45.33	0.0	19.8	2.6	33.0	2.6	33.0	2.6	33.0	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	18		48.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in

sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.67	0.000	0.079	0.079	0.091	0.091	4
					0.079		0.079	3
1	2	5.33	0.000	0.150	0.150	0.171	0.171	4
					0.150		0.150	3
1	3	8.00	0.000	0.211	0.211	0.240	0.240	4
					0.211		0.211	3
1	4	10.67	0.000	0.262	0.262	0.299	0.299	4
					0.262		0.262	3
1	5	13.33	0.000	0.287	0.287	0.328	0.328	4
					0.235		0.235	3
1	6	16.00	0.000	0.318	0.318	0.405	0.405	4
					0.261		0.261	3
1	7	18.67	0.000	0.339	0.339	0.432	0.432	4
					0.278		0.278	3
1	8	21.33	0.000	0.352	0.352	0.449	0.449	4
					0.289		0.289	3
1	9	24.00	0.000	0.357	0.357	0.454	0.454	4
					0.292		0.292	3
1	10	26.67	0.000	0.352	0.352	0.449	0.449	4
					0.289		0.289	3
1	11	29.33	0.000	0.339	0.339	0.432	0.432	4
					0.278		0.278	3
1	12	32.00	0.000	0.317	0.317	0.404	0.404	4
					0.260		0.260	3
1	13	34.67	0.000	0.287	0.287	0.365	0.365	4
					0.235		0.235	3
1	14	37.33	0.000	0.263	0.263	0.263	0.263	2
					0.263		0.263	3
1	15	40.00	0.000	0.211	0.211	0.211	0.211	2
					0.211		0.211	3
1	16	42.67	0.000	0.150	0.150	0.150	0.150	2
					0.150		0.150	3
1	17	45.33	0.000	0.080	0.080	0.080	0.080	2
					0.080		0.080	3
1	18	48.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2078.7	-	-	0.0	19.8	-	-	0--0
1	1		2.67	0	390.1	2078.7	-	-	0.0	19.8	-	-	0--0
1	2		5.33	0	727.1	2078.7	-	-	0.0	19.8	-	-	0--0
1	3		8.00	0	1011.1	2078.7	-	-	0.0	19.8	-	-	0--0
1	4		10.67	0	1238.8	2078.7	-	-	0.0	19.8	-	-	0--0
1	5		13.33	0	1416.7	2487.3	-	-	0.0	19.8	-	-	0--0
1	6		16.00	0	1554.8	2487.3	-	-	0.0	19.8	-	-	0--0
1	7		18.67	0	1647.3	2487.3	-	-	0.0	19.8	-	-	0--0
1	8		21.33	0	1681.1	2487.3	-	-	0.0	19.8	-	-	0--0
1	9		24.00	0	1690.4	2487.3	-	-	0.0	19.8	-	-	0--0
1	10		26.67	0	1681.0	2487.3	-	-	0.0	19.8	-	-	0--0
1	11		29.33	0	1647.1	2487.3	-	-	0.0	19.8	-	-	0--0
1	12		32.00	0	1554.4	2487.3	-	-	0.0	19.8	-	-	0--0
1	13		34.67	0	1416.9	2487.3	-	-	0.0	19.8	-	-	0--0
1	14		37.33	0	1239.3	2078.7	-	-	0.0	19.8	-	-	0--0
1	15		40.00	0	1011.4	2078.7	-	-	0.0	19.8	-	-	0--0
1	16		42.67	0	727.3	2078.7	-	-	0.0	19.8	-	-	0--0
1	17		45.33	0	390.2	2078.7	-	-	0.0	19.8	-	-	0--0
1	18		48.00	0	0.0	2078.7	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000			0.000		1
1	1		2.67	0	0.188			0.188		1

1	2	5.33	0	0.350	0.350	1
1	3	8.00	0	0.486	0.486	1
1	4	10.67	0	0.596	0.596	1
1	5	13.33	0	0.570	0.570	1
1	6	16.00	0	0.625	0.625	1
1	7	18.67	0	0.662	0.662	1
1	8	21.33	0	0.676	0.676	1
1	9	24.00	0	0.680	0.680	1
1	10	26.67	0	0.676	0.676	1
1	11	29.33	0	0.662	0.662	1
1	12	32.00	0	0.625	0.625	1
1	13	34.67	0	0.570	0.570	1
1	14	37.33	0	0.596	0.596	1
1	15	40.00	0	0.487	0.487	1
1	16	42.67	0	0.350	0.350	1
1	17	45.33	0	0.188	0.188	1
1	18	48.00	0	0.000	0.000	1

```
NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
      [2] = (PHI)f * Fnc for comp. flange of composite
            sections in positive flexure (Eq. 6.10.7.2.1-1)
          = (PHI)f * Rh * Fyt for tension flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
      [3] = 0.6*Fyt for composite sections in positive flexure
            or tension flange for composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
          = 0.6*Fyc for comp. flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
      [4] = (PHI)f * Fnt for non-compact tension flange
            of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
          = (PHI)f * Fnc for comp. flange of composite sections
            in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

      The governing number is listed as below.
        1 = (Mu+1/3flSxt) / [1]
        2 = fbu / [2]
        3 = fl / [3]
        4 = (fbu + 1/3fl) / [4]
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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	2.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	5.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	8.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	10.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	13.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	16.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	18.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	21.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	24.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	26.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	29.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	32.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	34.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	37.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	40.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	42.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	17	45.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	18	48.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
            = 5 for unstiffened beams and girders
            = 5 for stiffened girders when (d0/D) > 3, or
                                           (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	STRENGTH CATEGORY; SEE NOTE CATEGORY	OVERALL WEB
1	0	0.00	0	180.5	<	330.8	0	0
1	1	2.67	0	165.4	<	330.8	0	0
1	2	5.33	0	150.5	<	330.8	0	0
1	3	8.00	0	135.7	<	330.8	0	0
1	4	10.67	0	121.1	<	330.8	0	0
1	5	13.33	0	106.6	<	330.8	0	0
1	6	16.00	0	91.8	<	330.8	0	0
1	7	18.67	0	77.7	<	330.8	0	0
1	8	21.33	0	64.2	<	330.8	0	0
1	9	24.00	0	51.7	<	330.8	0	0
1	10	26.67	0	64.3	<	330.8	0	0
1	11	29.33	0	77.7	<	330.8	0	0
1	12	32.00	0	91.8	<	330.8	0	0
1	13	34.67	0	106.3	<	330.8	0	0
1	14	37.33	0	121.2	<	330.8	0	0
1	15	40.00	0	135.8	<	330.8	0	0
1	16	42.67	0	150.6	<	330.8	0	0
1	17	45.33	0	165.5	<	330.8	0	0

```
 1 18  48.00       0            180.5   <      330.8       0          0


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 NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

          For compact section  (composite sections in positive flexure)

            w/o long. stiff.

            D/tw <= 150 (Eq. 6.10.2.1.1-1)

            w / long. stiff.

            D/tw <= 300 (Eq. 6.10.2.1.2-1)

            2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

          For compact or non-compact sections specified in Art. 6.10.8

            (composite sections in negative flexure and
             non-composite sections)

            2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


       [2]: SHEAR CAPACITY ... web crippling control

          Vn = CVp .......... AASHTO Eq.6.10.9.2-1

          Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

          Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

          Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

        * For the detailed description of shear capacity,
          please refer to AASHTO Art.6.10.9.2 and .3

        * STRENGTH CATEGORY; 0 = compact section
                             2 = braced non-compact section
                             3 = slender section
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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                ***************************

          YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
       (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
-----------------------------------------------------------------------

 1  0   0.00   33.   180.47   330.76            0
 1  1   2.67   33.   165.42   330.76            0
 1  2   5.33   33.   150.51   330.76            0
 1  3   8.00   33.   135.74   330.76            0
 1  4  10.67   33.   121.11   330.76            0
 1  5  13.33   33.   106.63   330.76            0
 1  6  16.00   33.    91.78   330.76            0
 1  7  18.67   33.    77.66   330.76            0
                                                Page 37
```

1	8	21.33	33.	64.24	330.76	0
1	9	24.00	33.	51.65	330.76	0
1	10	26.67	33.	64.28	330.76	0
1	11	29.33	33.	77.71	330.76	0
1	12	32.00	33.	91.82	330.76	0
1	13	34.67	33.	106.31	330.76	0
1	14	37.33	33.	121.16	330.76	0
1	15	40.00	33.	135.78	330.76	0
1	16	42.67	33.	150.55	330.76	0
1	17	45.33	33.	165.46	330.76	0
1	18	48.00	33.	180.52	330.76	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D NO	L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3]	TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-		00-
1	1	2.67	2.3	-	31.4	7.1	31.4	-	-		00-
1	2	5.33	4.3	-	31.4	13.3	31.4	-	-		00-
1	3	8.00	6.0	-	31.4	18.5	31.4	-	-		00-
1	4	10.67	7.4	-	31.4	22.7	31.4	-	-		00-
1	5	13.33	8.5	-	31.4	20.3	31.4	-	-		00-
1	6	16.00	9.4	-	31.4	22.3	31.4	-	-		00-
1	7	18.67	10.1	-	31.4	23.7	31.4	-	-		00-
1	8	21.33	10.4	-	31.4	24.2	31.4	-	-		00-
1	9	24.00	10.5	-	31.4	24.3	31.4	-	-		00-
1	10	26.67	10.4	-	31.4	24.2	31.4	-	-		00-
1	11	29.33	10.1	-	31.4	23.7	31.4	-	-		00-
1	12	32.00	9.4	-	31.4	22.3	31.4	-	-		00-
1	13	34.67	8.5	-	31.4	20.3	31.4	-	-		00-
1	14	37.33	7.5	-	31.4	22.7	31.4	-	-		00-
1	15	40.00	6.0	-	31.4	18.5	31.4	-	-		00-
1	16	42.67	4.3	-	31.4	13.3	31.4	-	-		00-
1	17	45.33	2.3	-	31.4	7.1	31.4	-	-		00-
1	18	48.00	0.0	-	31.4	0.0	31.4	-	-		00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic
range. For composite sections, Dc shall be
determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on
factored loads, and shall be taken as positive in

```
          sign in all resistance equations (Art. 6.10.1.6)
1
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                 ---------------------------- ----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   2.67    TR     -0.1 A B B^C^C D E E^  TR      1.8 A B B^C^C D E E^
  1  2   5.33    TR     -0.2 A B B^C^C D E E^  TR      3.3 A B B^C^C D E
  1  3   8.00    TR     -0.3 A B B^C^C D E E^  TR      4.5 A B B^C^C D
  1  4  10.67    TR     -0.4 A B B^C^C D E E^  TR      5.3 A B B^C^C D
  1     11.50  SCG
  1  5  13.33    TR     -0.5 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  6  16.00    TR     -0.6 A B B^C^C D E E^  TR      5.1 A B B^C^C D
  1  7  18.67    TR     -0.6 A B B^C^C D E E^  TR      5.5 A B B^C^C D
  1  8  21.33    TR     -0.6 A B B^C^C D E E^  TR      5.6 A B B^C^C D
  1  9  24.00    TR     -0.6 A B B^C^C D E E^  TR      5.6 A B B^C^C D
  1 10  26.67    TR     -0.6 A B B^C^C D E E^  TR      5.6 A B B^C^C D
  1 11  29.33    TR     -0.6 A B B^C^C D E E^  TR      5.5 A B B^C^C D
  1 12  32.00    TR     -0.6 A B B^C^C D E E^  TR      5.1 A B B^C^C D
  1 13  34.67    TR     -0.5 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1     36.50  SCG
  1 14  37.33    TR     -0.4 A B B^C^C D E E^  TR      5.3 A B B^C^C D
  1 15  40.00    TR     -0.3 A B B^C^C D E E^  TR      4.5 A B B^C^C D
  1 16  42.67    TR     -0.2 A B B^C^C D E E^  TR      3.3 A B B^C^C D E
  1 17  45.33    TR     -0.1 A B B^C^C D E E^  TR      1.8 A B B^C^C D E E^
  1 18  48.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
```

SP NO	IN NO	D FROM L SUPT (ft)	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.67	TR	-0.1	A B B^C^C D E E^	TR	0.9	A B B^C^C D E E^
1	2	5.33	TR	-0.1	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	3	8.00	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	4	10.67	TR	-0.2	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1		11.50	SCG					
1	5	13.33	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	6	16.00	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	7	18.67	TR	-0.3	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	8	21.33	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	9	24.00	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	10	26.67	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	11	29.33	TR	-0.3	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	12	32.00	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	13	34.67	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1		36.50	SCG					
1	14	37.33	TR	-0.2	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	15	40.00	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	16	42.67	TR	-0.1	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	17	45.33	TR	-0.1	A B B^C^C D E E^	TR	0.9	A B B^C^C D E E^
1	18	48.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	AASHTO E6.10.10.1.2 Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	46.3	.29909E-01	1.4	TR 3.09	3	6.70
1	1	2.67	44.0	.29909E-01	1.3	TR 3.09	3	7.06
1	2	5.33	41.9	.29909E-01	1.3	TR 3.09	3	7.41
1	3	8.00	40.1	.29909E-01	1.2	TR 3.09	3	7.75
1	4	10.67	37.9	.29909E-01	1.1	TR 3.09	3	8.19
1	5	13.33	36.1	.28418E-01	1.0	TR 3.09	3	9.05
1	6	16.00	36.1	.28418E-01	1.0	TR 3.09	3	9.05
1	7	18.67	36.6	.28418E-01	1.0	TR 3.09	3	8.94
1	8	21.33	36.6	.28418E-01	1.0	TR 3.09	3	8.92
1	9	24.00	36.6	.28418E-01	1.0	TR 3.09	3	8.92
1	10	26.67	36.6	.28418E-01	1.0	TR 3.09	3	8.92
1	11	29.33	36.6	.28418E-01	1.0	TR 3.09	3	8.94
1	12	32.00	36.1	.28418E-01	1.0	TR 3.09	3	9.05
1	13	34.67	36.1	.28418E-01	1.0	TR 3.09	3	9.05
1	14	37.33	37.9	.29909E-01	1.1	TR 3.09	3	8.19
1	15	40.00	40.1	.29909E-01	1.2	TR 3.09	3	7.75
1	16	42.67	41.9	.29909E-01	1.3	TR 3.09	3	7.41

```
 1 17  45.33  44.0 .29909E-01    1.3      TR    3.09        3           7.06
 1 18  48.00  46.3 .29909E-01    1.4      TR    3.09        3           6.70


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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
           *** If ADTT Single Lane is less than 960, use Fatigue II Table.
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                   *******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-2     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ---------------------------------------------------------------------------

  1  0   0.00  23.1 .29909E-01    0.7      TR    1.36        3           5.91
  1  1   2.67  22.0 .29909E-01    0.7      TR    1.36        3           6.22
  1  2   5.33  21.0 .29909E-01    0.6      TR    1.36        3           6.53
  1  3   8.00  20.0 .29909E-01    0.6      TR    1.36        3           6.83
  1  4  10.67  18.9 .29909E-01    0.6      TR    1.36        3           7.22
  1  5  13.33  18.0 .28418E-01    0.5      TR    1.36        3           7.98
  1  6  16.00  18.1 .28418E-01    0.5      TR    1.36        3           7.97
  1  7  18.67  18.3 .28418E-01    0.5      TR    1.36        3           7.88
```

1	8	21.33	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	9	24.00	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	10	26.67	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	11	29.33	18.3	.28418E-01	0.5	TR	1.36	3	7.88
1	12	32.00	18.1	.28418E-01	0.5	TR	1.36	3	7.97
1	13	34.67	18.0	.28418E-01	0.5	TR	1.36	3	7.98
1	14	37.33	18.9	.29909E-01	0.6	TR	1.36	3	7.22
1	15	40.00	20.0	.29909E-01	0.6	TR	1.36	3	6.83
1	16	42.67	21.0	.29909E-01	0.6	TR	1.36	3	6.53
1	17	45.33	22.0	.29909E-01	0.7	TR	1.36	3	6.22
1	18	48.00	23.1	.29909E-01	0.7	TR	1.36	3	5.91

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

1

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

```
                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
       (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
-------------------------------------------------------------------------

 1  9  24.00   1 1276.3  1626.6          147         >  66
 1 18  48.00   1 1276.3  1626.6          147         >  66


-------------------------------------------------------------------------
 Please read NOTE on the Following Page

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     Strength Limit State (Article 6.10.10.4)

  [1] The number of shear connectors provided between the
       section of maximum positive moment and each adjacent
       point of 0.0 moment, or between each adjacent point
       of 0.0 moment and centerline of an interior support
       shall be checked to ensure that adequate connectors
       are provided for Strength Limit State.

  [2] The number of shear connectors required equal or exceed
       the number given by the formula :

             N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
       Where
         N1 = Number of connectors between points of maximum
              positive moment and each adjacent point of 0.0 moment
         N2 = number of connectors between each adjacent point of
              0.0 moment and the centerline of an interior support
         P = total nominal horizontal shear force
         Qr = factored shear resistance of one shear connector
            = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
         Qn = nominal shear resistance
         (Phi)sc = resistance factor for shear connectors

  [3] The total horizontal shear force between the point of
       maximum positive moment and each adjacent point of 0.0 moment
       shall be the lesser either:

            P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
       or
         P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
       Where
         f'c = specified 28-day compressive strength of the concrete
         bs = effective width of the slab
         bfc = width of compression flange
         bft = width of tension flange
         ts = slab thickness
         Fyw = specified minimum yield strength of the web
         Fyt = specified minimum yield strength of the tension flange
         Fyc = specified minimum yield strength of the compression flange
         D = web depth
         tft = thickness of tension flange
         tfc = thickness of compression flange
         tw = web thickness


  [4] For continuous-span composite sections, the total horizontal
       shear force between each adjacent point of 0.0 moment
       and the centerline of an interior support shall be taken as:

            P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
       Where
         bs = effective width of the concrete deck
         ts = thickness of a concrete deck

  [5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
                                                Page 43
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If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	24.000	6.000
1	24.000	48.000	6.000

1

```
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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	48.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2078.7	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.67	2078.7	52.2	27.5	163.4	0.0	8.95	HL-93	6.90	0
1	2	5.33	2078.7	98.4	51.8	303.5	0.0	4.59	HL-93	3.54	0
1	3	8.00	2078.7	138.4	72.8	420.2	0.0	3.18	HL-93	2.45	0
1	4	10.67	2078.7	172.5	90.6	511.7	0.0	2.51	HL-93	1.94	0
1	5	13.33	2487.3	200.5	105.2	581.6	0.0	2.66	HL-93	2.05	0
1	6	16.00	2487.3	222.1	116.5	635.9	0.0	2.38	HL-93	1.84	0
1	7	18.67	2487.3	237.2	124.6	671.5	0.0	2.22	HL-93	1.71	0
1	8	21.33	2487.3	246.3	129.4	680.4	0.0	2.17	HL-93	1.68	0
1	9	24.00	2487.3	249.2	131.0	682.3	0.0	2.16	HL-93	1.67	0
1	10	26.67	2487.3	246.2	129.4	680.4	0.0	2.17	HL-93	1.68	0
1	11	29.33	2487.3	237.0	124.6	671.5	0.0	2.22	HL-93	1.72	0
1	12	32.00	2487.3	221.8	116.5	635.9	0.0	2.38	HL-93	1.84	0
1	13	34.67	2487.3	200.6	105.2	581.6	0.0	2.66	HL-93	2.05	0
1	14	37.33	2078.7	172.9	90.6	511.7	0.0	2.51	HL-93	1.94	0
1	15	40.00	2078.7	138.7	72.8	420.2	0.0	3.18	HL-93	2.45	0
1	16	42.67	2078.7	98.5	51.8	303.5	0.0	4.59	HL-93	3.54	0
1	17	45.33	2078.7	52.3	27.5	163.4	0.0	8.95	HL-93	6.90	0
1	18	48.00	2078.7	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT		UNFACTORED L+I STRESS ---STEEL--- MAX. POS. TOP;ksi;BOT		--- STEEL--- MAX. NEG. TOP;ksi;BOT		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	2.67	31.4	-1.95	2.42	-0.25	3.62	0.00	0.00	8.00	HL-93	6.15
1	2	5.33	31.4	-3.67	4.55	-0.46	6.72	0.00	0.00	3.99	HL-93	3.07
1	3	8.00	31.4	-5.17	6.41	-0.63	9.30	0.00	0.00	2.68	HL-93	2.06
1	4	10.67	31.4	-6.44	7.98	-0.77	11.32	0.00	0.00	2.06	HL-93	1.59
1	5	13.33	31.4	-7.10	7.18	-1.11	10.10	0.00	0.00	2.39	HL-93	1.84
1	6	16.00	31.4	-7.87	7.96	-1.21	11.05	0.00	0.00	2.12	HL-93	1.63
1	7	18.67	31.4	-8.40	8.50	-1.28	11.67	0.00	0.00	1.96	HL-93	1.51
1	8	21.33	31.4	-8.72	8.83	-1.30	11.82	0.00	0.00	1.91	HL-93	1.47
1	9	24.00	31.4	-8.83	8.93	-1.30	11.85	0.00	0.00	1.89	HL-93	1.45
1	10	26.67	31.4	-8.72	8.82	-1.30	11.82	0.00	0.00	1.91	HL-93	1.47
1	11	29.33	31.4	-8.40	8.49	-1.28	11.67	0.00	0.00	1.96	HL-93	1.51
1	12	32.00	31.4	-7.86	7.95	-1.21	11.05	0.00	0.00	2.12	HL-93	1.63
1	13	34.67	31.4	-7.10	7.18	-1.11	10.10	0.00	0.00	2.39	HL-93	1.84
1	14	37.33	31.4	-6.46	7.99	-0.77	11.32	0.00	0.00	2.06	HL-93	1.59
1	15	40.00	31.4	-5.18	6.42	-0.63	9.30	0.00	0.00	2.68	HL-93	2.06
1	16	42.67	31.4	-3.68	4.56	-0.46	6.72	0.00	0.00	3.99	HL-93	3.07
1	17	45.33	31.4	-1.95	2.42	-0.25	3.62	0.00	0.00	8.00	HL-93	6.15
1	18	48.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

1	48.00	24.00	STRENGTH L.S.	2.16	1.67	HL-93
1	48.00	24.00	SERVICEBILITY	1.89	1.45	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.891
Inventory= 1.455

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	48.00	24.00	STRENGTH L.S.	2.98	667.33
1	48.00	24.00	SERVICEBILITY	2.48	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.479

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	20.7	10.9	79.5	0.0	2.70	HL-93	2.08
1	1	2.67	330.8	18.4	9.7	73.5	-2.1	2.96	HL-93	2.28
1	2	5.33	330.8	16.2	8.5	67.6	-4.2	3.27	HL-93	2.52
1	3	8.00	330.8	13.9	7.3	61.8	-7.5	3.63	HL-93	2.80
1	4	10.67	330.8	11.6	6.1	56.0	-11.0	4.07	HL-93	3.14
1	5	13.33	330.8	9.4	4.9	50.3	-14.5	4.59	HL-93	3.54
1	6	16.00	330.8	6.8	3.6	44.7	-18.1	5.26	HL-93	4.06
1	7	18.67	330.8	4.5	2.4	39.2	-21.8	6.08	HL-93	4.69
1	8	21.33	330.8	2.3	1.2	34.1	-25.6	7.08	HL-93	5.46
1	9	24.00	330.8	0.0	0.0	29.5	-29.5	8.30	HL-93	6.41
1	10	26.67	330.8	-2.3	-1.2	25.6	-34.1	7.08	HL-93	5.46
1	11	29.33	330.8	-4.6	-2.4	21.8	-39.2	6.08	HL-93	4.69
1	12	32.00	330.8	-6.8	-3.6	18.1	-44.7	5.26	HL-93	4.06
1	13	34.67	330.8	-9.1	-4.9	14.5	-50.3	4.60	HL-93	3.55
1	14	37.33	330.8	-11.7	-6.1	11.0	-56.0	4.07	HL-93	3.14
1	15	40.00	330.8	-13.9	-7.3	7.5	-61.8	3.63	HL-93	2.80
1	16	42.67	330.8	-16.2	-8.5	4.2	-67.6	3.27	HL-93	2.52
1	17	45.33	330.8	-18.5	-9.7	2.1	-73.5	2.96	HL-93	2.28
1	18	48.00	330.8	-20.7	-10.9	0.0	-79.5	2.70	HL-93	2.08

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	48.00	48.00	STRENGTH L.S.	2.70	2.08	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.696
Inventory= 2.079

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	48.00	48.00	STRENGTH L.S.	3.40	85.14

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.400

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	24.00	3 0.425	-0.23	HL-93	3.18
		4 0.433	-0.23		3.12

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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```
TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 3 Interior Fatigue   8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Interior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       15           1       AASHTO  2010     0     2            3
 ----------------------------------------------------------------------

 * output level    : 0 = basic output
                    1 = detailed output

 * span interval   : maximum = 20
                    default = 10

 * structural type : 1 = composite (default)
                    2 = noncomposite
                    3 = reinforced concrete
                    4 = prestressed concrete

 * type of unit    : 0 = English (default)
                    1 = Metric
                    2 = Metric input English output
                    3 = English input Metric output

 * design option   : 0 = WSD (default)
                    1 = LFD
                    2 = LRFD


 * program flow    : 0 = DL analysis only
                    1 = DL + LL analysis
                    2 = code check
                    3 = rating
                    4 = design
                    5 = design + code check
                    6 = design + recycle + code check
                    7 = DL stage only
                    8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              *****************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.15	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
48.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	PLATE GIRDER WEB THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.00001	0.00	0.4375		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.50				33.	33.	33.
2	2	2	25.00				33.	33.	33.
3	1	1	11.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.44	0.00	15.00	1 1 1 1 1	0.71	2 2 2 2 2
1	0.00	0	0.00	0.60	0.00	0.00	0 0 0 0 0	0.86	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

1

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	48.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	15.10
2	2	Diaphragm	0.000	0.00	0.00	0.29	35.85
3	1	Railing	0.087	0.00	48.00	0.00	0.00
4	1	Curb	0.076	0.00	48.00	0.00	0.00
5	0	Pavement	0.292	0.00	48.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN.	DIA. SEC (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	15.10	33.0	33.0	15.10
15.10	35.85	33.0	33.0	20.75
35.85	48.00	33.0	33.0	12.15

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.726       0.1258     0.8518


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      48.00

     1   DC2     0.087       0.000     48.000

     2   DC2     0.076       0.000     48.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1165.5	43.2	174.8	36.2	211.07

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.710	0.710	0.719	0.719	FOR STRENGTH POSITIVE MOMENT
	0.860				FOR STRENGTH POSITIVE SHEAR
1	0.443				FOR FATIGUE POSITIVE MOMENT
	0.601				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.20	5900.0	32.9	16.45	358.7	358.7
1	2	6.40	5900.0	32.9	16.45	358.7	358.7
1	3	9.60	5900.0	32.9	16.45	358.7	358.7
1	4	12.80	6981.8	32.9	15.02	464.8	381.2
1	5	16.00	6981.8	32.9	15.02	464.8	381.2
1	6	19.20	6981.8	32.9	15.02	464.8	381.2
1	7	22.40	6981.8	32.9	15.02	464.8	381.2
1	8	25.60	6981.8	32.9	15.02	464.8	381.2
1	9	28.80	6981.8	32.9	15.02	464.8	381.2
1	10	32.00	6981.8	32.9	15.02	464.8	381.2
1	11	35.20	6981.8	32.9	15.02	464.8	381.2
1	12	38.40	5900.0	32.9	16.45	358.7	358.7

1 13	41.60	5900.0	32.9	16.45	358.7	358.7
1 14	44.80	5900.0	32.9	16.45	358.7	358.7
1 15	48.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L (ft)	SUPT	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00		12412.8	0.024716	492.1	1617.3	783.9
1	1	3.20		12412.8	0.024716	492.1	1617.3	783.9
1	2	6.40		12412.8	0.024716	492.1	1617.3	783.9
1	3	9.60		12412.8	0.024716	492.1	1617.3	783.9
1	4	12.80		15053.7	0.023292	629.3	1598.8	856.5
1	5	16.00		15053.7	0.023292	629.3	1598.8	856.5
1	6	19.20		15053.7	0.023292	629.3	1598.8	856.5
1	7	22.40		15053.7	0.023292	629.3	1598.8	856.5
1	8	25.60		15053.7	0.023292	629.3	1598.8	856.5
1	9	28.80		15053.7	0.023292	629.3	1598.8	856.5
1	10	32.00		15053.7	0.023292	629.3	1598.8	856.5
1	11	35.20		15053.7	0.023292	629.3	1598.8	856.5
1	12	38.40		12412.8	0.024716	492.1	1617.3	783.9
1	13	41.60		12412.8	0.024716	492.1	1617.3	783.9
1	14	44.80		12412.8	0.024716	492.1	1617.3	783.9
1	15	48.00		12412.8	0.024716	492.1	1617.3	783.9

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L (ft)	SUPT	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00		16700.9	0.029909	542.3	7943.7	1627.4
1	1	3.20		16700.9	0.029909	542.3	7943.7	1627.4
1	2	6.40		16700.9	0.029909	542.3	7943.7	1627.4
1	3	9.60		16700.9	0.029909	542.3	7943.7	1627.4
1	4	12.80		20750.0	0.028418	690.7	6295.0	1811.2
1	5	16.00		20750.0	0.028418	690.7	6295.0	1811.2
1	6	19.20		20750.0	0.028418	690.7	6295.0	1811.2
1	7	22.40		20750.0	0.028418	690.7	6295.0	1811.2
1	8	25.60		20750.0	0.028418	690.7	6295.0	1811.2
1	9	28.80		20750.0	0.028418	690.7	6295.0	1811.2
1	10	32.00		20750.0	0.028418	690.7	6295.0	1811.2
1	11	35.20		20750.0	0.028418	690.7	6295.0	1811.2
1	12	38.40		16700.9	0.029909	542.3	7943.7	1627.4
1	13	41.60		16700.9	0.029909	542.3	7943.7	1627.4

1 14	44.80	16700.9	0.029909	542.3	7943.7	1627.4
1 15	48.00	16700.9	0.029909	542.3	7943.7	1627.4

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	3.20	9.0	52.0	0.9	0.0	61.9	
1	2	6.40	16.8	96.6	1.7	0.0	115.1	
1	3	9.60	23.2	133.8	2.6	0.0	159.6	
1	4	12.80	28.3	163.6	3.4	0.0	195.3	
1	5	16.00	32.2	185.9	4.1	0.0	222.1	
1	6	19.20	34.8	200.7	4.0	0.0	239.5	
1	7	22.40	36.1	208.2	3.9	0.0	248.2	
1	8	25.60	36.1	208.2	3.9	0.0	248.1	
1	9	28.80	34.8	200.7	3.8	0.0	239.3	
1	10	32.00	32.2	185.9	3.8	0.0	221.8	
1	11	35.20	28.3	163.6	3.7	0.0	195.6	
1	12	38.40	23.2	133.8	2.9	0.0	159.9	
1	13	41.60	16.8	96.6	1.9	0.0	115.3	
1	14	44.80	9.0	52.0	1.0	0.0	62.0	
1	15	48.00	0.0	0.0	0.0	0.0	0.0	

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.20	20.9	11.7	0.0	0.0	0.0	32.6
1	2	6.40	38.9	21.7	0.0	0.0	0.0	60.6
1	3	9.60	53.8	30.0	0.0	0.0	0.0	83.9
1	4	12.80	65.8	36.7	0.0	0.0	0.0	102.5
1	5	16.00	74.8	41.7	0.0	0.0	0.0	116.5
1	6	19.20	80.7	45.1	0.0	0.0	0.0	125.8
1	7	22.40	83.7	46.7	0.0	0.0	0.0	130.5
1	8	25.60	83.7	46.7	0.0	0.0	0.0	130.5
1	9	28.80	80.7	45.1	0.0	0.0	0.0	125.8
1	10	32.00	74.8	41.7	0.0	0.0	0.0	116.5
1	11	35.20	65.8	36.7	0.0	0.0	0.0	102.5
1	12	38.40	53.8	30.0	0.0	0.0	0.0	83.9
1	13	41.60	38.9	21.7	0.0	0.0	0.0	60.6
1	14	44.80	20.9	11.7	0.0	0.0	0.0	32.6
1	15	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.20	0.0	0.0	193.3	HL-93	0.0	HL-93
1	2	6.40	0.0	0.0	353.0	HL-93	0.0	HL-93
1	3	9.60	0.0	0.0	479.0	HL-93	0.0	HL-93
1	4	12.80	0.0	0.0	569.0	HL-93	0.0	HL-93
1	5	16.00	0.0	0.0	635.9	HL-93	0.0	HL-93
1	6	19.20	0.0	0.0	676.2	HL-93	0.0	HL-93
1	7	22.40	0.0	0.0	681.6	HL-93	0.0	HL-93
1	8	25.60	0.0	0.0	681.6	HL-93	0.0	HL-93
1	9	28.80	0.0	0.0	676.2	HL-93	0.0	HL-93
1	10	32.00	0.0	0.0	635.9	HL-93	0.0	HL-93
1	11	35.20	0.0	0.0	569.0	HL-93	0.0	HL-93
1	12	38.40	0.0	0.0	479.0	HL-93	0.0	HL-93
1	13	41.60	0.0	0.0	353.0	HL-93	0.0	HL-93
1	14	44.80	0.0	0.0	193.3	HL-93	0.0	HL-93
1	15	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.20	65.	0.	65.
1	2	6.40	116.	0.	116.
1	3	9.60	153.	0.	153.
1	4	12.80	173.	0.	173.
1	5	16.00	195.	0.	195.
1	6	19.20	212.	0.	212.
1	7	22.40	216.	0.	216.
1	8	25.60	216.	0.	216.
1	9	28.80	212.	0.	212.
1	10	32.00	195.	0.	195.
1	11	35.20	173.	0.	173.
1	12	38.40	153.	0.	153.
1	13	41.60	116.	0.	116.
1	14	44.80	65.	0.	65.
1	15	48.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.20	61.9	32.6	193.3	HL-93	0.0	HL-93	287.9	94.5
1	2	6.40	115.1	60.6	353.0	HL-93	0.0	HL-93	528.7	175.7
1	3	9.60	159.6	83.9	479.0	HL-93	0.0	HL-93	722.5	243.5
1	4	12.80	195.3	102.5	569.0	HL-93	0.0	HL-93	866.8	297.9
1	5	16.00	222.1	116.5	635.9	HL-93	0.0	HL-93	974.5	338.6
1	6	19.20	239.5	125.8	676.2	HL-93	0.0	HL-93	1041.5	365.3
1	7	22.40	248.2	130.5	681.6	HL-93	0.0	HL-93	1060.3	378.6
1	8	25.60	248.1	130.5	681.6	HL-93	0.0	HL-93	1060.2	378.6
1	9	28.80	239.3	125.8	676.2	HL-93	0.0	HL-93	1041.3	365.1
1	10	32.00	221.8	116.5	635.9	HL-93	0.0	HL-93	974.2	338.3
1	11	35.20	195.6	102.5	569.0	HL-93	0.0	HL-93	867.1	298.1
1	12	38.40	159.9	83.9	479.0	HL-93	0.0	HL-93	722.8	243.8
1	13	41.60	115.3	60.6	353.0	HL-93	0.0	HL-93	528.9	175.9
1	14	44.80	62.0	32.6	193.3	HL-93	0.0	HL-93	288.0	94.6
1	15	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)

1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.20	287.9	345.9	461.7	384.4	141.8	48.6
1	2	6.40	528.7	634.6	847.1	705.9	263.5	86.6
1	3	9.60	722.5	866.2	1156.0	964.4	365.2	114.4
1	4	12.80	866.8	1037.5	1384.5	1156.9	446.8	129.8
1	5	16.00	974.5	1165.3	1554.8	1300.4	507.9	146.6
1	6	19.20	1041.5	1244.4	1660.1	1389.7	548.0	158.9
1	7	22.40	1060.3	1264.8	1687.1	1414.4	567.9	161.6
1	8	25.60	1060.2	1264.7	1687.0	1414.4	567.9	161.6
1	9	28.80	1041.3	1244.2	1659.9	1389.5	547.7	158.9
1	10	32.00	974.2	1165.0	1554.4	1300.0	507.5	146.6
1	11	35.20	867.1	1037.8	1384.8	1157.2	447.2	129.8
1	12	38.40	722.8	866.5	1156.5	964.9	365.7	114.4
1	13	41.60	528.9	634.8	847.4	706.2	263.9	86.6
1	14	44.80	288.0	346.0	461.9	384.5	142.0	48.6
1	15	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.0	17.4	0.3	0.0	20.7
1	1	3.20	2.6	15.1	0.3	0.0	18.0
1	2	6.40	2.2	12.8	0.3	0.0	15.3
1	3	9.60	1.8	10.5	0.3	0.0	12.5
1	4	12.80	1.4	8.1	0.3	0.0	9.8
1	5	16.00	1.0	5.8	0.0	0.0	6.8
1	6	19.20	0.6	3.5	0.0	0.0	4.1
1	7	22.40	0.2	1.2	0.0	0.0	1.3
1	8	25.60	-0.2	-1.2	0.0	0.0	-1.4
1	9	28.80	-0.6	-3.5	0.0	0.0	-4.1
1	10	32.00	-1.0	-5.8	0.0	0.0	-6.8
1	11	35.20	-1.4	-8.1	0.0	0.0	-9.6
1	12	38.40	-1.8	-10.5	-0.3	0.0	-12.6
1	13	41.60	-2.2	-12.8	-0.3	0.0	-15.3
1	14	44.80	-2.6	-15.1	-0.3	0.0	-18.0
1	15	48.00	-3.0	-17.4	-0.3	0.0	-20.7

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.0	3.9	0.0	0.0	10.9	20.7	31.6
1	1	3.20	6.1	3.4	0.0	0.0	9.5	18.0	27.5
1	2	6.40	5.1	2.9	0.0	0.0	8.0	15.3	23.3
1	3	9.60	4.2	2.3	0.0	0.0	6.6	12.5	19.1

1	4	12.80	3.3	1.8	0.0	0.0	5.1	9.8	14.9
1	5	16.00	2.3	1.3	0.0	0.0	3.6	6.8	10.4
1	6	19.20	1.4	0.8	0.0	0.0	2.2	4.1	6.3
1	7	22.40	0.5	0.3	0.0	0.0	0.7	1.3	2.1
1	8	25.60	-0.5	-0.3	0.0	0.0	-0.7	-1.4	-2.1
1	9	28.80	-1.4	-0.8	0.0	0.0	-2.2	-4.1	-6.3
1	10	32.00	-2.3	-1.3	0.0	0.0	-3.6	-6.8	-10.5
1	11	35.20	-3.3	-1.8	0.0	0.0	-5.1	-9.6	-14.7
1	12	38.40	-4.2	-2.3	0.0	0.0	-6.6	-12.6	-19.1
1	13	41.60	-5.1	-2.9	0.0	0.0	-8.0	-15.3	-23.3
1	14	44.80	-6.1	-3.4	0.0	0.0	-9.5	-18.0	-27.5
1	15	48.00	-7.0	-3.9	0.0	0.0	-10.9	-20.7	-31.7

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	79.5	HL-93	0.0	HL-93
1	1	3.20	0.0	0.0	72.3	HL-93	-2.5	HL-93
1	2	6.40	0.0	0.0	65.3	HL-93	-5.5	HL-93
1	3	9.60	0.0	0.0	58.3	HL-93	-9.6	HL-93
1	4	12.80	0.0	0.0	51.5	HL-93	-13.8	HL-93
1	5	16.00	0.0	0.0	44.7	HL-93	-18.1	HL-93
1	6	19.20	0.0	0.0	38.1	HL-93	-22.6	HL-93
1	7	22.40	0.0	0.0	32.1	HL-93	-27.2	HL-93
1	8	25.60	0.0	0.0	27.2	HL-93	-32.1	HL-93
1	9	28.80	0.0	0.0	22.6	HL-93	-38.1	HL-93
1	10	32.00	0.0	0.0	18.1	HL-93	-44.7	HL-93
1	11	35.20	0.0	0.0	13.8	HL-93	-51.5	HL-93
1	12	38.40	0.0	0.0	9.6	HL-93	-58.3	HL-93
1	13	41.60	0.0	0.0	5.5	HL-93	-65.3	HL-93
1	14	44.80	0.0	0.0	2.5	HL-93	-72.3	HL-93
1	15	48.00	0.0	0.0	0.0	HL-93	-79.5	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	31.	0.	31.
1	1	3.20	28.	-1.	29.
1	2	6.40	25.	-3.	27.
1	3	9.60	22.	-4.	26.
1	4	12.80	18.	-6.	24.
1	5	16.00	16.	-8.	24.
1	6	19.20	15.	-9.	24.
1	7	22.40	13.	-11.	24.
1	8	25.60	11.	-13.	24.
1	9	28.80	9.	-15.	24.
1	10	32.00	8.	-16.	24.
1	11	35.20	6.	-18.	24.
1	12	38.40	4.	-22.	26.
1	13	41.60	3.	-25.	27.

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 1 14  44.80    1.   -28.    29.
 1 15  48.00    0.   -31.    31.


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 TABLE 1.2.6.3B=MINIMUM WELD SIZE
                 *****************

                           TOP FLANGE             BOTTOM FLANGE
                      ----------------------- -----------------------
 SP IN D FROM   Vr      Q/I    SHEAR  WELD      Q/I    SHEAR  WELD
 NO NO L SUPT  [1]      [2]    FLOW   SIZE             FLOW   SIZE
         (ft) (Kip)   (1/in)    [3]   (in)    (1/in)          (in)
 ------------------------------------------------------------------
  1  0   0.00  30.9 .31121E-01  0.48 0.2500*.14421E-01  0.22 0.2500*
  1  1   3.20  29.0 .31121E-01  0.45 0.2500*.14421E-01  0.21 0.2500*
  1  2   6.40  27.4 .31121E-01  0.43 0.2500*.14421E-01  0.20 0.2500*
  1  3   9.60  26.0 .31121E-01  0.40 0.2500*.14421E-01  0.19 0.2500*
  1  4  12.80  24.2 .29870E-01  0.36 0.2500*.11130E-01  0.13 0.2500*
  1  5  16.00  24.1 .29870E-01  0.36 0.2500*.11130E-01  0.13 0.2500*
  1  6  19.20  24.4 .29870E-01  0.36 0.2500*.11130E-01  0.14 0.2500*
  1  7  22.40  24.4 .29870E-01  0.36 0.2500*.11130E-01  0.14 0.2500*
  1  8  25.60  24.4 .29870E-01  0.36 0.2500*.11130E-01  0.14 0.2500*
  1  9  28.80  24.4 .29870E-01  0.36 0.2500*.11130E-01  0.14 0.2500*
  1 10  32.00  24.1 .29870E-01  0.36 0.2500*.11130E-01  0.13 0.2500*
  1 11  35.20  24.2 .29870E-01  0.36 0.2500*.11130E-01  0.13 0.2500*
  1 12  38.40  26.0 .31121E-01  0.40 0.2500*.14421E-01  0.19 0.2500*
  1 13  41.60  27.4 .31121E-01  0.43 0.2500*.14421E-01  0.20 0.2500*
  1 14  44.80  29.0 .31121E-01  0.45 0.2500*.14421E-01  0.21 0.2500*
  1 15  48.00  30.9 .31121E-01  0.48 0.2500*.14421E-01  0.22 0.2500*


 ------------------------------------------------------------------
   NOTE:  [1] Vr = range of shear due to live loads and impact

          [2] For non-composite construction:

              Q/I = (At * Dt) / Inc -- top flange
              Inc = moment of inertia of non-composite section
              At = area of top flange
              Dt = distance between the center of top flange and neutral axis
              Q/I = (Ab * Db) / Inc -- bottom flange
              Ab = area of bottom flange
              Db = distance between the center of bottom flange and neutral axis

              For composite construction:

              Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
              Q = statical moment about the neutral axis of the
                  composite section of the transformed compressive
                  concrete area or the area of reinforcement embedded
                  in the concrete for negative moment
              Ic = moment of inertia for composite section
              Q/I = (Ab * Db) / Ic -- bottom flange

          [3] shear flow = (Vr * Q) / (2 * I)

          * -- minimum weld size governs
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 TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
               ****************************************

                TOTAL     TOTAL L+I , LOAD TYPE = HL - 93
 SP IN D FROM ---------- ----------------------------------    MAX. SHEAR
 NO NO L SUPT DEAD LOADS POSITIVE GOVERN.  NEGATIVE GOVERN.       (kips)
```

		(ft)	(k)	(k)	LOAD TYPE	(k)	LOAD TYPE	POS.	NEG.
1	0	0.00	31.6	79.5	HL-93	0.0	HL-93	111.2	31.6
1	1	3.20	27.5	72.3	HL-93	-2.5	HL-93	99.8	25.0
1	2	6.40	23.3	65.3	HL-93	-5.5	HL-93	88.5	17.8
1	3	9.60	19.1	58.3	HL-93	-9.6	HL-93	77.4	9.5
1	4	12.80	14.9	51.5	HL-93	-13.8	HL-93	66.4	1.1
1	5	16.00	10.4	44.7	HL-93	-18.1	HL-93	55.1	-7.7
1	6	19.20	6.3	38.1	HL-93	-22.6	HL-93	44.4	-16.3
1	7	22.40	2.1	32.1	HL-93	-27.2	HL-93	34.2	-25.1
1	8	25.60	-2.1	27.2	HL-93	-32.1	HL-93	25.1	-34.2
1	9	28.80	-6.3	22.6	HL-93	-38.1	HL-93	16.3	-44.4
1	10	32.00	-10.5	18.1	HL-93	-44.7	HL-93	7.7	-55.1
1	11	35.20	-14.7	13.8	HL-93	-51.5	HL-93	-0.9	-66.1
1	12	38.40	-19.1	9.6	HL-93	-58.3	HL-93	-9.6	-77.4
1	13	41.60	-23.3	5.5	HL-93	-65.3	HL-93	-17.8	-88.6
1	14	44.80	-27.5	2.5	HL-93	-72.3	HL-93	-25.0	-99.8
1	15	48.00	-31.7	0.0	HL-93	-79.5	HL-93	-31.7	-111.2

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	111.2	135.0	180.5	148.7	47.4	23.1
1	1	3.20	99.8	121.5	162.4	133.5	41.2	21.8
1	2	6.40	88.5	108.1	144.6	118.5	34.9	20.6
1	3	9.60	77.4	94.9	126.9	103.6	28.6	19.5
1	4	12.80	66.4	81.8	109.5	88.9	22.4	18.2
1	5	16.00	55.1	68.5	91.8	73.9	15.7	18.1
1	6	19.20	44.4	55.8	74.9	59.7	9.4	18.3
1	7	22.40	34.2	43.9	59.0	46.1	3.1	18.3
1	8	25.60	-34.2	-43.9	-59.0	-46.1	-3.2	18.3
1	9	28.80	-44.4	-55.9	-75.0	-59.7	-9.4	18.3
1	10	32.00	-55.1	-68.5	-91.8	-74.0	-15.7	18.1
1	11	35.20	-66.1	-81.6	-109.2	-88.6	-22.0	18.2
1	12	38.40	-77.4	-94.9	-127.0	-103.7	-28.7	19.5
1	13	41.60	-88.6	-108.1	-144.6	-118.5	-35.0	20.6
1	14	44.80	-99.8	-121.5	-162.5	-133.5	-41.2	21.8
1	15	48.00	-111.2	-135.1	-180.5	-148.7	-47.5	23.1

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	31.63	0.00	HL-93	79.53	HL-93	ST1	26.72	180.47
						ST2	26.72	148.66
						ST4	36.94	47.45
						SE1	31.63	111.16

```
                                                   SE2    31.63        135.02

  2     31.67       0.00  HL-93      79.53  HL-93 ST1    26.75        180.52
                                                   ST2    26.75        148.71
                                                   ST4    36.99         47.50
                                                   SE1    31.67        111.20
                                                   SE2    31.67        135.06

 ---------------------------------------------------------------------------
 NOTE: [1] " - " Indicates Uplift

          ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.
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 TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
               ***************************************************

               NONCOMPOSITE DL     COMPOSITE DL           T O T A L
 SP IN D FROM ---------------- -------------------- --------------------------
 NO NO L SUPT   BEAM    SLAB   CONCENTRATED UNIFORM NONCOMPOSITE+COMPOSITE = DL
        (ft)    (in)    (in)       (in)      (in)       (in)       (in)     (in)
 ---------------------------------------------------------------------------

  1  0   0.00  0.0000  0.0000     0.0000     0.0000     0.0000   0.0000   0.0000
  1  1   3.20 -0.0165 -0.0953     0.0000    -0.0279    -0.1137  -0.0279  -0.1416
  1  2   6.40 -0.0321 -0.1852     0.0000    -0.0542    -0.2210  -0.0542  -0.2753
  1  3   9.60 -0.0460 -0.2653     0.0000    -0.0776    -0.3166  -0.0776  -0.3942
  1  4  12.80 -0.0575 -0.3318     0.0000    -0.0970    -0.3959  -0.0970  -0.4929
  1  5  16.00 -0.0665 -0.3836     0.0000    -0.1120    -0.4577  -0.1120  -0.5698
  1  6  19.20 -0.0727 -0.4192     0.0000    -0.1224    -0.5002  -0.1224  -0.6226
  1  7  22.40 -0.0758 -0.4373     0.0000    -0.1276    -0.5219  -0.1276  -0.6495
  1  8  25.60 -0.0758 -0.4373     0.0000    -0.1276    -0.5219  -0.1276  -0.6495
  1  9  28.80 -0.0727 -0.4192     0.0000    -0.1224    -0.5002  -0.1224  -0.6226
  1 10  32.00 -0.0665 -0.3836     0.0000    -0.1120    -0.4578  -0.1120  -0.5698
  1 11  35.20 -0.0575 -0.3318     0.0000    -0.0970    -0.3960  -0.0970  -0.4929
  1 12  38.40 -0.0460 -0.2653     0.0000    -0.0776    -0.3166  -0.0776  -0.3943
  1 13  41.60 -0.0321 -0.1852     0.0000    -0.0542    -0.2211  -0.0542  -0.2754
  1 14  44.80 -0.0165 -0.0953     0.0000    -0.0279    -0.1137  -0.0279  -0.1416
  1 15  48.00  0.0000  0.0000     0.0000     0.0000     0.0000   0.0000   0.0000


 ---------------------------------------------------------------------------
  NOTE: " - " Indicates downward deflections

  NOTE: The total noncomposite DL deflection is the sum of the deflections
        due to beam, slab, arbitrary DL uniform load and arbitrary DL
        concentrated load.

  NOTE: Due to space limit only beam deflections and slab deflections are
        printed out.
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 TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
               *****************

                NONCOMPOSITE  DEAD  LOADS       COMPOSITE DL      T O T A L
 SP IN D FROM --------------(in)-------------- ------(in)------ ------(in)------
 NO NO L SUPT STEEL  CAMBER    SLAB   CAMBER    DEFL.  CAMBER    DEFL.  CAMBER
        (ft)  DEFL.   SIZE     DEFL.   SIZE             SIZE             SIZE
 ---------------------------------------------------------------------------
  1  0   0.0  0.000    --      0.000    --      0.000    --      0.000    --
  1  1   3.2 -0.017  0   1/16 -0.097  0   1/ 8 -0.028  0   1/16 -0.142  0   3/16
  1  2   6.4 -0.032  0   1/16 -0.189  0   1/ 4 -0.054  0   1/16 -0.275  0   5/16
  1  3   9.6 -0.046  0   1/16 -0.271  0   5/16 -0.078  0   1/ 8 -0.394  0   7/16
  1  4  12.8 -0.057  0   1/16 -0.338  0   3/ 8 -0.097  0   1/ 8 -0.493  0   1/ 2
  1  5  16.0 -0.066  0   1/ 8 -0.391  0   7/16 -0.112  0   1/ 8 -0.570  0   5/ 8
  1  6  19.2 -0.073  0   1/ 8 -0.428  0   7/16 -0.122  0   1/ 8 -0.623  0   5/ 8
  1  7  22.4 -0.076  0   1/ 8 -0.446  0   1/ 2 -0.128  0   3/16 -0.649  0  11/16
```

```
 1  8  25.6 -0.076  0   1/ 8 -0.446  0   1/ 2 -0.128  0   3/16 -0.650  0  11/16
 1  9  28.8 -0.073  0   1/ 8 -0.428  0   7/16 -0.122  0   1/ 8 -0.623  0   5/ 8
 1 10  32.0 -0.066  0   1/ 8 -0.391  0   7/16 -0.112  0   1/ 8 -0.570  0   5/ 8
 1 11  35.2 -0.057  0   1/16 -0.338  0   3/ 8 -0.097  0   1/ 8 -0.493  0   1/ 2
 1 12  38.4 -0.046  0   1/16 -0.271  0   5/16 -0.078  0   1/ 8 -0.394  0   7/16
 1 13  41.6 -0.032  0   1/16 -0.189  0   1/ 4 -0.054  0   1/16 -0.275  0   5/16
 1 14  44.8 -0.017  0   1/16 -0.097  0   1/ 8 -0.028  0   1/16 -0.142  0   3/16
 1 15  48.0  0.000    --      0.000    --      0.000    --      0.000    --


 ------------------------------------------------------------------------------
 NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2
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     TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
                   ******************************************************
                    (UNFACTORED)

                  NUMBER OF LANE   LL + I.
           D FROM -----AND------ DEFLECTION  GOVERN.  1/800 OF SPAN L  ROTATION
     SPAN  L SUPT  DIST. FACTOR  ------------  LOAD   ---------------------
      NO.   (ft)   FOR LL DEFL.     (inch)     TYPE   AASHTO 2.5.2.6.2  [5] Rad.
     ---------------------------------------------------------------------

       1    24.00   3    0.425     -0.226 MAX HL-93     0.72            0.00227
                                   0.025 MIN
                                  -0.056 MAX LANE
                                   0.025 MIN LANE

                    4    0.433     -0.231 MAX HL-93     0.72            0.00227
                                   0.025 MIN
                                  -0.057 MAX LANE
                                   0.025 MIN LANE

                    3    0.425     -0.141 MAX FA
                                   0.000 MIN FA

     ----------------------------------------------------------------------
     NOTE: [1] " - " indicates downward deflection

           [2] The distribution factor for LL+I deflection is defined as

               DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

               where NL= no. of traffic lanes
                     Ng= no. of girders
                     (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

           [3] This table is based upon the optional criteria specified
               in AASHTO LRFD Art. 3.6.1.3.2

           [4] The number of traffic lanes is determined according to
               AASHTO LRFD Art.3.6.1.1.1.
               The 1st line is for the most probable number of lanes
               and the 2nd line is for the next probable number of lanes.

           [5] Max rotations at left (1st line) & right (2nd line)
               supports of the span without averaging, factor and impact

           [6] If ADTT is between 100 and 1000, multi-presence
               factor of 0.95 is applied. If ADTT is below 100, factor
               is 0.9 (AASHTO LRFD C3.6.1.1.2).

           [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
```

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	-0.30	0.30	-1.77	1.77	-2.07	2.07
1	2	6.40	-0.56	0.56	-3.29	3.29	-3.85	3.85
1	3	9.60	-0.78	0.78	-4.56	4.56	-5.34	5.34
1	4	12.80	-0.89	0.73	-5.26	4.31	-6.15	5.04
1	5	16.00	-1.01	0.83	-5.98	4.90	-6.99	5.73
1	6	19.20	-1.10	0.90	-6.44	5.29	-7.54	6.18
1	7	22.40	-1.14	0.93	-6.68	5.48	-7.81	6.41
1	8	25.60	-1.14	0.93	-6.68	5.47	-7.81	6.41
1	9	28.80	-1.10	0.90	-6.44	5.28	-7.53	6.18
1	10	32.00	-1.01	0.83	-5.97	4.90	-6.98	5.73
1	11	35.20	-0.89	0.73	-5.27	4.32	-6.16	5.05
1	12	38.40	-0.78	0.78	-4.58	4.58	-5.35	5.35
1	13	41.60	-0.56	0.56	-3.30	3.30	-3.86	3.86
1	14	44.80	-0.30	0.30	-1.77	1.77	-2.08	2.08
1	15	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	-0.02	-0.24	0.80	-0.02	-2.31	2.87
1	2	6.40	-0.03	-0.45	1.48	-0.03	-4.30	5.33
1	3	9.60	-0.05	-0.62	2.05	-0.05	-5.96	7.38
1	4	12.80	-0.05	-0.77	1.95	-0.05	-6.92	7.00
1	5	16.00	-0.06	-0.87	2.22	-0.06	-7.87	7.96
1	6	19.20	-0.07	-0.94	2.40	-0.07	-8.48	8.58
1	7	22.40	-0.07	-0.98	2.49	-0.07	-8.79	8.90
1	8	25.60	-0.07	-0.98	2.49	-0.07	-8.79	8.89
1	9	28.80	-0.07	-0.94	2.40	-0.07	-8.48	8.58
1	10	32.00	-0.06	-0.87	2.22	-0.06	-7.86	7.95
1	11	35.20	-0.05	-0.77	1.95	-0.05	-6.93	7.00
1	12	38.40	-0.05	-0.62	2.05	-0.05	-5.97	7.40
1	13	41.60	-0.03	-0.45	1.48	-0.03	-4.31	5.34
1	14	44.80	-0.02	-0.24	0.80	-0.02	-2.32	2.87
1	15	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)

MAXIMUM POSITIVE,(MPa)	MAXIMUM NEGATIVE,(ksi)

SP NO	IN NO	D FROM L SUPT (ft)	CONCR. TOP	STEEL TOP	STEEL BOT	CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	-0.16	-0.29	4.28	0.00	0.00	0.00
1	2	6.40	-0.29	-0.53	7.81	0.00	0.00	0.00
1	3	9.60	-0.39	-0.72	10.60	0.00	0.00	0.00
1	4	12.80	-0.42	-1.08	9.89	0.00	0.00	0.00
1	5	16.00	-0.47	-1.21	11.05	0.00	0.00	0.00
1	6	19.20	-0.50	-1.29	11.75	0.00	0.00	0.00
1	7	22.40	-0.50	-1.30	11.84	0.00	0.00	0.00
1	8	25.60	-0.50	-1.30	11.84	0.00	0.00	0.00
1	9	28.80	-0.50	-1.29	11.75	0.00	0.00	0.00
1	10	32.00	-0.47	-1.21	11.05	0.00	0.00	0.00
1	11	35.20	-0.42	-1.08	9.89	0.00	0.00	0.00
1	12	38.40	-0.39	-0.72	10.60	0.00	0.00	0.00
1	13	41.60	-0.29	-0.53	7.81	0.00	0.00	0.00
1	14	44.80	-0.16	-0.29	4.28	0.00	0.00	0.00
1	15	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.20	0.1	0.0	0.1	C	HL	1.4	0.0	1.4	T	HL
1	2	6.40	0.2	0.0	0.2	C	HL	2.6	0.0	2.6	T	HL
1	3	9.60	0.2	0.0	0.2	C	HL	3.4	0.0	3.4	T	HL
1	4	12.80	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	5	16.00	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	6	19.20	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	7	22.40	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	8	25.60	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	9	28.80	0.4	0.0	0.4	C	HL	3.7	0.0	3.7	T	HL
1	10	32.00	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	11	35.20	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	12	38.40	0.2	0.0	0.2	C	HL	3.4	0.0	3.4	T	HL
1	13	41.60	0.2	0.0	0.2	C	HL	2.6	0.0	2.6	T	HL
1	14	44.80	0.1	0.0	0.1	C	HL	1.4	0.0	1.4	T	HL
1	15	48.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.18	-2.61	7.15	-0.02	-2.31	2.87
1	2	6.40	33.	33.	-0.32	-4.83	13.14	-0.03	-4.30	5.33
1	3	9.60	33.	33.	-0.44	-6.69	17.98	-0.05	-5.96	7.38
1	4	12.80	33.	33.	-0.47	-8.00	16.88	-0.05	-6.92	7.00
1	5	16.00	33.	33.	-0.53	-9.08	19.00	-0.06	-7.87	7.96
1	6	19.20	33.	33.	-0.56	-9.77	20.33	-0.07	-8.48	8.58
1	7	22.40	33.	33.	-0.57	-10.09	20.74	-0.07	-8.79	8.90
1	8	25.60	33.	33.	-0.57	-10.09	20.74	-0.07	-8.79	8.89
1	9	28.80	33.	33.	-0.56	-9.77	20.33	-0.07	-8.48	8.58
1	10	32.00	33.	33.	-0.53	-9.07	19.00	-0.06	-7.86	7.95
1	11	35.20	33.	33.	-0.47	-8.01	16.89	-0.05	-6.93	7.00
1	12	38.40	33.	33.	-0.44	-6.70	18.00	-0.05	-5.97	7.40
1	13	41.60	33.	33.	-0.32	-4.84	13.15	-0.03	-4.31	5.34
1	14	44.80	33.	33.	-0.18	-2.61	7.15	-0.02	-2.32	2.87
1	15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.22	-2.69	8.43	-0.02	-2.31	2.87
1	2	6.40	33.	33.	-0.41	-4.99	15.48	-0.03	-4.30	5.33
1	3	9.60	33.	33.	-0.56	-6.90	21.16	-0.05	-5.96	7.38
1	4	12.80	33.	33.	-0.60	-8.33	19.85	-0.05	-6.92	7.00
1	5	16.00	33.	33.	-0.67	-9.44	22.32	-0.06	-7.87	7.96
1	6	19.20	33.	33.	-0.71	-10.16	23.85	-0.07	-8.48	8.58
1	7	22.40	33.	33.	-0.72	-10.48	24.29	-0.07	-8.79	8.90
1	8	25.60	33.	33.	-0.72	-10.48	24.29	-0.07	-8.79	8.89
1	9	28.80	33.	33.	-0.71	-10.15	23.85	-0.07	-8.48	8.58
1	10	32.00	33.	33.	-0.67	-9.43	22.31	-0.06	-7.86	7.95
1	11	35.20	33.	33.	-0.60	-8.34	19.86	-0.05	-6.93	7.00
1	12	38.40	33.	33.	-0.56	-6.91	21.18	-0.05	-5.97	7.40
1	13	41.60	33.	33.	-0.41	-5.00	15.49	-0.03	-4.31	5.34
1	14	44.80	33.	33.	-0.22	-2.70	8.43	-0.02	-2.32	2.87
1	15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.30	-3.44	11.20	-0.03	-2.93	3.71
1	2	6.40	33.	33.	-0.55	-6.38	20.57	-0.05	-5.45	6.90
1	3	9.60	33.	33.	-0.75	-8.82	28.11	-0.07	-7.55	9.56
1	4	12.80	33.	33.	-0.81	-10.67	26.36	-0.08	-8.77	9.06
1	5	16.00	33.	33.	-0.90	-12.10	29.63	-0.09	-9.97	10.30

1	6	19.20	33.	33.	-0.96	-13.01	31.67	-0.09	-10.76	11.11
1	7	22.40	33.	33.	-0.97	-13.42	32.24	-0.10	-11.15	11.52
1	8	25.60	33.	33.	-0.97	-13.42	32.24	-0.10	-11.14	11.52
1	9	28.80	33.	33.	-0.96	-13.01	31.67	-0.09	-10.75	11.11
1	10	32.00	33.	33.	-0.90	-12.08	29.63	-0.09	-9.96	10.29
1	11	35.20	33.	33.	-0.81	-10.68	26.37	-0.08	-8.78	9.07
1	12	38.40	33.	33.	-0.75	-8.83	28.12	-0.07	-7.57	9.57
1	13	41.60	33.	33.	-0.55	-6.39	20.58	-0.05	-5.46	6.91
1	14	44.80	33.	33.	-0.30	-3.45	11.20	-0.03	-2.94	3.72
1	15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.29	-2.55	9.94	-0.01	-2.04	2.45
1	2	6.40	33.	33.	-0.53	-4.73	18.23	-0.03	-3.80	4.56
1	3	9.60	33.	33.	-0.72	-6.53	24.87	-0.04	-5.27	6.32
1	4	12.80	33.	33.	-0.77	-8.00	23.28	-0.04	-6.10	5.98
1	5	16.00	33.	33.	-0.86	-9.06	26.14	-0.04	-6.94	6.80
1	6	19.20	33.	33.	-0.92	-9.74	27.90	-0.05	-7.48	7.34
1	7	22.40	33.	33.	-0.93	-10.03	28.33	-0.05	-7.76	7.61
1	8	25.60	33.	33.	-0.93	-10.03	28.33	-0.05	-7.75	7.61
1	9	28.80	33.	33.	-0.92	-9.74	27.89	-0.05	-7.48	7.34
1	10	32.00	33.	33.	-0.86	-9.05	26.13	-0.04	-6.93	6.80
1	11	35.20	33.	33.	-0.77	-8.01	23.29	-0.04	-6.11	5.99
1	12	38.40	33.	33.	-0.72	-6.54	24.88	-0.04	-5.28	6.33
1	13	41.60	33.	33.	-0.53	-4.74	18.24	-0.03	-3.81	4.57
1	14	44.80	33.	33.	-0.29	-2.56	9.94	-0.01	-2.05	2.46
1	15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.24	-3.33	9.49	-0.03	-2.93	3.71
1	2	6.40	33.	33.	-0.44	-6.17	17.44	-0.05	-5.45	6.90
1	3	9.60	33.	33.	-0.60	-8.53	23.87	-0.07	-7.55	9.56
1	4	12.80	33.	33.	-0.64	-10.24	22.41	-0.08	-8.77	9.06
1	5	16.00	33.	33.	-0.72	-11.61	25.22	-0.09	-9.97	10.30
1	6	19.20	33.	33.	-0.76	-12.50	26.97	-0.09	-10.76	11.11
1	7	22.40	33.	33.	-0.77	-12.90	27.51	-0.10	-11.15	11.52
1	8	25.60	33.	33.	-0.77	-12.90	27.50	-0.10	-11.14	11.52
1	9	28.80	33.	33.	-0.76	-12.49	26.97	-0.09	-10.75	11.11
1	10	32.00	33.	33.	-0.72	-11.60	25.21	-0.09	-9.96	10.29
1	11	35.20	33.	33.	-0.64	-10.25	22.41	-0.08	-8.78	9.07
1	12	38.40	33.	33.	-0.60	-8.54	23.88	-0.07	-7.57	9.57
1	13	41.60	33.	33.	-0.44	-6.18	17.45	-0.05	-5.46	6.91
1	14	44.80	33.	33.	-0.24	-3.33	9.49	-0.03	-2.94	3.72

1 15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.20	33.	33.	-0.23	-2.44	8.23	-0.01	-2.04	2.45
1	2	6.40	33.	33.	-0.42	-4.52	15.10	-0.03	-3.80	4.56
1	3	9.60	33.	33.	-0.56	-6.24	20.63	-0.04	-5.27	6.32
1	4	12.80	33.	33.	-0.60	-7.57	19.33	-0.04	-6.10	5.98
1	5	16.00	33.	33.	-0.68	-8.58	21.72	-0.04	-6.94	6.80
1	6	19.20	33.	33.	-0.72	-9.22	23.20	-0.05	-7.48	7.34
1	7	22.40	33.	33.	-0.73	-9.51	23.59	-0.05	-7.76	7.61
1	8	25.60	33.	33.	-0.73	-9.51	23.59	-0.05	-7.75	7.61
1	9	28.80	33.	33.	-0.72	-9.22	23.19	-0.05	-7.48	7.34
1	10	32.00	33.	33.	-0.68	-8.57	21.71	-0.04	-6.93	6.80
1	11	35.20	33.	33.	-0.60	-7.58	19.34	-0.04	-6.11	5.99
1	12	38.40	33.	33.	-0.56	-6.25	20.64	-0.04	-5.28	6.33
1	13	41.60	33.	33.	-0.42	-4.53	15.11	-0.03	-3.81	4.57
1	14	44.80	33.	33.	-0.23	-2.44	8.23	-0.01	-2.05	2.46
1	15	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.50	181.	-6.0	95.	-0.7	532.	-0.8	0.	0.0	-7.56
											0.80
	LB			6.0		2.3		11.8		0.0	20.15
											11.78
	RT			-5.7		-0.7		-1.0		0.0	-7.42
											1.01
	RB			4.7		1.8		9.3		0.0	15.73
											9.25
1	LT	36.50	181.	-5.7	95.	-0.7	532.	-1.0	0.	0.0	-7.43
											1.01
	LB			4.7		1.8		9.3		0.0	15.74
											9.25
	RT			-6.1		-0.7		-0.8		0.0	-7.57
											0.80
	RB			6.1		2.3		11.8		0.0	20.16
											11.78

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	25.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.4375
3	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 48.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1]) [3]/(12*[1])	<=>	LIMITATION 1/25 1/30	STATUS
1	11.50	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK
2	25.00	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK
3	11.50	6.75	32.90	[2]=	41.06	0.07128	>	0.04000	OK
				[3]=	32.90	0.05712	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

1

TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	1	3.20	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	2	6.40	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	3	9.60	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	4	12.80	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	5	16.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	6	19.20	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	7	22.40	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	8	25.60	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	9	28.80	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	10	32.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	11	35.20	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	12	38.40	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	13	41.60	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	14	44.80	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	15	48.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	3.20	0	1	31.42	0.550	57.13	168.97	2
1	2	6.40	0	1	31.42	0.550	57.13	168.97	2
1	3	9.60	0	1	31.42	0.550	57.13	168.97	2
1	4	12.80	0	1	31.42	0.550	63.98	168.97	2
1	5	16.00	0	1	31.42	0.550	63.98	168.97	2
1	6	19.20	0	1	31.42	0.550	63.98	168.97	2
1	7	22.40	0	1	31.42	0.550	63.98	168.97	2
1	8	25.60	0	1	31.42	0.550	63.98	168.97	2
1	9	28.80	0	1	31.42	0.550	63.98	168.97	2
1	10	32.00	0	1	31.42	0.550	63.98	168.97	2
1	11	35.20	0	1	31.42	0.550	63.98	168.97	2
1	12	38.40	0	1	31.42	0.550	57.13	168.97	2
1	13	41.60	0	1	31.42	0.550	57.13	168.97	2
1	14	44.80	0	1	31.42	0.550	57.13	168.97	2
1	15	48.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners

```
          1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ------------------------------------------------------------

  1  0   0.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  1   3.20   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  2   6.40   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  3   9.60   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  4  12.80   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  5  16.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  6  19.20   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  7  22.40   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  25.60   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  28.80   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  32.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  35.20   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  38.40   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  41.60   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  44.80   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  48.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
```

NO	FROM	TO	(ksi)	(ksi)	(ksi)	(ksi)	Cb
1	0.0	15.1	0.000	10.133	6.580	3.026	1.000
			0.000	0.000	0.000	0.000	1.000
2	15.1	35.8	8.991	10.133	11.717	13.300	1.000
			0.000	0.000	0.000	0.000	1.000
3	35.8	48.0	0.000	8.991	5.517	2.043	1.527
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length
from the smaller end and the ratio of the smaller
and larger lateral moment of inertia of the flange
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	1	3.20	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	2	6.40	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	3	9.60	1	7.77	11.26	19.84	0	15.10	7.09	26.62	2	2
1	4	12.80	1	7.77	11.26	19.84	0	15.10	6.98	26.22	2	2
1	5	16.00	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	6	19.20	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	7	22.40	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	8	25.60	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	9	28.80	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	10	32.00	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	11	35.20	1	7.77	11.26	19.84	0	20.75	6.98	26.22	2	2
1	12	38.40	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	13	41.60	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	14	44.80	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2
1	15	48.00	1	7.77	11.26	19.84	0	12.15	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.00	33.00	28.94	28.94
1	1	3.20	1	1.000	1.000	1.00	33.00	28.94	28.94
1	2	6.40	1	1.000	1.000	1.00	33.00	28.94	28.94
1	3	9.60	1	1.000	1.000	1.00	33.00	28.94	28.94
1	4	12.80	1	1.000	1.000	1.00	33.00	28.82	28.82
1	5	16.00	1	1.000	1.000	1.00	33.00	25.92	25.92
1	6	19.20	1	1.000	1.000	1.00	33.00	25.92	25.92
1	7	22.40	1	1.000	1.000	1.00	33.00	25.92	25.92
1	8	25.60	1	1.000	1.000	1.00	33.00	25.92	25.92
1	9	28.80	1	1.000	1.000	1.00	33.00	25.92	25.92
1	10	32.00	1	1.000	1.000	1.00	33.00	25.92	25.92
1	11	35.20	1	1.000	1.000	1.00	33.00	25.92	25.92
1	12	38.40	1	1.000	1.000	1.53	33.00	33.00	33.00
1	13	41.60	1	1.000	1.000	1.53	33.00	33.00	33.00
1	14	44.80	1	1.000	1.000	1.53	33.00	33.00	33.00
1	15	48.00	1	1.000	1.000	1.53	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0	0.00	1	4.70	17.25
1	1	3.20	1	4.70	17.25
1	2	6.40	1	4.70	17.25
1	3	9.60	1	4.70	17.25
1	4	12.80	1	5.30	17.43
1	5	16.00	1	5.30	17.43
1	6	19.20	1	5.30	17.43
1	7	22.40	1	5.30	17.43
1	8	25.60	1	5.30	17.43
1	9	28.80	1	5.30	17.43
1	10	32.00	1	5.30	17.43
1	11	35.20	1	5.30	17.43
1	12	38.40	1	4.70	17.25
1	13	41.60	1	4.70	17.25
1	14	44.80	1	4.70	17.25
1	15	48.00	1	4.70	17.25

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

S T R E N G T H C A T E G O R Y, Category

SP NO	IN NO	D FROM SUPT (ft)	Section Region	Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.20	1	0	2	0		2	0
1	2	6.40	1	0	2	0		2	0
1	3	9.60	1	0	2	0		2	0
1	4	12.80	1	0	2	0		2	0
1	5	16.00	1	0	2	0		2	0
1	6	19.20	1	0	2	0		2	0
1	7	22.40	1	0	2	0		2	0
1	8	25.60	1	0	2	0		2	0
1	9	28.80	1	0	2	0		2	0
1	10	32.00	1	0	2	0		2	0
1	11	35.20	1	0	2	0		2	0
1	12	38.40	1	0	2	0		2	0
1	13	41.60	1	0	2	0		2	0
1	14	44.80	1	0	2	0		2	0
1	15	48.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	28.9	0
			0.0	-	0.0	-	0.0	33.0	-	-	0
1	1	3.20	0.0	19.8	3.1	33.0	3.1	33.0	3.1	28.9	0
			0.0	-	3.1	-	3.1	33.0	-	-	0
1	2	6.40	0.0	19.8	5.8	33.0	5.8	33.0	5.8	28.9	0

			0.0	-	5.8	-	5.8	33.0	-	-	0
1	3	9.60	0.0	19.8	8.0	33.0	8.0	33.0	8.0	28.9	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	4	12.80	0.0	19.8	9.2	33.0	9.2	33.0	9.2	28.8	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	5	16.00	0.0	19.8	10.5	33.0	10.5	33.0	10.5	25.9	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	6	19.20	0.0	19.8	11.3	33.0	11.3	33.0	11.3	25.9	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	7	22.40	0.0	19.8	11.7	33.0	11.7	33.0	11.7	25.9	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	8	25.60	0.0	19.8	11.7	33.0	11.7	33.0	11.7	25.9	0
			0.0	-	9.6	-	9.6	33.0	-	-	0
1	9	28.80	0.0	19.8	11.3	33.0	11.3	33.0	11.3	25.9	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	10	32.00	0.0	19.8	10.5	33.0	10.5	33.0	10.5	25.9	0
			0.0	-	8.6	-	8.6	33.0	-	-	0
1	11	35.20	0.0	19.8	9.2	33.0	9.2	33.0	9.2	25.9	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	12	38.40	0.0	19.8	8.0	33.0	8.0	33.0	8.0	33.0	0
			0.0	-	8.0	-	8.0	33.0	-	-	0
1	13	41.60	0.0	19.8	5.8	33.0	5.8	33.0	5.8	33.0	0
			0.0	-	5.8	-	5.8	33.0	-	-	0
1	14	44.80	0.0	19.8	3.1	33.0	3.1	33.0	3.1	33.0	0
			0.0	-	3.1	-	3.1	33.0	-	-	0
1	15	48.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.20	0.000	0.094	0.094	0.107	0.107	4
					0.094		0.094	3
1	2	6.40	0.000	0.175	0.175	0.200	0.200	4
					0.175		0.175	3
1	3	9.60	0.000	0.243	0.243	0.277	0.277	4
					0.243		0.243	3
1	4	12.80	0.000	0.280	0.280	0.320	0.320	4
					0.229		0.229	3
1	5	16.00	0.000	0.318	0.318	0.405	0.405	4
					0.261		0.261	3
1	6	19.20	0.000	0.343	0.343	0.436	0.436	4
					0.281		0.281	3
1	7	22.40	0.000	0.355	0.355	0.452	0.452	4
					0.291		0.291	3

1	8	25.60	0.000	0.355	0.355	0.452	0.452	4
					0.291		0.291	3
1	9	28.80	0.000	0.342	0.342	0.436	0.436	4
					0.281		0.281	3
1	10	32.00	0.000	0.317	0.317	0.404	0.404	4
					0.260		0.260	3
1	11	35.20	0.000	0.280	0.280	0.356	0.356	4
					0.230		0.230	3
1	12	38.40	0.000	0.243	0.243	0.243	0.243	2
					0.243		0.243	3
1	13	41.60	0.000	0.175	0.175	0.175	0.175	2
					0.175		0.175	3
1	14	44.80	0.000	0.094	0.094	0.094	0.094	2
					0.094		0.094	3
1	15	48.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2078.7	-	-	0.0	19.8	-	-	0--0
1	1		3.20	0	461.7	2078.7	-	-	0.0	19.8	-	-	0--0
1	2		6.40	0	847.1	2078.7	-	-	0.0	19.8	-	-	0--0
1	3		9.60	0	1156.0	2078.7	-	-	0.0	19.8	-	-	0--0
1	4		12.80	0	1384.5	2487.3	-	-	0.0	19.8	-	-	0--0
1	5		16.00	0	1554.8	2487.3	-	-	0.0	19.8	-	-	0--0
1	6		19.20	0	1660.1	2487.3	-	-	0.0	19.8	-	-	0--0
1	7		22.40	0	1687.1	2487.3	-	-	0.0	19.8	-	-	0--0
1	8		25.60	0	1687.0	2487.3	-	-	0.0	19.8	-	-	0--0
1	9		28.80	0	1659.9	2487.3	-	-	0.0	19.8	-	-	0--0
1	10		32.00	0	1554.4	2487.3	-	-	0.0	19.8	-	-	0--0
1	11		35.20	0	1384.8	2487.3	-	-	0.0	19.8	-	-	0--0
1	12		38.40	0	1156.5	2078.7	-	-	0.0	19.8	-	-	0--0
1	13		41.60	0	847.4	2078.7	-	-	0.0	19.8	-	-	0--0
1	14		44.80	0	461.9	2078.7	-	-	0.0	19.8	-	-	0--0
1	15		48.00	0	0.0	2078.7	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange
of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections

in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000				0.000	1
1	1	3.20	0	0.222				0.222	1
1	2	6.40	0	0.408				0.408	1
1	3	9.60	0	0.556				0.556	1
1	4	12.80	0	0.557				0.557	1
1	5	16.00	0	0.625				0.625	1
1	6	19.20	0	0.667				0.667	1
1	7	22.40	0	0.678				0.678	1
1	8	25.60	0	0.678				0.678	1
1	9	28.80	0	0.667				0.667	1
1	10	32.00	0	0.625				0.625	1
1	11	35.20	0	0.557				0.557	1
1	12	38.40	0	0.556				0.556	1
1	13	41.60	0	0.408				0.408	1
1	14	44.80	0	0.222				0.222	1
1	15	48.00	0	0.000				0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.20	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.40	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	9.60	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	12.80	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	16.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	19.20	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	22.40	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	25.60	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	28.80	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	32.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	35.20	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	38.40	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	41.60	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	44.80	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	48.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
= 5 for unstiffened beams and girders
= 5 for stiffened girders when (d0/D) > 3, or
(d0/D) > (260/(D/tw))**2

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=	UNSTIFFENED SHEAR CAPA-CITY, [2]	STRENGTH CATEGORY; SEE NOTE CATEGORY	OVERALL WEB
1	0	0.00	0	180.5	<	330.8	0	0
1	1	3.20	0	162.4	<	330.8	0	0
1	2	6.40	0	144.6	<	330.8	0	0
1	3	9.60	0	126.9	<	330.8	0	0
1	4	12.80	0	109.5	<	330.8	0	0
1	5	16.00	0	91.8	<	330.8	0	0
1	6	19.20	0	74.9	<	330.8	0	0
1	7	22.40	0	59.0	<	330.8	0	0
1	8	25.60	0	59.0	<	330.8	0	0
1	9	28.80	0	75.0	<	330.8	0	0
1	10	32.00	0	91.8	<	330.8	0	0
1	11	35.20	0	109.2	<	330.8	0	0
1	12	38.40	0	127.0	<	330.8	0	0
1	13	41.60	0	144.6	<	330.8	0	0
1	14	44.80	0	162.5	<	330.8	0	0
1	15	48.00	0	180.5	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	180.47	330.76	0	
1	1	3.20	33.	162.43	330.76	0	
1	2	6.40	33.	144.58	330.76	0	
1	3	9.60	33.	126.95	330.76	0	
1	4	12.80	33.	109.51	330.76	0	
1	5	16.00	33.	91.78	330.76	0	
1	6	19.20	33.	74.91	330.76	0	
1	7	22.40	33.	58.96	330.76	0	
1	8	25.60	33.	59.00	330.76	0	
1	9	28.80	33.	74.95	330.76	0	
1	10	32.00	33.	91.82	330.76	0	
1	11	35.20	33.	109.20	330.76	0	
1	12	38.40	33.	126.99	330.76	0	
1	13	41.60	33.	144.63	330.76	0	
1	14	44.80	33.	162.47	330.76	0	
1	15	48.00	33.	180.52	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	BOT. FLANGE [1] ff+fl/2 (ksi)	BOT. FLANGE [2] ff+fl/2 (ksi)	COMP. FLG fc (ksi)	COMP. FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	3.20	2.7	-	31.4	8.4	31.4	-	-	00-
1	2	6.40	5.0	-	31.4	15.5	31.4	-	-	00-
1	3	9.60	6.9	-	31.4	21.2	31.4	-	-	00-
1	4	12.80	8.3	-	31.4	19.8	31.4	-	-	00-
1	5	16.00	9.4	-	31.4	22.3	31.4	-	-	00-
1	6	19.20	10.2	-	31.4	23.9	31.4	-	-	00-
1	7	22.40	10.5	-	31.4	24.3	31.4	-	-	00-
1	8	25.60	10.5	-	31.4	24.3	31.4	-	-	00-
1	9	28.80	10.2	-	31.4	23.8	31.4	-	-	00-
1	10	32.00	9.4	-	31.4	22.3	31.4	-	-	00-
1	11	35.20	8.3	-	31.4	19.9	31.4	-	-	00-
1	12	38.40	6.9	-	31.4	21.2	31.4	-	-	00-
1	13	41.60	5.0	-	31.4	15.5	31.4	-	-	00-
1	14	44.80	2.7	-	31.4	8.4	31.4	-	-	00-
1	15	48.00	0.0	-	31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
            but not to exceed the smaller of RhFyc and Fyw/0.7
            k = bending-buckling coefficient
              = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
            where:
            Dc = depth of the web in compression in the elastic
                 range. For composite sections, Dc shall be
                 determined as specified in Article D6.3.1

      "-" is N.A.

      Flag check - 0 = OK; 1 = NG
      T = Top Flange; B = Bottom Flange; C = Comp. Flange

      The values of fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)
1
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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

    (1) Main (Longitudinal) Load Carrying Members

    (2) Road Type = I --- Rural Interstate
        ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
               ---------------------------- ----------------------------
SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
-----------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   3.20    TR     -0.1 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1  2   6.40    TR     -0.3 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
 1  3   9.60    TR     -0.3 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1     11.50  SCG
 1  4  12.80    TR     -0.5 A B B^C^C D E E^  TR      4.5 A B B^C^C D
 1  5  16.00    TR     -0.6 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1  6  19.20    TR     -0.6 A B B^C^C D E E^  TR      5.5 A B B^C^C D
 1  7  22.40    TR     -0.6 A B B^C^C D E E^  TR      5.6 A B B^C^C D
 1  8  25.60    TR     -0.6 A B B^C^C D E E^  TR      5.6 A B B^C^C D
 1  9  28.80    TR     -0.6 A B B^C^C D E E^  TR      5.5 A B B^C^C D
 1 10  32.00    TR     -0.6 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1 11  35.20    TR     -0.5 A B B^C^C D E E^  TR      4.5 A B B^C^C D
 1     36.50  SCG
 1 12  38.40    TR     -0.3 A B B^C^C D E E^  TR      5.1 A B B^C^C D
 1 13  41.60    TR     -0.3 A B B^C^C D E E^  TR      3.8 A B B^C^C D E
 1 14  44.80    TR     -0.1 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1 15  48.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


-------------------------------------------------------------------------------
NOTE: Negative sign means live load stresses all in compression or the permanent
      load compressive stress more than twice the max. live load tensile stress.
-------------------------------------------------------------------------------
NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
      Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
             SCG = Section-change point
             POC = Dead load point of contraflexure

             I:P = Point where INT coincides with POC
             I:C = Point where INT coincides with SCG
             S:P = Point where SCG coincides with POC
```

1

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.20	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2	6.40	TR	-0.1	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	3	9.60	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1		11.50	SCG					
1	4	12.80	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	5	16.00	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	6	19.20	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	7	22.40	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	8	25.60	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	9	28.80	TR	-0.3	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	10	32.00	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	11	35.20	TR	-0.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1		36.50	SCG					
1	12	38.40	TR	-0.2	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	13	41.60	TR	-0.1	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	14	44.80	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	15	48.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

1

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	AASHTO E6.10.10.1.2 Vf [1]	Q/I [2]	Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	46.3	.29909E-01	1.4	TR	3.09	3	6.70
1	1	3.20	43.5	.29909E-01	1.3	TR	3.09	3	7.13
1	2	6.40	41.2	.29909E-01	1.2	TR	3.09	3	7.54

1	3	9.60	39.0	.29909E-01	1.2	TR	3.09	3	7.97
1	4	12.80	36.3	.28418E-01	1.0	TR	3.09	3	8.99
1	5	16.00	36.1	.28418E-01	1.0	TR	3.09	3	9.05
1	6	19.20	36.6	.28418E-01	1.0	TR	3.09	3	8.92
1	7	22.40	36.6	.28418E-01	1.0	TR	3.09	3	8.92
1	8	25.60	36.6	.28418E-01	1.0	TR	3.09	3	8.92
1	9	28.80	36.6	.28418E-01	1.0	TR	3.09	3	8.92
1	10	32.00	36.1	.28418E-01	1.0	TR	3.09	3	9.05
1	11	35.20	36.3	.28418E-01	1.0	TR	3.09	3	8.99
1	12	38.40	39.0	.29909E-01	1.2	TR	3.09	3	7.97
1	13	41.60	41.2	.29909E-01	1.2	TR	3.09	3	7.54
1	14	44.80	43.5	.29909E-01	1.3	TR	3.09	3	7.13
1	15	48.00	46.3	.29909E-01	1.4	TR	3.09	3	6.70

Please read NOTE on the following page

1

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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
           *** If ADTT Single Lane is less than 960, use Fatigue II Table.
1
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                   *********************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-2     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
```

		(ft)	[1]	[2]	[3]	[4],Zr <--Input	[5], input	(in),[6]
1	0	0.00	23.1	.29909E-01	0.7	TR	1.36	3	5.91
1	1	3.20	21.8	.29909E-01	0.7	TR	1.36	3	6.28
1	2	6.40	20.6	.29909E-01	0.6	TR	1.36	3	6.65
1	3	9.60	19.5	.29909E-01	0.6	TR	1.36	3	7.02
1	4	12.80	18.2	.28418E-01	0.5	TR	1.36	3	7.93
1	5	16.00	18.1	.28418E-01	0.5	TR	1.36	3	7.97
1	6	19.20	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	7	22.40	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	8	25.60	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	9	28.80	18.3	.28418E-01	0.5	TR	1.36	3	7.87
1	10	32.00	18.1	.28418E-01	0.5	TR	1.36	3	7.97
1	11	35.20	18.2	.28418E-01	0.5	TR	1.36	3	7.93
1	12	38.40	19.5	.29909E-01	0.6	TR	1.36	3	7.02
1	13	41.60	20.6	.29909E-01	0.6	TR	1.36	3	6.65
1	14	44.80	21.8	.29909E-01	0.7	TR	1.36	3	6.28
1	15	48.00	23.1	.29909E-01	0.7	TR	1.36	3	5.91

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

1

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D L (ft)	FROM SUPT	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k)	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE LRFD 6.10.10.4.2 N1	N2	STATUS FATIG. CRI. BLANK=OK CHECK=**
1	7	22.40		1	1276.3	1626.6		135	>	66		
1	15	48.00		1	1276.3	1626.6		135	>	66		

Please read NOTE on the Following Page

1

Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment

and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	22.400	6.000
1	22.400	48.000	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	48.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2078.7	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.20	2078.7	61.9	32.6	193.3	0.0	7.49	HL-93	5.78	0
1	2	6.40	2078.7	115.1	60.6	353.0	0.0	3.88	HL-93	2.99	0
1	3	9.60	2078.7	159.6	83.9	479.0	0.0	2.72	HL-93	2.10	0
1	4	12.80	2487.3	195.3	102.5	569.0	0.0	2.73	HL-93	2.11	0
1	5	16.00	2487.3	222.1	116.5	635.9	0.0	2.38	HL-93	1.84	0
1	6	19.20	2487.3	239.5	125.8	676.2	0.0	2.20	HL-93	1.70	0
1	7	22.40	2487.3	248.2	130.5	681.6	0.0	2.17	HL-93	1.67	0
1	8	25.60	2487.3	248.1	130.5	681.6	0.0	2.17	HL-93	1.67	0
1	9	28.80	2487.3	239.3	125.8	676.2	0.0	2.20	HL-93	1.70	0
1	10	32.00	2487.3	221.8	116.5	635.9	0.0	2.38	HL-93	1.84	0
1	11	35.20	2487.3	195.6	102.5	569.0	0.0	2.73	HL-93	2.11	0
1	12	38.40	2078.7	159.9	83.9	479.0	0.0	2.72	HL-93	2.10	0
1	13	41.60	2078.7	115.3	60.6	353.0	0.0	3.88	HL-93	2.99	0
1	14	44.80	2078.7	62.0	32.6	193.3	0.0	7.49	HL-93	5.78	0
1	15	48.00	2078.7	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

Please read NOTES on the following page

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	3.20	31.4	-2.31 2.87	-0.29 4.28	0.00 0.00	6.66	HL-93	5.12
1	2	6.40	31.4	-4.30 5.33	-0.53 7.81	0.00 0.00	3.33	HL-93	2.56
1	3	9.60	31.4	-5.96 7.38	-0.72 10.60	0.00 0.00	2.26	HL-93	1.74
1	4	12.80	31.4	-6.92 7.00	-1.08 9.89	0.00 0.00	2.46	HL-93	1.89
1	5	16.00	31.4	-7.87 7.96	-1.21 11.05	0.00 0.00	2.12	HL-93	1.63
1	6	19.20	31.4	-8.48 8.58	-1.29 11.75	0.00 0.00	1.94	HL-93	1.49
1	7	22.40	31.4	-8.79 8.90	-1.30 11.84	0.00 0.00	1.90	HL-93	1.46
1	8	25.60	31.4	-8.79 8.89	-1.30 11.84	0.00 0.00	1.90	HL-93	1.46
1	9	28.80	31.4	-8.48 8.58	-1.29 11.75	0.00 0.00	1.94	HL-93	1.49
1	10	32.00	31.4	-7.86 7.95	-1.21 11.05	0.00 0.00	2.12	HL-93	1.63
1	11	35.20	31.4	-6.93 7.00	-1.08 9.89	0.00 0.00	2.46	HL-93	1.89
1	12	38.40	31.4	-5.97 7.40	-0.72 10.60	0.00 0.00	2.26	HL-93	1.74
1	13	41.60	31.4	-4.31 5.34	-0.53 7.81	0.00 0.00	3.33	HL-93	2.56
1	14	44.80	31.4	-2.32 2.87	-0.29 4.28	0.00 0.00	6.66	HL-93	5.12
1	15	48.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH	CRITICAL LOCATION D FROM L SUPT	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

	(ft)	(ft)				
1	48.00	22.40	STRENGTH L.S.	2.17	1.67	HL-93
1	48.00	22.40	SERVICEBILITY	1.90	1.46	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.896
Inventory= 1.459

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	48.00	22.40	STRENGTH L.S.	2.99	665.93
1	48.00	22.40	SERVICEBILITY	2.49	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.489

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	20.7	10.9	79.5	0.0	2.70	HL-93	2.08
1	1	3.20	330.8	18.0	9.5	72.3	-2.5	3.02	HL-93	2.33
1	2	6.40	330.8	15.3	8.0	65.3	-5.5	3.41	HL-93	2.63
1	3	9.60	330.8	12.5	6.6	58.3	-9.6	3.89	HL-93	3.00
1	4	12.80	330.8	9.8	5.1	51.5	-13.8	4.48	HL-93	3.46
1	5	16.00	330.8	6.8	3.6	44.7	-18.1	5.26	HL-93	4.06
1	6	19.20	330.8	4.1	2.2	38.1	-22.6	6.27	HL-93	4.83
1	7	22.40	330.8	1.3	0.7	32.1	-27.2	7.56	HL-93	5.83
1	8	25.60	330.8	-1.4	-0.7	27.2	-32.1	7.56	HL-93	5.83
1	9	28.80	330.8	-4.1	-2.2	22.6	-38.1	6.26	HL-93	4.83
1	10	32.00	330.8	-6.8	-3.6	18.1	-44.7	5.26	HL-93	4.06
1	11	35.20	330.8	-9.6	-5.1	13.8	-51.5	4.49	HL-93	3.46
1	12	38.40	330.8	-12.6	-6.6	9.6	-58.3	3.89	HL-93	3.00
1	13	41.60	330.8	-15.3	-8.0	5.5	-65.3	3.41	HL-93	2.63
1	14	44.80	330.8	-18.0	-9.5	2.5	-72.3	3.02	HL-93	2.33
1	15	48.00	330.8	-20.7	-10.9	0.0	-79.5	2.70	HL-93	2.08

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN CRITICAL LOCATION GOVERNING GOVERNING

NO.	LENGTH (ft)	D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	LIVE LOAD TYPE
1	48.00	48.00	STRENGTH L.S.	2.70	2.08	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.696
Inventory= 2.079

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	48.00	48.00	STRENGTH L.S.	3.40	85.14

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 3.400

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	24.00	3 0.425	-0.23	HL-93	3.18
		4 0.433	-0.23		3.12

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95 NB Span 3 Exterior Strength	8/19/201

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Exterior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	14	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.15	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
48.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB ------- (lb/ft)	PLATE GIRDER WEB DEPTH THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.0000	7.50	0.3125		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	14.00				33.	33.	33.
2	2	2	20.00				33.	33.	33.
3	1	1	14.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.57	2 2 2 2 2
1	0.00	0	0.00	0.39	0.00	0.00	0 0 0 0 0	0.55	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.55	0.00	48.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	6.50
2	2	Diaphragm	0.000	0.00	0.00	0.14	27.25
3	1	Railing	0.087	0.00	48.00	0.00	0.00
4	1	Curb	0.076	0.00	48.00	0.00	0.00
5	0	Pavement	0.292	0.00	48.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING
UNIVERSITY OF MARYLAND

MERLIN V 10.4
COMPOSITE
LRF -- 2010
INPUT

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	6.50	33.0	33.0	6.50
6.50	27.25	33.0	33.0	20.75
27.25	48.00	33.0	33.0	20.75

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

FACTORS USED BY LRFD

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.554       0.1214     0.6754


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      48.00

     1   DC2     0.087       0.000     48.000

     2   DC2     0.076       0.000     48.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1165.5	43.2	174.8	35.0	209.79

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.571	0.571	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.550				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.391				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.43	5900.0	32.9	16.45	358.7	358.7
1	2	6.86	5900.0	32.9	16.45	358.7	358.7
1	3	10.29	5900.0	32.9	16.45	358.7	358.7
1	4	13.71	5900.0	32.9	16.45	358.7	358.7
1	5	17.14	6506.7	32.9	15.71	414.1	371.8
1	6	20.57	6506.7	32.9	15.71	414.1	371.8
1	7	24.00	6506.7	32.9	15.71	414.1	371.8
1	8	27.43	6506.7	32.9	15.71	414.1	371.8
1	9	30.86	6506.7	32.9	15.71	414.1	371.8
1	10	34.29	5900.0	32.9	16.45	358.7	358.7
1	11	37.71	5900.0	32.9	16.45	358.7	358.7
1	12	41.14	5900.0	32.9	16.45	358.7	358.7

1 13	44.57	5900.0	32.9	16.45	358.7	358.7
1 14	48.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11329.8	0.022745	476.1	1244.6	656.3
1	1	3.43	11329.8	0.022745	476.1	1244.6	656.3
1	2	6.86	11329.8	0.022745	476.1	1244.6	656.3
1	3	10.29	11329.8	0.022745	476.1	1244.6	656.3
1	4	13.71	11329.8	0.022745	476.1	1244.6	656.3
1	5	17.14	12619.3	0.021941	546.2	1248.3	690.7
1	6	20.57	12619.3	0.021941	546.2	1248.3	690.7
1	7	24.00	12619.3	0.021941	546.2	1248.3	690.7
1	8	27.43	12619.3	0.021941	546.2	1248.3	690.7
1	9	30.86	12619.3	0.021941	546.2	1248.3	690.7
1	10	34.29	11329.8	0.022745	476.1	1244.6	656.3
1	11	37.71	11329.8	0.022745	476.1	1244.6	656.3
1	12	41.14	11329.8	0.022745	476.1	1244.6	656.3
1	13	44.57	11329.8	0.022745	476.1	1244.6	656.3
1	14	48.00	11329.8	0.022745	476.1	1244.6	656.3

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15583.4	0.029038	529.9	4459.6	1337.1
1	1	3.43	15583.4	0.029038	529.9	4459.6	1337.1
1	2	6.86	15583.4	0.029038	529.9	4459.6	1337.1
1	3	10.29	15583.4	0.029038	529.9	4459.6	1337.1
1	4	13.71	15583.4	0.029038	529.9	4459.6	1337.1
1	5	17.14	17581.4	0.028154	606.8	4149.2	1418.2
1	6	20.57	17581.4	0.028154	606.8	4149.2	1418.2
1	7	24.00	17581.4	0.028154	606.8	4149.2	1418.2
1	8	27.43	17581.4	0.028154	606.8	4149.2	1418.2
1	9	30.86	17581.4	0.028154	606.8	4149.2	1418.2
1	10	34.29	15583.4	0.029038	529.9	4459.6	1337.1
1	11	37.71	15583.4	0.029038	529.9	4459.6	1337.1
1	12	41.14	15583.4	0.029038	529.9	4459.6	1337.1
1	13	44.57	15583.4	0.029038	529.9	4459.6	1337.1
1	14	48.00	15583.4	0.029038	529.9	4459.6	1337.1

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.43	9.3	42.3	0.6	0.0		52.2
1	2	6.86	17.1	78.1	1.2	0.0		96.5
1	3	10.29	23.5	107.5	1.4	0.0		132.4
1	4	13.71	28.5	130.2	1.5	0.0		160.3
1	5	17.14	32.1	146.5	1.7	0.0		180.3
1	6	20.57	34.2	156.3	1.8	0.0		192.4
1	7	24.00	35.0	159.6	2.0	0.0		196.5
1	8	27.43	34.2	156.3	2.1	0.0		192.6
1	9	30.86	32.1	146.5	1.7	0.0		180.4
1	10	34.29	28.5	130.2	1.4	0.0		160.2
1	11	37.71	23.5	107.5	1.0	0.0		132.0
1	12	41.14	17.1	78.1	0.7	0.0		96.0
1	13	44.57	9.3	42.3	0.3	0.0		52.0
1	14	48.00	0.0	0.0	0.0	0.0		0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.43	22.3	12.5	0.0	0.0	0.0	34.8
1	2	6.86	41.2	23.0	0.0	0.0	0.0	64.2
1	3	10.29	56.6	31.6	0.0	0.0	0.0	88.3
1	4	13.71	68.6	38.3	0.0	0.0	0.0	107.0
1	5	17.14	77.2	43.1	0.0	0.0	0.0	120.3
1	6	20.57	82.4	46.0	0.0	0.0	0.0	128.4
1	7	24.00	84.1	46.9	0.0	0.0	0.0	131.0
1	8	27.43	82.4	46.0	0.0	0.0	0.0	128.4
1	9	30.86	77.2	43.1	0.0	0.0	0.0	120.3
1	10	34.29	68.6	38.3	0.0	0.0	0.0	107.0
1	11	37.71	56.6	31.6	0.0	0.0	0.0	88.3
1	12	41.14	41.2	23.0	0.0	0.0	0.0	64.2
1	13	44.57	22.3	12.5	0.0	0.0	0.0	34.8
1	14	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.43	0.0	0.0	165.5	HL-93	0.0	HL-93
1	2	6.86	0.0	0.0	300.0	HL-93	0.0	HL-93
1	3	10.29	0.0	0.0	402.4	HL-93	0.0	HL-93
1	4	13.71	0.0	0.0	474.7	HL-93	0.0	HL-93
1	5	17.14	0.0	0.0	525.6	HL-93	0.0	HL-93
1	6	20.57	0.0	0.0	546.2	HL-93	0.0	HL-93
1	7	24.00	0.0	0.0	548.7	HL-93	0.0	HL-93
1	8	27.43	0.0	0.0	546.2	HL-93	0.0	HL-93
1	9	30.86	0.0	0.0	525.6	HL-93	0.0	HL-93
1	10	34.29	0.0	0.0	474.7	HL-93	0.0	HL-93
1	11	37.71	0.0	0.0	402.4	HL-93	0.0	HL-93
1	12	41.14	0.0	0.0	300.0	HL-93	0.0	HL-93
1	13	44.57	0.0	0.0	165.5	HL-93	0.0	HL-93
1	14	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.

1	1	3.43	57.	0.	57.
1	2	6.86	100.	0.	100.
1	3	10.29	130.	0.	130.
1	4	13.71	148.	0.	148.
1	5	17.14	167.	0.	167.
1	6	20.57	177.	0.	177.
1	7	24.00	178.	0.	178.
1	8	27.43	177.	0.	177.
1	9	30.86	167.	0.	167.
1	10	34.29	148.	0.	148.
1	11	37.71	130.	0.	130.
1	12	41.14	100.	0.	100.
1	13	44.57	57.	0.	57.
1	14	48.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.43	52.2	34.8	165.5	HL-93	0.0	HL-93	252.6	87.0
1	2	6.86	96.5	64.2	300.0	HL-93	0.0	HL-93	460.7	160.7
1	3	10.29	132.4	88.3	402.4	HL-93	0.0	HL-93	623.0	220.6
1	4	13.71	160.3	107.0	474.7	HL-93	0.0	HL-93	742.0	267.3
1	5	17.14	180.3	120.3	525.6	HL-93	0.0	HL-93	826.3	300.6
1	6	20.57	192.4	128.4	546.2	HL-93	0.0	HL-93	867.0	320.7
1	7	24.00	196.5	131.0	548.7	HL-93	0.0	HL-93	876.3	327.5
1	8	27.43	192.6	128.4	546.2	HL-93	0.0	HL-93	867.2	321.0
1	9	30.86	180.4	120.3	525.6	HL-93	0.0	HL-93	826.4	300.7
1	10	34.29	160.2	107.0	474.7	HL-93	0.0	HL-93	741.8	267.1
1	11	37.71	132.0	88.3	402.4	HL-93	0.0	HL-93	622.7	220.3
1	12	41.14	96.0	64.2	300.0	HL-93	0.0	HL-93	460.2	160.1
1	13	44.57	52.0	34.8	165.5	HL-93	0.0	HL-93	252.3	86.7
1	14	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.43	252.6	302.2	404.1	337.8	130.5	42.7
1	2	6.86	460.7	550.7	736.2	616.2	241.0	75.4
1	3	10.29	623.0	743.7	994.1	833.1	330.9	97.3
1	4	13.71	742.0	884.4	1182.0	992.1	400.9	110.8
1	5	17.14	826.3	984.0	1315.0	1104.7	451.0	125.5
1	6	20.57	867.0	1030.8	1377.4	1158.9	481.1	132.5
1	7	24.00	876.3	1040.9	1390.7	1171.2	491.3	133.8

1	8	27.43	867.2	1031.1	1377.7	1159.2	481.5	132.5
1	9	30.86	826.4	984.1	1315.1	1104.8	451.1	125.5
1	10	34.29	741.8	884.2	1181.8	991.9	400.7	110.8
1	11	37.71	622.7	743.4	993.7	832.7	330.4	97.3
1	12	41.14	460.2	550.2	735.5	615.5	240.2	75.4
1	13	44.57	252.3	301.9	403.7	337.5	130.1	42.7
1	14	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	2.9	13.3	0.2	0.0	16.4
1	1	3.43	2.5	11.4	0.2	0.0	14.1
1	2	6.86	2.1	9.5	0.0	0.0	11.6
1	3	10.29	1.7	7.6	0.0	0.0	9.3
1	4	13.71	1.2	5.7	0.0	0.0	7.0
1	5	17.14	0.8	3.8	0.0	0.0	4.7
1	6	20.57	0.4	1.9	0.0	0.0	2.4
1	7	24.00	0.0	0.0	0.0	0.0	0.0
1	8	27.43	-0.4	-1.9	-0.1	0.0	-2.4
1	9	30.86	-0.8	-3.8	-0.1	0.0	-4.7
1	10	34.29	-1.2	-5.7	-0.1	0.0	-7.0
1	11	37.71	-1.7	-7.6	-0.1	0.0	-9.4
1	12	41.14	-2.1	-9.5	-0.1	0.0	-11.7
1	13	44.57	-2.5	-11.4	-0.1	0.0	-14.0
1	14	48.00	-2.9	-13.3	-0.1	0.0	-16.3

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.0	3.9	0.0	0.0	10.9	16.4	27.3
1	1	3.43	6.0	3.4	0.0	0.0	9.4	14.1	23.4
1	2	6.86	5.0	2.8	0.0	0.0	7.8	11.6	19.4
1	3	10.29	4.0	2.2	0.0	0.0	6.2	9.3	15.5
1	4	13.71	3.0	1.7	0.0	0.0	4.7	7.0	11.7
1	5	17.14	2.0	1.1	0.0	0.0	3.1	4.7	7.8
1	6	20.57	1.0	0.6	0.0	0.0	1.6	2.4	3.9
1	7	24.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	8	27.43	-1.0	-0.6	0.0	0.0	-1.6	-2.4	-4.0
1	9	30.86	-2.0	-1.1	0.0	0.0	-3.1	-4.7	-7.9
1	10	34.29	-3.0	-1.7	0.0	0.0	-4.7	-7.0	-11.7
1	11	37.71	-4.0	-2.2	0.0	0.0	-6.2	-9.4	-15.6
1	12	41.14	-5.0	-2.8	0.0	0.0	-7.8	-11.7	-19.5
1	13	44.57	-6.0	-3.4	0.0	0.0	-9.4	-14.0	-23.4

```
 1 14  48.00    -7.0   -3.9     0.0    0.0    -10.9      -16.3       -27.2


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 TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N =  9.0 (UNFACTORED)
               ****************************

               SIDEWALK (MAXIMUM)     LL+I ,(kips), LOAD TYPE= HL -93
 SP IN D FROM  ------------------  -------------------------------------
 NO NO L SUPT  POSITIVE  NEGATIVE  MAXIMUM   GOVERN.   MAXIMUM   GOVERN.
        (ft)    (kips)    (kips)   POSITIVE LOAD TYPE  NEGATIVE LOAD TYPE
 ---------------------------------------------------------------------

  1  0   0.00     0.0       0.0       50.9   HL-93        0.0    HL-93
  1  1   3.43     0.0       0.0       45.9   HL-93       -1.7    HL-93
  1  2   6.86     0.0       0.0       41.1   HL-93       -3.9    HL-93
  1  3  10.29     0.0       0.0       36.3   HL-93       -6.7    HL-93
  1  4  13.71     0.0       0.0       31.7   HL-93       -9.6    HL-93
  1  5  17.14     0.0       0.0       27.0   HL-93      -12.6    HL-93
  1  6  20.57     0.0       0.0       22.7   HL-93      -15.7    HL-93
  1  7  24.00     0.0       0.0       18.9   HL-93      -18.9    HL-93
  1  8  27.43     0.0       0.0       15.7   HL-93      -22.7    HL-93
  1  9  30.86     0.0       0.0       12.6   HL-93      -27.0    HL-93
  1 10  34.29     0.0       0.0        9.6   HL-93      -31.7    HL-93
  1 11  37.71     0.0       0.0        6.7   HL-93      -36.3    HL-93
  1 12  41.14     0.0       0.0        3.9   HL-93      -41.1    HL-93
  1 13  44.57     0.0       0.0        1.7   HL-93      -45.9    HL-93
  1 14  48.00     0.0       0.0        0.0   HL-93      -50.9    HL-93


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 TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N =  9.0 (kips) (UNFACTORED)
                ****************************************

 SP IN D FROM     TRUCK   ONLY
 NO NO L SUPT -------------------
        (ft)    POS    NEG  RANGE
 --------------------------------
  1  0   0.00   20.     0.    20.
  1  1   3.43   18.    -1.    19.
  1  2   6.86   16.    -2.    18.
  1  3  10.29   14.    -3.    17.
  1  4  13.71   11.    -4.    16.
  1  5  17.14   10.    -5.    16.
  1  6  20.57    9.    -7.    16.
  1  7  24.00    8.    -8.    16.
  1  8  27.43    7.    -9.    16.
  1  9  30.86    5.   -10.    16.
  1 10  34.29    4.   -11.    16.
  1 11  37.71    3.   -14.    17.
  1 12  41.14    2.   -16.    18.
  1 13  44.57    1.   -18.    19.
  1 14  48.00    0.   -20.    20.


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 TABLE 1.2.6.3B=MINIMUM WELD SIZE
```

****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	20.1	.31052E-01	0.31	0.2500*	.14740E-01	0.15	0.2500*
1	1		3.43	18.8	.31052E-01	0.29	0.2500*	.14740E-01	0.14	0.2500*
1	2		6.86	17.7	.31052E-01	0.28	0.2500*	.14740E-01	0.13	0.2500*
1	3		10.29	16.6	.31052E-01	0.26	0.2500*	.14740E-01	0.12	0.2500*
1	4		13.71	15.6	.31052E-01	0.24	0.2500*	.14740E-01	0.11	0.2500*
1	5		17.14	15.8	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	6		20.57	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	7		24.00	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	8		27.43	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	9		30.86	15.8	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	10		34.29	15.6	.31052E-01	0.24	0.2500*	.14740E-01	0.11	0.2500*
1	11		37.71	16.6	.31052E-01	0.26	0.2500*	.14740E-01	0.12	0.2500*
1	12		41.14	17.7	.31052E-01	0.28	0.2500*	.14740E-01	0.13	0.2500*
1	13		44.57	18.8	.31052E-01	0.29	0.2500*	.14740E-01	0.14	0.2500*
1	14		48.00	20.1	.31052E-01	0.31	0.2500*	.14740E-01	0.15	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0		0.00	27.3	50.9	HL-93	0.0	HL-93	78.2	27.3
1	1		3.43	23.4	45.9	HL-93	-1.7	HL-93	69.4	21.7
1	2		6.86	19.4	41.1	HL-93	-3.9	HL-93	60.5	15.6
1	3		10.29	15.5	36.3	HL-93	-6.7	HL-93	51.9	8.9
1	4		13.71	11.7	31.7	HL-93	-9.6	HL-93	43.3	2.1
1	5		17.14	7.8	27.0	HL-93	-12.6	HL-93	34.8	-4.8
1	6		20.57	3.9	22.7	HL-93	-15.7	HL-93	26.7	-11.8
1	7		24.00	0.0	18.9	HL-93	-18.9	HL-93	18.9	-18.8
1	8		27.43	-4.0	15.7	HL-93	-22.7	HL-93	11.7	-26.7
1	9		30.86	-7.9	12.6	HL-93	-27.0	HL-93	4.8	-34.9
1	10		34.29	-11.7	9.6	HL-93	-31.7	HL-93	-2.1	-43.4

1	11	37.71	-15.6	6.7	HL-93	-36.3	HL-93	-8.9	-51.9
1	12	41.14	-19.5	3.9	HL-93	-41.1	HL-93	-15.6	-60.6
1	13	44.57	-23.4	1.7	HL-93	-45.9	HL-93	-21.6	-69.3
1	14	48.00	-27.2	0.0	HL-93	-50.9	HL-93	-27.2	-78.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN D NO	FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	78.2	93.4	124.9	104.6	41.0	15.1
1	1	3.43	69.4	83.2	111.2	92.8	35.2	14.1
1	2	6.86	60.5	72.8	97.4	81.0	29.1	13.3
1	3	10.29	51.9	62.8	84.0	69.5	23.3	12.4
1	4	13.71	43.3	52.9	70.8	58.1	17.5	11.7
1	5	17.14	34.8	42.9	57.6	46.7	11.7	11.8
1	6	20.57	26.7	33.5	44.9	35.8	5.9	11.9
1	7	24.00	18.9	24.6	33.1	25.5	0.1	11.9
1	8	27.43	-26.7	-33.5	-45.0	-35.9	-6.0	11.9
1	9	30.86	-34.9	-43.0	-57.6	-46.8	-11.8	11.8
1	10	34.29	-43.4	-52.9	-70.8	-58.2	-17.6	11.7
1	11	37.71	-51.9	-62.9	-84.1	-69.6	-23.4	12.4
1	12	41.14	-60.6	-72.9	-97.5	-81.1	-29.2	13.3
1	13	44.57	-69.3	-83.1	-111.1	-92.7	-35.0	14.1
1	14	48.00	-78.1	-93.4	-124.8	-104.5	-40.8	15.1

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	27.32	0.00	HL-93	50.86	HL-93	ST1	22.83	124.91
						ST2	22.83	104.56
						ST4	30.46	40.97
						SE1	27.32	78.18
						SE2	27.32	93.44
2	27.23	0.00	HL-93	50.86	HL-93	ST1	22.76	124.80
						ST2	22.76	104.46
						ST4	30.33	40.85
						SE1	27.23	78.09
						SE2	27.23	93.35

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	3.43	-0.0181	-0.0827	0.0000	-0.0352	-0.1018	-0.0352	-0.1370
1	2	6.86	-0.0351	-0.1604	0.0000	-0.0682	-0.1975	-0.0682	-0.2657
1	3	10.29	-0.0502	-0.2289	0.0000	-0.0973	-0.2818	-0.0973	-0.3791
1	4	13.71	-0.0624	-0.2847	0.0000	-0.1210	-0.3505	-0.1210	-0.4714
1	5	17.14	-0.0714	-0.3257	0.0000	-0.1383	-0.4010	-0.1383	-0.5393
1	6	20.57	-0.0769	-0.3511	0.0000	-0.1491	-0.4322	-0.1491	-0.5812
1	7	24.00	-0.0788	-0.3596	0.0000	-0.1527	-0.4427	-0.1527	-0.5954
1	8	27.43	-0.0769	-0.3511	0.0000	-0.1491	-0.4321	-0.1491	-0.5812
1	9	30.86	-0.0714	-0.3257	0.0000	-0.1383	-0.4009	-0.1383	-0.5393
1	10	34.29	-0.0624	-0.2847	0.0000	-0.1210	-0.3504	-0.1210	-0.4713
1	11	37.71	-0.0502	-0.2289	0.0000	-0.0973	-0.2817	-0.0973	-0.3789
1	12	41.14	-0.0351	-0.1604	0.0000	-0.0682	-0.1973	-0.0682	-0.2655
1	13	44.57	-0.0181	-0.0827	0.0000	-0.0352	-0.1017	-0.0352	-0.1369
1	14	48.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.4	-0.018	0 1/16	-0.084	0 1/ 8	-0.035	0 1/16	-0.137	0 3/16
1	2	6.9	-0.035	0 1/16	-0.162	0 3/16	-0.068	0 1/ 8	-0.266	0 5/16
1	3	10.3	-0.050	0 1/16	-0.232	0 1/ 4	-0.097	0 1/ 8	-0.379	0 7/16
1	4	13.7	-0.062	0 1/16	-0.288	0 5/16	-0.121	0 1/ 8	-0.471	0 1/ 2
1	5	17.1	-0.071	0 1/ 8	-0.330	0 3/ 8	-0.138	0 3/16	-0.539	0 9/16
1	6	20.6	-0.077	0 1/ 8	-0.355	0 3/ 8	-0.149	0 3/16	-0.581	0 5/ 8
1	7	24.0	-0.079	0 1/ 8	-0.364	0 3/ 8	-0.153	0 3/16	-0.595	0 5/ 8
1	8	27.4	-0.077	0 1/ 8	-0.355	0 3/ 8	-0.149	0 3/16	-0.581	0 5/ 8
1	9	30.9	-0.071	0 1/ 8	-0.330	0 3/ 8	-0.138	0 3/16	-0.539	0 9/16
1	10	34.3	-0.062	0 1/16	-0.288	0 5/16	-0.121	0 1/ 8	-0.471	0 1/ 2
1	11	37.7	-0.050	0 1/16	-0.231	0 1/ 4	-0.097	0 1/ 8	-0.379	0 7/16
1	12	41.1	-0.035	0 1/16	-0.162	0 3/16	-0.068	0 1/ 8	-0.266	0 5/16
1	13	44.6	-0.018	0 1/16	-0.084	0 1/ 8	-0.035	0 1/16	-0.137	0 3/16
1	14	48.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	24.00	3	-0.266	MAX HL-93	0.72	0.00265
			0.029	MIN		
			-0.066	MAX LANE		
			0.029	MIN LANE		
		4	-0.272	MAX HL-93	0.72	0.00265
			0.029	MIN		
			-0.067	MAX LANE		
			0.029	MIN LANE		
		3	-0.166	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	-0.31	0.31	-1.44	1.44	-1.75	1.75
1	2	6.86	-0.57	0.57	-2.66	2.66	-3.23	3.23
1	3	10.29	-0.79	0.79	-3.64	3.64	-4.43	4.43
1	4	13.71	-0.95	0.95	-4.41	4.41	-5.36	5.36
1	5	17.14	-1.04	0.93	-4.78	4.29	-5.82	5.22
1	6	20.57	-1.11	0.99	-5.10	4.58	-6.21	5.57
1	7	24.00	-1.13	1.01	-5.21	4.68	-6.34	5.69
1	8	27.43	-1.11	0.99	-5.11	4.59	-6.22	5.58

1	9	30.86	-1.04	0.93	-4.79	4.30	-5.82	5.23
1	10	34.29	-0.95	0.95	-4.40	4.40	-5.36	5.36
1	11	37.71	-0.79	0.79	-3.63	3.63	-4.42	4.42
1	12	41.14	-0.57	0.57	-2.64	2.64	-3.21	3.21
1	13	44.57	-0.31	0.31	-1.43	1.43	-1.74	1.74
1	14	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	-0.02	-0.34	0.88	-0.02	-2.08	2.62
1	2	6.86	-0.04	-0.62	1.62	-0.04	-3.85	4.85
1	3	10.29	-0.06	-0.85	2.22	-0.06	-5.28	6.65
1	4	13.71	-0.07	-1.03	2.70	-0.07	-6.39	8.06
1	5	17.14	-0.08	-1.16	2.64	-0.08	-6.98	7.87
1	6	20.57	-0.08	-1.23	2.82	-0.08	-7.44	8.39
1	7	24.00	-0.08	-1.26	2.88	-0.08	-7.60	8.57
1	8	27.43	-0.08	-1.23	2.82	-0.08	-7.45	8.40
1	9	30.86	-0.08	-1.16	2.64	-0.08	-6.98	7.87
1	10	34.29	-0.07	-1.03	2.70	-0.07	-6.39	8.06
1	11	37.71	-0.06	-0.85	2.22	-0.06	-5.27	6.64
1	12	41.14	-0.04	-0.62	1.62	-0.04	-3.83	4.83
1	13	44.57	-0.02	-0.34	0.88	-0.02	-2.07	2.61
1	14	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	-0.17	-0.45	3.75	0.00	0.00	0.00
1	2	6.86	-0.30	-0.81	6.79	0.00	0.00	0.00
1	3	10.29	-0.40	-1.08	9.11	0.00	0.00	0.00
1	4	13.71	-0.47	-1.28	10.75	0.00	0.00	0.00
1	5	17.14	-0.49	-1.52	10.40	0.00	0.00	0.00
1	6	20.57	-0.51	-1.58	10.80	0.00	0.00	0.00
1	7	24.00	-0.52	-1.59	10.85	0.00	0.00	0.00
1	8	27.43	-0.51	-1.58	10.80	0.00	0.00	0.00
1	9	30.86	-0.49	-1.52	10.40	0.00	0.00	0.00
1	10	34.29	-0.47	-1.28	10.75	0.00	0.00	0.00
1	11	37.71	-0.40	-1.08	9.11	0.00	0.00	0.00
1	12	41.14	-0.30	-0.81	6.79	0.00	0.00	0.00
1	13	44.57	-0.17	-0.45	3.75	0.00	0.00	0.00
1	14	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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 TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N =  9.0 (ksi) (UNFACTORED)
                 ************************************************


 SP IN D FROM  TOP OF TOP FLANGE   GOVERNING    BOT OF BOT FLANGE   GOVERNING
 NO NO L SUPT ------------------  STRESS  LOAD ------------------  STRESS  LOAD
        (ft)     TRUCK  ONLY     RANGE TP TYPE    TRUCK  ONLY     RANGE TP TYPE
 -------------------------------------------------------------------------------

  1  0   0.00     0.0     0.0      0.0 T   HL      0.0     0.0      0.0 T   HL
  1  1   3.43     0.2     0.0      0.2 C   HL      1.3     0.0      1.3 T   HL
  1  2   6.86     0.3     0.0      0.3 C   HL      2.3     0.0      2.3 T   HL
  1  3  10.29     0.3     0.0      0.3 C   HL      2.9     0.0      2.9 T   HL
  1  4  13.71     0.4     0.0      0.4 C   HL      3.3     0.0      3.3 T   HL
  1  5  17.14     0.5     0.0      0.5 C   HL      3.3     0.0      3.3 T   HL
  1  6  20.57     0.5     0.0      0.5 C   HL      3.5     0.0      3.5 T   HL
  1  7  24.00     0.5     0.0      0.5 C   HL      3.5     0.0      3.5 T   HL
  1  8  27.43     0.5     0.0      0.5 C   HL      3.5     0.0      3.5 T   HL
  1  9  30.86     0.5     0.0      0.5 C   HL      3.3     0.0      3.3 T   HL
  1 10  34.29     0.4     0.0      0.4 C   HL      3.3     0.0      3.3 T   HL
  1 11  37.71     0.3     0.0      0.3 C   HL      2.9     0.0      2.9 T   HL
  1 12  41.14     0.3     0.0      0.3 C   HL      2.3     0.0      2.3 T   HL
  1 13  44.57     0.2     0.0      0.2 C   HL      1.3     0.0      1.3 T   HL
  1 14  48.00     0.0     0.0      0.0 T   HL      0.0     0.0      0.0 T   HL


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 NOTE: LOAD TYPE:  HL = Fatigue Truck or Tandem
       STRESS TYPE:  R = Reversal, C = Compression, and T = Tension

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 TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
                 *****************************************

               YIELD      TOTAL POSITIVE ,(ksi)     TOTAL NEGATIVE ,(ksi)
               STRESS   ------------------------ ------------------------
 SP IN D FROM --------- CONCR.    STEEL BEAM      CONCR.    STEEL BEAM
 NO NO L SUPT  Fy (ksi)   TOP  ------------------   TOP  ------------------
        (ft)  TOP  BOT           TOP      BOT               TOP      BOT
 ---------------------------------------------------------------------------

  1  0   0.00  33.  33.   0.00     0.00   0.00      0.00     0.00     0.00
  1  1   3.43  33.  33.  -0.19    -2.53   6.37     -0.02    -2.08     2.62
  1  2   6.86  33.  33.  -0.34    -4.65  11.64     -0.04    -3.85     4.85
  1  3  10.29  33.  33.  -0.46    -6.36  15.76     -0.06    -5.28     6.65
  1  4  13.71  33.  33.  -0.55    -7.67  18.81     -0.07    -6.39     8.06
  1  5  17.14  33.  33.  -0.57    -8.50  18.26     -0.08    -6.98     7.87
  1  6  20.57  33.  33.  -0.60    -9.02  19.20     -0.08    -7.44     8.39
  1  7  24.00  33.  33.  -0.60    -9.19  19.42     -0.08    -7.60     8.57
  1  8  27.43  33.  33.  -0.60    -9.03  19.20     -0.08    -7.45     8.40
  1  9  30.86  33.  33.  -0.57    -8.50  18.27     -0.08    -6.98     7.87
  1 10  34.29  33.  33.  -0.55    -7.67  18.80     -0.07    -6.39     8.06
  1 11  37.71  33.  33.  -0.46    -6.35  15.75     -0.06    -5.27     6.64
  1 12  41.14  33.  33.  -0.34    -4.64  11.62     -0.04    -3.83     4.83
  1 13  44.57  33.  33.  -0.19    -2.52   6.36     -0.02    -2.07     2.61
  1 14  48.00  33.  33.   0.00     0.00   0.00      0.00     0.00     0.00


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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	33.	33.	-0.24	-2.66	7.50	-0.02	-2.08	2.62
1	2	6.86	33.	33.	-0.43	-4.90	13.68	-0.04	-3.85	4.85
1	3	10.29	33.	33.	-0.58	-6.69	18.50	-0.06	-5.28	6.65
1	4	13.71	33.	33.	-0.69	-8.06	22.03	-0.07	-6.39	8.06
1	5	17.14	33.	33.	-0.72	-8.95	21.38	-0.08	-6.98	7.87
1	6	20.57	33.	33.	-0.75	-9.50	22.44	-0.08	-7.44	8.39
1	7	24.00	33.	33.	-0.76	-9.66	22.68	-0.08	-7.60	8.57
1	8	27.43	33.	33.	-0.75	-9.50	22.45	-0.08	-7.45	8.40
1	9	30.86	33.	33.	-0.72	-8.95	21.38	-0.08	-6.98	7.87
1	10	34.29	33.	33.	-0.69	-8.05	22.03	-0.07	-6.39	8.06
1	11	37.71	33.	33.	-0.58	-6.68	18.49	-0.06	-5.27	6.64
1	12	41.14	33.	33.	-0.43	-4.88	13.66	-0.04	-3.83	4.83
1	13	44.57	33.	33.	-0.24	-2.65	7.49	-0.02	-2.07	2.61
1	14	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	33.	33.	-0.32	-3.44	9.98	-0.03	-2.66	3.42
1	2	6.86	33.	33.	-0.58	-6.32	18.21	-0.06	-4.91	6.32
1	3	10.29	33.	33.	-0.79	-8.63	24.62	-0.08	-6.74	8.67
1	4	13.71	33.	33.	-0.93	-10.39	29.32	-0.10	-8.16	10.51
1	5	17.14	33.	33.	-0.97	-11.57	28.45	-0.11	-8.91	10.26
1	6	20.57	33.	33.	-1.02	-12.27	29.85	-0.12	-9.50	10.95
1	7	24.00	33.	33.	-1.02	-12.48	30.17	-0.12	-9.70	11.18
1	8	27.43	33.	33.	-1.02	-12.28	29.86	-0.12	-9.51	10.95
1	9	30.86	33.	33.	-0.97	-11.57	28.45	-0.11	-8.91	10.26
1	10	34.29	33.	33.	-0.93	-10.39	29.31	-0.10	-8.15	10.50
1	11	37.71	33.	33.	-0.79	-8.62	24.60	-0.08	-6.72	8.66
1	12	41.14	33.	33.	-0.58	-6.30	18.18	-0.06	-4.89	6.30
1	13	44.57	33.	33.	-0.32	-3.43	9.97	-0.03	-2.65	3.41
1	14	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

YIELD STRESS — TOTAL POSITIVE ,(ksi) — TOTAL NEGATIVE ,(ksi)
SP IN D FROM --------- CONCR. STEEL BEAM — CONCR. STEEL BEAM

NO	NO	L SUPT (ft)	Fy (ksi) TOP	Fy (ksi) BOT	TOP TOP	TOP BOT	TOP TOP	TOP BOT	TOP TOP	TOP BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	33.	33.	-0.31	-2.60	8.78	-0.02	-1.82	2.22
1	2	6.86	33.	33.	-0.56	-4.78	15.99	-0.03	-3.36	4.10
1	3	10.29	33.	33.	-0.75	-6.51	21.58	-0.04	-4.62	5.63
1	4	13.71	33.	33.	-0.88	-7.83	25.63	-0.05	-5.59	6.82
1	5	17.14	33.	33.	-0.92	-8.75	24.85	-0.06	-6.09	6.66
1	6	20.57	33.	33.	-0.96	-9.26	26.01	-0.06	-6.50	7.10
1	7	24.00	33.	33.	-0.97	-9.42	26.24	-0.06	-6.64	7.25
1	8	27.43	33.	33.	-0.96	-9.27	26.01	-0.06	-6.51	7.11
1	9	30.86	33.	33.	-0.92	-8.76	24.85	-0.06	-6.10	6.66
1	10	34.29	33.	33.	-0.88	-7.82	25.63	-0.05	-5.59	6.82
1	11	37.71	33.	33.	-0.75	-6.50	21.57	-0.04	-4.61	5.62
1	12	41.14	33.	33.	-0.56	-4.76	15.97	-0.03	-3.35	4.09
1	13	44.57	33.	33.	-0.31	-2.59	8.77	-0.02	-1.81	2.21
1	14	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	33.	33.	-0.26	-3.26	8.48	-0.03	-2.66	3.42
1	2	6.86	33.	33.	-0.47	-6.00	15.49	-0.06	-4.91	6.32
1	3	10.29	33.	33.	-0.63	-8.20	20.97	-0.08	-6.74	8.67
1	4	13.71	33.	33.	-0.74	-9.88	25.02	-0.10	-8.16	10.51
1	5	17.14	33.	33.	-0.78	-10.96	24.29	-0.11	-8.91	10.26
1	6	20.57	33.	33.	-0.81	-11.63	25.53	-0.12	-9.50	10.95
1	7	24.00	33.	33.	-0.82	-11.85	25.83	-0.12	-9.70	11.18
1	8	27.43	33.	33.	-0.81	-11.64	25.54	-0.12	-9.51	10.95
1	9	30.86	33.	33.	-0.78	-10.96	24.30	-0.11	-8.91	10.26
1	10	34.29	33.	33.	-0.74	-9.88	25.01	-0.10	-8.15	10.50
1	11	37.71	33.	33.	-0.63	-8.18	20.96	-0.08	-6.72	8.66
1	12	41.14	33.	33.	-0.47	-5.98	15.47	-0.06	-4.89	6.30
1	13	44.57	33.	33.	-0.26	-3.25	8.47	-0.03	-2.65	3.41
1	14	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.43	33.	33.	-0.24	-2.42	7.28	-0.02	-1.82	2.22
1	2	6.86	33.	33.	-0.44	-4.45	13.27	-0.03	-3.36	4.10
1	3	10.29	33.	33.	-0.59	-6.08	17.93	-0.04	-4.62	5.63
1	4	13.71	33.	33.	-0.69	-7.31	21.33	-0.05	-5.59	6.82
1	5	17.14	33.	33.	-0.72	-8.15	20.69	-0.06	-6.09	6.66
1	6	20.57	33.	33.	-0.75	-8.63	21.69	-0.06	-6.50	7.10

1	7	24.00	33.	33.	-0.76	-8.78	21.90	-0.06	-6.64	7.25
1	8	27.43	33.	33.	-0.75	-8.64	21.69	-0.06	-6.51	7.11
1	9	30.86	33.	33.	-0.72	-8.15	20.69	-0.06	-6.10	6.66
1	10	34.29	33.	33.	-0.69	-7.31	21.33	-0.05	-5.59	6.82
1	11	37.71	33.	33.	-0.59	-6.07	17.92	-0.04	-4.61	5.62
1	12	41.14	33.	33.	-0.44	-4.44	13.26	-0.03	-3.35	4.09
1	13	44.57	33.	33.	-0.24	-2.41	7.27	-0.02	-1.81	2.21
1	14	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	14.00	162.	-5.4	108.	-1.0	479.	-1.3	0.	0.0	-7.75
											1.29
	LB			5.4		2.7		10.8		0.0	18.99
											10.84
	RT			-5.2		-1.0		-1.4		0.0	-7.65
											1.39
	RB			4.7		2.4		9.5		0.0	16.54
											9.47
1	LT	34.00	162.	-5.2	108.	-1.0	479.	-1.4	0.	0.0	-7.65
											1.39
	LB			4.7		2.4		9.5		0.0	16.54
											9.47
	RT			-5.4		-1.0		-1.3		0.0	-7.75
											1.29
	RB			5.4		2.7		10.8		0.0	18.98
											10.84

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	14.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	20.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	7.50	0.3125
3	14.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 48.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1]) [3]= GIRDER D. [3]/(12*[1]) (in)	<=>	LIMITATION 1/25 1/30	STATUS
1	14.00	6.75	32.90	[2]=	41.06	0.07128	> 0.04000	OK
				[3]=	32.90	0.05712	> 0.03300	OK
2	20.00	6.75	32.90	[2]=	41.06	0.07128	> 0.04000	OK
				[3]=	32.90	0.05712	> 0.03300	OK
3	14.00	6.75	32.90	[2]=	41.06	0.07128	> 0.04000	OK
				[3]=	32.90	0.05712	> 0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	2Dc/tw [4]	[5]	[6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	1	3.43	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	2	6.86	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	3	10.29	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	4	13.71	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	5	17.14	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	6	20.57	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	7	24.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	8	27.43	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	9	30.86	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	10	34.29	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	11	37.71	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	12	41.14	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	13	44.57	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	14	48.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0

NOTE:
[1] 0 - No long. stiffeners

```
          1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE  1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
                 ********************************

 SP IN D FROM  Lo Co       Web                   Web
 NO NO L SUPT  ng mp depth thick     2Dc/tw      Category
              [1]                  [2]   [3]
-------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.13 168.97    2
  1  1   3.43  0  1  31.42 0.550  57.13 168.97    2
  1  2   6.86  0  1  31.42 0.550  57.13 168.97    2
  1  3  10.29  0  1  31.42 0.550  57.13 168.97    2
  1  4  13.71  0  1  31.42 0.550  57.13 168.97    2
  1  5  17.14  0  1  31.42 0.550  60.99 168.97    2
  1  6  20.57  0  1  31.42 0.550  60.99 168.97    2
  1  7  24.00  0  1  31.42 0.550  60.99 168.97    2
  1  8  27.43  0  1  31.42 0.550  60.99 168.97    2
  1  9  30.86  0  1  31.42 0.550  60.99 168.97    2
  1 10  34.29  0  1  31.42 0.550  57.13 168.97    2
  1 11  37.71  0  1  31.42 0.550  57.13 168.97    2
  1 12  41.14  0  1  31.42 0.550  57.13 168.97    2
  1 13  44.57  0  1  31.42 0.550  57.13 168.97    2
  1 14  48.00  0  1  31.42 0.550  57.13 168.97    2


-------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  For non-composite sections, calculate Dc and 2*Dc/tw
      [3]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section

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 TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
                ************************

 SP NO D FROM
 NO NO L SUPT bf/2tf [1]   bf   [2]    tf    [3]   Iyc/Iyt FLAG
        (ft)              (in)  (in)  (in)   (in)
 ---------------------------------------------------------------

  1  0   0.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  1   3.43   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  2   6.86   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
```

```
  1  3  10.29   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  4  13.71   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  5  17.14   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  6  20.57   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  7  24.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  27.43   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  30.86   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  34.29   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  37.71   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  41.14   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  44.57   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  48.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0   6.5   0.000   4.612   2.485   0.359   1.670
                       0.000   0.000   0.000   0.000   1.000
    2      6.5  27.2   4.612   9.336   8.676   8.016   1.000
                       0.000   0.000   0.000   0.000   1.000
    3     27.2  48.0   0.000   9.336   6.663   3.991   1.000
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
         (ft)                         CAT  (ft)     (ft)     (ft)    CAT CAT
-------------------------------------------------------------------------------
```

1	0	0.00	1	7.77	11.26	19.84	0	6.50	7.09	26.62	0	0
1	1	3.43	1	7.77	11.26	19.84	0	6.50	7.09	26.62	0	0
1	2	6.86	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	3	10.29	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	4	13.71	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	5	17.14	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	6	20.57	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	7	24.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	8	27.43	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	9	30.86	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	10	34.29	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	11	37.71	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	12	41.14	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	13	44.57	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	14	48.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.67	33.00	33.00	33.00
1	1	3.43	1	1.000	1.000	1.67	33.00	33.00	33.00
1	2	6.86	1	1.000	1.000	1.00	33.00	26.07	26.07
1	3	10.29	1	1.000	1.000	1.00	33.00	26.07	26.07
1	4	13.71	1	1.000	1.000	1.00	33.00	26.07	26.07
1	5	17.14	1	1.000	1.000	1.00	33.00	25.98	25.98
1	6	20.57	1	1.000	1.000	1.00	33.00	25.98	25.98
1	7	24.00	1	1.000	1.000	1.00	33.00	25.98	25.98
1	8	27.43	1	1.000	1.000	1.00	33.00	25.98	25.98
1	9	30.86	1	1.000	1.000	1.00	33.00	25.98	25.98
1	10	34.29	1	1.000	1.000	1.00	33.00	26.07	26.07
1	11	37.71	1	1.000	1.000	1.00	33.00	26.07	26.07
1	12	41.14	1	1.000	1.000	1.00	33.00	26.07	26.07
1	13	44.57	1	1.000	1.000	1.00	33.00	26.07	26.07
1	14	48.00	1	1.000	1.000	1.00	33.00	26.07	26.07

Note: In the positive moment region, the result is for DL case

In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0	0.00	1	6.34	17.25
1	1	3.43	1	6.34	17.25
1	2	6.86	1	6.34	17.25
1	3	10.29	1	6.34	17.25
1	4	13.71	1	6.34	17.25
1	5	17.14	1	8.17	17.38
1	6	20.57	1	8.17	17.38
1	7	24.00	1	8.17	17.38
1	8	27.43	1	8.17	17.38
1	9	30.86	1	8.17	17.38
1	10	34.29	1	6.34	17.25
1	11	37.71	1	6.34	17.25
1	12	41.14	1	6.34	17.25
1	13	44.57	1	6.34	17.25
1	14	48.00	1	6.34	17.25

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	0	0		0	0
1	1	3.43	1	0	0	0		0	0
1	2	6.86	1	0	2	0		2	0
1	3	10.29	1	0	2	0		2	0
1	4	13.71	1	0	2	0		2	0
1	5	17.14	1	0	2	0		2	0
1	6	20.57	1	0	2	0		2	0
1	7	24.00	1	0	2	0		2	0
1	8	27.43	1	0	2	0		2	0
1	9	30.86	1	0	2	0		2	0
1	10	34.29	1	0	2	0		2	0
1	11	37.71	1	0	2	0		2	0
1	12	41.14	1	0	2	0		2	0
1	13	44.57	1	0	2	0		2	0
1	14	48.00	1	0	2	0		2	0

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1

[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.43	0.0	19.8	2.6	33.0	2.6	33.0	2.6	33.0	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	2		6.86	0.0	19.8	4.8	33.0	4.8	33.0	4.8	26.1	0
				0.0	-	4.8	-	4.8	33.0	-	-	0
1	3		10.29	0.0	19.8	6.6	33.0	6.6	33.0	6.6	26.1	0
				0.0	-	6.6	-	6.6	33.0	-	-	0
1	4		13.71	0.0	19.8	8.0	33.0	8.0	33.0	8.0	26.1	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	5		17.14	0.0	19.8	8.7	33.0	8.7	33.0	8.7	26.0	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	6		20.57	0.0	19.8	9.3	33.0	9.3	33.0	9.3	26.0	0
				0.0	-	8.4	-	8.4	33.0	-	-	0
1	7		24.00	0.0	19.8	9.5	33.0	9.5	33.0	9.5	26.0	0
				0.0	-	8.5	-	8.5	33.0	-	-	0
1	8		27.43	0.0	19.8	9.3	33.0	9.3	33.0	9.3	26.0	0
				0.0	-	8.4	-	8.4	33.0	-	-	0
1	9		30.86	0.0	19.8	8.7	33.0	8.7	33.0	8.7	26.0	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	10		34.29	0.0	19.8	8.0	33.0	8.0	33.0	8.0	26.1	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	11		37.71	0.0	19.8	6.6	33.0	6.6	33.0	6.6	26.1	0
				0.0	-	6.6	-	6.6	33.0	-	-	0
1	12		41.14	0.0	19.8	4.8	33.0	4.8	33.0	4.8	26.1	0
				0.0	-	4.8	-	4.8	33.0	-	-	0
1	13		44.57	0.0	19.8	2.6	33.0	2.6	33.0	2.6	26.1	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	14		48.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	26.1	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.43	0.000	0.079	0.079	0.079	0.079	2
					0.079		0.079	3
1	2	6.86	0.000	0.147	0.147	0.186	0.186	4
					0.147		0.147	3
1	3	10.29	0.000	0.201	0.201	0.255	0.255	4
					0.201		0.201	3
1	4	13.71	0.000	0.244	0.244	0.309	0.309	4
					0.244		0.244	3
1	5	17.14	0.000	0.265	0.265	0.336	0.336	4
					0.237		0.237	3
1	6	20.57	0.000	0.282	0.282	0.358	0.358	4
					0.253		0.253	3
1	7	24.00	0.000	0.288	0.288	0.366	0.366	4
					0.259		0.259	3
1	8	27.43	0.000	0.283	0.283	0.359	0.359	4
					0.254		0.254	3
1	9	30.86	0.000	0.265	0.265	0.336	0.336	4
					0.238		0.238	3
1	10	34.29	0.000	0.244	0.244	0.308	0.308	4
					0.244		0.244	3
1	11	37.71	0.000	0.201	0.201	0.254	0.254	4
					0.201		0.201	3
1	12	41.14	0.000	0.146	0.146	0.185	0.185	4
					0.146		0.146	3
1	13	44.57	0.000	0.079	0.079	0.100	0.100	4
					0.079		0.079	3
1	14	48.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1945.3	-	-	0.0	19.8	-	-	0--0
1	1		3.43	0	404.1	1945.3	-	-	0.0	19.8	-	-	0--0
1	2		6.86	0	736.2	1945.3	-	-	0.0	19.8	-	-	0--0
1	3		10.29	0	994.1	1945.3	-	-	0.0	19.8	-	-	0--0
1	4		13.71	0	1182.0	1945.3	-	-	0.0	19.8	-	-	0--0
1	5		17.14	0	1315.0	2092.6	-	-	0.0	19.8	-	-	0--0
1	6		20.57	0	1377.4	2092.6	-	-	0.0	19.8	-	-	0--0
1	7		24.00	0	1390.7	2092.6	-	-	0.0	19.8	-	-	0--0
1	8		27.43	0	1377.7	2092.6	-	-	0.0	19.8	-	-	0--0
1	9		30.86	0	1315.1	2092.6	-	-	0.0	19.8	-	-	0--0
1	10		34.29	0	1181.8	1945.3	-	-	0.0	19.8	-	-	0--0
1	11		37.71	0	993.7	1945.3	-	-	0.0	19.8	-	-	0--0
1	12		41.14	0	735.5	1945.3	-	-	0.0	19.8	-	-	0--0
1	13		44.57	0	403.7	1945.3	-	-	0.0	19.8	-	-	0--0
1	14		48.00	0	0.0	1945.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		3.43	0	0.208				0.208	1
1	2		6.86	0	0.378				0.378	1
1	3		10.29	0	0.511				0.511	1
1	4		13.71	0	0.608				0.608	1
1	5		17.14	0	0.628				0.628	1
1	6		20.57	0	0.658				0.658	1
1	7		24.00	0	0.665				0.665	1

1	8	27.43	0	0.658	0.658	1
1	9	30.86	0	0.628	0.628	1
1	10	34.29	0	0.608	0.608	1
1	11	37.71	0	0.511	0.511	1
1	12	41.14	0	0.378	0.378	1
1	13	44.57	0	0.208	0.208	1
1	14	48.00	0	0.000	0.000	1

```
NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
      [2] = (PHI)f * Fnc for comp. flange of composite
            sections in positive flexure (Eq. 6.10.7.2.1-1)
          = (PHI)f * Rh * Fyt for tension flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
      [3] = 0.6*Fyt for composite sections in positive flexure
            or tension flange for composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
          = 0.6*Fyc for comp. flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
      [4] = (PHI)f * Fnt for non-compact tension flange
            of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
          = (PHI)f * Fnc for comp. flange of composite sections
            in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

      The governing number is listed as below.
        1 = (Mu+1/3flSxt) / [1]
        2 = fbu / [2]
        3 = fl / [3]
        4 = (fbu + 1/3fl) / [4]
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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.43	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.86	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	10.29	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	13.71	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	17.14	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	20.57	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	24.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	27.43	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	30.86	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	34.29	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	37.71	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	41.14	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	44.57	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	48.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
            = 5 for unstiffened beams and girders
            = 5 for stiffened girders when (d0/D) > 3, or
                                           (d0/D) > (260/(D/tw))**2
```

[2] = 1.12 * sqrt(Ek / Fyw)

[3] = 1.40 * sqrt(Ek / Fyw)

[4] C = ratio of nominal shear resistance and plastic shear force, Vp

For D/tw < [2], C = 1.0

For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
..... AASHTO LRFD Eq.6.10.9.3.2-7

** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered stiffened
if (d0/D) < 3 without long. stiffener, or
if (d0/D) > 3 with both transv. and long. stiffeners.
For handling requirement, (d0/D) > (260/(D/tw))**2

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot count on post-buckling shear resistance from tension-field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPACITY, [2]	CATEGORY	OVERALL WEB
			S T R E N G T H C A T E G O R Y; SEE NOTE					
1	0	0.00	0	124.9	<	330.8	0	0
1	1	3.43	0	111.2	<	330.8	0	0
1	2	6.86	0	97.4	<	330.8	0	0
1	3	10.29	0	84.0	<	330.8	0	0
1	4	13.71	0	70.8	<	330.8	0	0
1	5	17.14	0	57.6	<	330.8	0	0
1	6	20.57	0	44.9	<	330.8	0	0
1	7	24.00	0	33.1	<	330.8	0	0
1	8	27.43	0	45.0	<	330.8	0	0
1	9	30.86	0	57.6	<	330.8	0	0
1	10	34.29	0	70.8	<	330.8	0	0
1	11	37.71	0	84.1	<	330.8	0	0
1	12	41.14	0	97.5	<	330.8	0	0
1	13	44.57	0	111.1	<	330.8	0	0
1	14	48.00	0	124.8	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN D NO	L FROM SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	124.91	330.76	0	
1	1	3.43	33.	111.19	330.76	0	
1	2	6.86	33.	97.45	330.76	0	
1	3	10.29	33.	84.04	330.76	0	
1	4	13.71	33.	70.77	330.76	0	
1	5	17.14	33.	57.56	330.76	0	
1	6	20.57	33.	44.93	330.76	0	
1	7	24.00	33.	33.07	330.76	0	
1	8	27.43	33.	45.00	330.76	0	
1	9	30.86	33.	57.63	330.76	0	
1	10	34.29	33.	70.85	330.76	0	
1	11	37.71	33.	84.11	330.76	0	
1	12	41.14	33.	97.52	330.76	0	
1	13	44.57	33.	111.09	330.76	0	
1	14	48.00	33.	124.80	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK

```
                    ************************

    SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                  ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
    NO NO L SUPT                                          [3]  TBC
            (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi) (ksi)
    ---------------------------------------------------------------

     1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
     1  1   3.43   2.7    -     31.4     7.5  31.4   -     -   00-
     1  2   6.86   4.9    -     31.4    13.7  31.4   -     -   00-
     1  3  10.29   6.7    -     31.4    18.5  31.4   -     -   00-
     1  4  13.71   8.1    -     31.4    22.0  31.4   -     -   00-
     1  5  17.14   9.0    -     31.4    21.4  31.4   -     -   00-
     1  6  20.57   9.5    -     31.4    22.4  31.4   -     -   00-
     1  7  24.00   9.7    -     31.4    22.7  31.4   -     -   00-
     1  8  27.43   9.5    -     31.4    22.4  31.4   -     -   00-
     1  9  30.86   9.0    -     31.4    21.4  31.4   -     -   00-
     1 10  34.29   8.1    -     31.4    22.0  31.4   -     -   00-
     1 11  37.71   6.7    -     31.4    18.5  31.4   -     -   00-
     1 12  41.14   4.9    -     31.4    13.7  31.4   -     -   00-
     1 13  44.57   2.7    -     31.4     7.5  31.4   -     -   00-
     1 14  48.00   0.0    -     31.4     0.0  31.4   -     -   00-


    ---------------------------------------------------------------
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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   3.43    TR     -0.2 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
```

```
 1  2   6.86    TR     -0.4 A B B^C^C D E E^  TR      3.4 A B B^C^C D E
 1  3  10.29    TR     -0.5 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1  4  13.71    TR     -0.6 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1     14.00  SCG
 1  5  17.14    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1  6  20.57    TR     -0.8 A B B^C^C D E E^  TR      5.2 A B B^C^C D
 1  7  24.00    TR     -0.8 A B B^C^C D E E^  TR      5.3 A B B^C^C D
 1  8  27.43    TR     -0.8 A B B^C^C D E E^  TR      5.2 A B B^C^C D
 1  9  30.86    TR     -0.7 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1     34.00  SCG
 1 10  34.29    TR     -0.6 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 11  37.71    TR     -0.5 A B B^C^C D E E^  TR      4.4 A B B^C^C D E
 1 12  41.14    TR     -0.4 A B B^C^C D E E^  TR      3.4 A B B^C^C D E
 1 13  44.57    TR     -0.2 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
 1 14  48.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                   ************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                  ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ki)    CATEGORY             (ki)    CATEGORY
 ---------------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   3.43    TR     -0.1 A B B^C^C D E E^  TR      1.0 A B B^C^C D E E^
 1  2   6.86    TR     -0.2 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1  3  10.29    TR     -0.3 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1  4  13.71    TR     -0.3 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1     14.00  SCG
 1  5  17.14    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1  6  20.57    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1  7  24.00    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1  8  27.43    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1  9  30.86    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1     34.00  SCG
 1 10  34.29    TR     -0.3 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1 11  37.71    TR     -0.3 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1 12  41.14    TR     -0.2 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1 13  44.57    TR     -0.1 A B B^C^C D E E^  TR      1.0 A B B^C^C D E E^
 1 14  48.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
```

Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	30.1	.29038E-01	0.9	TR 3.09	3	10.61
1	1	3.43	28.2	.29038E-01	0.8	TR 3.09	3	11.33
1	2	6.86	26.6	.29038E-01	0.8	TR 3.09	3	12.03
1	3	10.29	24.9	.29038E-01	0.7	TR 3.09	3	12.85
1	4	13.71	23.4	.29038E-01	0.7	TR 3.09	3	13.68
1	5	17.14	23.6	.28154E-01	0.7	TR 3.09	3	13.95
1	6	20.57	23.8	.28154E-01	0.7	TR 3.09	3	13.84
1	7	24.00	23.8	.28154E-01	0.7	TR 3.09	3	13.84
1	8	27.43	23.8	.28154E-01	0.7	TR 3.09	3	13.84
1	9	30.86	23.6	.28154E-01	0.7	TR 3.09	3	13.95
1	10	34.29	23.4	.29038E-01	0.7	TR 3.09	3	13.68
1	11	37.71	24.9	.29038E-01	0.7	TR 3.09	3	12.85
1	12	41.14	26.6	.29038E-01	0.8	TR 3.09	3	12.03
1	13	44.57	28.2	.29038E-01	0.8	TR 3.09	3	11.33
1	14	48.00	30.1	.29038E-01	0.9	TR 3.09	3	10.61

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees

Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	15.1	.29038E-01	0.4	TR 1.36	3	9.36
1	1	3.43	14.1	.29038E-01	0.4	TR 1.36	3	9.99
1	2	6.86	13.3	.29038E-01	0.4	TR 1.36	3	10.60
1	3	10.29	12.4	.29038E-01	0.4	TR 1.36	3	11.33
1	4	13.71	11.7	.29038E-01	0.3	TR 1.36	3	12.06
1	5	17.14	11.8	.28154E-01	0.3	TR 1.36	3	12.30
1	6	20.57	11.9	.28154E-01	0.3	TR 1.36	3	12.20
1	7	24.00	11.9	.28154E-01	0.3	TR 1.36	3	12.20
1	8	27.43	11.9	.28154E-01	0.3	TR 1.36	3	12.20
1	9	30.86	11.8	.28154E-01	0.3	TR 1.36	3	12.30
1	10	34.29	11.7	.29038E-01	0.3	TR 1.36	3	12.06
1	11	37.71	12.4	.29038E-01	0.4	TR 1.36	3	11.33
1	12	41.14	13.3	.29038E-01	0.4	TR 1.36	3	10.60
1	13	44.57	14.1	.29038E-01	0.4	TR 1.36	3	9.99
1	14	48.00	15.1	.29038E-01	0.4	TR 1.36	3	9.36

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under

```
                consideration.
              = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
        Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
        Ffat = radial fatigue shear range per unit length
             = Ffat2 = Frc/w for skews exceeding 20 degrees
        Frc = 25.0 kips for both exterior and interior girders
        w = the effective length of the deck
          = 48 inches except at end supports where w is taken
            as 24 inches

    [4] Zr = allowable range of horizontal shear, in Kips, on an individual
             connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

        * Default ALPHA value based on 7/8" diameter and input road type


    [5] No. of shear connectors per transverse section. This value is from
        input data type 12032.

    [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

        ** The connector spacing shown here is calculated based upon
           AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
           where ADTT Single Lane= 1140 & No. of Cycles=   31207500
       *** If ADTT Single Lane greater than or equal to 960, refer
           to Fatigue I Table.
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ***************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  7  24.00   1 1209.3  1204.9           87         >  63
      1 14  48.00   1 1209.3  1204.9           87         >  63


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
           shall be checked to ensure that adequate connectors
           are provided for Strength Limit State.

      [2] The number of shear connectors required equal or exceed
           the number given by the formula :

                 N1 (N2) = P / Qr  - - - - - - (Eq. 6.10.10.4.1-2)
           Where
             N1 = Number of connectors between points of maximum
                  positive moment and each adjacent point of 0.0 moment
             N2 = number of connectors between each adjacent point of
                  0.0 moment and the centerline of an interior support
             P = total nominal horizontal shear force
```

```
Qr = factored shear resistance of one shear connector
   = (Phi)sc Qn    - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of
     maximum positive moment and each adjacent point of 0.0 moment
     shall be the lesser either:

         P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)
     or
       P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)
     Where
       f'c = specified 28-day compressive strength of the concrete
       bs = effective width of the slab
       bfc = width of compression flange
       bft = width of tension flange
       ts = slab thickness
       Fyw = specified minimum yield strength of the web
       Fyt = specified minimum yield strength of the tension flange
       Fyc = specified minimum yield strength of the compression flange
       D = web depth
       tft = thickness of tension flange
       tfc = thickness of compression flange
       tw = web thickness


[4] For continuous-span composite sections, the total horizontal
     shear force between each adjacent point of 0.0 moment
     and the centerline of an interior support shall be taken as:

         P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)
     Where
       bs = effective width of the concrete deck
       ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
    If ADTT Single Lane is less than 960, the pitch should use Fatigue II.
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 TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
                 ****************************************

                SPAN     CURRENT SPAN     REQUIRED
                 NO.     FROM      TO        PITCH
                         (ft)     (ft)        (in)
               ----------------------------------

                   1     0.000    24.000    10.000

                   1    24.000    48.000    10.000

               ----------------------------------
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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                 **********************************

                SPAN     CURRENT SPAN    T
                 NO.     FROM      TO    C
                         (ft)     (ft)   R
               ---------------------------

                   1       0.0      48.0 C

               ---------------------------
```

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1945.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	3.43	1945.3	52.2	34.8	165.5	0.0	8.19	HL-93	6.32	0
1	2	6.86	1945.3	96.5	64.2	300.0	0.0	4.28	HL-93	3.30	0
1	3	10.29	1945.3	132.4	88.3	402.4	0.0	3.05	HL-93	2.35	0
1	4	13.71	1945.3	160.3	107.0	474.7	0.0	2.49	HL-93	1.92	0
1	5	17.14	2092.6	180.3	120.3	525.6	0.0	2.39	HL-93	1.85	0
1	6	20.57	2092.6	192.4	128.4	546.2	0.0	2.27	HL-93	1.75	0
1	7	24.00	2092.6	196.5	131.0	548.7	0.0	2.24	HL-93	1.73	0
1	8	27.43	2092.6	192.6	128.4	546.2	0.0	2.27	HL-93	1.75	0
1	9	30.86	2092.6	180.4	120.3	525.6	0.0	2.39	HL-93	1.85	0
1	10	34.29	1945.3	160.2	107.0	474.7	0.0	2.49	HL-93	1.92	0
1	11	37.71	1945.3	132.0	88.3	402.4	0.0	3.05	HL-93	2.35	0
1	12	41.14	1945.3	96.0	64.2	300.0	0.0	4.28	HL-93	3.30	0
1	13	44.57	1945.3	52.0	34.8	165.5	0.0	8.19	HL-93	6.32	0
1	14	48.00	1945.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

Please read NOTES on the following page

1

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD STRESS TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. NEG. STEEL TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. RATING LIVE LOAD FACTOR TYPE
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68
1	1	3.43	31.4	-2.08 2.62	-0.45 3.75	0.00 0.00	7.66	HL-93 5.89
1	2	6.86	31.4	-3.85 4.85	-0.81 6.79	0.00 0.00	3.90	HL-93 3.00
1	3	10.29	31.4	-5.28 6.65	-1.08 9.11	0.00 0.00	2.71	HL-93 2.09
1	4	13.71	31.4	-6.39 8.06	-1.28 10.75	0.00 0.00	2.17	HL-93 1.67
1	5	17.14	31.4	-6.98 7.87	-1.52 10.40	0.00 0.00	2.26	HL-93 1.74
1	6	20.57	31.4	-7.44 8.39	-1.58 10.80	0.00 0.00	2.13	HL-93 1.63
1	7	24.00	31.4	-7.60 8.57	-1.59 10.85	0.00 0.00	2.10	HL-93 1.61
1	8	27.43	31.4	-7.45 8.40	-1.58 10.80	0.00 0.00	2.12	HL-93 1.63
1	9	30.86	31.4	-6.98 7.87	-1.52 10.40	0.00 0.00	2.26	HL-93 1.74
1	10	34.29	31.4	-6.39 8.06	-1.28 10.75	0.00 0.00	2.17	HL-93 1.67
1	11	37.71	31.4	-5.27 6.64	-1.08 9.11	0.00 0.00	2.71	HL-93 2.09
1	12	41.14	31.4	-3.83 4.83	-0.81 6.79	0.00 0.00	3.90	HL-93 3.00
1	13	44.57	31.4	-2.07 2.61	-0.45 3.75	0.00 0.00	7.67	HL-93 5.90
1	14	48.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68

Please read NOTES on the following page

NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	48.00	24.00	STRENGTH L.S.	2.24	1.73	HL-93
1	48.00	24.00	SERVICEBILITY	2.10	1.61	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.099
Inventory= 1.615

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	48.00	24.00	STRENGTH L.S.	3.10	536.68
1	48.00	24.00	SERVICEBILITY	2.75	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.752

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	16.4	10.9	50.9	0.0	4.29	HL-93	3.31
1	1	3.43	330.8	14.1	9.4	45.9	-1.7	4.84	HL-93	3.73
1	2	6.86	330.8	11.6	7.8	41.1	-3.9	5.50	HL-93	4.24
1	3	10.29	330.8	9.3	6.2	36.3	-6.7	6.32	HL-93	4.88
1	4	13.71	330.8	7.0	4.7	31.7	-9.6	7.38	HL-93	5.69

1	5	17.14	330.8	4.7	3.1	27.0	-12.6	8.78	HL-93	6.77
1	6	20.57	330.8	2.4	1.6	22.7	-15.7	9.99	HL-93	8.19
1	7	24.00	330.8	0.0	0.0	18.9	-18.9	9.99	HL-93	9.99
1	8	27.43	330.8	-2.4	-1.6	15.7	-22.7	9.99	HL-93	8.18
1	9	30.86	330.8	-4.7	-3.1	12.6	-27.0	8.78	HL-93	6.77
1	10	34.29	330.8	-7.0	-4.7	9.6	-31.7	7.37	HL-93	5.69
1	11	37.71	330.8	-9.4	-6.2	6.7	-36.3	6.32	HL-93	4.88
1	12	41.14	330.8	-11.7	-7.8	3.9	-41.1	5.50	HL-93	4.24
1	13	44.57	330.8	-14.0	-9.4	1.7	-45.9	4.84	HL-93	3.73
1	14	48.00	330.8	-16.3	-10.9	0.0	-50.9	4.30	HL-93	3.31

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	48.00	0.00	STRENGTH L.S.	4.29	3.31	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.294
Inventory= 3.313

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	48.00	0.00	STRENGTH L.S.	5.42	54.45

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.416

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	24.00	3	0.425	-0.27	HL-93	2.70
		4	0.433	-0.27		2.65

NOTE: [1] "-" indicates downward deflection

```
[2] RF * (live load defl.) = span length/800, therefore

    RF = L/(800 * LL Defl.), if = 99, No rating
```

1

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95 NB Span 3 Exterior Fatigue	8/19/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Exterior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	12	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.15	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
48.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	PLATE GIRDER WEB THICK. (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.0000	7.50	0.3125		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	14.00				33.	33.	33.
2	2	2	20.00				33.	33.	33.
3	1	1	14.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.57	2 2 2 2 2
1	0.00	0	0.00	0.39	0.00	0.00	0 0 0 0 0	0.55	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

1

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.55	0.00	48.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	6.50
2	2	Diaphragm	0.000	0.00	0.00	0.14	27.25
3	1	Railing	0.087	0.00	48.00	0.00	0.00
4	1	Curb	0.076	0.00	48.00	0.00	0.00
5	0	Pavement	0.292	0.00	48.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN.	DIA. SEC (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	6.50	33.0	33.0	6.50
6.50	27.25	33.0	33.0	20.75
27.25	48.00	33.0	33.0	20.75

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

FACTORS USED BY LRFD

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.554       0.1214     0.6754


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      48.00

     1   DC2     0.087       0.000     48.000

     2   DC2     0.076       0.000     48.000


   ----------------------------------------
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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1165.5	43.2	174.8	35.0	209.79

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.571	0.571	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.550				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.391				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	4.00	5900.0	32.9	16.45	358.7	358.7
1	2	8.00	5900.0	32.9	16.45	358.7	358.7
1	3	12.00	5900.0	32.9	16.45	358.7	358.7
1	4	16.00	6506.7	32.9	15.71	414.1	371.8
1	5	20.00	6506.7	32.9	15.71	414.1	371.8
1	6	24.00	6506.7	32.9	15.71	414.1	371.8
1	7	28.00	6506.7	32.9	15.71	414.1	371.8
1	8	32.00	6506.7	32.9	15.71	414.1	371.8
1	9	36.00	5900.0	32.9	16.45	358.7	358.7
1	10	40.00	5900.0	32.9	16.45	358.7	358.7
1	11	44.00	5900.0	32.9	16.45	358.7	358.7
1	12	48.00	5900.0	32.9	16.45	358.7	358.7

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 ----------------------------------------------------------------------
  NOTE: For rolled section, the 5th column is the depth  d (inch)
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 TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
               ****************************************


              MOMENT OF    Q/Ix       ELASTIC SECTION MODULUS,(in**3)
 SP IN D FROM  INERTIA -------------- --------------------------------
 NO NO L SUPT --------- Q=1ST. MOMENT      STEEL       CONCRETE(SLAB)
       (ft)    Ix         OF INERTIA ---------------- ---------------
              (in**4)     (1/in)        BOT.    TOP.         TOP.
 ----------------------------------------------------------------------

  1  0   0.00   11329.8   0.022745     476.1     1244.6      656.3
  1  1   4.00   11329.8   0.022745     476.1     1244.6      656.3
  1  2   8.00   11329.8   0.022745     476.1     1244.6      656.3
  1  3  12.00   11329.8   0.022745     476.1     1244.6      656.3
  1  4  16.00   12619.3   0.021941     546.2     1248.3      690.7
  1  5  20.00   12619.3   0.021941     546.2     1248.3      690.7
  1  6  24.00   12619.3   0.021941     546.2     1248.3      690.7
  1  7  28.00   12619.3   0.021941     546.2     1248.3      690.7
  1  8  32.00   12619.3   0.021941     546.2     1248.3      690.7
  1  9  36.00   11329.8   0.022745     476.1     1244.6      656.3
  1 10  40.00   11329.8   0.022745     476.1     1244.6      656.3
  1 11  44.00   11329.8   0.022745     476.1     1244.6      656.3
  1 12  48.00   11329.8   0.022745     476.1     1244.6      656.3


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 TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N =  9.00
               ****************************************


              MOMENT OF    Q/Ix       ELASTIC SECTION MODULUS,(in**3)
 SP IN D FROM  INERTIA -------------- --------------------------------
 NO NO L SUPT --------- Q=1ST. MOMENT      STEEL       CONCRETE(SLAB)
       (ft)    Ix         OF INERTIA ---------------- ---------------
              (in**4)     (1/in)        BOT.    TOP.         TOP.
 ----------------------------------------------------------------------

  1  0   0.00   15583.4   0.029038     529.9     4459.6     1337.1
  1  1   4.00   15583.4   0.029038     529.9     4459.6     1337.1
  1  2   8.00   15583.4   0.029038     529.9     4459.6     1337.1
  1  3  12.00   15583.4   0.029038     529.9     4459.6     1337.1
  1  4  16.00   17581.4   0.028154     606.8     4149.2     1418.2
  1  5  20.00   17581.4   0.028154     606.8     4149.2     1418.2
  1  6  24.00   17581.4   0.028154     606.8     4149.2     1418.2
  1  7  28.00   17581.4   0.028154     606.8     4149.2     1418.2
  1  8  32.00   17581.4   0.028154     606.8     4149.2     1418.2
  1  9  36.00   15583.4   0.029038     529.9     4459.6     1337.1
  1 10  40.00   15583.4   0.029038     529.9     4459.6     1337.1
  1 11  44.00   15583.4   0.029038     529.9     4459.6     1337.1
  1 12  48.00   15583.4   0.029038     529.9     4459.6     1337.1


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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	4.00	10.7	48.8	0.7	0.0	60.2
1	2	8.00	19.4	88.6	1.3	0.0	109.3
1	3	12.00	26.2	119.7	1.4	0.0	147.3
1	4	16.00	31.1	141.8	1.6	0.0	174.5
1	5	20.00	34.0	155.1	1.8	0.0	190.9
1	6	24.00	35.0	159.6	2.0	0.0	196.5
1	7	28.00	34.0	155.1	2.0	0.0	191.1
1	8	32.00	31.1	141.8	1.6	0.0	174.5
1	9	36.00	26.2	119.7	1.2	0.0	147.1
1	10	40.00	19.4	88.6	0.8	0.0	108.9
1	11	44.00	10.7	48.8	0.4	0.0	59.8
1	12	48.00	0.0	0.0	0.0	0.0	0.0

TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft)	CONCENTRATED LOADS (k-ft)	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD

			DW	DC2	DW	DC2		
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	4.00	25.7	14.3	0.0	0.0	0.0	40.0
1	2	8.00	46.7	26.1	0.0	0.0	0.0	72.8
1	3	12.00	63.1	35.2	0.0	0.0	0.0	98.3
1	4	16.00	74.8	41.7	0.0	0.0	0.0	116.5
1	5	20.00	81.8	45.6	0.0	0.0	0.0	127.4
1	6	24.00	84.1	46.9	0.0	0.0	0.0	131.0
1	7	28.00	81.8	45.6	0.0	0.0	0.0	127.4
1	8	32.00	74.8	41.7	0.0	0.0	0.0	116.5
1	9	36.00	63.1	35.2	0.0	0.0	0.0	98.3
1	10	40.00	46.7	26.1	0.0	0.0	0.0	72.8
1	11	44.00	25.7	14.3	0.0	0.0	0.0	40.0
1	12	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	SIDEWALK (MAXIMUM) NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	4.00	0.0	0.0	190.1	HL-93	0.0	HL-93
1	2	8.00	0.0	0.0	337.9	HL-93	0.0	HL-93
1	3	12.00	0.0	0.0	441.3	HL-93	0.0	HL-93
1	4	16.00	0.0	0.0	511.4	HL-93	0.0	HL-93
1	5	20.00	0.0	0.0	545.3	HL-93	0.0	HL-93
1	6	24.00	0.0	0.0	548.7	HL-93	0.0	HL-93
1	7	28.00	0.0	0.0	545.3	HL-93	0.0	HL-93
1	8	32.00	0.0	0.0	511.4	HL-93	0.0	HL-93
1	9	36.00	0.0	0.0	441.3	HL-93	0.0	HL-93
1	10	40.00	0.0	0.0	337.9	HL-93	0.0	HL-93
1	11	44.00	0.0	0.0	190.1	HL-93	0.0	HL-93
1	12	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	4.00	65.	0.	65.
1	2	8.00	112.	0.	112.
1	3	12.00	139.	0.	139.
1	4	16.00	161.	0.	161.
1	5	20.00	176.	0.	176.
1	6	24.00	178.	0.	178.
1	7	28.00	176.	0.	176.
1	8	32.00	161.	0.	161.
1	9	36.00	139.	0.	139.
1	10	40.00	112.	0.	112.
1	11	44.00	65.	0.	65.
1	12	48.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	DEAD LOAD COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	4.00	60.2	40.0	190.1	HL-93	0.0	HL-93	290.3	100.2
1	2	8.00	109.3	72.8	337.9	HL-93	0.0	HL-93	520.1	182.1
1	3	12.00	147.3	98.3	441.3	HL-93	0.0	HL-93	686.9	245.6
1	4	16.00	174.5	116.5	511.4	HL-93	0.0	HL-93	802.4	291.0
1	5	20.00	190.9	127.4	545.3	HL-93	0.0	HL-93	863.6	318.3
1	6	24.00	196.5	131.0	548.7	HL-93	0.0	HL-93	876.3	327.5
1	7	28.00	191.1	127.4	545.3	HL-93	0.0	HL-93	863.9	318.5
1	8	32.00	174.5	116.5	511.4	HL-93	0.0	HL-93	802.4	291.0
1	9	36.00	147.1	98.3	441.3	HL-93	0.0	HL-93	686.7	245.4
1	10	40.00	108.9	72.8	337.9	HL-93	0.0	HL-93	519.6	181.7
1	11	44.00	59.8	40.0	190.1	HL-93	0.0	HL-93	290.0	99.9
1	12	48.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	4.00	290.3	347.4	464.4	388.4	150.3	48.8
1	2	8.00	520.1	621.5	830.7	695.6	273.2	84.1
1	3	12.00	686.9	819.3	1095.1	918.6	368.4	104.1
1	4	16.00	802.4	955.8	1277.4	1072.8	436.5	121.1
1	5	20.00	863.6	1027.2	1372.6	1154.5	477.5	132.1
1	6	24.00	876.3	1040.9	1390.7	1171.2	491.3	133.8
1	7	28.00	863.9	1027.4	1372.9	1154.8	477.8	132.1
1	8	32.00	802.4	955.8	1277.4	1072.8	436.5	121.1
1	9	36.00	686.7	819.1	1094.8	918.3	368.1	104.1
1	10	40.00	519.6	621.0	830.2	695.0	272.5	84.1
1	11	44.00	290.0	347.0	464.0	387.9	149.8	48.8
1	12	48.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	2.9	13.3	0.2	0.0	16.4
1	1	4.00	2.4	11.1	0.2	0.0	13.7
1	2	8.00	1.9	8.9	0.0	0.0	10.8
1	3	12.00	1.5	6.6	0.0	0.0	8.1
1	4	16.00	1.0	4.4	0.0	0.0	5.4
1	5	20.00	0.5	2.2	0.0	0.0	2.7
1	6	24.00	0.0	0.0	0.0	0.0	0.0
1	7	28.00	-0.5	-2.2	-0.1	0.0	-2.8
1	8	32.00	-1.0	-4.4	-0.1	0.0	-5.5
1	9	36.00	-1.5	-6.6	-0.1	0.0	-8.2
1	10	40.00	-1.9	-8.9	-0.1	0.0	-10.9
1	11	44.00	-2.4	-11.1	-0.1	0.0	-13.6
1	12	48.00	-2.9	-13.3	-0.1	0.0	-16.3

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.0	3.9	0.0	0.0	10.9	16.4	27.3
1	1	4.00	5.8	3.3	0.0	0.0	9.1	13.7	22.8
1	2	8.00	4.7	2.6	0.0	0.0	7.3	10.8	18.1
1	3	12.00	3.5	2.0	0.0	0.0	5.5	8.1	13.6
1	4	16.00	2.3	1.3	0.0	0.0	3.6	5.4	9.1
1	5	20.00	1.2	0.7	0.0	0.0	1.8	2.7	4.6
1	6	24.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	7	28.00	-1.2	-0.7	0.0	0.0	-1.8	-2.8	-4.6
1	8	32.00	-2.3	-1.3	0.0	0.0	-3.6	-5.5	-9.1
1	9	36.00	-3.5	-2.0	0.0	0.0	-5.5	-8.2	-13.7
1	10	40.00	-4.7	-2.6	0.0	0.0	-7.3	-10.9	-18.2
1	11	44.00	-5.8	-3.3	0.0	0.0	-9.1	-13.6	-22.7
1	12	48.00	-7.0	-3.9	0.0	0.0	-10.9	-16.3	-27.2

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	50.9	HL-93	0.0	HL-93

1	1	4.00	0.0	0.0	45.1	HL-93	-2.0	HL-93
1	2	8.00	0.0	0.0	39.5	HL-93	-4.8	HL-93
1	3	12.00	0.0	0.0	34.0	HL-93	-8.1	HL-93
1	4	16.00	0.0	0.0	28.6	HL-93	-11.6	HL-93
1	5	20.00	0.0	0.0	23.4	HL-93	-15.2	HL-93
1	6	24.00	0.0	0.0	18.9	HL-93	-18.9	HL-93
1	7	28.00	0.0	0.0	15.2	HL-93	-23.4	HL-93
1	8	32.00	0.0	0.0	11.6	HL-93	-28.6	HL-93
1	9	36.00	0.0	0.0	8.1	HL-93	-34.0	HL-93
1	10	40.00	0.0	0.0	4.8	HL-93	-39.5	HL-93
1	11	44.00	0.0	0.0	2.0	HL-93	-45.1	HL-93
1	12	48.00	0.0	0.0	0.0	HL-93	-50.9	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	20.	0.	20.
1	1	4.00	17.	-1.	19.
1	2	8.00	15.	-2.	17.
1	3	12.00	12.	-4.	16.
1	4	16.00	11.	-5.	16.
1	5	20.00	9.	-6.	16.
1	6	24.00	8.	-8.	16.
1	7	28.00	6.	-9.	16.
1	8	32.00	5.	-11.	16.
1	9	36.00	4.	-12.	16.
1	10	40.00	2.	-15.	17.
1	11	44.00	1.	-17.	19.
1	12	48.00	0.	-20.	20.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	20.1	.31052E-01	0.31	0.2500*	.14740E-01	0.15	0.2500*
1	1	4.00	18.6	.31052E-01	0.29	0.2500*	.14740E-01	0.14	0.2500*
1	2	8.00	17.4	.31052E-01	0.27	0.2500*	.14740E-01	0.13	0.2500*
1	3	12.00	16.0	.31052E-01	0.25	0.2500*	.14740E-01	0.12	0.2500*
1	4	16.00	15.7	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	5	20.00	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	6	24.00	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	7	28.00	15.9	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	8	32.00	15.7	.30292E-01	0.24	0.2500*	.12712E-01	0.10	0.2500*
1	9	36.00	16.0	.31052E-01	0.25	0.2500*	.14740E-01	0.12	0.2500*
1	10	40.00	17.4	.31052E-01	0.27	0.2500*	.14740E-01	0.13	0.2500*
1	11	44.00	18.6	.31052E-01	0.29	0.2500*	.14740E-01	0.14	0.2500*
1	12	48.00	20.1	.31052E-01	0.31	0.2500*	.14740E-01	0.15	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	27.3	50.9	HL-93	0.0	HL-93	78.2	27.3
1	1	4.00	22.8	45.1	HL-93	-2.0	HL-93	67.9	20.8
1	2	8.00	18.1	39.5	HL-93	-4.8	HL-93	57.6	13.3
1	3	12.00	13.6	34.0	HL-93	-8.1	HL-93	47.6	5.5
1	4	16.00	9.1	28.6	HL-93	-11.6	HL-93	37.6	-2.5
1	5	20.00	4.6	23.4	HL-93	-15.2	HL-93	28.0	-10.6
1	6	24.00	0.0	18.9	HL-93	-18.9	HL-93	18.9	-18.8
1	7	28.00	-4.6	15.2	HL-93	-23.4	HL-93	10.6	-28.0
1	8	32.00	-9.1	11.6	HL-93	-28.6	HL-93	2.5	-37.7
1	9	36.00	-13.7	8.1	HL-93	-34.0	HL-93	-5.5	-47.7
1	10	40.00	-18.2	4.8	HL-93	-39.5	HL-93	-13.4	-57.7
1	11	44.00	-22.7	2.0	HL-93	-45.1	HL-93	-20.7	-67.8
1	12	48.00	-27.2	0.0	HL-93	-50.9	HL-93	-27.2	-78.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	78.2	93.4	124.9	104.6	41.0	15.1
1	1	4.00	67.9	81.5	108.9	90.9	34.2	14.0
1	2	8.00	57.6	69.5	93.0	77.2	27.2	13.0
1	3	12.00	47.6	57.8	77.4	63.8	20.4	12.0
1	4	16.00	37.6	46.2	61.9	50.5	13.6	11.7
1	5	20.00	28.0	35.0	47.0	37.6	6.8	11.9

1	6	24.00	18.9	24.6	33.1	25.5	0.1	11.9
1	7	28.00	-28.0	-35.1	-47.1	-37.7	-6.9	11.9
1	8	32.00	-37.7	-46.3	-62.0	-50.6	-13.7	11.7
1	9	36.00	-47.7	-57.9	-77.5	-63.9	-20.5	12.0
1	10	40.00	-57.7	-69.5	-93.0	-77.2	-27.3	13.0
1	11	44.00	-67.8	-81.4	-108.8	-90.8	-34.1	14.0
1	12	48.00	-78.1	-93.4	-124.8	-104.5	-40.8	15.1

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	27.32	0.00	HL-93	50.86	HL-93	ST1	22.83	124.91
						ST2	22.83	104.56
						ST4	30.46	40.97
						SE1	27.32	78.18
						SE2	27.32	93.44
2	27.23	0.00	HL-93	50.86	HL-93	ST1	22.76	124.80
						ST2	22.76	104.46
						ST4	30.33	40.85
						SE1	27.23	78.09
						SE2	27.23	93.35

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	+COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	4.00	-0.0211	-0.0961	0.0000	-0.0409	-0.1183	-0.0409	-0.1592
1	2	8.00	-0.0404	-0.1844	0.0000	-0.0784	-0.2271	-0.0784	-0.3055
1	3	12.00	-0.0567	-0.2586	0.0000	-0.1099	-0.3183	-0.1099	-0.4282
1	4	16.00	-0.0687	-0.3137	0.0000	-0.1333	-0.3862	-0.1333	-0.5195
1	5	20.00	-0.0763	-0.3480	0.0000	-0.1478	-0.4284	-0.1478	-0.5761
1	6	24.00	-0.0788	-0.3596	0.0000	-0.1527	-0.4427	-0.1527	-0.5954
1	7	28.00	-0.0763	-0.3480	0.0000	-0.1478	-0.4283	-0.1478	-0.5761
1	8	32.00	-0.0687	-0.3137	0.0000	-0.1333	-0.3861	-0.1333	-0.5194
1	9	36.00	-0.0567	-0.2586	0.0000	-0.1099	-0.3182	-0.1099	-0.4281
1	10	40.00	-0.0404	-0.1844	0.0000	-0.0784	-0.2269	-0.0784	-0.3053
1	11	44.00	-0.0211	-0.0961	0.0000	-0.0409	-0.1182	-0.0409	-0.1591
1	12	48.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections

due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	4.0	-0.021	0 1/16	-0.097	0 1/ 8	-0.041	0 1/16	-0.159	0 3/16
1	2	8.0	-0.040	0 1/16	-0.187	0 3/16	-0.078	0 1/ 8	-0.306	0 5/16
1	3	12.0	-0.057	0 1/16	-0.262	0 5/16	-0.110	0 1/ 8	-0.428	0 7/16
1	4	16.0	-0.069	0 1/ 8	-0.317	0 3/ 8	-0.133	0 3/16	-0.519	0 9/16
1	5	20.0	-0.076	0 1/ 8	-0.352	0 3/ 8	-0.148	0 3/16	-0.576	0 5/ 8
1	6	24.0	-0.079	0 1/ 8	-0.364	0 3/ 8	-0.153	0 3/16	-0.595	0 5/ 8
1	7	28.0	-0.076	0 1/ 8	-0.352	0 3/ 8	-0.148	0 3/16	-0.576	0 5/ 8
1	8	32.0	-0.069	0 1/ 8	-0.317	0 3/ 8	-0.133	0 3/16	-0.519	0 9/16
1	9	36.0	-0.057	0 1/16	-0.262	0 5/16	-0.110	0 1/ 8	-0.428	0 7/16
1	10	40.0	-0.040	0 1/16	-0.187	0 3/16	-0.078	0 1/ 8	-0.305	0 5/16
1	11	44.0	-0.021	0 1/16	-0.097	0 1/ 8	-0.041	0 1/16	-0.159	0 3/16
1	12	48.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	FACTOR	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	24.00	3	0.425	-0.266	MAX HL-93	0.72	0.00265
				0.029	MIN		
				-0.066	MAX LANE		
				0.029	MIN LANE		
		4	0.433	-0.272	MAX HL-93	0.72	0.00265
				0.029	MIN		
				-0.067	MAX LANE		
				0.029	MIN LANE		
		3	0.425	-0.166	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	-0.36	0.36	-1.66	1.66	-2.01	2.01
1	2	8.00	-0.65	0.65	-3.01	3.01	-3.66	3.66
1	3	12.00	-0.88	0.88	-4.05	4.05	-4.93	4.93
1	4	16.00	-1.00	0.90	-4.63	4.16	-5.63	5.06
1	5	20.00	-1.10	0.98	-5.06	4.55	-6.16	5.53
1	6	24.00	-1.13	1.01	-5.21	4.68	-6.34	5.69
1	7	28.00	-1.10	0.98	-5.07	4.55	-6.17	5.54
1	8	32.00	-1.00	0.90	-4.63	4.16	-5.63	5.06
1	9	36.00	-0.88	0.88	-4.04	4.04	-4.92	4.92
1	10	40.00	-0.65	0.65	-2.99	2.99	-3.64	3.64
1	11	44.00	-0.36	0.36	-1.64	1.64	-2.00	2.00
1	12	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	-0.03	-0.39	1.01	-0.03	-2.40	3.02
1	2	8.00	-0.05	-0.70	1.83	-0.05	-4.36	5.49
1	3	12.00	-0.07	-0.95	2.48	-0.07	-5.88	7.41
1	4	16.00	-0.07	-1.12	2.56	-0.07	-6.75	7.62
1	5	20.00	-0.08	-1.22	2.80	-0.08	-7.39	8.33
1	6	24.00	-0.08	-1.26	2.88	-0.08	-7.60	8.57
1	7	28.00	-0.08	-1.22	2.80	-0.08	-7.39	8.34
1	8	32.00	-0.07	-1.12	2.56	-0.07	-6.75	7.62
1	9	36.00	-0.07	-0.95	2.48	-0.07	-5.87	7.40

1 10	40.00	-0.05	-0.70	1.83	-0.05	-4.34	5.48
1 11	44.00	-0.03	-0.39	1.01	-0.03	-2.39	3.01
1 12	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	-0.19	-0.51	4.31	0.00	0.00	0.00
1	2	8.00	-0.34	-0.91	7.65	0.00	0.00	0.00
1	3	12.00	-0.44	-1.19	9.99	0.00	0.00	0.00
1	4	16.00	-0.48	-1.48	10.11	0.00	0.00	0.00
1	5	20.00	-0.51	-1.58	10.78	0.00	0.00	0.00
1	6	24.00	-0.52	-1.59	10.85	0.00	0.00	0.00
1	7	28.00	-0.51	-1.58	10.78	0.00	0.00	0.00
1	8	32.00	-0.48	-1.48	10.11	0.00	0.00	0.00
1	9	36.00	-0.44	-1.19	9.99	0.00	0.00	0.00
1	10	40.00	-0.34	-0.91	7.65	0.00	0.00	0.00
1	11	44.00	-0.19	-0.51	4.31	0.00	0.00	0.00
1	12	48.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE	BOT OF BOT FLANGE TRUCK	ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	4.00	0.2	0.0	0.2	C	HL	1.5	0.0	1.5	T	HL
1	2	8.00	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T	HL
1	3	12.00	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	4	16.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL
1	5	20.00	0.5	0.0	0.5	C	HL	3.5	0.0	3.5	T	HL
1	6	24.00	0.5	0.0	0.5	C	HL	3.5	0.0	3.5	T	HL
1	7	28.00	0.5	0.0	0.5	C	HL	3.5	0.0	3.5	T	HL
1	8	32.00	0.5	0.0	0.5	C	HL	3.2	0.0	3.2	T	HL
1	9	36.00	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	10	40.00	0.3	0.0	0.3	C	HL	2.5	0.0	2.5	T	HL
1	11	44.00	0.2	0.0	0.2	C	HL	1.5	0.0	1.5	T	HL
1	12	48.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.22	-2.91	7.33	-0.03	-2.40	3.02
1	2	8.00	33.	33.	-0.39	-5.27	13.15	-0.05	-4.36	5.49
1	3	12.00	33.	33.	-0.51	-7.06	17.40	-0.07	-5.88	7.41
1	4	16.00	33.	33.	-0.56	-8.23	17.73	-0.07	-6.75	7.62
1	5	20.00	33.	33.	-0.59	-8.96	19.12	-0.08	-7.39	8.33
1	6	24.00	33.	33.	-0.60	-9.19	19.42	-0.08	-7.60	8.57
1	7	28.00	33.	33.	-0.59	-8.97	19.12	-0.08	-7.39	8.34
1	8	32.00	33.	33.	-0.56	-8.23	17.73	-0.07	-6.75	7.62
1	9	36.00	33.	33.	-0.51	-7.06	17.39	-0.07	-5.87	7.40
1	10	40.00	33.	33.	-0.39	-5.25	13.13	-0.05	-4.34	5.48
1	11	44.00	33.	33.	-0.22	-2.90	7.32	-0.03	-2.39	3.01
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.27	-3.06	8.62	-0.03	-2.40	3.02
1	2	8.00	33.	33.	-0.49	-5.54	15.44	-0.05	-4.36	5.49
1	3	12.00	33.	33.	-0.64	-7.42	20.40	-0.07	-5.88	7.41
1	4	16.00	33.	33.	-0.70	-8.68	20.76	-0.07	-6.75	7.62
1	5	20.00	33.	33.	-0.75	-9.44	22.35	-0.08	-7.39	8.33
1	6	24.00	33.	33.	-0.76	-9.66	22.68	-0.08	-7.60	8.57
1	7	28.00	33.	33.	-0.75	-9.44	22.36	-0.08	-7.39	8.34
1	8	32.00	33.	33.	-0.70	-8.68	20.76	-0.07	-6.75	7.62
1	9	36.00	33.	33.	-0.64	-7.41	20.39	-0.07	-5.87	7.40
1	10	40.00	33.	33.	-0.49	-5.53	15.43	-0.05	-4.34	5.48
1	11	44.00	33.	33.	-0.27	-3.05	8.61	-0.03	-2.39	3.01
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT

1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.37	-3.96	11.47	-0.04	-3.06	3.94
1	2	8.00	33.	33.	-0.66	-7.15	20.55	-0.07	-5.56	7.16
1	3	12.00	33.	33.	-0.86	-9.58	27.14	-0.09	-7.50	9.66
1	4	16.00	33.	33.	-0.95	-11.21	27.63	-0.11	-8.62	9.93
1	5	20.00	33.	33.	-1.01	-12.19	29.74	-0.12	-9.43	10.86
1	6	24.00	33.	33.	-1.02	-12.48	30.17	-0.12	-9.70	11.18
1	7	28.00	33.	33.	-1.01	-12.20	29.74	-0.12	-9.44	10.87
1	8	32.00	33.	33.	-0.95	-11.21	27.63	-0.11	-8.62	9.93
1	9	36.00	33.	33.	-0.86	-9.57	27.13	-0.09	-7.49	9.65
1	10	40.00	33.	33.	-0.66	-7.13	20.53	-0.07	-5.54	7.14
1	11	44.00	33.	33.	-0.37	-3.94	11.46	-0.04	-3.05	3.93
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.35	-2.99	10.09	-0.02	-2.10	2.56
1	2	8.00	33.	33.	-0.63	-5.40	18.04	-0.04	-3.81	4.65
1	3	12.00	33.	33.	-0.82	-7.22	23.76	-0.05	-5.14	6.27
1	4	16.00	33.	33.	-0.90	-8.49	24.14	-0.06	-5.90	6.44
1	5	20.00	33.	33.	-0.96	-9.21	25.92	-0.06	-6.45	7.05
1	6	24.00	33.	33.	-0.97	-9.42	26.24	-0.06	-6.64	7.25
1	7	28.00	33.	33.	-0.96	-9.22	25.93	-0.06	-6.46	7.05
1	8	32.00	33.	33.	-0.90	-8.49	24.14	-0.06	-5.90	6.44
1	9	36.00	33.	33.	-0.82	-7.21	23.75	-0.05	-5.13	6.26
1	10	40.00	33.	33.	-0.63	-5.39	18.03	-0.04	-3.80	4.64
1	11	44.00	33.	33.	-0.35	-2.98	10.08	-0.02	-2.09	2.55
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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COMPOSITE
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ANALYSIS

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.29	-3.75	9.75	-0.04	-3.06	3.94
1	2	8.00	33.	33.	-0.52	-6.79	17.49	-0.07	-5.56	7.16
1	3	12.00	33.	33.	-0.69	-9.10	23.15	-0.09	-7.50	9.66
1	4	16.00	33.	33.	-0.75	-10.62	23.58	-0.11	-8.62	9.93
1	5	20.00	33.	33.	-0.81	-11.56	25.42	-0.12	-9.43	10.86
1	6	24.00	33.	33.	-0.82	-11.85	25.83	-0.12	-9.70	11.18
1	7	28.00	33.	33.	-0.81	-11.57	25.43	-0.12	-9.44	10.87
1	8	32.00	33.	33.	-0.75	-10.62	23.58	-0.11	-8.62	9.93
1	9	36.00	33.	33.	-0.69	-9.09	23.14	-0.09	-7.49	9.65
1	10	40.00	33.	33.	-0.52	-6.77	17.47	-0.07	-5.54	7.14
1	11	44.00	33.	33.	-0.29	-3.74	9.74	-0.04	-3.05	3.93
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.00	33.	33.	-0.28	-2.79	8.37	-0.02	-2.10	2.56
1	2	8.00	33.	33.	-0.49	-5.04	14.98	-0.04	-3.81	4.65
1	3	12.00	33.	33.	-0.64	-6.74	19.76	-0.05	-5.14	6.27
1	4	16.00	33.	33.	-0.70	-7.89	20.10	-0.06	-5.90	6.44
1	5	20.00	33.	33.	-0.75	-8.58	21.61	-0.06	-6.45	7.05
1	6	24.00	33.	33.	-0.76	-8.78	21.90	-0.06	-6.64	7.25
1	7	28.00	33.	33.	-0.75	-8.59	21.61	-0.06	-6.46	7.05
1	8	32.00	33.	33.	-0.70	-7.89	20.10	-0.06	-5.90	6.44
1	9	36.00	33.	33.	-0.64	-6.73	19.75	-0.05	-5.13	6.26
1	10	40.00	33.	33.	-0.49	-5.03	14.97	-0.04	-3.80	4.64
1	11	44.00	33.	33.	-0.28	-2.78	8.36	-0.02	-2.09	2.55
1	12	48.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	14.00	161.	-5.4	107.	-1.0	476.	-1.3	0.	0.0	-7.70
											1.28
	LB			5.4		2.7		10.8		0.0	18.88
											10.79
	RT			-5.2		-1.0		-1.4		0.0	-7.60
											1.38
	RB			4.7		2.4		9.4		0.0	16.44
											9.42
1	LT	34.00	161.	-5.2	107.	-1.0	476.	-1.4	0.	0.0	-7.60
											1.38
	LB			4.7		2.4		9.4		0.0	16.44
											9.42
	RT			-5.4		-1.0		-1.3		0.0	-7.70
											1.28
	RB			5.4		2.7		10.8		0.0	18.87
											10.79

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

```
                 **********************************

                                                          COVER PLATE -- (in)
 MEMBER          TOP FLANGE     WEB       BOT FLANGE  -------------------------
--------- STEEL ----------- ------------ ------------     TOP          BOT
NO LENGTH  TYPE WIDTH THICK DEPTH THICK  WIDTH THICK  -----------  ------------
    (ft)           (in)        (in)          (in)     WIDTH THICK  WIDTH THICK
-------------------------------------------------------------------------------

 1  14.00    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

 2  20.00    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000  7.50 0.3125

 3  14.00    W  11.5 0.7400  31.4 0.5500 11.5 0.7400  11.50 0.0000 11.50 0.0000

-------------------------------------------------------------------------------
NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
               ************

 ** [1] = Span length (ft) =   48.0
                                                          LIMITATION
  MEMBER   SLAB  GIRDER [2]=OVERALL D. [2]/(12*[1])        1/25
--------- THICK. DEPTH  -------------- ------------ <=> ---------- STATUS
NO LENGTH ------ ------ [3]= GIRDER D. [3]/(12*[1])        1/30
    (ft)   (in)   (in)       (in)
-------------------------------------------------------------------------

 1  14.00  6.75   32.90 [2]=   41.06     0.07128      >   0.04000    OK

                        [3]=   32.90     0.05712      >   0.03300    OK

 2  20.00  6.75   32.90 [2]=   41.06     0.07128      >   0.04000    OK

                        [3]=   32.90     0.05712      >   0.03300    OK

 3  14.00  6.75   32.90 [2]=   41.06     0.07128      >   0.04000    OK

                        [3]=   32.90     0.05712      >   0.03300    OK

-------------------------------------------------------------------------

 NOTE : [1] -- Span length or Average of two adjacent span lengths

        [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

            * For composite girders, the minimum overall depth
              of girder (concrete slab plus haunch & girder) preferably should not
              be less than 0.040*Length of span (or 0.032*L if continuous spans).
              and the depth of steel girder alone preferably should not be
              less than 0.033*Length of span (or 0.027*L if continuous)

        [3] -- same as NOTE [2]

         ** -- cover plates not taken into account

         ** -- these criteria are related to the structural stability
               during construction and the limitation for the live load
               deflection.
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TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                 ******************************

SP IN D FROM  Lo Co       Web                                          Web
NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
```

			[1]					[2]	[3]	[4]	[5]	[6]
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	1	4.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	2	8.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	3	12.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	4	16.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	5	20.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	6	24.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	7	28.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	8	32.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	9	36.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	10	40.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	11	44.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	12	48.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp	depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0		0.00	0	1	31.42	0.550	57.13	168.97	2
1	1		4.00	0	1	31.42	0.550	57.13	168.97	2
1	2		8.00	0	1	31.42	0.550	57.13	168.97	2
1	3		12.00	0	1	31.42	0.550	57.13	168.97	2
1	4		16.00	0	1	31.42	0.550	60.99	168.97	2
1	5		20.00	0	1	31.42	0.550	60.99	168.97	2
1	6		24.00	0	1	31.42	0.550	60.99	168.97	2
1	7		28.00	0	1	31.42	0.550	60.99	168.97	2
1	8		32.00	0	1	31.42	0.550	60.99	168.97	2
1	9		36.00	0	1	31.42	0.550	57.13	168.97	2
1	10		40.00	0	1	31.42	0.550	57.13	168.97	2
1	11		44.00	0	1	31.42	0.550	57.13	168.97	2
1	12		48.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 1	4.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 2	8.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 3	12.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 4	16.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 5	20.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 6	24.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 7	28.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 8	32.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 9	36.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 10	40.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 11	44.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1 12	48.00	7.77	12.	11.5	5.2	0.740	0.605	
		7.77	12.	11.5	5.2	0.740	0.605	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)
[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	6.5	0.000	4.562	2.454	0.346	1.672
			0.000	0.000	0.000	0.000	1.000
2	6.5	27.2	4.562	9.302	8.623	7.943	1.000
			0.000	0.000	0.000	0.000	1.000
3	27.2	48.0	0.000	9.302	6.603	3.904	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange

is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	6.50	7.09	26.62	0	0
1	1	4.00	1	7.77	11.26	19.84	0	6.50	7.09	26.62	0	0
1	2	8.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	3	12.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	4	16.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	5	20.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	6	24.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	7	28.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	8	32.00	1	7.77	11.26	19.84	0	20.75	7.03	26.39	2	2
1	9	36.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	10	40.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	11	44.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2
1	12	48.00	1	7.77	11.26	19.84	0	20.75	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

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UNIVERSITY OF MARYLAND

MERLIN V 10.4
COMPOSITE
LRF -- 2010
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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.67	33.00	33.00	33.00
1	1	4.00	1	1.000	1.000	1.67	33.00	33.00	33.00
1	2	8.00	1	1.000	1.000	1.00	33.00	26.07	26.07
1	3	12.00	1	1.000	1.000	1.00	33.00	26.07	26.07
1	4	16.00	1	1.000	1.000	1.00	33.00	25.98	25.98
1	5	20.00	1	1.000	1.000	1.00	33.00	25.98	25.98
1	6	24.00	1	1.000	1.000	1.00	33.00	25.98	25.98

1	7	28.00	1	1.000	1.000	1.00	33.00	25.98	25.98
1	8	32.00	1	1.000	1.000	1.00	33.00	25.98	25.98
1	9	36.00	1	1.000	1.000	1.00	33.00	26.07	26.07
1	10	40.00	1	1.000	1.000	1.00	33.00	26.07	26.07
1	11	44.00	1	1.000	1.000	1.00	33.00	26.07	26.07
1	12	48.00	1	1.000	1.000	1.00	33.00	26.07	26.07

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	6.34	17.25
1	1		4.00	1	6.34	17.25
1	2		8.00	1	6.34	17.25
1	3		12.00	1	6.34	17.25
1	4		16.00	1	8.17	17.38
1	5		20.00	1	8.17	17.38
1	6		24.00	1	8.17	17.38
1	7		28.00	1	8.17	17.38
1	8		32.00	1	8.17	17.38
1	9		36.00	1	6.34	17.25
1	10		40.00	1	6.34	17.25
1	11		44.00	1	6.34	17.25
1	12		48.00	1	6.34	17.25

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0		0.00	1	0	0	0		0	0
1	1		4.00	1	0	0	0		0	0
1	2		8.00	1	0	2	0		2	0
1	3		12.00	1	0	2	0		2	0
1	4		16.00	1	0	2	0		2	0
1	5		20.00	1	0	2	0		2	0
1	6		24.00	1	0	2	0		2	0
1	7		28.00	1	0	2	0		2	0
1	8		32.00	1	0	2	0		2	0
1	9		36.00	1	0	2	0		2	0
1	10		40.00	1	0	2	0		2	0
1	11		44.00	1	0	2	0		2	0
1	12		48.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

1

TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		4.00	0.0	19.8	3.0	33.0	3.0	33.0	3.0	33.0	0
				0.0	-	3.0	-	3.0	33.0	-	-	0
1	2		8.00	0.0	19.8	5.5	33.0	5.5	33.0	5.5	26.1	0
				0.0	-	5.5	-	5.5	33.0	-	-	0
1	3		12.00	0.0	19.8	7.4	33.0	7.4	33.0	7.4	26.1	0
				0.0	-	7.4	-	7.4	33.0	-	-	0
1	4		16.00	0.0	19.8	8.4	33.0	8.4	33.0	8.4	26.0	0
				0.0	-	7.6	-	7.6	33.0	-	-	0
1	5		20.00	0.0	19.8	9.2	33.0	9.2	33.0	9.2	26.0	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	6		24.00	0.0	19.8	9.5	33.0	9.5	33.0	9.5	26.0	0
				0.0	-	8.5	-	8.5	33.0	-	-	0
1	7		28.00	0.0	19.8	9.3	33.0	9.3	33.0	9.3	26.0	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	8		32.00	0.0	19.8	8.4	33.0	8.4	33.0	8.4	26.0	0
				0.0	-	7.6	-	7.6	33.0	-	-	0
1	9		36.00	0.0	19.8	7.4	33.0	7.4	33.0	7.4	26.1	0
				0.0	-	7.4	-	7.4	33.0	-	-	0
1	10		40.00	0.0	19.8	5.5	33.0	5.5	33.0	5.5	26.1	0
				0.0	-	5.5	-	5.5	33.0	-	-	0
1	11		44.00	0.0	19.8	3.0	33.0	3.0	33.0	3.0	26.1	0
				0.0	-	3.0	-	3.0	33.0	-	-	0
1	12		48.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	26.1	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	4.00	0.000	0.092	0.092	0.092	0.092	2
					0.092		0.092	3
1	2	8.00	0.000	0.166	0.166	0.210	0.210	4
					0.166		0.166	3
1	3	12.00	0.000	0.224	0.224	0.284	0.284	4
					0.224		0.224	3
1	4	16.00	0.000	0.256	0.256	0.325	0.325	4
					0.230		0.230	3
1	5	20.00	0.000	0.280	0.280	0.356	0.356	4
					0.251		0.251	3
1	6	24.00	0.000	0.288	0.288	0.366	0.366	4
					0.259		0.259	3
1	7	28.00	0.000	0.280	0.280	0.356	0.356	4
					0.252		0.252	3
1	8	32.00	0.000	0.256	0.256	0.325	0.325	4
					0.230		0.230	3
1	9	36.00	0.000	0.224	0.224	0.283	0.283	4
					0.224		0.224	3
1	10	40.00	0.000	0.166	0.166	0.210	0.210	4
					0.166		0.166	3
1	11	44.00	0.000	0.091	0.091	0.115	0.115	4
					0.091		0.091	3
1	12	48.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1945.3	-	-	0.0	19.8	-	-	0--0
1	1		4.00	0	464.4	1945.3	-	-	0.0	19.8	-	-	0--0
1	2		8.00	0	830.7	1945.3	-	-	0.0	19.8	-	-	0--0
1	3		12.00	0	1095.1	1945.3	-	-	0.0	19.8	-	-	0--0
1	4		16.00	0	1277.4	2092.6	-	-	0.0	19.8	-	-	0--0
1	5		20.00	0	1372.6	2092.6	-	-	0.0	19.8	-	-	0--0
1	6		24.00	0	1390.7	2092.6	-	-	0.0	19.8	-	-	0--0
1	7		28.00	0	1372.9	2092.6	-	-	0.0	19.8	-	-	0--0
1	8		32.00	0	1277.4	2092.6	-	-	0.0	19.8	-	-	0--0
1	9		36.00	0	1094.8	1945.3	-	-	0.0	19.8	-	-	0--0
1	10		40.00	0	830.2	1945.3	-	-	0.0	19.8	-	-	0--0
1	11		44.00	0	464.0	1945.3	-	-	0.0	19.8	-	-	0--0
1	12		48.00	0	0.0	1945.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		4.00	0	0.239				0.239	1
1	2		8.00	0	0.427				0.427	1
1	3		12.00	0	0.563				0.563	1
1	4		16.00	0	0.610				0.610	1
1	5		20.00	0	0.656				0.656	1
1	6		24.00	0	0.665				0.665	1
1	7		28.00	0	0.656				0.656	1
1	8		32.00	0	0.610				0.610	1
1	9		36.00	0	0.563				0.563	1
1	10		40.00	0	0.427				0.427	1

```
 1 11  44.00  0        0.239                                  0.239     1
 1 12  48.00  0        0.000                                  0.000     1


 ----------------------------------------------------------------------

 NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
       [2] = (PHI)f * Fnc for comp. flange of composite
             sections in positive flexure (Eq. 6.10.7.2.1-1)
           = (PHI)f * Rh * Fyt for tension flange of composite sections in
             negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
       [3] = 0.6*Fyt for composite sections in positive flexure
             or tension flange for composite sections in
             negative flexure and non-composite sections (Eq. 6.10.1.6-1)
           = 0.6*Fyc for comp. flange of composite sections in
             negative flexure and non-composite sections (Eq. 6.10.1.6-1)
       [4] = (PHI)f * Fnt for non-compact tension flange
             of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
           = (PHI)f * Fnc for comp. flange of composite sections
             in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

       The governing number is listed as below.
         1 = (Mu+1/3flSxt) / [1]
         2 = fbu / [2]
         3 = fl / [3]
         4 = (fbu + 1/3fl) / [4]

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 TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
                 *********************************

 SP IN D FROM  Fy     k     Art.6.10.9.2         C     Vp        SHEAR CAPACITY
 NO NO L SUPT ----- ----- -------------- D/tw  ----- ----- ----- -Eq.6.10.9.2--
        (ft)  (ksi)  [1]    [2]    [3]          [4]  (ksi)        Vn = C x Vp
 ---------------------------------------------------------------------------

  1  0   0.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  1   4.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  2   8.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  3  12.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  4  16.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  5  20.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  6  24.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  7  28.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  8  32.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1  9  36.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1 10  40.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1 11  44.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8
  1 12  48.00  33.0  5.0    74.2   92.8  57.1  1.000   330.8     330.8


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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0
```

```
For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                ..... AASHTO LRFD Eq.6.10.9.3.2-6

For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                ..... AASHTO LRFD Eq.6.10.9.3.2-7


** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
   stiffened
   if (d0/D) < 3 without long. stiffener, or
   if (d0/D) > 3 with both transv. and long. stiffeners.
   For handling requirement, (d0/D) > (260/(D/tw))**2

        Vu = CVp

   where D = clear, unsupported distance between flange components

        d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot
   count on post-buckling shear resistance from tension-
   field action for an unstiffened girder.
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    TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                   ********************************

                         S T R E N G T H  C A T E G O R Y; SEE NOTE
                  ------------------------------------------------------------
    SP IN D FROM WEB STABILITY    LFD/LRFD      UNSTIFFENED            OVERALL
    NO NO L SUPT UNSTIFFENED,[1] MAX. SHEAR <=> SHEAR CAPA- CATEGORY     WEB
           (ft)  ---- D/tw -----    (kip)       CITY,   [2]
    ---------------------------------------------------------------------------

     1  0   0.00       0            124.9   <      330.8       0          0
     1  1   4.00       0            108.9   <      330.8       0          0
     1  2   8.00       0             93.0   <      330.8       0          0
     1  3  12.00       0             77.4   <      330.8       0          0
     1  4  16.00       0             61.9   <      330.8       0          0
     1  5  20.00       0             47.0   <      330.8       0          0
     1  6  24.00       0             33.1   <      330.8       0          0
     1  7  28.00       0             47.1   <      330.8       0          0
     1  8  32.00       0             62.0   <      330.8       0          0
     1  9  36.00       0             77.5   <      330.8       0          0
     1 10  40.00       0             93.0   <      330.8       0          0
     1 11  44.00       0            108.8   <      330.8       0          0
     1 12  48.00       0            124.8   <      330.8       0          0


    ---------------------------------------------------------------------------
    Please read NOTE on the following page
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     NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

               For compact section  (composite sections in positive flexure)

                 w/o long. stiff.

                 D/tw <= 150 (Eq. 6.10.2.1.1-1)

                 w / long. stiff.

                 D/tw <= 300 (Eq. 6.10.2.1.2-1)

                 2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1
```

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	124.91	330.76	0	
1	1	4.00	33.	108.92	330.76	0	
1	2	8.00	33.	92.96	330.76	0	
1	3	12.00	33.	77.39	330.76	0	
1	4	16.00	33.	61.92	330.76	0	
1	5	20.00	33.	46.98	330.76	0	
1	6	24.00	33.	33.07	330.76	0	
1	7	28.00	33.	47.06	330.76	0	
1	8	32.00	33.	61.99	330.76	0	
1	9	36.00	33.	77.46	330.76	0	
1	10	40.00	33.	93.03	330.76	0	
1	11	44.00	33.	108.82	330.76	0	
1	12	48.00	33.	124.80	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3]	TBC
1	0	0.00	0.0	-		31.4	0.0	31.4	-	-	00-
1	1	4.00	3.1	-		31.4	8.6	31.4	-	-	00-
1	2	8.00	5.5	-		31.4	15.4	31.4	-	-	00-

1	3	12.00	7.4	-	31.4	20.4	31.4	-	-	00-
1	4	16.00	8.7	-	31.4	20.8	31.4	-	-	00-
1	5	20.00	9.4	-	31.4	22.4	31.4	-	-	00-
1	6	24.00	9.7	-	31.4	22.7	31.4	-	-	00-
1	7	28.00	9.4	-	31.4	22.4	31.4	-	-	00-
1	8	32.00	8.7	-	31.4	20.8	31.4	-	-	00-
1	9	36.00	7.4	-	31.4	20.4	31.4	-	-	00-
1	10	40.00	5.5	-	31.4	15.4	31.4	-	-	00-
1	11	44.00	3.1	-	31.4	8.6	31.4	-	-	00-
1	12	48.00	0.0	-	31.4	0.0	31.4	-	-	00-

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```
NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
          = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
      [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
            but not to exceed the smaller of RhFyc and Fyw/0.7
            k = bending-buckling coefficient
              = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
            where:
            Dc = depth of the web in compression in the elastic
                 range. For composite sections, Dc shall be
                 determined as specified in Article D6.3.1

      "-" is N.A.

      Flag check - 0 = OK; 1 = NG
      T = Top Flange; B = Bottom Flange; C = Comp. Flange

      The values of fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)
```

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.00	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	2	8.00	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	3	12.00	TR	-0.6	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1		14.00	SCG					
1	4	16.00	TR	-0.7	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	5	20.00	TR	-0.8	A B B^C^C D E E^	TR	5.2	A B B^C^C D
1	6	24.00	TR	-0.8	A B B^C^C D E E^	TR	5.3	A B B^C^C D
1	7	28.00	TR	-0.8	A B B^C^C D E E^	TR	5.2	A B B^C^C D
1	8	32.00	TR	-0.7	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1		34.00	SCG					
1	9	36.00	TR	-0.6	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	10	40.00	TR	-0.5	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	11	44.00	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	12	48.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

```
 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                  ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                     TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
       (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ------------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   4.00    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1  2   8.00    TR     -0.2 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
  1  3  12.00    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1     14.00  SCG
  1  4  16.00    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1  5  20.00    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
  1  6  24.00    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
  1  7  28.00    TR     -0.4 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
  1  8  32.00    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1     34.00  SCG
  1  9  36.00    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
  1 10  40.00    TR     -0.2 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
  1 11  44.00    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
  1 12  48.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	30.1	.29038E-01	0.9	TR	3.09	3	10.61
1	1	4.00	27.9	.29038E-01	0.8	TR	3.09	3	11.45
1	2	8.00	26.1	.29038E-01	0.8	TR	3.09	3	12.26
1	3	12.00	24.0	.29038E-01	0.7	TR	3.09	3	13.34
1	4	16.00	23.5	.28154E-01	0.7	TR	3.09	3	14.03
1	5	20.00	23.8	.28154E-01	0.7	TR	3.09	3	13.84
1	6	24.00	23.8	.28154E-01	0.7	TR	3.09	3	13.84
1	7	28.00	23.8	.28154E-01	0.7	TR	3.09	3	13.84
1	8	32.00	23.5	.28154E-01	0.7	TR	3.09	3	14.03
1	9	36.00	24.0	.29038E-01	0.7	TR	3.09	3	13.34
1	10	40.00	26.1	.29038E-01	0.8	TR	3.09	3	12.26
1	11	44.00	27.9	.29038E-01	0.8	TR	3.09	3	11.45
1	12	48.00	30.1	.29038E-01	0.9	TR	3.09	3	10.61

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr	<--Input [5], input	SHEAR CONN. PER TRANS. SECTION	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	15.1	.29038E-01	0.4	TR	1.36	3	9.36
1	1	4.00	14.0	.29038E-01	0.4	TR	1.36	3	10.10
1	2	8.00	13.0	.29038E-01	0.4	TR	1.36	3	10.81
1	3	12.00	12.0	.29038E-01	0.3	TR	1.36	3	11.77
1	4	16.00	11.7	.28154E-01	0.3	TR	1.36	3	12.37
1	5	20.00	11.9	.28154E-01	0.3	TR	1.36	3	12.20
1	6	24.00	11.9	.28154E-01	0.3	TR	1.36	3	12.20
1	7	28.00	11.9	.28154E-01	0.3	TR	1.36	3	12.20
1	8	32.00	11.7	.28154E-01	0.3	TR	1.36	3	12.37
1	9	36.00	12.0	.29038E-01	0.3	TR	1.36	3	11.77
1	10	40.00	13.0	.29038E-01	0.4	TR	1.36	3	10.81
1	11	44.00	14.0	.29038E-01	0.4	TR	1.36	3	10.10
1	12	48.00	15.1	.29038E-01	0.4	TR	1.36	3	9.36

Please read NOTE on the following page

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

```
** The connector spacing shown here is calculated based upon
   AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
   where ADTT Single Lane= 1140 & No. of Cycles=   31207500
*** If ADTT Single Lane greater than or equal to 960, refer
   to Fatigue I Table.
```

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k)	P1p (k)	P2n (k)	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	N2	STATUS BLANK=OK CHECK=**
				LRFD 6.10.10.4.2				FATIG. CRI.				
1	6		24.00	1	1209.3	1204.9		87	>	63		
1	12		48.00	1	1209.3	1204.9		87	>	63		

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
- N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
- N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
- P = total nominal horizontal shear force
- Qr = factored shear resistance of one shear connector
  = (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
- Qn = nominal shear resistance
- (Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
- f'c = specified 28-day compressive strength of the concrete
- bs = effective width of the slab
- bfc = width of compression flange
- bft = width of tension flange
- ts = slab thickness
- Fyw = specified minimum yield strength of the web
- Fyt = specified minimum yield strength of the tension flange
- Fyc = specified minimum yield strength of the compression flange

D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	24.000	10.000
1	24.000	48.000	10.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	48.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK

OK 1.2.22.14=STRENGTH LIMIT STATE CHECK

OK 1.2.22.17=TRANSVERSE STIFFENER SPACING

OK 1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft ksi	D2	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1945.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	4.00	1945.3	60.2	40.0	190.1	0.0	7.07	HL-93	5.45	0
1	2	8.00	1945.3	109.3	72.8	337.9	0.0	3.74	HL-93	2.88	0
1	3	12.00	1945.3	147.3	98.3	441.3	0.0	2.72	HL-93	2.10	0
1	4	16.00	2092.6	174.5	116.5	511.4	0.0	2.48	HL-93	1.91	0
1	5	20.00	2092.6	190.9	127.4	545.3	0.0	2.27	HL-93	1.75	0
1	6	24.00	2092.6	196.5	131.0	548.7	0.0	2.24	HL-93	1.73	0
1	7	28.00	2092.6	191.1	127.4	545.3	0.0	2.27	HL-93	1.75	0
1	8	32.00	2092.6	174.5	116.5	511.4	0.0	2.48	HL-93	1.91	0
1	9	36.00	1945.3	147.1	98.3	441.3	0.0	2.72	HL-93	2.10	0
1	10	40.00	1945.3	108.9	72.8	337.9	0.0	3.74	HL-93	2.89	0
1	11	44.00	1945.3	59.8	40.0	190.1	0.0	7.07	HL-93	5.45	0
1	12	48.00	1945.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD RATING FACTOR TYPE
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68
1	1	4.00	31.4	-2.40 3.02	-0.51 4.31	0.00 0.00	6.58	HL-93 5.06
1	2	8.00	31.4	-4.36 5.49	-0.91 7.65	0.00 0.00	3.38	HL-93 2.60
1	3	12.00	31.4	-5.88 7.41	-1.19 9.99	0.00 0.00	2.40	HL-93 1.84
1	4	16.00	31.4	-6.75 7.62	-1.48 10.11	0.00 0.00	2.35	HL-93 1.81
1	5	20.00	31.4	-7.39 8.33	-1.58 10.78	0.00 0.00	2.13	HL-93 1.64
1	6	24.00	31.4	-7.60 8.57	-1.59 10.85	0.00 0.00	2.10	HL-93 1.61
1	7	28.00	31.4	-7.39 8.34	-1.58 10.78	0.00 0.00	2.13	HL-93 1.64
1	8	32.00	31.4	-6.75 7.62	-1.48 10.11	0.00 0.00	2.35	HL-93 1.81
1	9	36.00	31.4	-5.87 7.40	-1.19 9.99	0.00 0.00	2.40	HL-93 1.84
1	10	40.00	31.4	-4.34 5.48	-0.91 7.65	0.00 0.00	3.38	HL-93 2.60
1	11	44.00	31.4	-2.39 3.01	-0.51 4.31	0.00 0.00	6.58	HL-93 5.06
1	12	48.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93 7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN CRITICAL LOCATION GOVERNING GOVERNING

NO.	LENGTH (ft)	D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	LIVE LOAD TYPE
1	48.00	24.00	STRENGTH L.S.	2.24	1.73	HL-93
1	48.00	24.00	SERVICEBILITY	2.10	1.61	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.099
Inventory= 1.615

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	48.00	24.00	STRENGTH L.S.	3.10	536.68
1	48.00	24.00	SERVICEBILITY	2.75	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.752

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I D2	MAX. SHEAR POS.	MAX. SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	16.4	10.9	50.9	0.0	4.29	HL-93	3.31
1	1	4.00	330.8	13.7	9.1	45.1	-2.0	4.94	HL-93	3.81
1	2	8.00	330.8	10.8	7.3	39.5	-4.8	5.76	HL-93	4.44
1	3	12.00	330.8	8.1	5.5	34.0	-8.1	6.82	HL-93	5.26
1	4	16.00	330.8	5.4	3.6	28.6	-11.6	8.27	HL-93	6.38
1	5	20.00	330.8	2.7	1.8	23.4	-15.2	9.99	HL-93	7.92
1	6	24.00	330.8	0.0	0.0	18.9	-18.9	9.99	HL-93	9.99
1	7	28.00	330.8	-2.8	-1.8	15.2	-23.4	9.99	HL-93	7.92
1	8	32.00	330.8	-5.5	-3.6	11.6	-28.6	8.27	HL-93	6.38
1	9	36.00	330.8	-8.2	-5.5	8.1	-34.0	6.81	HL-93	5.26
1	10	40.00	330.8	-10.9	-7.3	4.8	-39.5	5.75	HL-93	4.44
1	11	44.00	330.8	-13.6	-9.1	2.0	-45.1	4.94	HL-93	3.81
1	12	48.00	330.8	-16.3	-10.9	0.0	-50.9	4.30	HL-93	3.31

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	RATING	TYPE
1	48.00	0.00	STRENGTH L.S.	4.29	3.31	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.294
Inventory= 3.313

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING CRITERION	RATING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	48.00	0.00	STRENGTH L.S.	5.42		54.45

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.416

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	24.00	3 0.425	-0.27	HL-93	2.70
		4 0.433	-0.27		2.65

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95 NB Span 4 Interior Strength	8/19/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Interior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	16	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.27	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
51.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	STANDARD SECTN NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	PLATE GIRDER WEB THICK. DEPTH (in)	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118		0.00	0.0000	0.00	0.0000		
2	W	33	118		0.00	0.00001	0.00	0.6250		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	10.00				33.	33.	33.
2	2	2	31.00				33.	33.	33.
3	1	1	10.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO		AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES — OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.44	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.60	0.00	0.00	0 0 0 0 0	0.86	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD or fraction of an axle load for LRFD

0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	51.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	16.10
2	2	Diaphragm	0.000	0.00	0.00	0.29	37.85
3	1	Railing	0.087	0.00	51.00	0.00	0.00
4	1	Curb	0.076	0.00	51.00	0.00	0.00
5	0	Pavement	0.292	0.00	51.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	16.10	33.0	33.0	16.10
16.10	37.85	33.0	33.0	21.75
37.85	51.00	33.0	33.0	13.15

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.726       0.1310     0.8570


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      51.00

     1   DC2     0.087       0.000     51.000

     2   DC2     0.076       0.000     51.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1238.4	45.9	185.8	40.1	225.84

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.705	0.705	0.714	0.714	FOR STRENGTH POSITIVE MOMENT
	0.860				FOR STRENGTH POSITIVE SHEAR
1	0.437				FOR FATIGUE POSITIVE MOMENT
	0.600				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.19	5900.0	32.9	16.45	358.7	358.7
1	2	6.38	5900.0	32.9	16.45	358.7	358.7
1	3	9.56	5900.0	32.9	16.45	358.7	358.7
1	4	12.75	7391.8	32.9	14.52	509.2	388.9
1	5	15.94	7391.8	32.9	14.52	509.2	388.9
1	6	19.12	7391.8	32.9	14.52	509.2	388.9
1	7	22.31	7391.8	32.9	14.52	509.2	388.9
1	8	25.50	7391.8	32.9	14.52	509.2	388.9
1	9	28.69	7391.8	32.9	14.52	509.2	388.9
1	10	31.87	7391.8	32.9	14.52	509.2	388.9
1	11	35.06	7391.8	32.9	14.52	509.2	388.9
1	12	38.25	7391.8	32.9	14.52	509.2	388.9

1 13	41.44	5900.0	32.9	16.45	358.7	358.7
1 14	44.62	5900.0	32.9	16.45	358.7	358.7
1 15	47.81	5900.0	32.9	16.45	358.7	358.7
1 16	51.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L (ft)	SUPT	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00		12487.6	0.024729	493.9	1639.5	785.8
1	1	3.19		12487.6	0.024729	493.9	1639.5	785.8
1	2	6.38		12487.6	0.024729	493.9	1639.5	785.8
1	3	9.56		12487.6	0.024729	493.9	1639.5	785.8
1	4	12.75		16197.6	0.022851	689.8	1612.6	884.2
1	5	15.94		16197.6	0.022851	689.8	1612.6	884.2
1	6	19.12		16197.6	0.022851	689.8	1612.6	884.2
1	7	22.31		16197.6	0.022851	689.8	1612.6	884.2
1	8	25.50		16197.6	0.022851	689.8	1612.6	884.2
1	9	28.69		16197.6	0.022851	689.8	1612.6	884.2
1	10	31.87		16197.6	0.022851	689.8	1612.6	884.2
1	11	35.06		16197.6	0.022851	689.8	1612.6	884.2
1	12	38.25		16197.6	0.022851	689.8	1612.6	884.2
1	13	41.44		12487.6	0.024729	493.9	1639.5	785.8
1	14	44.62		12487.6	0.024729	493.9	1639.5	785.8
1	15	47.81		12487.6	0.024729	493.9	1639.5	785.8
1	16	51.00		12487.6	0.024729	493.9	1639.5	785.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L (ft)	SUPT	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00		16816.5	0.029869	544.6	8325.5	1633.5
1	1	3.19		16816.5	0.029869	544.6	8325.5	1633.5
1	2	6.38		16816.5	0.029869	544.6	8325.5	1633.5
1	3	9.56		16816.5	0.029869	544.6	8325.5	1633.5
1	4	12.75		22573.1	0.027904	756.9	6094.9	1884.5
1	5	15.94		22573.1	0.027904	756.9	6094.9	1884.5
1	6	19.12		22573.1	0.027904	756.9	6094.9	1884.5
1	7	22.31		22573.1	0.027904	756.9	6094.9	1884.5
1	8	25.50		22573.1	0.027904	756.9	6094.9	1884.5
1	9	28.69		22573.1	0.027904	756.9	6094.9	1884.5
1	10	31.87		22573.1	0.027904	756.9	6094.9	1884.5
1	11	35.06		22573.1	0.027904	756.9	6094.9	1884.5

1 12	38.25	22573.1	0.027904	756.9	6094.9	1884.5
1 13	41.44	16816.5	0.029869	544.6	8325.5	1633.5
1 14	44.62	16816.5	0.029869	544.6	8325.5	1633.5
1 15	47.81	16816.5	0.029869	544.6	8325.5	1633.5
1 16	51.00	16816.5	0.029869	544.6	8325.5	1633.5

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	3.19	10.0	55.3	0.9	0.0	66.2
1	2	6.38	18.6	103.3	1.7	0.0	123.6
1	3	9.56	26.0	143.8	2.6	0.0	172.4
1	4	12.75	31.9	177.0	3.4	0.0	212.4
1	5	15.94	36.6	202.8	4.3	0.0	243.8
1	6	19.12	39.9	221.3	4.3	0.0	265.5
1	7	22.31	41.9	232.4	4.3	0.0	278.5
1	8	25.50	42.6	236.0	4.2	0.0	282.8
1	9	28.69	41.9	232.4	4.1	0.0	278.4
1	10	31.87	39.9	221.3	4.1	0.0	265.3
1	11	35.06	36.6	202.8	4.0	0.0	243.5
1	12	38.25	31.9	177.0	3.9	0.0	212.8
1	13	41.44	26.0	143.8	2.9	0.0	172.7
1	14	44.62	18.6	103.3	1.9	0.0	123.8
1	15	47.81	10.0	55.3	1.0	0.0	66.3
1	16	51.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.19	22.3	12.4	0.0	0.0	0.0	34.7
1	2	6.38	41.5	23.2	0.0	0.0	0.0	64.7
1	3	9.56	57.9	32.3	0.0	0.0	0.0	90.1
1	4	12.75	71.2	39.7	0.0	0.0	0.0	110.9
1	5	15.94	81.6	45.5	0.0	0.0	0.0	127.1
1	6	19.12	89.0	49.7	0.0	0.0	0.0	138.7
1	7	22.31	93.5	52.2	0.0	0.0	0.0	145.6
1	8	25.50	94.9	53.0	0.0	0.0	0.0	147.9
1	9	28.69	93.5	52.2	0.0	0.0	0.0	145.6
1	10	31.87	89.0	49.7	0.0	0.0	0.0	138.7
1	11	35.06	81.6	45.5	0.0	0.0	0.0	127.1
1	12	38.25	71.2	39.7	0.0	0.0	0.0	110.9
1	13	41.44	57.9	32.3	0.0	0.0	0.0	90.1
1	14	44.62	41.5	23.2	0.0	0.0	0.0	64.7
1	15	47.81	22.3	12.4	0.0	0.0	0.0	34.7
1	16	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.19	0.0	0.0	196.7	HL-93	0.0	HL-93
1	2	6.38	0.0	0.0	362.0	HL-93	0.0	HL-93
1	3	9.56	0.0	0.0	495.8	HL-93	0.0	HL-93
1	4	12.75	0.0	0.0	595.4	HL-93	0.0	HL-93
1	5	15.94	0.0	0.0	670.9	HL-93	0.0	HL-93
1	6	19.12	0.0	0.0	723.0	HL-93	0.0	HL-93
1	7	22.31	0.0	0.0	741.5	HL-93	0.0	HL-93
1	8	25.50	0.0	0.0	744.9	HL-93	0.0	HL-93
1	9	28.69	0.0	0.0	741.5	HL-93	0.0	HL-93
1	10	31.87	0.0	0.0	723.0	HL-93	0.0	HL-93
1	11	35.06	0.0	0.0	670.9	HL-93	0.0	HL-93
1	12	38.25	0.0	0.0	595.4	HL-93	0.0	HL-93
1	13	41.44	0.0	0.0	495.8	HL-93	0.0	HL-93
1	14	44.62	0.0	0.0	362.0	HL-93	0.0	HL-93
1	15	47.81	0.0	0.0	196.7	HL-93	0.0	HL-93
1	16	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.19	67.	0.	67.
1	2	6.38	119.	0.	119.
1	3	9.56	159.	0.	159.
1	4	12.75	188.	0.	188.
1	5	15.94	205.	0.	205.
1	6	19.12	213.	0.	213.
1	7	22.31	223.	0.	223.
1	8	25.50	228.	0.	228.
1	9	28.69	223.	0.	223.
1	10	31.87	213.	0.	213.
1	11	35.06	205.	0.	205.
1	12	38.25	188.	0.	188.
1	13	41.44	159.	0.	159.
1	14	44.62	119.	0.	119.
1	15	47.81	67.	0.	67.
1	16	51.00	0.	0.	0.

TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.19	66.2	34.7	196.7	HL-93	0.0	HL-93	297.6	100.8
1	2	6.38	123.6	64.7	362.0	HL-93	0.0	HL-93	550.3	188.3
1	3	9.56	172.4	90.1	495.8	HL-93	0.0	HL-93	758.3	262.5
1	4	12.75	212.4	110.9	595.4	HL-93	0.0	HL-93	918.8	323.4
1	5	15.94	243.8	127.1	670.9	HL-93	0.0	HL-93	1041.8	370.9
1	6	19.12	265.5	138.7	723.0	HL-93	0.0	HL-93	1127.2	404.2
1	7	22.31	278.5	145.6	741.5	HL-93	0.0	HL-93	1165.6	424.2
1	8	25.50	282.8	147.9	744.9	HL-93	0.0	HL-93	1175.7	430.8
1	9	28.69	278.4	145.6	741.5	HL-93	0.0	HL-93	1165.5	424.0
1	10	31.87	265.3	138.7	723.0	HL-93	0.0	HL-93	1127.0	404.0
1	11	35.06	243.5	127.1	670.9	HL-93	0.0	HL-93	1041.5	370.6
1	12	38.25	212.8	110.9	595.4	HL-93	0.0	HL-93	919.2	323.8
1	13	41.44	172.7	90.1	495.8	HL-93	0.0	HL-93	758.6	262.8
1	14	44.62	123.8	64.7	362.0	HL-93	0.0	HL-93	550.6	188.6
1	15	47.81	66.3	34.7	196.7	HL-93	0.0	HL-93	297.7	100.9
1	16	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.19	297.6	356.6	475.9	397.2	151.3	50.0
1	2	6.38	550.3	658.9	879.3	734.5	282.5	89.4
1	3	9.56	758.3	907.0	1210.2	1011.9	393.8	119.5
1	4	12.75	918.8	1097.4	1464.0	1225.9	485.1	140.8
1	5	15.94	1041.8	1243.1	1658.1	1389.8	556.3	153.8
1	6	19.12	1127.2	1344.1	1792.7	1503.5	606.3	159.7
1	7	22.31	1165.6	1388.1	1851.1	1554.5	636.2	167.6
1	8	25.50	1175.7	1399.1	1865.8	1567.8	646.1	171.1
1	9	28.69	1165.5	1388.0	1851.0	1554.4	636.1	167.6
1	10	31.87	1127.0	1343.9	1792.4	1503.2	606.0	159.7
1	11	35.06	1041.5	1242.8	1657.8	1389.4	555.9	153.8
1	12	38.25	919.2	1097.9	1464.6	1226.4	485.7	140.8
1	13	41.44	758.6	907.3	1210.6	1012.3	394.3	119.5
1	14	44.62	550.6	659.2	879.6	734.8	282.8	89.4
1	15	47.81	297.7	356.7	476.0	397.3	151.4	50.0
1	16	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.3	18.5	0.3	0.0	22.1
1	1	3.19	2.9	16.2	0.3	0.0	19.4
1	2	6.38	2.5	13.9	0.3	0.0	16.7
1	3	9.56	2.1	11.6	0.3	0.0	13.9
1	4	12.75	1.7	9.3	0.3	0.0	11.2
1	5	15.94	1.3	6.9	0.3	0.0	8.5
1	6	19.12	0.8	4.6	0.0	0.0	5.4
1	7	22.31	0.4	2.3	0.0	0.0	2.7
1	8	25.50	0.0	0.0	0.0	0.0	0.0
1	9	28.69	-0.4	-2.3	0.0	0.0	-2.7
1	10	31.87	-0.8	-4.6	0.0	0.0	-5.5
1	11	35.06	-1.3	-6.9	0.0	0.0	-8.2
1	12	38.25	-1.7	-9.3	-0.3	0.0	-11.2
1	13	41.44	-2.1	-11.6	-0.3	0.0	-14.0
1	14	44.62	-2.5	-13.9	-0.3	0.0	-16.7
1	15	47.81	-2.9	-16.2	-0.3	0.0	-19.4
1	16	51.00	-3.3	-18.5	-0.3	0.0	-22.2

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)

COMPOSITE (k) TOTAL TOTAL

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	COMPOSITE (k)	NONCOMPOSITE (k)	DEAD LOADS (k)
1	0	0.00	7.4	4.2	0.0	0.0	11.6	22.1	33.7
1	1	3.19	6.5	3.6	0.0	0.0	10.2	19.4	29.5
1	2	6.38	5.6	3.1	0.0	0.0	8.7	16.7	25.4
1	3	9.56	4.7	2.6	0.0	0.0	7.3	13.9	21.2
1	4	12.75	3.7	2.1	0.0	0.0	5.8	11.2	17.0
1	5	15.94	2.8	1.6	0.0	0.0	4.4	8.5	12.8
1	6	19.12	1.9	1.0	0.0	0.0	2.9	5.4	8.3
1	7	22.31	0.9	0.5	0.0	0.0	1.5	2.7	4.2
1	8	25.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	9	28.69	-0.9	-0.5	0.0	0.0	-1.5	-2.7	-4.2
1	10	31.87	-1.9	-1.0	0.0	0.0	-2.9	-5.5	-8.4
1	11	35.06	-2.8	-1.6	0.0	0.0	-4.4	-8.2	-12.6
1	12	38.25	-3.7	-2.1	0.0	0.0	-5.8	-11.2	-17.0
1	13	41.44	-4.7	-2.6	0.0	0.0	-7.3	-14.0	-21.2
1	14	44.62	-5.6	-3.1	0.0	0.0	-8.7	-16.7	-25.4
1	15	47.81	-6.5	-3.6	0.0	0.0	-10.2	-19.4	-29.6
1	16	51.00	-7.4	-4.2	0.0	0.0	-11.6	-22.2	-33.8

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	81.3	HL-93	0.0	HL-93
1	1	3.19	0.0	0.0	74.5	HL-93	-2.3	HL-93
1	2	6.38	0.0	0.0	67.7	HL-93	-5.1	HL-93
1	3	9.56	0.0	0.0	61.1	HL-93	-9.0	HL-93
1	4	12.75	0.0	0.0	54.6	HL-93	-12.9	HL-93
1	5	15.94	0.0	0.0	48.1	HL-93	-17.0	HL-93
1	6	19.12	0.0	0.0	41.8	HL-93	-21.2	HL-93
1	7	22.31	0.0	0.0	35.8	HL-93	-25.5	HL-93
1	8	25.50	0.0	0.0	30.0	HL-93	-30.0	HL-93
1	9	28.69	0.0	0.0	25.5	HL-93	-35.8	HL-93
1	10	31.87	0.0	0.0	21.2	HL-93	-41.8	HL-93
1	11	35.06	0.0	0.0	17.0	HL-93	-48.1	HL-93
1	12	38.25	0.0	0.0	12.9	HL-93	-54.6	HL-93
1	13	41.44	0.0	0.0	9.0	HL-93	-61.1	HL-93
1	14	44.62	0.0	0.0	5.1	HL-93	-67.7	HL-93
1	15	47.81	0.0	0.0	2.3	HL-93	-74.5	HL-93
1	16	51.00	0.0	0.0	0.0	HL-93	-81.3	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	TRUCK ONLY NEG	TRUCK ONLY RANGE
1	0	0.00	32.	0.	32.
1	1	3.19	29.	-1.	30.

1	2	6.38	26.	-3.	29.
1	3	9.56	23.	-4.	27.
1	4	12.75	20.	-6.	26.
1	5	15.94	17.	-7.	24.
1	6	19.12	14.	-9.	23.
1	7	22.31	13.	-11.	24.
1	8	25.50	12.	-12.	25.
1	9	28.69	11.	-13.	24.
1	10	31.87	9.	-14.	23.
1	11	35.06	7.	-17.	24.
1	12	38.25	6.	-20.	26.
1	13	41.44	4.	-23.	27.
1	14	44.62	3.	-26.	29.
1	15	47.81	1.	-29.	30.
1	16	51.00	0.	-32.	32.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	31.9	.31072E-01	0.50	0.2500*	.14322E-01	0.23	0.2500*
1	1		3.19	30.1	.31072E-01	0.47	0.2500*	.14322E-01	0.22	0.2500*
1	2		6.38	28.5	.31072E-01	0.44	0.2500*	.14322E-01	0.20	0.2500*
1	3		9.56	27.0	.31072E-01	0.42	0.2500*	.14322E-01	0.19	0.2500*
1	4		12.75	25.7	.29418E-01	0.38	0.2500*	.10052E-01	0.13	0.2500*
1	5		15.94	24.3	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	6		19.12	23.1	.29418E-01	0.34	0.2500*	.10052E-01	0.12	0.2500*
1	7		22.31	24.0	.29418E-01	0.35	0.2500*	.10052E-01	0.12	0.2500*
1	8		25.50	24.6	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	9		28.69	24.0	.29418E-01	0.35	0.2500*	.10052E-01	0.12	0.2500*
1	10		31.87	23.1	.29418E-01	0.34	0.2500*	.10052E-01	0.12	0.2500*
1	11		35.06	24.3	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	12		38.25	25.7	.29418E-01	0.38	0.2500*	.10052E-01	0.13	0.2500*
1	13		41.44	27.0	.31072E-01	0.42	0.2500*	.14322E-01	0.19	0.2500*
1	14		44.62	28.5	.31072E-01	0.44	0.2500*	.14322E-01	0.20	0.2500*
1	15		47.81	30.1	.31072E-01	0.47	0.2500*	.14322E-01	0.22	0.2500*
1	16		51.00	31.9	.31072E-01	0.50	0.2500*	.14322E-01	0.23	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	33.7	81.3	HL-93	0.0	HL-93	115.0	33.7
1	1	3.19	29.5	74.5	HL-93	-2.3	HL-93	104.0	27.2
1	2	6.38	25.4	67.7	HL-93	-5.1	HL-93	93.1	20.2
1	3	9.56	21.2	61.1	HL-93	-9.0	HL-93	82.3	12.2
1	4	12.75	17.0	54.6	HL-93	-12.9	HL-93	71.6	4.1
1	5	15.94	12.8	48.1	HL-93	-17.0	HL-93	61.0	-4.2
1	6	19.12	8.3	41.8	HL-93	-21.2	HL-93	50.2	-12.8
1	7	22.31	4.2	35.8	HL-93	-25.5	HL-93	40.0	-21.3
1	8	25.50	0.0	30.0	HL-93	-30.0	HL-93	30.0	-30.1
1	9	28.69	-4.2	25.5	HL-93	-35.8	HL-93	21.3	-40.0
1	10	31.87	-8.4	21.2	HL-93	-41.8	HL-93	12.8	-50.2
1	11	35.06	-12.6	17.0	HL-93	-48.1	HL-93	4.4	-60.7
1	12	38.25	-17.0	12.9	HL-93	-54.6	HL-93	-4.1	-71.6
1	13	41.44	-21.2	9.0	HL-93	-61.1	HL-93	-12.3	-82.3
1	14	44.62	-25.4	5.1	HL-93	-67.7	HL-93	-20.3	-93.1
1	15	47.81	-29.6	2.3	HL-93	-74.5	HL-93	-27.2	-104.0
1	16	51.00	-33.8	0.0	HL-93	-81.3	HL-93	-33.8	-115.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	115.0	139.4	186.3	153.8	50.6	23.9
1	1	3.19	104.0	126.3	168.9	139.1	44.3	22.6
1	2	6.38	93.1	113.4	151.6	124.5	38.0	21.4
1	3	9.56	82.3	100.6	134.5	110.1	31.8	20.3
1	4	12.75	71.6	87.9	117.7	95.9	25.5	19.2
1	5	15.94	61.0	75.4	101.0	81.7	19.2	18.2
1	6	19.12	50.2	62.7	84.1	67.4	12.5	17.3
1	7	22.31	40.0	50.7	68.1	53.8	6.2	18.0
1	8	25.50	-30.1	-39.1	-52.6	-40.6	0.0	18.4
1	9	28.69	-40.0	-50.7	-68.1	-53.8	-6.3	18.0
1	10	31.87	-50.2	-62.8	-84.1	-67.4	-12.6	17.3
1	11	35.06	-60.7	-75.1	-100.6	-81.4	-18.8	18.2
1	12	38.25	-71.6	-88.0	-117.7	-95.9	-25.5	19.2
1	13	41.44	-82.3	-100.6	-134.6	-110.2	-31.8	20.3
1	14	44.62	-93.1	-113.4	-151.6	-124.6	-38.1	21.4
1	15	47.81	-104.0	-126.4	-168.9	-139.1	-44.4	22.6
1	16	51.00	-115.1	-139.5	-186.3	-153.8	-50.6	23.9

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	33.73	0.00	HL-93	81.30	HL-93	ST1	28.49	186.30
						ST2	28.49	153.78
						ST4	39.42	50.59
						SE1	33.73	115.03
						SE2	33.73	139.42
2	33.76	0.00	HL-93	81.30	HL-93	ST1	28.52	186.34
						ST2	28.52	153.82
						ST4	39.47	50.64
						SE1	33.76	115.06
						SE2	33.76	139.45

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D FROM NO L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1 0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 1	3.19	-0.0194	-0.1074	0.0000	-0.0310	-0.1287	-0.0310	-0.1597
1 2	6.38	-0.0377	-0.2091	0.0000	-0.0602	-0.2508	-0.0602	-0.3110
1 3	9.56	-0.0542	-0.3003	0.0000	-0.0864	-0.3602	-0.0864	-0.4466
1 4	12.75	-0.0682	-0.3780	0.0000	-0.1087	-0.4534	-0.1087	-0.5621
1 5	15.94	-0.0796	-0.4413	0.0000	-0.1268	-0.5293	-0.1268	-0.6561
1 6	19.12	-0.0881	-0.4880	0.0000	-0.1401	-0.5853	-0.1401	-0.7254
1 7	22.31	-0.0932	-0.5166	0.0000	-0.1483	-0.6196	-0.1483	-0.7679
1 8	25.50	-0.0950	-0.5263	0.0000	-0.1510	-0.6312	-0.1510	-0.7822
1 9	28.69	-0.0932	-0.5166	0.0000	-0.1483	-0.6196	-0.1483	-0.7679
1 10	31.87	-0.0881	-0.4880	0.0000	-0.1401	-0.5853	-0.1401	-0.7254
1 11	35.06	-0.0796	-0.4413	0.0000	-0.1268	-0.5294	-0.1268	-0.6561
1 12	38.25	-0.0682	-0.3780	0.0000	-0.1087	-0.4535	-0.1087	-0.5622
1 13	41.44	-0.0542	-0.3003	0.0000	-0.0864	-0.3603	-0.0864	-0.4467
1 14	44.62	-0.0377	-0.2091	0.0000	-0.0602	-0.2508	-0.0602	-0.3111
1 15	47.81	-0.0194	-0.1074	0.0000	-0.0310	-0.1288	-0.0310	-0.1597
1 16	51.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.2	-0.019	0 1/16	-0.109	0 1/ 8	-0.031	0 1/16	-0.160	0 3/16
1	2	6.4	-0.038	0 1/16	-0.213	0 1/ 4	-0.060	0 1/16	-0.311	0 5/16
1	3	9.6	-0.054	0 1/16	-0.306	0 5/16	-0.086	0 1/ 8	-0.447	0 1/ 2
1	4	12.7	-0.068	0 1/ 8	-0.385	0 7/16	-0.109	0 1/ 8	-0.562	0 9/16
1	5	15.9	-0.080	0 1/ 8	-0.450	0 1/ 2	-0.127	0 3/16	-0.656	0 11/16
1	6	19.1	-0.088	0 1/ 8	-0.497	0 1/ 2	-0.140	0 3/16	-0.725	0 3/ 4
1	7	22.3	-0.093	0 1/ 8	-0.526	0 9/16	-0.148	0 3/16	-0.768	0 13/16
1	8	25.5	-0.095	0 1/ 8	-0.536	0 9/16	-0.151	0 3/16	-0.782	0 13/16
1	9	28.7	-0.093	0 1/ 8	-0.526	0 9/16	-0.148	0 3/16	-0.768	0 13/16
1	10	31.9	-0.088	0 1/ 8	-0.497	0 1/ 2	-0.140	0 3/16	-0.725	0 3/ 4
1	11	35.1	-0.080	0 1/ 8	-0.450	0 1/ 2	-0.127	0 3/16	-0.656	0 11/16
1	12	38.2	-0.068	0 1/ 8	-0.385	0 7/16	-0.109	0 1/ 8	-0.562	0 9/16
1	13	41.4	-0.054	0 1/16	-0.306	0 5/16	-0.086	0 1/ 8	-0.447	0 1/ 2
1	14	44.6	-0.038	0 1/16	-0.213	0 1/ 4	-0.060	0 1/16	-0.311	0 5/16
1	15	47.8	-0.019	0 1/16	-0.109	0 1/ 8	-0.031	0 1/16	-0.160	0 3/16
1	16	51.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.50	3 0.425	-0.253	MAX HL-93	0.76	0.00241
			0.026	MIN		
			-0.065	MAX LANE		
			0.026	MIN LANE		
		4 0.433	-0.258	MAX HL-93	0.76	0.00241
			0.026	MIN		
			-0.066	MAX LANE		
			0.026	MIN LANE		
		3 0.425	-0.159	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.33	0.33	-1.88	1.88	-2.21	2.21
1	2	6.38	-0.62	0.62	-3.51	3.51	-4.14	4.14
1	3	9.56	-0.87	0.87	-4.90	4.90	-5.77	5.77
1	4	12.75	-0.99	0.75	-5.57	4.25	-6.56	5.01
1	5	15.94	-1.13	0.86	-6.39	4.88	-7.52	5.74
1	6	19.12	-1.23	0.94	-6.96	5.32	-8.19	6.26
1	7	22.31	-1.29	0.99	-7.30	5.58	-8.60	6.56
1	8	25.50	-1.31	1.00	-7.41	5.66	-8.73	6.67
1	9	28.69	-1.29	0.99	-7.30	5.57	-8.59	6.56
1	10	31.87	-1.23	0.94	-6.95	5.31	-8.19	6.25
1	11	35.06	-1.13	0.86	-6.38	4.88	-7.51	5.74
1	12	38.25	-0.99	0.75	-5.58	4.26	-6.57	5.02
1	13	41.44	-0.87	0.87	-4.91	4.91	-5.78	5.78
1	14	44.62	-0.62	0.62	-3.52	3.52	-4.14	4.14
1	15	47.81	-0.33	0.33	-1.88	1.88	-2.22	2.22
1	16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.02	-0.25	0.84	-0.02	-2.47	3.06
1	2	6.38	-0.04	-0.47	1.57	-0.04	-4.61	5.71
1	3	9.56	-0.05	-0.66	2.19	-0.05	-6.43	7.96
1	4	12.75	-0.06	-0.83	1.93	-0.06	-7.38	6.94
1	5	15.94	-0.06	-0.95	2.21	-0.06	-8.47	7.96
1	6	19.12	-0.07	-1.03	2.41	-0.07	-9.23	8.67
1	7	22.31	-0.07	-1.08	2.53	-0.07	-9.68	9.10
1	8	25.50	-0.07	-1.10	2.57	-0.07	-9.83	9.24
1	9	28.69	-0.07	-1.08	2.53	-0.07	-9.68	9.09
1	10	31.87	-0.07	-1.03	2.41	-0.07	-9.22	8.66
1	11	35.06	-0.06	-0.95	2.21	-0.06	-8.46	7.95
1	12	38.25	-0.06	-0.83	1.93	-0.06	-7.39	6.95

1 13	41.44	-0.05	-0.66	2.19	-0.05	-6.44	7.97
1 14	44.62	-0.04	-0.47	1.57	-0.04	-4.62	5.72
1 15	47.81	-0.02	-0.25	0.84	-0.02	-2.47	3.06
1 16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.16	-0.28	4.34	0.00	0.00	0.00
1	2	6.38	-0.30	-0.52	7.98	0.00	0.00	0.00
1	3	9.56	-0.40	-0.71	10.92	0.00	0.00	0.00
1	4	12.75	-0.42	-1.17	9.44	0.00	0.00	0.00
1	5	15.94	-0.47	-1.32	10.64	0.00	0.00	0.00
1	6	19.12	-0.51	-1.42	11.46	0.00	0.00	0.00
1	7	22.31	-0.52	-1.46	11.75	0.00	0.00	0.00
1	8	25.50	-0.53	-1.47	11.81	0.00	0.00	0.00
1	9	28.69	-0.52	-1.46	11.75	0.00	0.00	0.00
1	10	31.87	-0.51	-1.42	11.46	0.00	0.00	0.00
1	11	35.06	-0.47	-1.32	10.64	0.00	0.00	0.00
1	12	38.25	-0.42	-1.17	9.44	0.00	0.00	0.00
1	13	41.44	-0.40	-0.71	10.92	0.00	0.00	0.00
1	14	44.62	-0.30	-0.52	7.98	0.00	0.00	0.00
1	15	47.81	-0.16	-0.28	4.34	0.00	0.00	0.00
1	16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.19	0.1	0.0	0.1	C	HL	1.5	0.0	1.5	T	HL
1	2	6.38	0.2	0.0	0.2	C	HL	2.6	0.0	2.6	T	HL
1	3	9.56	0.2	0.0	0.2	C	HL	3.5	0.0	3.5	T	HL
1	4	12.75	0.4	0.0	0.4	C	HL	3.0	0.0	3.0	T	HL
1	5	15.94	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	6	19.12	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	7	22.31	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	8	25.50	0.4	0.0	0.4	C	HL	3.6	0.0	3.6	T	HL
1	9	28.69	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	10	31.87	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	11	35.06	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	12	38.25	0.4	0.0	0.4	C	HL	3.0	0.0	3.0	T	HL
1	13	41.44	0.2	0.0	0.2	C	HL	3.5	0.0	3.5	T	HL
1	14	44.62	0.2	0.0	0.2	C	HL	2.6	0.0	2.6	T	HL
1	15	47.81	0.1	0.0	0.1	C	HL	1.5	0.0	1.5	T	HL
1	16	51.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.18	-2.75	7.39	-0.02	-2.47	3.06
1	2	6.38	33.	33.	-0.33	-5.13	13.69	-0.04	-4.61	5.71
1	3	9.56	33.	33.	-0.46	-7.14	18.88	-0.05	-6.43	7.96
1	4	12.75	33.	33.	-0.48	-8.55	16.38	-0.06	-7.38	6.94
1	5	15.94	33.	33.	-0.54	-9.79	18.59	-0.06	-8.47	7.96
1	6	19.12	33.	33.	-0.58	-10.65	20.13	-0.07	-9.23	8.67
1	7	22.31	33.	33.	-0.60	-11.14	20.85	-0.07	-9.68	9.10
1	8	25.50	33.	33.	-0.60	-11.30	21.05	-0.07	-9.83	9.24
1	9	28.69	33.	33.	-0.60	-11.14	20.85	-0.07	-9.68	9.09
1	10	31.87	33.	33.	-0.58	-10.64	20.13	-0.07	-9.22	8.66
1	11	35.06	33.	33.	-0.54	-9.78	18.59	-0.06	-8.46	7.95
1	12	38.25	33.	33.	-0.48	-8.57	16.39	-0.06	-7.39	6.95
1	13	41.44	33.	33.	-0.46	-7.15	18.89	-0.05	-6.44	7.97
1	14	44.62	33.	33.	-0.33	-5.14	13.69	-0.04	-4.62	5.72
1	15	47.81	33.	33.	-0.18	-2.75	7.40	-0.02	-2.47	3.06
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.23	-2.84	8.69	-0.02	-2.47	3.06
1	2	6.38	33.	33.	-0.42	-5.29	16.08	-0.04	-4.61	5.71
1	3	9.56	33.	33.	-0.58	-7.36	22.16	-0.05	-6.43	7.96
1	4	12.75	33.	33.	-0.60	-8.90	19.21	-0.06	-7.38	6.94
1	5	15.94	33.	33.	-0.68	-10.19	21.78	-0.06	-8.47	7.96
1	6	19.12	33.	33.	-0.73	-11.08	23.57	-0.07	-9.23	8.67
1	7	22.31	33.	33.	-0.76	-11.58	24.38	-0.07	-9.68	9.10
1	8	25.50	33.	33.	-0.76	-11.74	24.59	-0.07	-9.83	9.24
1	9	28.69	33.	33.	-0.76	-11.57	24.38	-0.07	-9.68	9.09
1	10	31.87	33.	33.	-0.73	-11.07	23.56	-0.07	-9.22	8.66
1	11	35.06	33.	33.	-0.68	-10.18	21.78	-0.06	-8.46	7.95
1	12	38.25	33.	33.	-0.60	-8.92	19.22	-0.06	-7.39	6.95
1	13	41.44	33.	33.	-0.58	-7.37	22.17	-0.05	-6.44	7.97
1	14	44.62	33.	33.	-0.42	-5.30	16.09	-0.04	-4.62	5.72
1	15	47.81	33.	33.	-0.23	-2.84	8.70	-0.02	-2.47	3.06
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.31	-3.62	11.54	-0.03	-3.13	3.96
1	2	6.38	33.	33.	-0.57	-6.75	21.35	-0.05	-5.84	7.39
1	3	9.56	33.	33.	-0.78	-9.39	29.42	-0.07	-8.14	10.30
1	4	12.75	33.	33.	-0.82	-11.41	25.50	-0.08	-9.36	8.98
1	5	15.94	33.	33.	-0.92	-13.05	28.91	-0.09	-10.74	10.30
1	6	19.12	33.	33.	-0.99	-14.19	31.28	-0.10	-11.70	11.22
1	7	22.31	33.	33.	-1.02	-14.83	32.35	-0.10	-12.27	11.78
1	8	25.50	33.	33.	-1.03	-15.03	32.63	-0.10	-12.46	11.96
1	9	28.69	33.	33.	-1.02	-14.82	32.35	-0.10	-12.27	11.77
1	10	31.87	33.	33.	-0.99	-14.18	31.28	-0.10	-11.69	11.22
1	11	35.06	33.	33.	-0.92	-13.04	28.91	-0.09	-10.73	10.29
1	12	38.25	33.	33.	-0.82	-11.43	25.51	-0.08	-9.37	8.99
1	13	41.44	33.	33.	-0.78	-9.40	29.43	-0.07	-8.15	10.31
1	14	44.62	33.	33.	-0.57	-6.76	21.36	-0.05	-5.85	7.40
1	15	47.81	33.	33.	-0.31	-3.63	11.55	-0.03	-3.13	3.96
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.30	-2.68	10.20	-0.01	-2.18	2.62
1	2	6.38	33.	33.	-0.54	-4.99	18.85	-0.03	-4.07	4.89
1	3	9.56	33.	33.	-0.75	-6.93	25.93	-0.04	-5.68	6.81
1	4	12.75	33.	33.	-0.78	-8.56	22.45	-0.04	-6.51	5.93
1	5	15.94	33.	33.	-0.88	-9.78	25.42	-0.05	-7.47	6.81
1	6	19.12	33.	33.	-0.95	-10.63	27.47	-0.05	-8.14	7.42
1	7	22.31	33.	33.	-0.97	-11.09	28.35	-0.05	-8.54	7.78
1	8	25.50	33.	33.	-0.98	-11.24	28.57	-0.05	-8.67	7.90
1	9	28.69	33.	33.	-0.97	-11.09	28.35	-0.05	-8.53	7.78
1	10	31.87	33.	33.	-0.95	-10.62	27.47	-0.05	-8.13	7.41
1	11	35.06	33.	33.	-0.88	-9.77	25.41	-0.05	-7.46	6.80
1	12	38.25	33.	33.	-0.78	-8.57	22.46	-0.04	-6.52	5.94
1	13	41.44	33.	33.	-0.75	-6.94	25.94	-0.04	-5.69	6.82
1	14	44.62	33.	33.	-0.54	-4.99	18.85	-0.03	-4.08	4.89
1	15	47.81	33.	33.	-0.30	-2.68	10.21	-0.01	-2.18	2.62
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.24	-3.51	9.81	-0.03	-3.13	3.96
1	2	6.38	33.	33.	-0.45	-6.54	18.16	-0.05	-5.84	7.39
1	3	9.56	33.	33.	-0.62	-9.10	25.05	-0.07	-8.14	10.30
1	4	12.75	33.	33.	-0.65	-10.94	21.72	-0.08	-9.36	8.98
1	5	15.94	33.	33.	-0.73	-12.52	24.66	-0.09	-10.74	10.30
1	6	19.12	33.	33.	-0.79	-13.62	26.70	-0.10	-11.70	11.22
1	7	22.31	33.	33.	-0.81	-14.24	27.65	-0.10	-12.27	11.78
1	8	25.50	33.	33.	-0.82	-14.44	27.90	-0.10	-12.46	11.96
1	9	28.69	33.	33.	-0.81	-14.24	27.64	-0.10	-12.27	11.77
1	10	31.87	33.	33.	-0.79	-13.61	26.69	-0.10	-11.69	11.22
1	11	35.06	33.	33.	-0.73	-12.51	24.65	-0.09	-10.73	10.29
1	12	38.25	33.	33.	-0.65	-10.96	21.74	-0.08	-9.37	8.99
1	13	41.44	33.	33.	-0.62	-9.12	25.06	-0.07	-8.15	10.31
1	14	44.62	33.	33.	-0.45	-6.55	18.17	-0.05	-5.85	7.40
1	15	47.81	33.	33.	-0.24	-3.51	9.81	-0.03	-3.13	3.96
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.23	-2.56	8.47	-0.01	-2.18	2.62
1	2	6.38	33.	33.	-0.43	-4.78	15.65	-0.03	-4.07	4.89
1	3	9.56	33.	33.	-0.58	-6.64	21.56	-0.04	-5.68	6.81
1	4	12.75	33.	33.	-0.61	-8.09	18.68	-0.04	-6.51	5.93
1	5	15.94	33.	33.	-0.69	-9.25	21.16	-0.05	-7.47	6.81
1	6	19.12	33.	33.	-0.74	-10.06	22.89	-0.05	-8.14	7.42
1	7	22.31	33.	33.	-0.76	-10.51	23.65	-0.05	-8.54	7.78
1	8	25.50	33.	33.	-0.77	-10.65	23.84	-0.05	-8.67	7.90
1	9	28.69	33.	33.	-0.76	-10.50	23.65	-0.05	-8.53	7.78
1	10	31.87	33.	33.	-0.74	-10.05	22.88	-0.05	-8.13	7.41
1	11	35.06	33.	33.	-0.69	-9.25	21.16	-0.05	-7.46	6.80
1	12	38.25	33.	33.	-0.61	-8.10	18.69	-0.04	-6.52	5.94
1	13	41.44	33.	33.	-0.58	-6.65	21.57	-0.04	-5.69	6.82
1	14	44.62	33.	33.	-0.43	-4.78	15.66	-0.03	-4.08	4.89
1	15	47.81	33.	33.	-0.23	-2.57	8.47	-0.01	-2.18	2.62
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP	JOINT	D FROM	DL1, N=INF.	DL2, N=27.0	POS. LL+I	NEG. LL+I	TOTAL STRESS --AND--

NO	TYPE	L SUPT (ft)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	STRESS RANGE
1	LT	10.00	178.	-6.0	93.	-0.7	509.	-0.7	0.	0.0	-7.37
											0.73
	LB		6.0		2.3		11.2			0.0	19.44
											11.23
	RT		-5.5		-0.7		-1.0			0.0	-7.18
											1.00
	RB		4.2		1.6		8.1			0.0	13.89
											8.08
1	LT	41.00	178.	-5.5	93.	-0.7	509.	-1.0	0.	0.0	-7.19
											1.00
	LB		4.2		1.6		8.1			0.0	13.89
											8.08
	RT		-6.0		-0.7		-0.7			0.0	-7.38
											0.73
	RB		6.0		2.3		11.2			0.0	19.45
											11.23

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	10.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	31.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.6250
3	10.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 51.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. / [3]= GIRDER D.	[2]/(12*[1]) / [3]/(12*[1])	<=>	LIMITATION 1/25 / 1/30	STATUS
1	10.00	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK
				[3]=	32.90	0.05376	>	0.03300	OK
2	31.00	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK
				[3]=	32.90	0.05376	>	0.03300	OK
3	10.00	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK

[3]= 32.90 0.05376 > 0.03300 OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN D NO	L FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	1	3.19	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	2	6.38	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	3	9.56	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	4	12.75	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	5	15.94	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	6	19.12	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	7	22.31	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	8	25.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	9	28.69	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	10	31.87	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	11	35.06	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	12	38.25	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	13	41.44	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	14	44.62	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	15	47.81	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	16	51.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0

NOTE:

[1] 0 - No long. stiffeners
1 - long. stiffeners

[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)

[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0

[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)

[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw

[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	[3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	3.19	0	1	31.42	0.550	57.13	168.97	2
1	2	6.38	0	1	31.42	0.550	57.13	168.97	2
1	3	9.56	0	1	31.42	0.550	57.13	168.97	2
1	4	12.75	0	1	31.42	0.550	66.52	168.97	2
1	5	15.94	0	1	31.42	0.550	66.52	168.97	2
1	6	19.12	0	1	31.42	0.550	66.52	168.97	2
1	7	22.31	0	1	31.42	0.550	66.52	168.97	2
1	8	25.50	0	1	31.42	0.550	66.52	168.97	2
1	9	28.69	0	1	31.42	0.550	66.52	168.97	2
1	10	31.87	0	1	31.42	0.550	66.52	168.97	2
1	11	35.06	0	1	31.42	0.550	66.52	168.97	2
1	12	38.25	0	1	31.42	0.550	66.52	168.97	2
1	13	41.44	0	1	31.42	0.550	57.13	168.97	2
1	14	44.62	0	1	31.42	0.550	57.13	168.97	2
1	15	47.81	0	1	31.42	0.550	57.13	168.97	2
1	16	51.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	1	3.19	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	2	6.38	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	3	9.56	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	4	12.75	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	5	15.94	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	6	19.12	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	7	22.31	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	8	25.50	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	9	28.69	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	10	31.87	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	11	35.06	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	12	38.25	7.77	12.	11.5	5.2	0.740	0.605	

```
                 7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  41.44    7.77 12.  11.5   5.2  0.740  0.605
                 7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  44.62    7.77 12.  11.5   5.2  0.740  0.605
                 7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  47.81    7.77 12.  11.5   5.2  0.740  0.605
                 7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 16  51.00    7.77 12.  11.5   5.2  0.740  0.605
                 7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
1
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0  16.1   0.000  11.334   7.490   3.646   1.000
                       0.000   0.000   0.000   0.000   1.000
    2     16.1  37.8  10.030  11.334  12.997  14.660   1.000
                       0.000   0.000   0.000   0.000   1.000
    3     37.8  51.0   0.000  10.030   6.369   2.707   1.000
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
1
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
         (ft)                          CAT  (ft)     (ft)     (ft)    CAT CAT
-------------------------------------------------------------------------------

  1  0   0.00  1  7.77  11.26  19.84   0    16.10     7.09    26.62   2   2
  1  1   3.19  1  7.77  11.26  19.84   0    16.10     7.09    26.62   2   2
  1  2   6.38  1  7.77  11.26  19.84   0    16.10     7.09    26.62   2   2
  1  3   9.56  1  7.77  11.26  19.84   0    16.10     7.09    26.62   2   2
  1  4  12.75  1  7.77  11.26  19.84   0    16.10     6.95    26.08   2   2
  1  5  15.94  1  7.77  11.26  19.84   0    16.10     6.95    26.08   2   2
  1  6  19.12  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1  7  22.31  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1  8  25.50  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1  9  28.69  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1 10  31.87  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1 11  35.06  1  7.77  11.26  19.84   0    21.75     6.95    26.08   2   2
  1 12  38.25  1  7.77  11.26  19.84   0    13.15     6.95    26.08   2   2
  1 13  41.44  1  7.77  11.26  19.84   0    13.15     7.09    26.62   2   2
```

1	14	44.62	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2
1	15	47.81	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2
1	16	51.00	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2

```
 NOTE: LMDAf = slenderness ratio for the comp. flange
             = bfc / 2tfc (Eq. 6.10.8.2.2-3)
       LMDApf = limiting slenderness ratio for a compact flange
              = 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
       LMDArf = limiting slenderness ration for a non-compct flange
              = 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
       Lb = unbraced length
       Lp = limiting unbraced length to achieve the nominal flexural
            of RbRhFyc under uniform bending
          = 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
       Lr = limiting unbraced length to achieve the onset of
            nominal yielding in either flange under uniform
            bending with consideration of comp. flange
            residual stress effects
          = pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

       In negative moment region, the fist line is for non-composite sections
       and the second line is for composite sections

       Flange Category
        0 = compact section
        2 = non-compact section
        3 = slender section
```

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.00	33.00	28.43	28.43
1	1	3.19	1	1.000	1.000	1.00	33.00	28.43	28.43
1	2	6.38	1	1.000	1.000	1.00	33.00	28.43	28.43
1	3	9.56	1	1.000	1.000	1.00	33.00	28.43	28.43
1	4	12.75	1	1.000	1.000	1.00	33.00	28.26	28.26
1	5	15.94	1	1.000	1.000	1.00	33.00	28.26	28.26
1	6	19.12	1	1.000	1.000	1.00	33.00	25.34	25.34
1	7	22.31	1	1.000	1.000	1.00	33.00	25.34	25.34
1	8	25.50	1	1.000	1.000	1.00	33.00	25.34	25.34
1	9	28.69	1	1.000	1.000	1.00	33.00	25.34	25.34
1	10	31.87	1	1.000	1.000	1.00	33.00	25.34	25.34
1	11	35.06	1	1.000	1.000	1.00	33.00	25.34	25.34
1	12	38.25	1	1.000	1.000	1.00	33.00	29.79	29.79
1	13	41.44	1	1.000	1.000	1.00	33.00	29.93	29.93
1	14	44.62	1	1.000	1.000	1.00	33.00	29.93	29.93
1	15	47.81	1	1.000	1.000	1.00	33.00	29.93	29.93
1	16	51.00	1	1.000	1.000	1.00	33.00	29.93	29.93

```
 Note: In the positive moment region, the result is for DL case
       In the negative moment region, the 1st line is for DL case
       and the 2nd line is for LL case
```

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0	0.00	1	4.70	17.29
1	1	3.19	1	4.70	17.29
1	2	6.38	1	4.70	17.29
1	3	9.56	1	4.70	17.29
1	4	12.75	1	5.55	17.56
1	5	15.94	1	5.55	17.56
1	6	19.12	1	5.55	17.56
1	7	22.31	1	5.55	17.56
1	8	25.50	1	5.55	17.56
1	9	28.69	1	5.55	17.56
1	10	31.87	1	5.55	17.56
1	11	35.06	1	5.55	17.56
1	12	38.25	1	5.55	17.56
1	13	41.44	1	4.70	17.29
1	14	44.62	1	4.70	17.29
1	15	47.81	1	4.70	17.29
1	16	51.00	1	4.70	17.29

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.19	1	0	2	0		2	0
1	2	6.38	1	0	2	0		2	0
1	3	9.56	1	0	2	0		2	0
1	4	12.75	1	0	2	0		2	0
1	5	15.94	1	0	2	0		2	0
1	6	19.12	1	0	2	0		2	0
1	7	22.31	1	0	2	0		2	0
1	8	25.50	1	0	2	0		2	0
1	9	28.69	1	0	2	0		2	0
1	10	31.87	1	0	2	0		2	0
1	11	35.06	1	0	2	0		2	0
1	12	38.25	1	0	2	0		2	0
1	13	41.44	1	0	2	0		2	0
1	14	44.62	1	0	2	0		2	0
1	15	47.81	1	0	2	0		2	0
1	16	51.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute

section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	28.4	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.19	0.0	19.8	3.3	33.0	3.3	33.0	3.3	28.4	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	2		6.38	0.0	19.8	6.2	33.0	6.2	33.0	6.2	28.4	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	3		9.56	0.0	19.8	8.7	33.0	8.7	33.0	8.7	28.4	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	4		12.75	0.0	19.8	9.8	33.0	9.8	33.0	9.8	28.3	0
				0.0	-	7.5	-	7.5	33.0	-	-	0
1	5		15.94	0.0	19.8	11.3	33.0	11.3	33.0	11.3	28.3	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	6		19.12	0.0	19.8	12.3	33.0	12.3	33.0	12.3	25.3	0
				0.0	-	9.4	-	9.4	33.0	-	-	0
1	7		22.31	0.0	19.8	12.9	33.0	12.9	33.0	12.9	25.3	0
				0.0	-	9.8	-	9.8	33.0	-	-	0
1	8		25.50	0.0	19.8	13.1	33.0	13.1	33.0	13.1	25.3	0
				0.0	-	10.0	-	10.0	33.0	-	-	0
1	9		28.69	0.0	19.8	12.9	33.0	12.9	33.0	12.9	25.3	0
				0.0	-	9.8	-	9.8	33.0	-	-	0
1	10		31.87	0.0	19.8	12.3	33.0	12.3	33.0	12.3	25.3	0
				0.0	-	9.4	-	9.4	33.0	-	-	0
1	11		35.06	0.0	19.8	11.3	33.0	11.3	33.0	11.3	25.3	0
				0.0	-	8.6	-	8.6	33.0	-	-	0
1	12		38.25	0.0	19.8	9.9	33.0	9.9	33.0	9.9	29.8	0
				0.0	-	7.5	-	7.5	33.0	-	-	0
1	13		41.44	0.0	19.8	8.7	33.0	8.7	33.0	8.7	29.9	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	14		44.62	0.0	19.8	6.2	33.0	6.2	33.0	6.2	29.9	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	15		47.81	0.0	19.8	3.3	33.0	3.3	33.0	3.3	29.9	0
				0.0	-	3.3	-	3.3	33.0	-	-	0
1	16		51.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	29.9	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.19	0.000	0.101	0.101	0.117	0.117	4
					0.101		0.101	3
1	2	6.38	0.000	0.188	0.188	0.218	0.218	4
					0.188		0.188	3
1	3	9.56	0.000	0.262	0.262	0.304	0.304	4
					0.262		0.262	3
1	4	12.75	0.000	0.298	0.298	0.348	0.348	4
					0.228		0.228	3
1	5	15.94	0.000	0.342	0.342	0.399	0.399	4
					0.261		0.261	3
1	6	19.12	0.000	0.372	0.372	0.485	0.485	4
					0.284		0.284	3
1	7	22.31	0.000	0.391	0.391	0.509	0.509	4
					0.298		0.298	3
1	8	25.50	0.000	0.397	0.397	0.517	0.517	4
					0.303		0.303	3
1	9	28.69	0.000	0.391	0.391	0.509	0.509	4
					0.298		0.298	3
1	10	31.87	0.000	0.372	0.372	0.485	0.485	4
					0.284		0.284	3
1	11	35.06	0.000	0.342	0.342	0.445	0.445	4
					0.261		0.261	3
1	12	38.25	0.000	0.299	0.299	0.331	0.331	4
					0.228		0.228	3
1	13	41.44	0.000	0.263	0.263	0.290	0.290	4
					0.263		0.263	3
1	14	44.62	0.000	0.188	0.188	0.208	0.208	4
					0.188		0.188	3
1	15	47.81	0.000	0.101	0.101	0.111	0.111	4
					0.101		0.101	3
1	16	51.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2089.9	-	-	0.0	19.8	-	-	0--0
1	1		3.19	0	475.9	2089.9	-	-	0.0	19.8	-	-	0--0
1	2		6.38	0	879.3	2089.9	-	-	0.0	19.8	-	-	0--0
1	3		9.56	0	1210.2	2089.9	-	-	0.0	19.8	-	-	0--0
1	4		12.75	0	1464.0	2672.8	-	-	0.0	19.8	-	-	0--0
1	5		15.94	0	1658.1	2672.8	-	-	0.0	19.8	-	-	0--0
1	6		19.12	0	1792.7	2672.8	-	-	0.0	19.8	-	-	0--0
1	7		22.31	0	1851.1	2672.8	-	-	0.0	19.8	-	-	0--0
1	8		25.50	0	1865.8	2672.8	-	-	0.0	19.8	-	-	0--0
1	9		28.69	0	1851.0	2672.8	-	-	0.0	19.8	-	-	0--0
1	10		31.87	0	1792.4	2672.8	-	-	0.0	19.8	-	-	0--0
1	11		35.06	0	1657.8	2672.8	-	-	0.0	19.8	-	-	0--0
1	12		38.25	0	1464.6	2672.8	-	-	0.0	19.8	-	-	0--0
1	13		41.44	0	1210.6	2089.9	-	-	0.0	19.8	-	-	0--0
1	14		44.62	0	879.6	2089.9	-	-	0.0	19.8	-	-	0--0
1	15		47.81	0	476.0	2089.9	-	-	0.0	19.8	-	-	0--0
1	16		51.00	0	0.0	2089.9	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		3.19	0	0.228				0.228	1
1	2		6.38	0	0.421				0.421	1
1	3		9.56	0	0.579				0.579	1
1	4		12.75	0	0.548				0.548	1

1	5	15.94	0	0.620	0.620	1
1	6	19.12	0	0.671	0.671	1
1	7	22.31	0	0.693	0.693	1
1	8	25.50	0	0.698	0.698	1
1	9	28.69	0	0.693	0.693	1
1	10	31.87	0	0.671	0.671	1
1	11	35.06	0	0.620	0.620	1
1	12	38.25	0	0.548	0.548	1
1	13	41.44	0	0.579	0.579	1
1	14	44.62	0	0.421	0.421	1
1	15	47.81	0	0.228	0.228	1
1	16	51.00	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

1

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.19	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.38	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	9.56	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	12.75	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	15.94	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	19.12	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	22.31	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	25.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	28.69	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	31.87	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	35.06	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	38.25	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	41.44	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	44.62	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	47.81	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	51.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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```
NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
           = 5 for unstiffened beams and girders
           = 5 for stiffened girders when (d0/D) > 3, or
                                          (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                     ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                     ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
```

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	186.3	<	330.8	0	0
1	1	3.19	0	168.9	<	330.8	0	0
1	2	6.38	0	151.6	<	330.8	0	0
1	3	9.56	0	134.5	<	330.8	0	0
1	4	12.75	0	117.7	<	330.8	0	0
1	5	15.94	0	101.0	<	330.8	0	0
1	6	19.12	0	84.1	<	330.8	0	0
1	7	22.31	0	68.1	<	330.8	0	0
1	8	25.50	0	52.6	<	330.8	0	0
1	9	28.69	0	68.1	<	330.8	0	0
1	10	31.87	0	84.1	<	330.8	0	0
1	11	35.06	0	100.6	<	330.8	0	0
1	12	38.25	0	117.7	<	330.8	0	0
1	13	41.44	0	134.6	<	330.8	0	0
1	14	44.62	0	151.6	<	330.8	0	0
1	15	47.81	0	168.9	<	330.8	0	0
1	16	51.00	0	186.3	<	330.8	0	0

Please read NOTE on the following page

```
     NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

               For compact section  (composite sections in positive flexure)

                 w/o long. stiff.

                 D/tw <= 150 (Eq. 6.10.2.1.1-1)

                 w / long. stiff.

                 D/tw <= 300 (Eq. 6.10.2.1.2-1)

                 2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

               For compact or non-compact sections specified in Art. 6.10.8

                 (composite sections in negative flexure and
                  non-composite sections)

                 2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


          [2]: SHEAR CAPACITY ... web crippling control

               Vn = CVp .......... AASHTO Eq.6.10.9.2-1

               Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

    Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

               Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

    Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

               Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

             * For the detailed description of shear capacity,
               please refer to AASHTO Art.6.10.9.2 and .3

             * STRENGTH CATEGORY; 0 = compact section
                                  2 = braced non-compact section
                                  3 = slender section
```


TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	186.30	330.76	0	
1	1	3.19	33.	168.85	330.76	0	
1	2	6.38	33.	151.60	330.76	0	
1	3	9.56	33.	134.55	330.76	0	
1	4	12.75	33.	117.68	330.76	0	
1	5	15.94	33.	100.96	330.76	0	
1	6	19.12	33.	84.09	330.76	0	
1	7	22.31	33.	68.07	330.76	0	
1	8	25.50	33.	52.60	330.76	0	
1	9	28.69	33.	68.12	330.76	0	
1	10	31.87	33.	84.13	330.76	0	
1	11	35.06	33.	100.64	330.76	0	
1	12	38.25	33.	117.72	330.76	0	
1	13	41.44	33.	134.59	330.76	0	
1	14	44.62	33.	151.64	330.76	0	
1	15	47.81	33.	168.89	330.76	0	
1	16	51.00	33.	186.34	330.76	0	

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     TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
                     *************************

     SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                   ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
     NO NO L SUPT                                          [3]  TBC
             (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi)
     ------------------------------------------------------------

      1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
      1  1   3.19   2.8    -     31.4     8.7  31.4   -     -   00-
      1  2   6.38   5.3    -     31.4    16.1  31.4   -     -   00-
      1  3   9.56   7.4    -     31.4    22.2  31.4   -     -   00-
      1  4  12.75   8.9    -     31.4    19.2  31.4   -     -   00-
      1  5  15.94  10.2    -     31.4    21.8  31.4   -     -   00-
      1  6  19.12  11.1    -     31.4    23.6  31.4   -     -   00-
      1  7  22.31  11.6    -     31.4    24.4  31.4   -     -   00-
      1  8  25.50  11.7    -     31.4    24.6  31.4   -     -   00-
      1  9  28.69  11.6    -     31.4    24.4  31.4   -     -   00-
      1 10  31.87  11.1    -     31.4    23.6  31.4   -     -   00-
      1 11  35.06  10.2    -     31.4    21.8  31.4   -     -   00-
      1 12  38.25   8.9    -     31.4    19.2  31.4   -     -   00-
      1 13  41.44   7.4    -     31.4    22.2  31.4   -     -   00-
      1 14  44.62   5.3    -     31.4    16.1  31.4   -     -   00-
      1 15  47.81   2.8    -     31.4     8.7  31.4   -     -   00-
      1 16  51.00   0.0    -     31.4     0.0  31.4   -     -   00-


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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members
```

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.19	TR	-0.1	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	2	6.38	TR	-0.3	A B B^C^C D E E^	TR	3.9	A B B^C^C D E
1	3	9.56	TR	-0.3	A B B^C^C D E E^	TR	5.3	A B B^C^C D
1		10.00	SCG					
1	4	12.75	TR	-0.6	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	5	15.94	TR	-0.6	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	6	19.12	TR	-0.6	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1	7	22.31	TR	-0.7	A B B^C^C D E E^	TR	5.3	A B B^C^C D
1	8	25.50	TR	-0.7	A B B^C^C D E E^	TR	5.4	A B B^C^C D
1	9	28.69	TR	-0.7	A B B^C^C D E E^	TR	5.3	A B B^C^C D
1	10	31.87	TR	-0.6	A B B^C^C D E E^	TR	5.1	A B B^C^C D
1	11	35.06	TR	-0.6	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	12	38.25	TR	-0.6	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1		41.00	SCG					
1	13	41.44	TR	-0.3	A B B^C^C D E E^	TR	5.3	A B B^C^C D
1	14	44.62	TR	-0.3	A B B^C^C D E E^	TR	3.9	A B B^C^C D E
1	15	47.81	TR	-0.1	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	16	51.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.19	TR	-0.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2	6.38	TR	-0.1	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	3	9.56	TR	-0.2	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1		10.00	SCG					
1	4	12.75	TR	-0.3	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	5	15.94	TR	-0.3	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	6	19.12	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	7	22.31	TR	-0.3	A B B^C^C D E E^	TR	2.7	A B B^C^C D E

```
 1  8  25.50    TR     -0.3 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
 1  9  28.69    TR     -0.3 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
 1 10  31.87    TR     -0.3 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1 11  35.06    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 12  38.25    TR     -0.3 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1     41.00  SCG
 1 13  41.44    TR     -0.2 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1 14  44.62    TR     -0.1 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1 15  47.81    TR     -0.1 A B B^C^C D E E^  TR      1.1 A B B^C^C D E E^
 1 16  51.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   *******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-1     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

  1  0   0.00  47.9 .29869E-01    1.4      TR    3.09        3           6.49
  1  1   3.19  45.2 .29869E-01    1.4      TR    3.09        3           6.87
  1  2   6.38  42.8 .29869E-01    1.3      TR    3.09        3           7.26
  1  3   9.56  40.5 .29869E-01    1.2      TR    3.09        3           7.67
  1  4  12.75  38.5 .27904E-01    1.1      TR    3.09        3           8.64
  1  5  15.94  36.4 .27904E-01    1.0      TR    3.09        3           9.13
  1  6  19.12  34.7 .27904E-01    1.0      TR    3.09        3           9.59
  1  7  22.31  35.9 .27904E-01    1.0      TR    3.09        3           9.26
  1  8  25.50  36.8 .27904E-01    1.0      TR    3.09        3           9.03
  1  9  28.69  35.9 .27904E-01    1.0      TR    3.09        3           9.26
  1 10  31.87  34.7 .27904E-01    1.0      TR    3.09        3           9.59
  1 11  35.06  36.4 .27904E-01    1.0      TR    3.09        3           9.13
  1 12  38.25  38.5 .27904E-01    1.1      TR    3.09        3           8.64
  1 13  41.44  40.5 .29869E-01    1.2      TR    3.09        3           7.67
  1 14  44.62  42.8 .29869E-01    1.3      TR    3.09        3           7.26
  1 15  47.81  45.2 .29869E-01    1.4      TR    3.09        3           6.87
  1 16  51.00  47.9 .29869E-01    1.4      TR    3.09        3           6.49


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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2
```

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	23.9	.29869E-01	0.7	TR 1.36	3	5.72
1	1	3.19	22.6	.29869E-01	0.7	TR 1.36	3	6.06
1	2	6.38	21.4	.29869E-01	0.6	TR 1.36	3	6.40
1	3	9.56	20.3	.29869E-01	0.6	TR 1.36	3	6.76
1	4	12.75	19.2	.27904E-01	0.5	TR 1.36	3	7.62
1	5	15.94	18.2	.27904E-01	0.5	TR 1.36	3	8.05
1	6	19.12	17.3	.27904E-01	0.5	TR 1.36	3	8.45
1	7	22.31	18.0	.27904E-01	0.5	TR 1.36	3	8.16
1	8	25.50	18.4	.27904E-01	0.5	TR 1.36	3	7.96
1	9	28.69	18.0	.27904E-01	0.5	TR 1.36	3	8.16
1	10	31.87	17.3	.27904E-01	0.5	TR 1.36	3	8.45
1	11	35.06	18.2	.27904E-01	0.5	TR 1.36	3	8.05
1	12	38.25	19.2	.27904E-01	0.5	TR 1.36	3	7.62
1	13	41.44	20.3	.29869E-01	0.6	TR 1.36	3	6.76
1	14	44.62	21.4	.29869E-01	0.6	TR 1.36	3	6.40
1	15	47.81	22.6	.29869E-01	0.7	TR 1.36	3	6.06
1	16	51.00	23.9	.29869E-01	0.7	TR 1.36	3	5.72

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```
 NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                section, the range of shear shall be taken as the difference
                in the minimum and maximum shear envelopes (excluding dead
                loads); AASHTO LRFD Eqn.6.10.10.1.2-2

       [2] Q/I : Q= statical moment about the neutral axis of the composite
                   section of the transformed compressive concrete area or
                   the area of reinforcement embedded in the concrete for
                   negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                 I= moment of inertia of the transformed composite girder in
                   positive moment regions or the moment of inertia provided
                   by the steel beam including or excluding the area of
                   reinforcement embedded in the concrete in negative moment
                   regions, in inches to the fourth power.

       [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                 of the slab and girder at the point in the span under
                 consideration.
               = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
           Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
           Ffat = radial fatigue shear range per unit length
                = Ffat2 = Frc/w for skews exceeding 20 degrees
           Frc = 25.0 kips for both exterior and interior girders
           w = the effective length of the deck
             = 48 inches except at end supports where w is taken
               as 24 inches

       [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

           * Default ALPHA value based on 7/8" diameter and input road type


       [5] No. of shear connectors per transverse section. This value is from
           input data type 12032.

       [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

           ** The connector spacing shown here is calculated based upon
              AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
              where ADTT Single Lane= 1140 & No. of Cycles=   31207500
          *** If ADTT Single Lane greater than or equal to 960, refer
              to Fatigue I Table.
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```
 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  25.50   1 1338.2  1626.6          156         >  70
      1 16  51.00   1 1338.2  1626.6          156         >  70


     ---------------------------------------------------------------------------
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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	25.500	6.000
1	25.500	51.000	6.000

```
              ----------------------------------
1
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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                 *********************************

               SPAN     CURRENT SPAN    T
                NO.     FROM      TO    C
                        (ft)     (ft)   R
              ---------------------------

                  1       0.0      51.0 C

              ---------------------------

              NOTE: T: TENSION AREA, C: COMPRESSION AREA,
              R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
              STRESSES
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     TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
                    ************************

 STATUS                 TABLE NO.
-------------------------------------------------------------------------------

  OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

  OK     1.2.22.4=DEPTH RATIOS

  OK     1.2.22.10=CONSTRUCBILITY CHECK

  OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

  OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

  OK     1.2.22.21=SERVICE LIMIT STATE CHECK


         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
1
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
                *********************************

                           UNFACTORED   UNFACTORED L+I
                           DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT        MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
 NO NO L SUPT  CAP.                      --------------- FACTOR  LOAD  FACTOR  T
       (ft)  (k-ft)       D1; k-ft  D2     POS.     NEG.  [3]   TYPE
           or STRESS          ksi        MAX. STRESS
 ------------------------------------------------------------------------------

  1  0   0.00   2089.9      0.0      0.0      0.0      0.0  9.99  HL-93  9.99  0
```

1	1	3.19	2089.9	66.2	34.7	196.7	0.0	7.37	HL-93	5.69	0
1	2	6.38	2089.9	123.6	64.7	362.0	0.0	3.77	HL-93	2.91	0
1	3	9.56	2089.9	172.4	90.1	495.8	0.0	2.61	HL-93	2.01	0
1	4	12.75	2672.8	212.4	110.9	595.4	0.0	2.80	HL-93	2.16	0
1	5	15.94	2672.8	243.8	127.1	670.9	0.0	2.42	HL-93	1.86	0
1	6	19.12	2672.8	265.5	138.7	723.0	0.0	2.20	HL-93	1.70	0
1	7	22.31	2672.8	278.5	145.6	741.5	0.0	2.12	HL-93	1.63	0
1	8	25.50	2672.8	282.8	147.9	744.9	0.0	2.10	HL-93	1.62	0
1	9	28.69	2672.8	278.4	145.6	741.5	0.0	2.12	HL-93	1.63	0
1	10	31.87	2672.8	265.3	138.7	723.0	0.0	2.20	HL-93	1.70	0
1	11	35.06	2672.8	243.5	127.1	670.9	0.0	2.42	HL-93	1.86	0
1	12	38.25	2672.8	212.8	110.9	595.4	0.0	2.80	HL-93	2.16	0
1	13	41.44	2089.9	172.7	90.1	495.8	0.0	2.61	HL-93	2.01	0
1	14	44.62	2089.9	123.8	64.7	362.0	0.0	3.77	HL-93	2.91	0
1	15	47.81	2089.9	66.3	34.7	196.7	0.0	7.37	HL-93	5.69	0
1	16	51.00	2089.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	3.19	31.4	-2.47 3.06	-0.28 4.34	0.00 0.00	6.53	HL-93	5.02
1	2	6.38	31.4	-4.61 5.71	-0.52 7.98	0.00 0.00	3.21	HL-93	2.47
1	3	9.56	31.4	-6.43 7.96	-0.71 10.92	0.00 0.00	2.14	HL-93	1.65
1	4	12.75	31.4	-7.38 6.94	-1.17 9.44	0.00 0.00	2.59	HL-93	1.99
1	5	15.94	31.4	-8.47 7.96	-1.32 10.64	0.00 0.00	2.20	HL-93	1.69
1	6	19.12	31.4	-9.23 8.67	-1.42 11.46	0.00 0.00	1.98	HL-93	1.52
1	7	22.31	31.4	-9.68 9.10	-1.46 11.75	0.00 0.00	1.89	HL-93	1.46
1	8	25.50	31.4	-9.83 9.24	-1.47 11.81	0.00 0.00	1.87	HL-93	1.44
1	9	28.69	31.4	-9.68 9.09	-1.46 11.75	0.00 0.00	1.89	HL-93	1.46

1	10	31.87	31.4	-9.22	8.66	-1.42	11.46	0.00	0.00	1.98	HL-93	1.52
1	11	35.06	31.4	-8.46	7.95	-1.32	10.64	0.00	0.00	2.20	HL-93	1.69
1	12	38.25	31.4	-7.39	6.95	-1.17	9.44	0.00	0.00	2.59	HL-93	1.99
1	13	41.44	31.4	-6.44	7.97	-0.71	10.92	0.00	0.00	2.14	HL-93	1.65
1	14	44.62	31.4	-4.62	5.72	-0.52	7.98	0.00	0.00	3.21	HL-93	2.47
1	15	47.81	31.4	-2.47	3.06	-0.28	4.34	0.00	0.00	6.53	HL-93	5.02
1	16	51.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

Please read NOTES on the following page

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	25.50	STRENGTH L.S.	2.10	1.62	HL-93
1	51.00	25.50	SERVICEBILITY	1.87	1.44	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.872
Inventory= 1.440

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	51.00	25.50	STRENGTH L.S.	2.97	709.50
1	51.00	25.50	SERVICEBILITY	2.52	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 2.521

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I SHEAR D2	MAX. SHEAR POS.	MAX. SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	22.1	11.6	81.3	0.0	2.61	HL-93	2.02
1	1	3.19	330.8	19.4	10.2	74.5	-2.3	2.91	HL-93	2.24
1	2	6.38	330.8	16.7	8.7	67.7	-5.1	3.26	HL-93	2.51
1	3	9.56	330.8	13.9	7.3	61.1	-9.0	3.68	HL-93	2.84
1	4	12.75	330.8	11.2	5.8	54.6	-12.9	4.19	HL-93	3.23
1	5	15.94	330.8	8.5	4.4	48.1	-17.0	4.83	HL-93	3.73
1	6	19.12	330.8	5.4	2.9	41.8	-21.2	5.67	HL-93	4.37
1	7	22.31	330.8	2.7	1.5	35.8	-25.5	6.73	HL-93	5.19
1	8	25.50	330.8	0.0	0.0	30.0	-30.0	8.15	HL-93	6.29
1	9	28.69	330.8	-2.7	-1.5	25.5	-35.8	6.73	HL-93	5.19
1	10	31.87	330.8	-5.5	-2.9	21.2	-41.8	5.66	HL-93	4.37
1	11	35.06	330.8	-8.2	-4.4	17.0	-48.1	4.84	HL-93	3.73
1	12	38.25	330.8	-11.2	-5.8	12.9	-54.6	4.19	HL-93	3.23
1	13	41.44	330.8	-14.0	-7.3	9.0	-61.1	3.67	HL-93	2.83
1	14	44.62	330.8	-16.7	-8.7	5.1	-67.7	3.26	HL-93	2.51
1	15	47.81	330.8	-19.4	-10.2	2.3	-74.5	2.91	HL-93	2.24
1	16	51.00	330.8	-22.2	-11.6	0.0	-81.3	2.61	HL-93	2.02

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	51.00	STRENGTH L.S.	2.61	2.02	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.612
Inventory= 2.015

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	51.00	51.00	STRENGTH L.S.	3.25	88.20

```
 ------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
                Legal Load Rating= 3.250
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                                                                    RATING
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 TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
                ***************************

                  NUMBER OF LANE   LL + I.
     SPAN  D FROM -----AND------ DEFLECTION GOVERN. RATING FACTOR FOR LL DEFL.
      NO.  L SUPT  DIST. FACTOR  ----------  LOAD   --------------------------
            (ft)   FOR LL DEFL.     (inch)   TYPE      RF = L/(800 * LL DEFL.)
     ---------------------------------------------------------------------------

       1    25.50    3   0.425       -0.25    HL-93            3.02

                     4   0.433       -0.26                     2.96

     ---------------------------------------------------------------------------
     NOTE: [1] "-" indicates downward deflection

           [2] RF * (live load defl.) = span length/800, therefore

               RF = L/(800 * LL Defl.), if = 99, No rating
```

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```
TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 4 Interior Fatigue  8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Interior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       19           1       AASHTO  2010     0     2            3
 ---------------------------------------------------------------------

 * output level    : 0 = basic output
                    1 = detailed output

 * span interval   : maximum = 20
                    default = 10

 * structural type : 1 = composite (default)
                    2 = noncomposite
                    3 = reinforced concrete
                    4 = prestressed concrete

 * type of unit    : 0 = English (default)
                    1 = Metric
                    2 = Metric input English output
                    3 = English input Metric output

 * design option   : 0 = WSD (default)
                    1 = LFD
                    2 = LRFD


 * program flow    : 0 = DL analysis only
                    1 = DL + LL analysis
                    2 = code check
                    3 = rating
                    4 = design
                    5 = design + code check
                    6 = design + recycle + code check
                    7 = DL stage only
                    8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
1
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              *****************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	1	40.00	1.90	1.58	2.27	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
51.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT DEPTH (in)	WEB (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.6250		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	10.00				33.	33.	33.
2	2	2	31.00				33.	33.	33.
3	1	1	10.00				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C DISTRIBUTION FACTORS
1	33.00	0	0.00	0.44	0.00	15.00	1 1 1 1 1	0.70	2 2 2 2 2
1	0.00	0	0.00	0.60	0.00	0.00	0 0 0 0 0	0.86	3 3 3 3 3

OPTIONAL CALCULATION OF FACTOR

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.73	0.00	51.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

1

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.29	16.10
2	2	Diaphragm	0.000	0.00	0.00	0.29	37.85
3	1	Railing	0.087	0.00	51.00	0.00	0.00
4	1	Curb	0.076	0.00	51.00	0.00	0.00
5	0	Pavement	0.292	0.00	51.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or
Superimposed Loads for composite construction
(In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as
determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

L O C A T I O N		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	16.10	33.0	33.0	16.10
16.10	37.85	33.0	33.0	21.75
37.85	51.00	33.0	33.0	13.15

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	D E S C R I P T I O N S
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

F A C T O R S U S E D B Y L R F D

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.726       0.1310     0.8570


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      51.00

     1   DC2     0.087       0.000     51.000

     2   DC2     0.076       0.000     51.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1238.4	45.9	185.8	40.1	225.84

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	94.50

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.705	0.705	0.714	0.714	FOR STRENGTH POSITIVE MOMENT
	0.860				FOR STRENGTH POSITIVE SHEAR
1	0.437				FOR FATIGUE POSITIVE MOMENT
	0.600				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.68	5900.0	32.9	16.45	358.7	358.7
1	2	5.37	5900.0	32.9	16.45	358.7	358.7
1	3	8.05	5900.0	32.9	16.45	358.7	358.7
1	4	10.74	7391.8	32.9	14.52	509.2	388.9
1	5	13.42	7391.8	32.9	14.52	509.2	388.9
1	6	16.11	7391.8	32.9	14.52	509.2	388.9
1	7	18.79	7391.8	32.9	14.52	509.2	388.9
1	8	21.47	7391.8	32.9	14.52	509.2	388.9
1	9	24.16	7391.8	32.9	14.52	509.2	388.9
1	10	26.84	7391.8	32.9	14.52	509.2	388.9
1	11	29.53	7391.8	32.9	14.52	509.2	388.9
1	12	32.21	7391.8	32.9	14.52	509.2	388.9

1 13	34.89	7391.8	32.9	14.52	509.2	388.9
1 14	37.58	7391.8	32.9	14.52	509.2	388.9
1 15	40.26	7391.8	32.9	14.52	509.2	388.9
1 16	42.95	5900.0	32.9	16.45	358.7	358.7
1 17	45.63	5900.0	32.9	16.45	358.7	358.7
1 18	48.32	5900.0	32.9	16.45	358.7	358.7
1 19	51.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	12487.6	0.024729	493.9	1639.5	785.8
1	1	2.68	12487.6	0.024729	493.9	1639.5	785.8
1	2	5.37	12487.6	0.024729	493.9	1639.5	785.8
1	3	8.05	12487.6	0.024729	493.9	1639.5	785.8
1	4	10.74	16197.6	0.022851	689.8	1612.6	884.2
1	5	13.42	16197.6	0.022851	689.8	1612.6	884.2
1	6	16.11	16197.6	0.022851	689.8	1612.6	884.2
1	7	18.79	16197.6	0.022851	689.8	1612.6	884.2
1	8	21.47	16197.6	0.022851	689.8	1612.6	884.2
1	9	24.16	16197.6	0.022851	689.8	1612.6	884.2
1	10	26.84	16197.6	0.022851	689.8	1612.6	884.2
1	11	29.53	16197.6	0.022851	689.8	1612.6	884.2
1	12	32.21	16197.6	0.022851	689.8	1612.6	884.2
1	13	34.89	16197.6	0.022851	689.8	1612.6	884.2
1	14	37.58	16197.6	0.022851	689.8	1612.6	884.2
1	15	40.26	16197.6	0.022851	689.8	1612.6	884.2
1	16	42.95	12487.6	0.024729	493.9	1639.5	785.8
1	17	45.63	12487.6	0.024729	493.9	1639.5	785.8
1	18	48.32	12487.6	0.024729	493.9	1639.5	785.8
1	19	51.00	12487.6	0.024729	493.9	1639.5	785.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	16816.5	0.029869	544.6	8325.5	1633.5
1	1	2.68	16816.5	0.029869	544.6	8325.5	1633.5
1	2	5.37	16816.5	0.029869	544.6	8325.5	1633.5
1	3	8.05	16816.5	0.029869	544.6	8325.5	1633.5
1	4	10.74	22573.1	0.027904	756.9	6094.9	1884.5
1	5	13.42	22573.1	0.027904	756.9	6094.9	1884.5

1	6	16.11	22573.1	0.027904	756.9	6094.9	1884.5
1	7	18.79	22573.1	0.027904	756.9	6094.9	1884.5
1	8	21.47	22573.1	0.027904	756.9	6094.9	1884.5
1	9	24.16	22573.1	0.027904	756.9	6094.9	1884.5
1	10	26.84	22573.1	0.027904	756.9	6094.9	1884.5
1	11	29.53	22573.1	0.027904	756.9	6094.9	1884.5
1	12	32.21	22573.1	0.027904	756.9	6094.9	1884.5
1	13	34.89	22573.1	0.027904	756.9	6094.9	1884.5
1	14	37.58	22573.1	0.027904	756.9	6094.9	1884.5
1	15	40.26	22573.1	0.027904	756.9	6094.9	1884.5
1	16	42.95	16816.5	0.029869	544.6	8325.5	1633.5
1	17	45.63	16816.5	0.029869	544.6	8325.5	1633.5
1	18	48.32	16816.5	0.029869	544.6	8325.5	1633.5
1	19	51.00	16816.5	0.029869	544.6	8325.5	1633.5

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	2.68	8.5	47.1	0.7	0.0	56.3
1	2	5.37	16.0	88.9	1.5	0.0	106.4
1	3	8.05	22.7	125.5	2.2	0.0	150.4
1	4	10.74	28.3	156.9	2.9	0.0	188.1
1	5	13.42	33.0	183.1	3.6	0.0	219.7
1	6	16.11	36.8	204.0	4.4	0.0	245.2
1	7	18.79	39.6	219.7	4.3	0.0	263.6
1	8	21.47	41.5	230.2	4.3	0.0	276.0
1	9	24.16	42.5	235.4	4.2	0.0	282.1

1 10	26.84	42.5	235.4	4.2	0.0	282.0
1 11	29.53	41.5	230.2	4.1	0.0	275.8
1 12	32.21	39.6	219.7	4.1	0.0	263.4
1 13	34.89	36.8	204.0	4.0	0.0	244.9
1 14	37.58	33.0	183.1	4.0	0.0	220.1
1 15	40.26	28.3	156.9	3.3	0.0	188.5
1 16	42.95	22.7	125.5	2.4	0.0	150.6
1 17	45.63	16.0	88.9	1.6	0.0	106.6
1 18	48.32	8.5	47.1	0.8	0.0	56.4
1 19	51.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.68	18.9	10.6	0.0	0.0	0.0	29.5
1	2	5.37	35.8	20.0	0.0	0.0	0.0	55.7
1	3	8.05	50.5	28.2	0.0	0.0	0.0	78.7
1	4	10.74	63.1	35.2	0.0	0.0	0.0	98.3
1	5	13.42	73.6	41.1	0.0	0.0	0.0	114.7
1	6	16.11	82.1	45.8	0.0	0.0	0.0	127.9
1	7	18.79	88.4	49.3	0.0	0.0	0.0	137.7
1	8	21.47	92.6	51.7	0.0	0.0	0.0	144.2
1	9	24.16	94.7	52.8	0.0	0.0	0.0	147.5
1	10	26.84	94.7	52.8	0.0	0.0	0.0	147.5
1	11	29.53	92.6	51.7	0.0	0.0	0.0	144.2
1	12	32.21	88.4	49.3	0.0	0.0	0.0	137.7
1	13	34.89	82.1	45.8	0.0	0.0	0.0	127.9
1	14	37.58	73.6	41.1	0.0	0.0	0.0	114.7
1	15	40.26	63.1	35.2	0.0	0.0	0.0	98.3
1	16	42.95	50.5	28.2	0.0	0.0	0.0	78.7
1	17	45.63	35.8	20.0	0.0	0.0	0.0	55.7
1	18	48.32	18.9	10.6	0.0	0.0	0.0	29.5
1	19	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.68	0.0	0.0	167.8	HL-93	0.0	HL-93
1	2	5.37	0.0	0.0	313.2	HL-93	0.0	HL-93
1	3	8.05	0.0	0.0	436.3	HL-93	0.0	HL-93
1	4	10.74	0.0	0.0	536.1	HL-93	0.0	HL-93
1	5	13.42	0.0	0.0	613.1	HL-93	0.0	HL-93
1	6	16.11	0.0	0.0	674.3	HL-93	0.0	HL-93
1	7	18.79	0.0	0.0	718.8	HL-93	0.0	HL-93

1 8	21.47	0.0	0.0	739.4	HL-93	0.0	HL-93
1 9	24.16	0.0	0.0	744.3	HL-93	0.0	HL-93
1 10	26.84	0.0	0.0	744.3	HL-93	0.0	HL-93
1 11	29.53	0.0	0.0	739.4	HL-93	0.0	HL-93
1 12	32.21	0.0	0.0	718.8	HL-93	0.0	HL-93
1 13	34.89	0.0	0.0	674.3	HL-93	0.0	HL-93
1 14	37.58	0.0	0.0	613.1	HL-93	0.0	HL-93
1 15	40.26	0.0	0.0	536.1	HL-93	0.0	HL-93
1 16	42.95	0.0	0.0	436.3	HL-93	0.0	HL-93
1 17	45.63	0.0	0.0	313.2	HL-93	0.0	HL-93
1 18	48.32	0.0	0.0	167.8	HL-93	0.0	HL-93
1 19	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.68	57.	0.	57.
1	2	5.37	104.	0.	104.
1	3	8.05	142.	0.	142.
1	4	10.74	171.	0.	171.
1	5	13.42	192.	0.	192.
1	6	16.11	205.	0.	205.
1	7	18.79	212.	0.	212.
1	8	21.47	221.	0.	221.
1	9	24.16	227.	0.	227.
1	10	26.84	227.	0.	227.
1	11	29.53	221.	0.	221.
1	12	32.21	212.	0.	212.
1	13	34.89	205.	0.	205.
1	14	37.58	192.	0.	192.
1	15	40.26	171.	0.	171.
1	16	42.95	142.	0.	142.
1	17	45.63	104.	0.	104.
1	18	48.32	57.	0.	57.
1	19	51.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 COMP. N=27.0 (k-ft)	MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	TOTAL MAXIMUM NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.68	56.3	29.5	167.8	HL-93	0.0	HL-93	253.6	85.8
1	2	5.37	106.4	55.7	313.2	HL-93	0.0	HL-93	475.4	162.2
1	3	8.05	150.4	78.7	436.3	HL-93	0.0	HL-93	665.4	229.0
1	4	10.74	188.1	98.3	536.1	HL-93	0.0	HL-93	822.6	286.5
1	5	13.42	219.7	114.7	613.1	HL-93	0.0	HL-93	947.6	334.5
1	6	16.11	245.2	127.9	674.3	HL-93	0.0	HL-93	1047.4	373.0
1	7	18.79	263.6	137.7	718.8	HL-93	0.0	HL-93	1120.1	401.3

1	8	21.47	276.0	144.2	739.4	HL-93	0.0	HL-93	1159.6	420.2
1	9	24.16	282.1	147.5	744.3	HL-93	0.0	HL-93	1173.9	429.6
1	10	26.84	282.0	147.5	744.3	HL-93	0.0	HL-93	1173.9	429.6
1	11	29.53	275.8	144.2	739.4	HL-93	0.0	HL-93	1159.5	420.1
1	12	32.21	263.4	137.7	718.8	HL-93	0.0	HL-93	1119.9	401.1
1	13	34.89	244.9	127.9	674.3	HL-93	0.0	HL-93	1047.1	372.7
1	14	37.58	220.1	114.7	613.1	HL-93	0.0	HL-93	948.0	334.9
1	15	40.26	188.5	98.3	536.1	HL-93	0.0	HL-93	823.0	286.8
1	16	42.95	150.6	78.7	436.3	HL-93	0.0	HL-93	665.6	229.3
1	17	45.63	106.6	55.7	313.2	HL-93	0.0	HL-93	475.5	162.3
1	18	48.32	56.4	29.5	167.8	HL-93	0.0	HL-93	253.7	85.9
1	19	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.68	253.6	303.9	405.6	338.5	128.7	42.7
1	2	5.37	475.4	569.3	759.8	634.5	243.2	78.2
1	3	8.05	665.4	796.3	1062.5	888.0	343.6	106.3
1	4	10.74	822.6	983.5	1312.1	1097.7	429.7	128.3
1	5	13.42	947.6	1131.6	1509.5	1264.3	501.7	144.3
1	6	16.11	1047.4	1249.7	1666.9	1397.2	559.5	154.1
1	7	18.79	1120.1	1335.7	1781.6	1494.1	602.0	159.1
1	8	21.47	1159.6	1381.5	1842.4	1546.6	630.3	165.5
1	9	24.16	1173.9	1397.2	1863.2	1565.5	644.4	170.5
1	10	26.84	1173.9	1397.1	1863.1	1565.4	644.3	170.5
1	11	29.53	1159.5	1381.3	1842.2	1546.4	630.1	165.5
1	12	32.21	1119.9	1335.5	1781.3	1493.8	601.7	159.1
1	13	34.89	1047.1	1249.4	1666.5	1396.8	559.1	154.1
1	14	37.58	948.0	1131.9	1510.0	1264.7	502.3	144.3
1	15	40.26	823.0	983.8	1312.6	1098.1	430.3	128.3
1	16	42.95	665.6	796.5	1062.8	888.3	344.0	106.3
1	17	45.63	475.5	569.5	760.0	634.7	243.5	78.2
1	18	48.32	253.7	304.0	405.7	338.6	128.8	42.7
1	19	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.3	18.5	0.3	0.0	22.1
1	1	2.68	3.0	16.6	0.3	0.0	19.8
1	2	5.37	2.6	14.6	0.3	0.0	17.5
1	3	8.05	2.3	12.7	0.3	0.0	15.2
1	4	10.74	1.9	10.7	0.3	0.0	12.9
1	5	13.42	1.6	8.8	0.3	0.0	10.6

1	6	16.11	1.2	6.8	0.0	0.0	8.0
1	7	18.79	0.9	4.9	0.0	0.0	5.7
1	8	21.47	0.5	2.9	0.0	0.0	3.4
1	9	24.16	0.2	1.0	0.0	0.0	1.1
1	10	26.84	-0.2	-1.0	0.0	0.0	-1.2
1	11	29.53	-0.5	-2.9	0.0	0.0	-3.5
1	12	32.21	-0.9	-4.9	0.0	0.0	-5.8
1	13	34.89	-1.2	-6.8	0.0	0.0	-8.1
1	14	37.58	-1.6	-8.8	0.0	0.0	-10.4
1	15	40.26	-1.9	-10.7	-0.3	0.0	-13.0
1	16	42.95	-2.3	-12.7	-0.3	0.0	-15.3
1	17	45.63	-2.6	-14.6	-0.3	0.0	-17.6
1	18	48.32	-3.0	-16.6	-0.3	0.0	-19.9
1	19	51.00	-3.3	-18.5	-0.3	0.0	-22.2

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.4	4.2	0.0	0.0	11.6	22.1	33.7
1	1	2.68	6.7	3.7	0.0	0.0	10.4	19.8	30.2
1	2	5.37	5.9	3.3	0.0	0.0	9.2	17.5	26.7
1	3	8.05	5.1	2.8	0.0	0.0	7.9	15.2	23.2
1	4	10.74	4.3	2.4	0.0	0.0	6.7	12.9	19.6
1	5	13.42	3.5	2.0	0.0	0.0	5.5	10.6	16.1
1	6	16.11	2.7	1.5	0.0	0.0	4.3	8.0	12.3
1	7	18.79	2.0	1.1	0.0	0.0	3.1	5.7	8.8
1	8	21.47	1.2	0.7	0.0	0.0	1.8	3.4	5.3
1	9	24.16	0.4	0.2	0.0	0.0	0.6	1.1	1.7
1	10	26.84	-0.4	-0.2	0.0	0.0	-0.6	-1.2	-1.8
1	11	29.53	-1.2	-0.7	0.0	0.0	-1.8	-3.5	-5.3
1	12	32.21	-2.0	-1.1	0.0	0.0	-3.1	-5.8	-8.8
1	13	34.89	-2.7	-1.5	0.0	0.0	-4.3	-8.1	-12.3
1	14	37.58	-3.5	-2.0	0.0	0.0	-5.5	-10.4	-15.9
1	15	40.26	-4.3	-2.4	0.0	0.0	-6.7	-13.0	-19.7
1	16	42.95	-5.1	-2.8	0.0	0.0	-7.9	-15.3	-23.2
1	17	45.63	-5.9	-3.3	0.0	0.0	-9.2	-17.6	-26.7
1	18	48.32	-6.7	-3.7	0.0	0.0	-10.4	-19.9	-30.2
1	19	51.00	-7.4	-4.2	0.0	0.0	-11.6	-22.2	-33.8

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	81.3	HL-93	0.0	HL-93
1	1	2.68	0.0	0.0	75.5	HL-93	-2.0	HL-93
1	2	5.37	0.0	0.0	69.8	HL-93	-4.0	HL-93

1	3	8.05	0.0	0.0	64.2	HL-93	-7.1	HL-93
1	4	10.74	0.0	0.0	58.7	HL-93	-10.4	HL-93
1	5	13.42	0.0	0.0	53.2	HL-93	-13.8	HL-93
1	6	16.11	0.0	0.0	47.8	HL-93	-17.2	HL-93
1	7	18.79	0.0	0.0	42.5	HL-93	-20.7	HL-93
1	8	21.47	0.0	0.0	37.4	HL-93	-24.3	HL-93
1	9	24.16	0.0	0.0	32.4	HL-93	-28.0	HL-93
1	10	26.84	0.0	0.0	28.0	HL-93	-32.4	HL-93
1	11	29.53	0.0	0.0	24.3	HL-93	-37.4	HL-93
1	12	32.21	0.0	0.0	20.7	HL-93	-42.5	HL-93
1	13	34.89	0.0	0.0	17.2	HL-93	-47.8	HL-93
1	14	37.58	0.0	0.0	13.8	HL-93	-53.2	HL-93
1	15	40.26	0.0	0.0	10.4	HL-93	-58.7	HL-93
1	16	42.95	0.0	0.0	7.1	HL-93	-64.2	HL-93
1	17	45.63	0.0	0.0	4.0	HL-93	-69.8	HL-93
1	18	48.32	0.0	0.0	2.0	HL-93	-75.5	HL-93
1	19	51.00	0.0	0.0	0.0	HL-93	-81.3	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	32.	0.	32.
1	1	2.68	29.	-1.	30.
1	2	5.37	27.	-2.	29.
1	3	8.05	24.	-3.	28.
1	4	10.74	22.	-5.	27.
1	5	13.42	20.	-6.	25.
1	6	16.11	17.	-7.	24.
1	7	18.79	15.	-9.	23.
1	8	21.47	13.	-10.	24.
1	9	24.16	13.	-12.	24.
1	10	26.84	12.	-13.	24.
1	11	29.53	10.	-13.	24.
1	12	32.21	9.	-15.	23.
1	13	34.89	7.	-17.	24.
1	14	37.58	6.	-20.	25.
1	15	40.26	5.	-22.	27.
1	16	42.95	3.	-24.	28.
1	17	45.63	2.	-27.	29.
1	18	48.32	1.	-29.	30.
1	19	51.00	0.	-32.	32.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	31.9	.31072E-01	0.50	0.2500*	.14322E-01	0.23	0.2500*
1	1	2.68	30.4	.31072E-01	0.47	0.2500*	.14322E-01	0.22	0.2500*
1	2	5.37	29.0	.31072E-01	0.45	0.2500*	.14322E-01	0.21	0.2500*
1	3	8.05	27.7	.31072E-01	0.43	0.2500*	.14322E-01	0.20	0.2500*
1	4	10.74	26.5	.29418E-01	0.39	0.2500*	.10052E-01	0.13	0.2500*
1	5	13.42	25.4	.29418E-01	0.37	0.2500*	.10052E-01	0.13	0.2500*

1	6	16.11	24.2	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	7	18.79	23.2	.29418E-01	0.34	0.2500*	.10052E-01	0.12	0.2500*
1	8	21.47	23.6	.29418E-01	0.35	0.2500*	.10052E-01	0.12	0.2500*
1	9	24.16	24.4	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	10	26.84	24.4	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	11	29.53	23.6	.29418E-01	0.35	0.2500*	.10052E-01	0.12	0.2500*
1	12	32.21	23.2	.29418E-01	0.34	0.2500*	.10052E-01	0.12	0.2500*
1	13	34.89	24.2	.29418E-01	0.36	0.2500*	.10052E-01	0.12	0.2500*
1	14	37.58	25.4	.29418E-01	0.37	0.2500*	.10052E-01	0.13	0.2500*
1	15	40.26	26.5	.29418E-01	0.39	0.2500*	.10052E-01	0.13	0.2500*
1	16	42.95	27.7	.31072E-01	0.43	0.2500*	.14322E-01	0.20	0.2500*
1	17	45.63	29.0	.31072E-01	0.45	0.2500*	.14322E-01	0.21	0.2500*
1	18	48.32	30.4	.31072E-01	0.47	0.2500*	.14322E-01	0.22	0.2500*
1	19	51.00	31.9	.31072E-01	0.50	0.2500*	.14322E-01	0.23	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	33.7	81.3	HL-93	0.0	HL-93	115.0	33.7
1	1	2.68	30.2	75.5	HL-93	-2.0	HL-93	105.7	28.2
1	2	5.37	26.7	69.8	HL-93	-4.0	HL-93	96.5	22.7
1	3	8.05	23.2	64.2	HL-93	-7.1	HL-93	87.4	16.0
1	4	10.74	19.6	58.7	HL-93	-10.4	HL-93	78.3	9.2
1	5	13.42	16.1	53.2	HL-93	-13.8	HL-93	69.3	2.4
1	6	16.11	12.3	47.8	HL-93	-17.2	HL-93	60.1	-4.9
1	7	18.79	8.8	42.5	HL-93	-20.7	HL-93	51.3	-11.9
1	8	21.47	5.3	37.4	HL-93	-24.3	HL-93	42.6	-19.0
1	9	24.16	1.7	32.4	HL-93	-28.0	HL-93	34.2	-26.2
1	10	26.84	-1.8	28.0	HL-93	-32.4	HL-93	26.2	-34.2
1	11	29.53	-5.3	24.3	HL-93	-37.4	HL-93	19.0	-42.6
1	12	32.21	-8.8	20.7	HL-93	-42.5	HL-93	11.9	-51.3
1	13	34.89	-12.3	17.2	HL-93	-47.8	HL-93	4.9	-60.1
1	14	37.58	-15.9	13.8	HL-93	-53.2	HL-93	-2.1	-69.1
1	15	40.26	-19.7	10.4	HL-93	-58.7	HL-93	-9.3	-78.3
1	16	42.95	-23.2	7.1	HL-93	-64.2	HL-93	-16.1	-87.4
1	17	45.63	-26.7	4.0	HL-93	-69.8	HL-93	-22.7	-96.5
1	18	48.32	-30.2	2.0	HL-93	-75.5	HL-93	-28.3	-105.8
1	19	51.00	-33.8	0.0	HL-93	-81.3	HL-93	-33.8	-115.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	115.0	139.4	186.3	153.8	50.6	23.9
1	1	2.68	105.7	128.4	171.6	141.4	45.3	22.8
1	2	5.37	96.5	117.5	157.0	129.1	40.0	21.8
1	3	8.05	87.4	106.6	142.6	116.9	34.7	20.8
1	4	10.74	78.3	95.9	128.3	104.8	29.5	19.9
1	5	13.42	69.3	85.3	114.2	92.9	24.2	19.0
1	6	16.11	60.1	74.4	99.7	80.6	18.5	18.1
1	7	18.79	51.3	64.0	85.8	68.8	13.2	17.4
1	8	21.47	42.6	53.8	72.2	57.3	7.9	17.7
1	9	24.16	34.2	43.9	59.0	46.1	2.6	18.3
1	10	26.84	-34.2	-43.9	-59.1	-46.1	-2.7	18.3
1	11	29.53	-42.6	-53.9	-72.3	-57.3	-7.9	17.7
1	12	32.21	-51.3	-64.0	-85.8	-68.9	-13.2	17.4
1	13	34.89	-60.1	-74.5	-99.8	-80.6	-18.5	18.1
1	14	37.58	-69.1	-85.0	-113.8	-92.6	-23.8	19.0
1	15	40.26	-78.3	-96.0	-128.4	-104.9	-29.5	19.9
1	16	42.95	-87.4	-106.7	-142.6	-117.0	-34.8	20.8
1	17	45.63	-96.5	-117.5	-157.1	-129.1	-40.1	21.8
1	18	48.32	-105.8	-128.4	-171.6	-141.4	-45.4	22.8
1	19	51.00	-115.1	-139.5	-186.3	-153.8	-50.6	23.9

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	33.73	0.00	HL-93	81.30	HL-93	ST1	28.49	186.30
						ST2	28.49	153.78
						ST4	39.42	50.59
						SE1	33.73	115.03
						SE2	33.73	139.42
2	33.76	0.00	HL-93	81.30	HL-93	ST1	28.52	186.34
						ST2	28.52	153.82
						ST4	39.47	50.64
						SE1	33.76	115.06
						SE2	33.76	139.45

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 NOTE: [1] " - " Indicates Uplift

           ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.
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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	NONCOMPOSITE DL SLAB (in)	COMPOSITE DL CONCENTRATED (in)	COMPOSITE DL UNIFORM (in)	T O T A L NONCOMPOSITE (in)	T O T A L COMPOSITE (in)	T O T A L = DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.68	-0.0164	-0.0906	0.0000	-0.0261	-0.1087	-0.0261	-0.1348
1	2	5.37	-0.0321	-0.1779	0.0000	-0.0513	-0.2133	-0.0513	-0.2646
1	3	8.05	-0.0467	-0.2587	0.0000	-0.0745	-0.3102	-0.0745	-0.3847
1	4	10.74	-0.0596	-0.3305	0.0000	-0.0951	-0.3964	-0.0951	-0.4914
1	5	13.42	-0.0709	-0.3926	0.0000	-0.1128	-0.4709	-0.1128	-0.5838
1	6	16.11	-0.0802	-0.4442	0.0000	-0.1276	-0.5328	-0.1276	-0.6604
1	7	18.79	-0.0873	-0.4839	0.0000	-0.1389	-0.5804	-0.1389	-0.7193
1	8	21.47	-0.0922	-0.5109	0.0000	-0.1467	-0.6128	-0.1467	-0.7594
1	9	24.16	-0.0947	-0.5245	0.0000	-0.1506	-0.6291	-0.1506	-0.7797
1	10	26.84	-0.0947	-0.5245	0.0000	-0.1506	-0.6291	-0.1506	-0.7797
1	11	29.53	-0.0922	-0.5109	0.0000	-0.1467	-0.6128	-0.1467	-0.7594
1	12	32.21	-0.0873	-0.4839	0.0000	-0.1389	-0.5804	-0.1389	-0.7194
1	13	34.89	-0.0802	-0.4442	0.0000	-0.1276	-0.5328	-0.1276	-0.6604
1	14	37.58	-0.0709	-0.3926	0.0000	-0.1128	-0.4710	-0.1128	-0.5839
1	15	40.26	-0.0596	-0.3305	0.0000	-0.0951	-0.3965	-0.0951	-0.4915
1	16	42.95	-0.0467	-0.2587	0.0000	-0.0745	-0.3103	-0.0745	-0.3848
1	17	45.63	-0.0321	-0.1779	0.0000	-0.0513	-0.2134	-0.0513	-0.2647
1	18	48.32	-0.0164	-0.0906	0.0000	-0.0261	-0.1087	-0.0261	-0.1349
1	19	51.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS STEEL DEFL. (in)	CAMBER SIZE	NONCOMPOSITE DEAD LOADS SLAB DEFL. (in)	CAMBER SIZE	COMPOSITE DL DEFL. (in)	CAMBER SIZE	T O T A L DEFL. (in)	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.7	-0.016	0 1/16	-0.092	0 1/ 8	-0.026	0 1/16	-0.135	0 3/16
1	2	5.4	-0.032	0 1/16	-0.181	0 3/16	-0.051	0 1/16	-0.265	0 5/16
1	3	8.1	-0.047	0 1/16	-0.264	0 5/16	-0.074	0 1/ 8	-0.385	0 7/16
1	4	10.7	-0.060	0 1/16	-0.337	0 3/ 8	-0.095	0 1/ 8	-0.491	0 1/ 2
1	5	13.4	-0.071	0 1/ 8	-0.400	0 7/16	-0.113	0 1/ 8	-0.584	0 5/ 8
1	6	16.1	-0.080	0 1/ 8	-0.453	0 1/ 2	-0.128	0 3/16	-0.660	0 11/16
1	7	18.8	-0.087	0 1/ 8	-0.493	0 1/ 2	-0.139	0 3/16	-0.719	0 3/ 4
1	8	21.5	-0.092	0 1/ 8	-0.521	0 9/16	-0.147	0 3/16	-0.759	0 13/16
1	9	24.2	-0.095	0 1/ 8	-0.534	0 9/16	-0.151	0 3/16	-0.780	0 13/16
1	10	26.8	-0.095	0 1/ 8	-0.534	0 9/16	-0.151	0 3/16	-0.780	0 13/16
1	11	29.5	-0.092	0 1/ 8	-0.521	0 9/16	-0.147	0 3/16	-0.759	0 13/16
1	12	32.2	-0.087	0 1/ 8	-0.493	0 1/ 2	-0.139	0 3/16	-0.719	0 3/ 4
1	13	34.9	-0.080	0 1/ 8	-0.453	0 1/ 2	-0.128	0 3/16	-0.660	0 11/16
1	14	37.6	-0.071	0 1/ 8	-0.400	0 7/16	-0.113	0 1/ 8	-0.584	0 5/ 8
1	15	40.3	-0.060	0 1/16	-0.337	0 3/ 8	-0.095	0 1/ 8	-0.492	0 1/ 2
1	16	42.9	-0.047	0 1/16	-0.264	0 5/16	-0.074	0 1/ 8	-0.385	0 7/16
1	17	45.6	-0.032	0 1/16	-0.181	0 3/16	-0.051	0 1/16	-0.265	0 5/16
1	18	48.3	-0.016	0 1/16	-0.092	0 1/ 8	-0.026	0 1/16	-0.135	0 3/16

```
 1 19  51.0  0.000    --      0.000    --      0.000    --      0.000    --


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 NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2
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     TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
                   ******************************************************
                    (UNFACTORED)

                  NUMBER OF LANE   LL + I.
           D FROM -----AND------ DEFLECTION  GOVERN.  1/800 OF SPAN L  ROTATION
     SPAN  L SUPT  DIST. FACTOR  ------------  LOAD   ---------------------
      NO.   (ft)   FOR LL DEFL.     (inch)     TYPE   AASHTO 2.5.2.6.2  [5] Rad.
     ---------------------------------------------------------------------

       1    25.50   3    0.425     -0.253 MAX HL-93     0.76            0.00241
                                   0.026 MIN
                                  -0.065 MAX LANE
                                   0.026 MIN LANE

                    4    0.433     -0.258 MAX HL-93     0.76            0.00241
                                   0.026 MIN
                                  -0.066 MAX LANE
                                   0.026 MIN LANE

                    3    0.425     -0.159 MAX FA
                                   0.000 MIN FA

     ---------------------------------------------------------------------
     NOTE: [1] " - " indicates downward deflection

           [2] The distribution factor for LL+I deflection is defined as

               DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

               where NL= no. of traffic lanes
                     Ng= no. of girders
                     (RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

           [3] This table is based upon the optional criteria specified
               in AASHTO LRFD Art. 3.6.1.3.2

           [4] The number of traffic lanes is determined according to
               AASHTO LRFD Art.3.6.1.1.1.
               The 1st line is for the most probable number of lanes
               and the 2nd line is for the next probable number of lanes.

           [5] Max rotations at left (1st line) & right (2nd line)
               supports of the span without averaging, factor and impact

           [6] If ADTT is between 100 and 1000, multi-presence
               factor of 0.95 is applied. If ADTT is below 100, factor
               is 0.9 (AASHTO LRFD C3.6.1.1.2).

           [7] For truck rating the most probable number of lanes is assumed for avereaging.
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 TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
               *********************************************

              STEEL DEAD LOAD    OTHER DEAD LOAD   TOTAL DEAD LOAD
 SP IN D FROM ----------------  ----------------  ----------------
 NO NO L SUPT    STEEL BEAM        STEEL BEAM        STEEL BEAM
               TOP      BOT      TOP      BOT      TOP      BOT
       (ft)       (ksi)             (ksi)              (ksi)
```

1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.68	-0.28	0.28	-1.60	1.60	-1.88	1.88
1	2	5.37	-0.54	0.54	-3.02	3.02	-3.56	3.56
1	3	8.05	-0.76	0.76	-4.27	4.27	-5.03	5.03
1	4	10.74	-0.87	0.67	-4.93	3.77	-5.81	4.43
1	5	13.42	-1.02	0.78	-5.76	4.40	-6.78	5.18
1	6	16.11	-1.14	0.87	-6.43	4.91	-7.57	5.78
1	7	18.79	-1.22	0.93	-6.91	5.28	-8.14	6.21
1	8	21.47	-1.28	0.98	-7.23	5.52	-8.52	6.50
1	9	24.16	-1.31	1.00	-7.39	5.65	-8.70	6.65
1	10	26.84	-1.31	1.00	-7.39	5.65	-8.70	6.65
1	11	29.53	-1.28	0.98	-7.23	5.52	-8.51	6.50
1	12	32.21	-1.22	0.93	-6.91	5.27	-8.13	6.21
1	13	34.89	-1.14	0.87	-6.42	4.90	-7.56	5.77
1	14	37.58	-1.02	0.78	-5.77	4.41	-6.79	5.19
1	15	40.26	-0.87	0.67	-4.94	3.77	-5.82	4.44
1	16	42.95	-0.76	0.76	-4.28	4.28	-5.04	5.04
1	17	45.63	-0.54	0.54	-3.03	3.03	-3.57	3.57
1	18	48.32	-0.28	0.28	-1.60	1.60	-1.89	1.89
1	19	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.68	-0.02	-0.22	0.72	-0.02	-2.10	2.60
1	2	5.37	-0.03	-0.41	1.35	-0.03	-3.97	4.91
1	3	8.05	-0.04	-0.58	1.91	-0.04	-5.61	6.94
1	4	10.74	-0.05	-0.73	1.71	-0.05	-6.54	6.14
1	5	13.42	-0.06	-0.85	2.00	-0.06	-7.63	7.17
1	6	16.11	-0.06	-0.95	2.22	-0.06	-8.52	8.00
1	7	18.79	-0.07	-1.02	2.40	-0.07	-9.16	8.61
1	8	21.47	-0.07	-1.07	2.51	-0.07	-9.59	9.01
1	9	24.16	-0.07	-1.10	2.57	-0.07	-9.80	9.21
1	10	26.84	-0.07	-1.10	2.57	-0.07	-9.80	9.21
1	11	29.53	-0.07	-1.07	2.51	-0.07	-9.58	9.01
1	12	32.21	-0.07	-1.02	2.40	-0.07	-9.15	8.60
1	13	34.89	-0.06	-0.95	2.22	-0.06	-8.51	7.99
1	14	37.58	-0.06	-0.85	2.00	-0.06	-7.65	7.18
1	15	40.26	-0.05	-0.73	1.71	-0.05	-6.55	6.15
1	16	42.95	-0.04	-0.58	1.91	-0.04	-5.62	6.95
1	17	45.63	-0.03	-0.41	1.35	-0.03	-3.97	4.92
1	18	48.32	-0.02	-0.22	0.72	-0.02	-2.10	2.60
1	19	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

MAXIMUM POSITIVE,(MPa) MAXIMUM NEGATIVE,(ksi)

SP NO	IN NO	D FROM L SUPT (ft)	CONCR. TOP	STEEL TOP	STEEL BOT	CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.68	-0.14	-0.24	3.70	0.00	0.00	0.00
1	2	5.37	-0.26	-0.45	6.90	0.00	0.00	0.00
1	3	8.05	-0.36	-0.63	9.61	0.00	0.00	0.00
1	4	10.74	-0.38	-1.06	8.50	0.00	0.00	0.00
1	5	13.42	-0.43	-1.21	9.72	0.00	0.00	0.00
1	6	16.11	-0.48	-1.33	10.69	0.00	0.00	0.00
1	7	18.79	-0.51	-1.42	11.39	0.00	0.00	0.00
1	8	21.47	-0.52	-1.46	11.72	0.00	0.00	0.00
1	9	24.16	-0.53	-1.47	11.80	0.00	0.00	0.00
1	10	26.84	-0.53	-1.47	11.80	0.00	0.00	0.00
1	11	29.53	-0.52	-1.46	11.72	0.00	0.00	0.00
1	12	32.21	-0.51	-1.42	11.39	0.00	0.00	0.00
1	13	34.89	-0.48	-1.33	10.69	0.00	0.00	0.00
1	14	37.58	-0.43	-1.21	9.72	0.00	0.00	0.00
1	15	40.26	-0.38	-1.06	8.50	0.00	0.00	0.00
1	16	42.95	-0.36	-0.63	9.61	0.00	0.00	0.00
1	17	45.63	-0.26	-0.45	6.90	0.00	0.00	0.00
1	18	48.32	-0.14	-0.24	3.70	0.00	0.00	0.00
1	19	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	2.68	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	2	5.37	0.2	0.0	0.2	C	HL	2.3	0.0	2.3	T	HL
1	3	8.05	0.2	0.0	0.2	C	HL	3.1	0.0	3.1	T	HL
1	4	10.74	0.3	0.0	0.3	C	HL	2.7	0.0	2.7	T	HL
1	5	13.42	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	6	16.11	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	7	18.79	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	8	21.47	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	9	24.16	0.4	0.0	0.4	C	HL	3.6	0.0	3.6	T	HL
1	10	26.84	0.4	0.0	0.4	C	HL	3.6	0.0	3.6	T	HL
1	11	29.53	0.4	0.0	0.4	C	HL	3.5	0.0	3.5	T	HL
1	12	32.21	0.4	0.0	0.4	C	HL	3.4	0.0	3.4	T	HL
1	13	34.89	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	14	37.58	0.4	0.0	0.4	C	HL	3.1	0.0	3.1	T	HL
1	15	40.26	0.3	0.0	0.3	C	HL	2.7	0.0	2.7	T	HL
1	16	42.95	0.2	0.0	0.2	C	HL	3.1	0.0	3.1	T	HL
1	17	45.63	0.2	0.0	0.2	C	HL	2.3	0.0	2.3	T	HL
1	18	48.32	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	19	51.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN D NO L FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	2.68	33.	33.	-0.15	-2.34	6.30	-0.02	-2.10	2.60
1 2	5.37	33.	33.	-0.29	-4.42	11.82	-0.03	-3.97	4.91
1 3	8.05	33.	33.	-0.40	-6.24	16.56	-0.04	-5.61	6.94
1 4	10.74	33.	33.	-0.43	-7.59	14.64	-0.05	-6.54	6.14
1 5	13.42	33.	33.	-0.49	-8.84	16.89	-0.06	-7.63	7.17
1 6	16.11	33.	33.	-0.54	-9.84	18.69	-0.06	-8.52	8.00
1 7	18.79	33.	33.	-0.58	-10.58	20.00	-0.07	-9.16	8.61
1 8	21.47	33.	33.	-0.60	-11.04	20.73	-0.07	-9.59	9.01
1 9	24.16	33.	33.	-0.60	-11.27	21.01	-0.07	-9.80	9.21
1 10	26.84	33.	33.	-0.60	-11.27	21.01	-0.07	-9.80	9.21
1 11	29.53	33.	33.	-0.60	-11.04	20.73	-0.07	-9.58	9.01
1 12	32.21	33.	33.	-0.58	-10.57	20.00	-0.07	-9.15	8.60
1 13	34.89	33.	33.	-0.54	-9.83	18.69	-0.06	-8.51	7.99
1 14	37.58	33.	33.	-0.49	-8.85	16.90	-0.06	-7.65	7.18
1 15	40.26	33.	33.	-0.43	-7.60	14.65	-0.05	-6.55	6.15
1 16	42.95	33.	33.	-0.40	-6.24	16.57	-0.04	-5.62	6.95
1 17	45.63	33.	33.	-0.29	-4.43	11.82	-0.03	-3.97	4.92
1 18	48.32	33.	33.	-0.15	-2.34	6.30	-0.02	-2.10	2.60
1 19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	2.68	33.	33.	-0.19	-2.41	7.41	-0.02	-2.10	2.60
1 2	5.37	33.	33.	-0.36	-4.56	13.89	-0.03	-3.97	4.91
1 3	8.05	33.	33.	-0.51	-6.42	19.44	-0.04	-5.61	6.94
1 4	10.74	33.	33.	-0.54	-7.91	17.19	-0.05	-6.54	6.14
1 5	13.42	33.	33.	-0.62	-9.20	19.81	-0.06	-7.63	7.17
1 6	16.11	33.	33.	-0.68	-10.24	21.90	-0.06	-8.52	8.00
1 7	18.79	33.	33.	-0.73	-11.00	23.42	-0.07	-9.16	8.61
1 8	21.47	33.	33.	-0.75	-11.48	24.25	-0.07	-9.59	9.01
1 9	24.16	33.	33.	-0.76	-11.71	24.55	-0.07	-9.80	9.21
1 10	26.84	33.	33.	-0.76	-11.71	24.55	-0.07	-9.80	9.21
1 11	29.53	33.	33.	-0.75	-11.48	24.25	-0.07	-9.58	9.01
1 12	32.21	33.	33.	-0.73	-10.99	23.42	-0.07	-9.15	8.60
1 13	34.89	33.	33.	-0.68	-10.23	21.89	-0.06	-8.51	7.99
1 14	37.58	33.	33.	-0.62	-9.22	19.82	-0.06	-7.65	7.18
1 15	40.26	33.	33.	-0.54	-7.92	17.20	-0.05	-6.55	6.15
1 16	42.95	33.	33.	-0.51	-6.43	19.45	-0.04	-5.62	6.95
1 17	45.63	33.	33.	-0.36	-4.56	13.89	-0.03	-3.97	4.92
1 18	48.32	33.	33.	-0.19	-2.42	7.41	-0.02	-2.10	2.60
1 19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	2.68	33.	33.	-0.26	-3.08	9.84	-0.02	-2.66	3.37
1 2	5.37	33.	33.	-0.49	-5.82	18.44	-0.04	-5.03	6.36
1 3	8.05	33.	33.	-0.69	-8.20	25.81	-0.06	-7.10	8.98
1 4	10.74	33.	33.	-0.73	-10.14	22.83	-0.07	-8.29	7.96
1 5	13.42	33.	33.	-0.84	-11.79	26.30	-0.08	-9.68	9.29
1 6	16.11	33.	33.	-0.93	-13.12	29.07	-0.09	-10.80	10.36
1 7	18.79	33.	33.	-0.99	-14.09	31.09	-0.10	-11.61	11.14
1 8	21.47	33.	33.	-1.02	-14.71	32.18	-0.10	-12.16	11.67
1 9	24.16	33.	33.	-1.03	-14.99	32.58	-0.10	-12.43	11.93
1 10	26.84	33.	33.	-1.03	-14.99	32.58	-0.10	-12.43	11.93
1 11	29.53	33.	33.	-1.02	-14.70	32.18	-0.10	-12.15	11.66
1 12	32.21	33.	33.	-0.99	-14.08	31.08	-0.10	-11.61	11.14
1 13	34.89	33.	33.	-0.93	-13.11	29.06	-0.09	-10.79	10.35
1 14	37.58	33.	33.	-0.84	-11.81	26.31	-0.08	-9.69	9.30
1 15	40.26	33.	33.	-0.73	-10.15	22.84	-0.07	-8.30	7.97
1 16	42.95	33.	33.	-0.69	-8.21	25.82	-0.06	-7.11	9.00
1 17	45.63	33.	33.	-0.49	-5.82	18.45	-0.04	-5.03	6.37
1 18	48.32	33.	33.	-0.26	-3.09	9.84	-0.02	-2.66	3.37
1 19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	2.68	33.	33.	-0.25	-2.28	8.69	-0.01	-1.85	2.23
1 2	5.37	33.	33.	-0.47	-4.30	16.28	-0.02	-3.51	4.21
1 3	8.05	33.	33.	-0.66	-6.05	22.77	-0.03	-4.95	5.94
1 4	10.74	33.	33.	-0.70	-7.61	20.13	-0.04	-5.77	5.26
1 5	13.42	33.	33.	-0.80	-8.85	23.15	-0.04	-6.73	6.14
1 6	16.11	33.	33.	-0.88	-9.84	25.55	-0.05	-7.51	6.84
1 7	18.79	33.	33.	-0.94	-10.56	27.30	-0.05	-8.08	7.36
1 8	21.47	33.	33.	-0.97	-11.01	28.22	-0.05	-8.46	7.71
1 9	24.16	33.	33.	-0.98	-11.21	28.53	-0.05	-8.65	7.88
1 10	26.84	33.	33.	-0.98	-11.21	28.53	-0.05	-8.64	7.88
1 11	29.53	33.	33.	-0.97	-11.00	28.22	-0.05	-8.45	7.71
1 12	32.21	33.	33.	-0.94	-10.55	27.30	-0.05	-8.07	7.36
1 13	34.89	33.	33.	-0.88	-9.83	25.55	-0.05	-7.50	6.84
1 14	37.58	33.	33.	-0.80	-8.86	23.16	-0.04	-6.74	6.14
1 15	40.26	33.	33.	-0.70	-7.62	20.14	-0.04	-5.78	5.26
1 16	42.95	33.	33.	-0.66	-6.06	22.78	-0.03	-4.96	5.95
1 17	45.63	33.	33.	-0.47	-4.30	16.29	-0.02	-3.51	4.21
1 18	48.32	33.	33.	-0.25	-2.28	8.70	-0.01	-1.86	2.23
1 19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.68	33.	33.	-0.21	-2.99	8.36	-0.02	-2.66	3.37
1	2	5.37	33.	33.	-0.39	-5.64	15.68	-0.04	-5.03	6.36
1	3	8.05	33.	33.	-0.54	-7.95	21.96	-0.06	-7.10	8.98
1	4	10.74	33.	33.	-0.58	-9.71	19.43	-0.07	-8.29	7.96
1	5	13.42	33.	33.	-0.67	-11.31	22.41	-0.08	-9.68	9.29
1	6	16.11	33.	33.	-0.73	-12.59	24.79	-0.09	-10.80	10.36
1	7	18.79	33.	33.	-0.78	-13.53	26.53	-0.10	-11.61	11.14
1	8	21.47	33.	33.	-0.81	-14.12	27.49	-0.10	-12.16	11.67
1	9	24.16	33.	33.	-0.82	-14.41	27.86	-0.10	-12.43	11.93
1	10	26.84	33.	33.	-0.82	-14.41	27.86	-0.10	-12.43	11.93
1	11	29.53	33.	33.	-0.81	-14.12	27.49	-0.10	-12.15	11.66
1	12	32.21	33.	33.	-0.78	-13.52	26.52	-0.10	-11.61	11.14
1	13	34.89	33.	33.	-0.73	-12.58	24.78	-0.09	-10.79	10.35
1	14	37.58	33.	33.	-0.67	-11.32	22.42	-0.08	-9.69	9.30
1	15	40.26	33.	33.	-0.58	-9.73	19.44	-0.07	-8.30	7.97
1	16	42.95	33.	33.	-0.54	-7.96	21.98	-0.06	-7.11	9.00
1	17	45.63	33.	33.	-0.39	-5.64	15.69	-0.04	-5.03	6.37
1	18	48.32	33.	33.	-0.21	-2.99	8.36	-0.02	-2.66	3.37
1	19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.68	33.	33.	-0.20	-2.18	7.22	-0.01	-1.85	2.23
1	2	5.37	33.	33.	-0.37	-4.12	13.52	-0.02	-3.51	4.21
1	3	8.05	33.	33.	-0.51	-5.80	18.92	-0.03	-4.95	5.94
1	4	10.74	33.	33.	-0.55	-7.19	16.73	-0.04	-5.77	5.26
1	5	13.42	33.	33.	-0.63	-8.36	19.26	-0.04	-6.73	6.14
1	6	16.11	33.	33.	-0.69	-9.30	21.28	-0.05	-7.51	6.84
1	7	18.79	33.	33.	-0.74	-9.99	22.75	-0.05	-8.08	7.36
1	8	21.47	33.	33.	-0.76	-10.42	23.53	-0.05	-8.46	7.71
1	9	24.16	33.	33.	-0.77	-10.62	23.81	-0.05	-8.65	7.88
1	10	26.84	33.	33.	-0.77	-10.62	23.81	-0.05	-8.64	7.88
1	11	29.53	33.	33.	-0.76	-10.42	23.53	-0.05	-8.45	7.71
1	12	32.21	33.	33.	-0.74	-9.98	22.74	-0.05	-8.07	7.36
1	13	34.89	33.	33.	-0.69	-9.30	21.27	-0.05	-7.50	6.84
1	14	37.58	33.	33.	-0.63	-8.37	19.27	-0.04	-6.74	6.14
1	15	40.26	33.	33.	-0.55	-7.20	16.74	-0.04	-5.78	5.26
1	16	42.95	33.	33.	-0.51	-5.81	18.93	-0.03	-4.96	5.95
1	17	45.63	33.	33.	-0.37	-4.12	13.53	-0.02	-3.51	4.21
1	18	48.32	33.	33.	-0.20	-2.18	7.22	-0.01	-1.86	2.23
1	19	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	DL1, N=INF. STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	DL2, N=27.0 STRESS (ksi)	POS. LL+I MOM. (k-ft)	POS. LL+I STRESS (ksi)	NEG. LL+I MOM. (k-ft)	NEG. LL+I STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	10.00	178.	-5.9	93.	-0.7	509.	-0.7	0.	0.0	-7.36
											0.73
	LB			5.9		2.3		11.2		0.0	19.42
											11.21
	RT			-5.5		-0.7		-1.0		0.0	-7.18
											1.00
	RB			4.2		1.6		8.1		0.0	13.87
											8.07
1	LT	41.00	178.	-5.5	93.	-0.7	509.	-1.0	0.	0.0	-7.19
											1.00
	LB			4.2		1.6		8.1		0.0	13.88
											8.07
	RT			-6.0		-0.7		-0.7		0.0	-7.37
											0.73
	RB			6.0		2.3		11.2		0.0	19.43
											11.21

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	COVER PLATE -- (in) TOP THICK	COVER PLATE -- (in) BOT WIDTH	COVER PLATE -- (in) BOT THICK
1	10.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	31.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.6250
3	10.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS

** [1] = Span length (ft) = 51.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1])	<=>	LIMITATION 1/25	STATUS
				[3]= GIRDER D. [3]/(12*[1])			1/30	
1	10.00	6.75	32.90	[2]=	41.18	0.06728	> 0.04000	OK
				[3]=	32.90	0.05376	> 0.03300	OK
2	31.00	6.75	32.90	[2]=	41.18	0.06728	> 0.04000	OK

```
                        [3]=   32.90     0.05376      >   0.03300    OK

  3  10.00  6.75   32.90 [2]=   41.18     0.06728      >   0.04000    OK

                        [3]=   32.90     0.05376      >   0.03300    OK

 ---------------------------------------------------------------------------

  NOTE : [1] -- Span length or Average of two adjacent span lengths

         [2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

             * For composite girders, the minimum overall depth
               of girder (concrete slab plus haunch & girder) preferably should not
               be less than 0.040*Length of span (or 0.032*L if continuous spans).
               and the depth of steel girder alone preferably should not be
               less than 0.033*Length of span (or 0.027*L if continuous)

         [3] -- same as NOTE [2]

          ** -- cover plates not taken into account

          ** -- these criteria are related to the structural stability
                during construction and the limitation for the live load
                deflection.
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 TABLE  1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
                  ******************************

 SP IN D FROM  Lo Co       Web                                                Web
 NO NO L SUPT  ng mp depth thick  D/tw            2Dcp/tw        2Dc/tw     Cat
               [1]                          [2]    [3]    [4]    [5]    [6]
-------------------------------------------------------------------------------

  1  0   0.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  1   2.68  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  2   5.37  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  3   8.05  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  4  10.74  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  5  13.42  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  6  16.11  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  7  18.79  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  8  21.47  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1  9  24.16  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 10  26.84  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 11  29.53  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 12  32.21  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 13  34.89  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 14  37.58  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 15  40.26  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 16  42.95  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 17  45.63  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 18  48.32  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0
  1 19  51.00  0  1  31.42 0.550  57.1  150.0    0.00 111.46                0


-------------------------------------------------------------------------------
 NOTE:
      [1]  0 - No long. stiffeners
           1 - long. stiffeners
      [2]  D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
      [3]  For composite sections in positive flexure, use Article D6.3.2
           to calculate Dcp and 2*Dcp/tw
           Note: If the plastic N.A. is not in the web, Dcp = 0
      [4]  2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
      [5]  Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
      [6]  2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

      Web Category
       0 = compact section
       2 = non-compact section
       3 = slender section
```

1

```
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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.68	0	1	31.42	0.550	57.13	168.97	2
1	2	5.37	0	1	31.42	0.550	57.13	168.97	2
1	3	8.05	0	1	31.42	0.550	57.13	168.97	2
1	4	10.74	0	1	31.42	0.550	66.52	168.97	2
1	5	13.42	0	1	31.42	0.550	66.52	168.97	2
1	6	16.11	0	1	31.42	0.550	66.52	168.97	2
1	7	18.79	0	1	31.42	0.550	66.52	168.97	2
1	8	21.47	0	1	31.42	0.550	66.52	168.97	2
1	9	24.16	0	1	31.42	0.550	66.52	168.97	2
1	10	26.84	0	1	31.42	0.550	66.52	168.97	2
1	11	29.53	0	1	31.42	0.550	66.52	168.97	2
1	12	32.21	0	1	31.42	0.550	66.52	168.97	2
1	13	34.89	0	1	31.42	0.550	66.52	168.97	2
1	14	37.58	0	1	31.42	0.550	66.52	168.97	2
1	15	40.26	0	1	31.42	0.550	66.52	168.97	2
1	16	42.95	0	1	31.42	0.550	57.13	168.97	2
1	17	45.63	0	1	31.42	0.550	57.13	168.97	2
1	18	48.32	0	1	31.42	0.550	57.13	168.97	2
1	19	51.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

1

```
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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	1	2.68	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	2	5.37	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	3	8.05	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	4	10.74	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	5	13.42	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0
1	6	16.11	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00	0

```
  1  7  18.79   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  8  21.47   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1  9  24.16   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 10  26.84   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 11  29.53   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 12  32.21   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  34.89   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  37.58   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  40.26   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 16  42.95   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 17  45.63   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 18  48.32   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 19  51.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
1
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                  ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0  16.1   0.000  11.346   7.544   3.743   1.000
                       0.000   0.000   0.000   0.000   1.000
    2     16.1  37.8  10.041  11.346  13.041  14.735   1.000
                       0.000   0.000   0.000   0.000   1.000
    3     37.8  51.0   0.000  10.041   6.343   2.646   1.000
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
1
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
```

		(ft)					CAT	(ft)	(ft)	(ft)	CAT	CAT
1	0	0.00	1	7.77	11.26	19.84	0	16.10	7.09	26.62	2	2
1	1	2.68	1	7.77	11.26	19.84	0	16.10	7.09	26.62	2	2
1	2	5.37	1	7.77	11.26	19.84	0	16.10	7.09	26.62	2	2
1	3	8.05	1	7.77	11.26	19.84	0	16.10	7.09	26.62	2	2
1	4	10.74	1	7.77	11.26	19.84	0	16.10	6.95	26.08	2	2
1	5	13.42	1	7.77	11.26	19.84	0	16.10	6.95	26.08	2	2
1	6	16.11	1	7.77	11.26	19.84	0	16.10	6.95	26.08	2	2
1	7	18.79	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	8	21.47	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	9	24.16	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	10	26.84	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	11	29.53	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	12	32.21	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	13	34.89	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	14	37.58	1	7.77	11.26	19.84	0	21.75	6.95	26.08	2	2
1	15	40.26	1	7.77	11.26	19.84	0	13.15	6.95	26.08	2	2
1	16	42.95	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2
1	17	45.63	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2
1	18	48.32	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2
1	19	51.00	1	7.77	11.26	19.84	0	13.15	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.00	33.00	28.43	28.43
1	1	2.68	1	1.000	1.000	1.00	33.00	28.43	28.43
1	2	5.37	1	1.000	1.000	1.00	33.00	28.43	28.43
1	3	8.05	1	1.000	1.000	1.00	33.00	28.43	28.43
1	4	10.74	1	1.000	1.000	1.00	33.00	28.26	28.26
1	5	13.42	1	1.000	1.000	1.00	33.00	28.26	28.26
1	6	16.11	1	1.000	1.000	1.00	33.00	28.26	28.26
1	7	18.79	1	1.000	1.000	1.00	33.00	25.34	25.34
1	8	21.47	1	1.000	1.000	1.00	33.00	25.34	25.34
1	9	24.16	1	1.000	1.000	1.00	33.00	25.34	25.34
1	10	26.84	1	1.000	1.000	1.00	33.00	25.34	25.34
1	11	29.53	1	1.000	1.000	1.00	33.00	25.34	25.34
1	12	32.21	1	1.000	1.000	1.00	33.00	25.34	25.34

1 13	34.89	1	1.000	1.000	1.00	33.00	25.34	25.34
1 14	37.58	1	1.000	1.000	1.00	33.00	25.34	25.34
1 15	40.26	1	1.000	1.000	1.00	33.00	29.79	29.79
1 16	42.95	1	1.000	1.000	1.00	33.00	29.93	29.93
1 17	45.63	1	1.000	1.000	1.00	33.00	29.93	29.93
1 18	48.32	1	1.000	1.000	1.00	33.00	29.93	29.93
1 19	51.00	1	1.000	1.000	1.49	33.00	33.00	33.00

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0	0.00	1	4.70	17.29
1	1	2.68	1	4.70	17.29
1	2	5.37	1	4.70	17.29
1	3	8.05	1	4.70	17.29
1	4	10.74	1	5.55	17.56
1	5	13.42	1	5.55	17.56
1	6	16.11	1	5.55	17.56
1	7	18.79	1	5.55	17.56
1	8	21.47	1	5.55	17.56
1	9	24.16	1	5.55	17.56
1	10	26.84	1	5.55	17.56
1	11	29.53	1	5.55	17.56
1	12	32.21	1	5.55	17.56
1	13	34.89	1	5.55	17.56
1	14	37.58	1	5.55	17.56
1	15	40.26	1	5.55	17.56
1	16	42.95	1	4.70	17.29
1	17	45.63	1	4.70	17.29
1	18	48.32	1	4.70	17.29
1	19	51.00	1	4.70	17.29

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	2.68	1	0	2	0		2	0
1	2	5.37	1	0	2	0		2	0
1	3	8.05	1	0	2	0		2	0
1	4	10.74	1	0	2	0		2	0
1	5	13.42	1	0	2	0		2	0
1	6	16.11	1	0	2	0		2	0
1	7	18.79	1	0	2	0		2	0
1	8	21.47	1	0	2	0		2	0
1	9	24.16	1	0	2	0		2	0
1	10	26.84	1	0	2	0		2	0

1	11	29.53	1	0	2	0	2	0
1	12	32.21	1	0	2	0	2	0
1	13	34.89	1	0	2	0	2	0
1	14	37.58	1	0	2	0	2	0
1	15	40.26	1	0	2	0	2	0
1	16	42.95	1	0	2	0	2	0
1	17	45.63	1	0	2	0	2	0
1	18	48.32	1	0	2	0	2	0
1	19	51.00	1	0	2	0	2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	28.4	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		2.68	0.0	19.8	2.8	33.0	2.8	33.0	2.8	28.4	0
				0.0	-	2.8	-	2.8	33.0	-	-	0
1	2		5.37	0.0	19.8	5.3	33.0	5.3	33.0	5.3	28.4	0
				0.0	-	5.3	-	5.3	33.0	-	-	0
1	3		8.05	0.0	19.8	7.5	33.0	7.5	33.0	7.5	28.4	0
				0.0	-	7.5	-	7.5	33.0	-	-	0
1	4		10.74	0.0	19.8	8.7	33.0	8.7	33.0	8.7	28.3	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	5		13.42	0.0	19.8	10.2	33.0	10.2	33.0	10.2	28.3	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	6		16.11	0.0	19.8	11.3	33.0	11.3	33.0	11.3	28.3	0
				0.0	-	8.7	-	8.7	33.0	-	-	0
1	7		18.79	0.0	19.8	12.2	33.0	12.2	33.0	12.2	25.3	0
				0.0	-	9.3	-	9.3	33.0	-	-	0
1	8		21.47	0.0	19.8	12.8	33.0	12.8	33.0	12.8	25.3	0
				0.0	-	9.8	-	9.8	33.0	-	-	0
1	9		24.16	0.0	19.8	13.1	33.0	13.1	33.0	13.1	25.3	0

		0.0	-	10.0	-	10.0	33.0	-	-	0
1	10	26.84	0.0	19.8	13.1	33.0	13.1	33.0	13.1	25.3	0
			0.0	-	10.0	-	10.0	33.0	-	-	0
1	11	29.53	0.0	19.8	12.8	33.0	12.8	33.0	12.8	25.3	0
			0.0	-	9.8	-	9.8	33.0	-	-	0
1	12	32.21	0.0	19.8	12.2	33.0	12.2	33.0	12.2	25.3	0
			0.0	-	9.3	-	9.3	33.0	-	-	0
1	13	34.89	0.0	19.8	11.3	33.0	11.3	33.0	11.3	25.3	0
			0.0	-	8.7	-	8.7	33.0	-	-	0
1	14	37.58	0.0	19.8	10.2	33.0	10.2	33.0	10.2	25.3	0
			0.0	-	7.8	-	7.8	33.0	-	-	0
1	15	40.26	0.0	19.8	8.7	33.0	8.7	33.0	8.7	29.8	0
			0.0	-	6.7	-	6.7	33.0	-	-	0
1	16	42.95	0.0	19.8	7.6	33.0	7.6	33.0	7.6	29.9	0
			0.0	-	7.6	-	7.6	33.0	-	-	0
1	17	45.63	0.0	19.8	5.3	33.0	5.3	33.0	5.3	29.9	0
			0.0	-	5.3	-	5.3	33.0	-	-	0
1	18	48.32	0.0	19.8	2.8	33.0	2.8	33.0	2.8	29.9	0
			0.0	-	2.8	-	2.8	33.0	-	-	0
1	19	51.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
			0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.68	0.000	0.086	0.086	0.099	0.099	4
					0.086		0.086	3
1	2	5.37	0.000	0.162	0.162	0.188	0.188	4
					0.162		0.162	3
1	3	8.05	0.000	0.229	0.229	0.265	0.265	4
					0.229		0.229	3
1	4	10.74	0.000	0.264	0.264	0.308	0.308	4
					0.202		0.202	3
1	5	13.42	0.000	0.308	0.308	0.360	0.360	4
					0.235		0.235	3
1	6	16.11	0.000	0.344	0.344	0.402	0.402	4
					0.263		0.263	3
1	7	18.79	0.000	0.370	0.370	0.482	0.482	4
					0.282		0.282	3
1	8	21.47	0.000	0.387	0.387	0.504	0.504	4
					0.296		0.296	3
1	9	24.16	0.000	0.396	0.396	0.515	0.515	4
					0.302		0.302	3
1	10	26.84	0.000	0.396	0.396	0.515	0.515	4
					0.302		0.302	3

1	11	29.53	0.000	0.387	0.387	0.504	0.504	4
					0.295		0.295	3
1	12	32.21	0.000	0.369	0.369	0.481	0.481	4
					0.282		0.282	3
1	13	34.89	0.000	0.343	0.343	0.447	0.447	4
					0.262		0.262	3
1	14	37.58	0.000	0.309	0.309	0.402	0.402	4
					0.236		0.236	3
1	15	40.26	0.000	0.264	0.264	0.293	0.293	4
					0.202		0.202	3
1	16	42.95	0.000	0.229	0.229	0.253	0.253	4
					0.229		0.229	3
1	17	45.63	0.000	0.162	0.162	0.179	0.179	4
					0.162		0.162	3
1	18	48.32	0.000	0.086	0.086	0.095	0.095	4
					0.086		0.086	3
1	19	51.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	2089.9	-	-	0.0	19.8	-	-	0--0
1	1		2.68	0	405.6	2089.9	-	-	0.0	19.8	-	-	0--0
1	2		5.37	0	759.8	2089.9	-	-	0.0	19.8	-	-	0--0
1	3		8.05	0	1062.5	2089.9	-	-	0.0	19.8	-	-	0--0
1	4		10.74	0	1312.1	2672.8	-	-	0.0	19.8	-	-	0--0
1	5		13.42	0	1509.5	2672.8	-	-	0.0	19.8	-	-	0--0
1	6		16.11	0	1666.9	2672.8	-	-	0.0	19.8	-	-	0--0
1	7		18.79	0	1781.6	2672.8	-	-	0.0	19.8	-	-	0--0
1	8		21.47	0	1842.4	2672.8	-	-	0.0	19.8	-	-	0--0
1	9		24.16	0	1863.2	2672.8	-	-	0.0	19.8	-	-	0--0
1	10		26.84	0	1863.1	2672.8	-	-	0.0	19.8	-	-	0--0
1	11		29.53	0	1842.2	2672.8	-	-	0.0	19.8	-	-	0--0
1	12		32.21	0	1781.3	2672.8	-	-	0.0	19.8	-	-	0--0
1	13		34.89	0	1666.5	2672.8	-	-	0.0	19.8	-	-	0--0
1	14		37.58	0	1510.0	2672.8	-	-	0.0	19.8	-	-	0--0
1	15		40.26	0	1312.6	2672.8	-	-	0.0	19.8	-	-	0--0
1	16		42.95	0	1062.8	2089.9	-	-	0.0	19.8	-	-	0--0
1	17		45.63	0	760.0	2089.9	-	-	0.0	19.8	-	-	0--0
1	18		48.32	0	405.7	2089.9	-	-	0.0	19.8	-	-	0--0
1	19		51.00	0	0.0	2089.9	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite
sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in
negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure
or tension flange for composite sections in

```
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
          = 0.6*Fyc for comp. flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
      [4] = (PHI)f * Fnt for non-compact tension flange
            of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
          = (PHI)f * Fnc for comp. flange of composite sections
            in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

      "-" is N.A.

      Under FLAG Column, 0 = OK; 1= NG
      M = Moment; C = Comp. Flange; T = Tension Flange
      D = Ductility

      For negative moment region or non-compact sections in
      positive moment region, the 1st line is for top
      flange and the 2nd line is for bottom flange

      The values of fbu, Mu and fl shall be determined based on
      factored loads, and shall be taken as positive in
      sign in all resistance equations (Art. 6.10.1.6)

      The value of fbu is the actual stress in this table,
      the users can use the maximum value within the
      unbraced length to do their own check
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  TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                  ***********************************************

  SP IN D FROM ID Mu+1/3flSxt/       fbu/        fl/ fbu+1/3fl/  MAX.
                          [1]        [2]        [3]        [4]  RAT. GOVN.
  ---------------------------------------------------------------------------

   1  0   0.00  0        0.000                                   0.000     1
   1  1   2.68  0        0.194                                   0.194     1
   1  2   5.37  0        0.364                                   0.364     1
   1  3   8.05  0        0.508                                   0.508     1
   1  4  10.74  0        0.491                                   0.491     1
   1  5  13.42  0        0.565                                   0.565     1
   1  6  16.11  0        0.624                                   0.624     1
   1  7  18.79  0        0.667                                   0.667     1
   1  8  21.47  0        0.689                                   0.689     1
   1  9  24.16  0        0.697                                   0.697     1
   1 10  26.84  0        0.697                                   0.697     1
   1 11  29.53  0        0.689                                   0.689     1
   1 12  32.21  0        0.666                                   0.666     1
   1 13  34.89  0        0.624                                   0.624     1
   1 14  37.58  0        0.565                                   0.565     1
   1 15  40.26  0        0.491                                   0.491     1
   1 16  42.95  0        0.509                                   0.509     1
   1 17  45.63  0        0.364                                   0.364     1
   1 18  48.32  0        0.194                                   0.194     1
   1 19  51.00  0        0.000                                   0.000     1


  ---------------------------------------------------------------------------

  NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
        [2] = (PHI)f * Fnc for comp. flange of composite
              sections in positive flexure (Eq. 6.10.7.2.1-1)
            = (PHI)f * Rh * Fyt for tension flange of composite sections in
              negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
        [3] = 0.6*Fyt for composite sections in positive flexure
              or tension flange for composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
            = 0.6*Fyc for comp. flange of composite sections in
              negative flexure and non-composite sections (Eq. 6.10.1.6-1)
        [4] = (PHI)f * Fnt for non-compact tension flange
              of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
            = (PHI)f * Fnc for comp. flange of composite sections
              in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

        The governing number is listed as below.
```

```
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]
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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN D NO	FROM L (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	2.68	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	5.37	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	8.05	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	10.74	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	13.42	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	16.11	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	18.79	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	21.47	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	24.16	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	26.84	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	29.53	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	32.21	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	34.89	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	37.58	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	40.26	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	42.95	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	17	45.63	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	18	48.32	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	19	51.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
            = 5 for unstiffened beams and girders
            = 5 for stiffened girders when (d0/D) > 3, or
                                           (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                  ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp
```

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

** This Article (6.10.9.2) indicates that a designer cannot
count on post-buckling shear resistance from tension-
field action for an unstiffened girder.

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	186.3	<	330.8	0	0
1	1	2.68	0	171.6	<	330.8	0	0
1	2	5.37	0	157.0	<	330.8	0	0
1	3	8.05	0	142.6	<	330.8	0	0
1	4	10.74	0	128.3	<	330.8	0	0
1	5	13.42	0	114.2	<	330.8	0	0
1	6	16.11	0	99.7	<	330.8	0	0
1	7	18.79	0	85.8	<	330.8	0	0
1	8	21.47	0	72.2	<	330.8	0	0
1	9	24.16	0	59.0	<	330.8	0	0
1	10	26.84	0	59.1	<	330.8	0	0
1	11	29.53	0	72.3	<	330.8	0	0
1	12	32.21	0	85.8	<	330.8	0	0
1	13	34.89	0	99.8	<	330.8	0	0
1	14	37.58	0	113.8	<	330.8	0	0
1	15	40.26	0	128.4	<	330.8	0	0
1	16	42.95	0	142.6	<	330.8	0	0
1	17	45.63	0	157.1	<	330.8	0	0
1	18	48.32	0	171.6	<	330.8	0	0
1	19	51.00	0	186.3	<	330.8	0	0

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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	186.30	330.76	0	
1	1	2.68	33.	171.59	330.76	0	
1	2	5.37	33.	157.03	330.76	0	
1	3	8.05	33.	142.60	330.76	0	
1	4	10.74	33.	128.31	330.76	0	
1	5	13.42	33.	114.15	330.76	0	
1	6	16.11	33.	99.72	330.76	0	
1	7	18.79	33.	85.80	330.76	0	
1	8	21.47	33.	72.24	330.76	0	
1	9	24.16	33.	59.03	330.76	0	
1	10	26.84	33.	59.07	330.76	0	
1	11	29.53	33.	72.28	330.76	0	
1	12	32.21	33.	85.85	330.76	0	
1	13	34.89	33.	99.76	330.76	0	
1	14	37.58	33.	113.84	330.76	0	
1	15	40.26	33.	128.35	330.76	0	
1	16	42.95	33.	142.64	330.76	0	
1	17	45.63	33.	157.07	330.76	0	
1	18	48.32	33.	171.64	330.76	0	
1	19	51.00	33.	186.34	330.76	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	TOP FLANGE ff+fl/2 (ksi)	BOT. FLANGE ff+fl/2 [1] (ksi)	BOT. FLANGE ff+fl/2 [2] (ksi)	COMP. FLG fc (ksi)	COMP. FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-	31.4	0.0	31.4	-	-	00-
1	1	2.68	2.4	-	31.4	7.4	31.4	-	-	00-
1	2	5.37	4.6	-	31.4	13.9	31.4	-	-	00-
1	3	8.05	6.4	-	31.4	19.4	31.4	-	-	00-

```
      1  4  10.74   7.9    -     31.4    17.2  31.4   -     -   00-
      1  5  13.42   9.2    -     31.4    19.8  31.4   -     -   00-
      1  6  16.11  10.2    -     31.4    21.9  31.4   -     -   00-
      1  7  18.79  11.0    -     31.4    23.4  31.4   -     -   00-
      1  8  21.47  11.5    -     31.4    24.3  31.4   -     -   00-
      1  9  24.16  11.7    -     31.4    24.6  31.4   -     -   00-
      1 10  26.84  11.7    -     31.4    24.6  31.4   -     -   00-
      1 11  29.53  11.5    -     31.4    24.2  31.4   -     -   00-
      1 12  32.21  11.0    -     31.4    23.4  31.4   -     -   00-
      1 13  34.89  10.2    -     31.4    21.9  31.4   -     -   00-
      1 14  37.58   9.2    -     31.4    19.8  31.4   -     -   00-
      1 15  40.26   7.9    -     31.4    17.2  31.4   -     -   00-
      1 16  42.95   6.4    -     31.4    19.5  31.4   -     -   00-
      1 17  45.63   4.6    -     31.4    13.9  31.4   -     -   00-
      1 18  48.32   2.4    -     31.4     7.4  31.4   -     -   00-
      1 19  51.00   0.0    -     31.4     0.0  31.4   -     -   00-


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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                      TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                  ----------------------------- -----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ---------------------------------------------------------------------------

  1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
  1  1   2.68    TR     -0.1 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
  1  2   5.37    TR     -0.2 A B B^C^C D E E^  TR      3.4 A B B^C^C D E
  1  3   8.05    TR     -0.3 A B B^C^C D E E^  TR      4.7 A B B^C^C D
  1     10.00  SCG
  1  4  10.74    TR     -0.5 A B B^C^C D E E^  TR      4.1 A B B^C^C D E
  1  5  13.42    TR     -0.6 A B B^C^C D E E^  TR      4.6 A B B^C^C D
  1  6  16.11    TR     -0.6 A B B^C^C D E E^  TR      4.9 A B B^C^C D
  1  7  18.79    TR     -0.6 A B B^C^C D E E^  TR      5.0 A B B^C^C D
```

```
 1  8  21.47    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
 1  9  24.16    TR     -0.7 A B B^C^C D E E^  TR      5.4 A B B^C^C D
 1 10  26.84    TR     -0.7 A B B^C^C D E E^  TR      5.4 A B B^C^C D
 1 11  29.53    TR     -0.7 A B B^C^C D E E^  TR      5.2 A B B^C^C D
 1 12  32.21    TR     -0.6 A B B^C^C D E E^  TR      5.0 A B B^C^C D
 1 13  34.89    TR     -0.6 A B B^C^C D E E^  TR      4.9 A B B^C^C D
 1 14  37.58    TR     -0.6 A B B^C^C D E E^  TR      4.6 A B B^C^C D
 1 15  40.26    TR     -0.5 A B B^C^C D E E^  TR      4.1 A B B^C^C D E
 1     41.00  SCG
 1 16  42.95    TR     -0.3 A B B^C^C D E E^  TR      4.7 A B B^C^C D
 1 17  45.63    TR     -0.2 A B B^C^C D E E^  TR      3.4 A B B^C^C D E
 1 18  48.32    TR     -0.1 A B B^C^C D E E^  TR      1.9 A B B^C^C D E E^
 1 19  51.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue I Limit State with 1.5 load factor.
       Design for Infinite Life

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
                   ***************************************

     (1) Main (Longitudinal) Load Carrying Members

     (2) Road Type = I --- Rural Interstate
         ---------


                       TOP OF TOP FLANGE          BOTTOM OF BOTTOM FLANGE
                  ---------------------------- ----------------------------
 SP IN D FROM  GOVERN. STRESS   ACCEPTABLE   GOVERN. STRESS   ACCEPTABLE
 NO NO L SUPT  LOADING RANGE      STRESS     LOADING RANGE      STRESS
        (ft)            (ksi)    CATEGORY             (ksi)    CATEGORY
 ------------------------------------------------------------------------

 1  0   0.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^
 1  1   2.68    TR     -0.1 A B B^C^C D E E^  TR      0.9 A B B^C^C D E E^
 1  2   5.37    TR     -0.1 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1  3   8.05    TR     -0.2 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1     10.00  SCG
 1  4  10.74    TR     -0.3 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1  5  13.42    TR     -0.3 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1  6  16.11    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1  7  18.79    TR     -0.3 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1  8  21.47    TR     -0.3 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1  9  24.16    TR     -0.3 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
 1 10  26.84    TR     -0.3 A B B^C^C D E E^  TR      2.7 A B B^C^C D E
 1 11  29.53    TR     -0.3 A B B^C^C D E E^  TR      2.6 A B B^C^C D E
 1 12  32.21    TR     -0.3 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1 13  34.89    TR     -0.3 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 14  37.58    TR     -0.3 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1 15  40.26    TR     -0.3 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1     41.00  SCG
 1 16  42.95    TR     -0.2 A B B^C^C D E E^  TR      2.3 A B B^C^C D E
 1 17  45.63    TR     -0.1 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1 18  48.32    TR     -0.1 A B B^C^C D E E^  TR      0.9 A B B^C^C D E E^
 1 19  51.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
```

load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr	<--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	47.9	.29869E-01	1.4	TR	3.09	3	6.49
1	1	2.68	45.6	.29869E-01	1.4	TR	3.09	3	6.81
1	2	5.37	43.5	.29869E-01	1.3	TR	3.09	3	7.14
1	3	8.05	41.6	.29869E-01	1.2	TR	3.09	3	7.47
1	4	10.74	39.8	.27904E-01	1.1	TR	3.09	3	8.37
1	5	13.42	38.1	.27904E-01	1.1	TR	3.09	3	8.73
1	6	16.11	36.3	.27904E-01	1.0	TR	3.09	3	9.16
1	7	18.79	34.8	.27904E-01	1.0	TR	3.09	3	9.56
1	8	21.47	35.4	.27904E-01	1.0	TR	3.09	3	9.40
1	9	24.16	36.7	.27904E-01	1.0	TR	3.09	3	9.07
1	10	26.84	36.7	.27904E-01	1.0	TR	3.09	3	9.07
1	11	29.53	35.4	.27904E-01	1.0	TR	3.09	3	9.40
1	12	32.21	34.8	.27904E-01	1.0	TR	3.09	3	9.56
1	13	34.89	36.3	.27904E-01	1.0	TR	3.09	3	9.16
1	14	37.58	38.1	.27904E-01	1.1	TR	3.09	3	8.73
1	15	40.26	39.8	.27904E-01	1.1	TR	3.09	3	8.37
1	16	42.95	41.6	.29869E-01	1.2	TR	3.09	3	7.47
1	17	45.63	43.5	.29869E-01	1.3	TR	3.09	3	7.14
1	18	48.32	45.6	.29869E-01	1.4	TR	3.09	3	6.81
1	19	51.00	47.9	.29869E-01	1.4	TR	3.09	3	6.49

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	23.9	.29869E-01	0.7	TR 1.36	3	5.72
1	1	2.68	22.8	.29869E-01	0.7	TR 1.36	3	6.00
1	2	5.37	21.8	.29869E-01	0.7	TR 1.36	3	6.29
1	3	8.05	20.8	.29869E-01	0.6	TR 1.36	3	6.59
1	4	10.74	19.9	.27904E-01	0.6	TR 1.36	3	7.38
1	5	13.42	19.0	.27904E-01	0.5	TR 1.36	3	7.70
1	6	16.11	18.1	.27904E-01	0.5	TR 1.36	3	8.08
1	7	18.79	17.4	.27904E-01	0.5	TR 1.36	3	8.43
1	8	21.47	17.7	.27904E-01	0.5	TR 1.36	3	8.29
1	9	24.16	18.3	.27904E-01	0.5	TR 1.36	3	8.00
1	10	26.84	18.3	.27904E-01	0.5	TR 1.36	3	8.00
1	11	29.53	17.7	.27904E-01	0.5	TR 1.36	3	8.29
1	12	32.21	17.4	.27904E-01	0.5	TR 1.36	3	8.43
1	13	34.89	18.1	.27904E-01	0.5	TR 1.36	3	8.08
1	14	37.58	19.0	.27904E-01	0.5	TR 1.36	3	7.70
1	15	40.26	19.9	.27904E-01	0.6	TR 1.36	3	7.38
1	16	42.95	20.8	.29869E-01	0.6	TR 1.36	3	6.59
1	17	45.63	21.8	.29869E-01	0.7	TR 1.36	3	6.29
1	18	48.32	22.8	.29869E-01	0.7	TR 1.36	3	6.00
1	19	51.00	23.9	.29869E-01	0.7	TR 1.36	3	5.72

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

```
[2] Q/I : Q= statical moment about the neutral axis of the composite
             section of the transformed compressive concrete area or
             the area of reinforcement embedded in the concrete for
             negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

          I= moment of inertia of the transformed composite girder in
             positive moment regions or the moment of inertia provided
             by the steel beam including or excluding the area of
             reinforcement embedded in the concrete in negative moment
             regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction
          of the slab and girder at the point in the span under
          consideration.
        = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
    Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
    Ffat = radial fatigue shear range per unit length
         = Ffat2 = Frc/w for skews exceeding 20 degrees
    Frc = 25.0 kips for both exterior and interior girders
    w = the effective length of the deck
      = 48 inches except at end supports where w is taken
        as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual
         connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

    * Default ALPHA value based on 7/8" diameter and input road type


[5] No. of shear connectors per transverse section. This value is from
    input data type 12032.

[6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

    ** The connector spacing shown here is calculated based upon
       AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
       where ADTT Single Lane= 1140 & No. of Cycles=   31207500
   *** If ADTT Single Lane greater than or equal to 960, refer
       to Fatigue I Table.
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 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                  ************************************

                  MOMENT
                  REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  9  24.16   1 1338.2  1626.6          147         >  70
      1 19  51.00   1 1338.2  1626.6          147         >  70


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

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          Strength Limit State (Article 6.10.10.4)

      [1] The number of shear connectors provided between the
           section of maximum positive moment and each adjacent
           point of 0.0 moment, or between each adjacent point
           of 0.0 moment and centerline of an interior support
```

shall be checked to ensure that adequate connectors
are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed
the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum
positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of
0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of
maximum positive moment and each adjacent point of 0.0 moment
shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or
P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal
shear force between each adjacent point of 0.0 moment
and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	24.158	6.000
1	24.158	51.000	6.000

1

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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	51.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

1

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	2089.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.68	2089.9	56.3	29.5	167.8	0.0	8.73	HL-93	6.74	0
1	2	5.37	2089.9	106.4	55.7	313.2	0.0	4.44	HL-93	3.43	0
1	3	8.05	2089.9	150.4	78.7	436.3	0.0	3.04	HL-93	2.35	0
1	4	10.74	2672.8	188.1	98.3	536.1	0.0	3.18	HL-93	2.45	0
1	5	13.42	2672.8	219.7	114.7	613.1	0.0	2.70	HL-93	2.08	0
1	6	16.11	2672.8	245.2	127.9	674.3	0.0	2.40	HL-93	1.85	0
1	7	18.79	2672.8	263.6	137.7	718.8	0.0	2.21	HL-93	1.71	0
1	8	21.47	2672.8	276.0	144.2	739.4	0.0	2.13	HL-93	1.64	0

1	9	24.16	2672.8	282.1	147.5	744.3	0.0	2.10	HL-93	1.62	0
1	10	26.84	2672.8	282.0	147.5	744.3	0.0	2.10	HL-93	1.62	0
1	11	29.53	2672.8	275.8	144.2	739.4	0.0	2.13	HL-93	1.64	0
1	12	32.21	2672.8	263.4	137.7	718.8	0.0	2.21	HL-93	1.71	0
1	13	34.89	2672.8	244.9	127.9	674.3	0.0	2.40	HL-93	1.85	0
1	14	37.58	2672.8	220.1	114.7	613.1	0.0	2.70	HL-93	2.08	0
1	15	40.26	2672.8	188.5	98.3	536.1	0.0	3.18	HL-93	2.45	0
1	16	42.95	2089.9	150.6	78.7	436.3	0.0	3.04	HL-93	2.35	0
1	17	45.63	2089.9	106.6	55.7	313.2	0.0	4.44	HL-93	3.43	0
1	18	48.32	2089.9	56.4	29.5	167.8	0.0	8.73	HL-93	6.74	0
1	19	51.00	2089.9	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	2.68	31.4	-2.10 2.60	-0.24 3.70	0.00 0.00	7.78	HL-93	5.98
1	2	5.37	31.4	-3.97 4.91	-0.45 6.90	0.00 0.00	3.83	HL-93	2.95
1	3	8.05	31.4	-5.61 6.94	-0.63 9.61	0.00 0.00	2.54	HL-93	1.95
1	4	10.74	31.4	-6.54 6.14	-1.06 8.50	0.00 0.00	2.97	HL-93	2.28
1	5	13.42	31.4	-7.63 7.17	-1.21 9.72	0.00 0.00	2.49	HL-93	1.91
1	6	16.11	31.4	-8.52 8.00	-1.33 10.69	0.00 0.00	2.18	HL-93	1.68
1	7	18.79	31.4	-9.16 8.61	-1.42 11.39	0.00 0.00	2.00	HL-93	1.54
1	8	21.47	31.4	-9.59 9.01	-1.46 11.72	0.00 0.00	1.91	HL-93	1.47
1	9	24.16	31.4	-9.80 9.21	-1.47 11.80	0.00 0.00	1.88	HL-93	1.44
1	10	26.84	31.4	-9.80 9.21	-1.47 11.80	0.00 0.00	1.88	HL-93	1.44
1	11	29.53	31.4	-9.58 9.01	-1.46 11.72	0.00 0.00	1.91	HL-93	1.47
1	12	32.21	31.4	-9.15 8.60	-1.42 11.39	0.00 0.00	2.00	HL-93	1.54
1	13	34.89	31.4	-8.51 7.99	-1.33 10.69	0.00 0.00	2.18	HL-93	1.68
1	14	37.58	31.4	-7.65 7.18	-1.21 9.72	0.00 0.00	2.49	HL-93	1.91

1 15	40.26	31.4	-6.55	6.15	-1.06	8.50	0.00	0.00	2.96	HL-93	2.28
1 16	42.95	31.4	-5.62	6.95	-0.63	9.61	0.00	0.00	2.54	HL-93	1.95
1 17	45.63	31.4	-3.97	4.92	-0.45	6.90	0.00	0.00	3.83	HL-93	2.95
1 18	48.32	31.4	-2.10	2.60	-0.24	3.70	0.00	0.00	7.78	HL-93	5.98
1 19	51.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	24.16	STRENGTH L.S.	2.10	1.62	HL-93
1	51.00	24.16	SERVICEBILITY	1.88	1.44	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 1.876
Inventory= 1.443

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	51.00	24.16	STRENGTH L.S.	2.99	706.91
1	51.00	24.16	SERVICEBILITY	2.53	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341

Legal Load Rating= 2.533

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	22.1	11.6	81.3	0.0	2.61	HL-93	2.02
1	1	2.68	330.8	19.8	10.4	75.5	-2.0	2.86	HL-93	2.20
1	2	5.37	330.8	17.5	9.2	69.8	-4.0	3.14	HL-93	2.42
1	3	8.05	330.8	15.2	7.9	64.2	-7.1	3.47	HL-93	2.67
1	4	10.74	330.8	12.9	6.7	58.7	-10.4	3.85	HL-93	2.97
1	5	13.42	330.8	10.6	5.5	53.2	-13.8	4.31	HL-93	3.33
1	6	16.11	330.8	8.0	4.3	47.8	-17.2	4.88	HL-93	3.76
1	7	18.79	330.8	5.7	3.1	42.5	-20.7	5.57	HL-93	4.30
1	8	21.47	330.8	3.4	1.8	37.4	-24.3	6.42	HL-93	4.96
1	9	24.16	330.8	1.1	0.6	32.4	-28.0	7.50	HL-93	5.79
1	10	26.84	330.8	-1.2	-0.6	28.0	-32.4	7.50	HL-93	5.79
1	11	29.53	330.8	-3.5	-1.8	24.3	-37.4	6.42	HL-93	4.95
1	12	32.21	330.8	-5.8	-3.1	20.7	-42.5	5.57	HL-93	4.29
1	13	34.89	330.8	-8.1	-4.3	17.2	-47.8	4.88	HL-93	3.76
1	14	37.58	330.8	-10.4	-5.5	13.8	-53.2	4.32	HL-93	3.33
1	15	40.26	330.8	-13.0	-6.7	10.4	-58.7	3.85	HL-93	2.97
1	16	42.95	330.8	-15.3	-7.9	7.1	-64.2	3.47	HL-93	2.67
1	17	45.63	330.8	-17.6	-9.2	4.0	-69.8	3.14	HL-93	2.42
1	18	48.32	330.8	-19.9	-10.4	2.0	-75.5	2.86	HL-93	2.20
1	19	51.00	330.8	-22.2	-11.6	0.0	-81.3	2.61	HL-93	2.02

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	51.00	STRENGTH L.S.	2.61	2.02	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.612
Inventory= 2.015

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	51.00	51.00	STRENGTH L.S.	3.25	88.20

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 ------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
                Legal Load Rating= 3.250
1
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                                                                    RATING
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 TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
                ***************************

                  NUMBER OF LANE   LL + I.
     SPAN  D FROM -----AND------ DEFLECTION GOVERN. RATING FACTOR FOR LL DEFL.
      NO.  L SUPT  DIST. FACTOR  ----------  LOAD   --------------------------
            (ft)   FOR LL DEFL.     (inch)   TYPE      RF = L/(800 * LL DEFL.)
     ---------------------------------------------------------------------------

       1    25.50    3   0.425       -0.25    HL-93            3.02

                     4   0.433       -0.26                     2.96

     ---------------------------------------------------------------------------
     NOTE: [1] "-" indicates downward deflection

           [2] RF * (live load defl.) = span length/800, therefore

               RF = L/(800 * LL Defl.), if = 99, No rating
```

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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5969 I-95 NB Span 4 Exterior Strength	8/19/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5969	Exterior	DGB/TDM	TDM/KSW		

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
****

OUTPUT LEVEL (0,1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	PROGRAM FLOW CONTROL
1	18	1	AASHTO	2010	0	2	3

* output level : 0 = basic output
  1 = detailed output

* span interval : maximum = 20
  default = 10

* structural type : 1 = composite (default)
  2 = noncomposite
  3 = reinforced concrete
  4 = prestressed concrete

* type of unit : 0 = English (default)
  1 = Metric
  2 = Metric input English output
  3 = English input Metric output

* design option : 0 = WSD (default)
  1 = LFD
  2 = LRFD

* program flow : 0 = DL analysis only
  1 = DL + LL analysis
  2 = code check
  3 = rating
  4 = design
  5 = design + code check
  6 = design + recycle + code check
  7 = DL stage only
  8 = DL stage + LL

* EFFECTIVE FLANGE WITH OPTION = 0
  0 - DEFAULT (2008)
  1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

NUMBER	BEAM POSITION	WIDTH BETWEEN CURBS OR	OVERHANG WIDTH	EDGE OF SLAB TO CURB	HAUNCH	COMPOSITE PERCENTAGE AT NEG MOM	STEEL LOAD DETAIL

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.27	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
51.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT (in)	WEB DEPTH (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.5000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.50				33.	33.	33.
2	2	2	28.00				33.	33.	33.
3	1	1	11.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	OPTIONAL LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.57	2 2 2 2 2
1	0.00	0	0.00	0.39	0.00	0.00	0 0 0 0 0	0.55	3 3 3 3 3

OPTIONAL CALCULATION OF FACTOR LOADING TYPES DISTRIBUTION FACTORS

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.55	0.00	51.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	7.50
2	2	Diaphragm	0.000	0.00	0.00	0.14	29.25
3	1	Railing	0.087	0.00	51.00	0.00	0.00
4	1	Curb	0.076	0.00	51.00	0.00	0.00
5	0	Pavement	0.292	0.00	51.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	7.50	33.0	33.0	7.50
7.50	29.25	33.0	33.0	21.75
29.25	51.00	33.0	33.0	21.75

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

1

TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

FACTORS USED BY LRFD

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.554       0.1274     0.6814


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ------------------------------------------

     1   DW       0.29        0.00      51.00

     1   DC2     0.087       0.000     51.000

     2   DC2     0.076       0.000     51.000


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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1238.4	45.9	185.8	39.0	224.74

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.567	0.567	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.550				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.390				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN D NO L	FROM SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	2.83	5900.0	32.9	16.45	358.7	358.7
1	2	5.67	5900.0	32.9	16.45	358.7	358.7
1	3	8.50	5900.0	32.9	16.45	358.7	358.7
1	4	11.33	5900.0	32.9	16.45	358.7	358.7
1	5	14.17	7121.5	32.9	14.85	479.7	383.8
1	6	17.00	7121.5	32.9	14.85	479.7	383.8
1	7	19.83	7121.5	32.9	14.85	479.7	383.8
1	8	22.67	7121.5	32.9	14.85	479.7	383.8
1	9	25.50	7121.5	32.9	14.85	479.7	383.8
1	10	28.33	7121.5	32.9	14.85	479.7	383.8
1	11	31.17	7121.5	32.9	14.85	479.7	383.8
1	12	34.00	7121.5	32.9	14.85	479.7	383.8

1 13	36.83	7121.5	32.9	14.85	479.7	383.8
1 14	39.67	5900.0	32.9	16.45	358.7	358.7
1 15	42.50	5900.0	32.9	16.45	358.7	358.7
1 16	45.33	5900.0	32.9	16.45	358.7	358.7
1 17	48.17	5900.0	32.9	16.45	358.7	358.7
1 18	51.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	CONCRETE(SLAB) TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11395.1	0.022778	477.8	1259.3	657.8
1	1	2.83	11395.1	0.022778	477.8	1259.3	657.8
1	2	5.67	11395.1	0.022778	477.8	1259.3	657.8
1	3	8.50	11395.1	0.022778	477.8	1259.3	657.8
1	4	11.33	11395.1	0.022778	477.8	1259.3	657.8
1	5	14.17	14054.0	0.021272	630.4	1265.2	725.1
1	6	17.00	14054.0	0.021272	630.4	1265.2	725.1
1	7	19.83	14054.0	0.021272	630.4	1265.2	725.1
1	8	22.67	14054.0	0.021272	630.4	1265.2	725.1
1	9	25.50	14054.0	0.021272	630.4	1265.2	725.1
1	10	28.33	14054.0	0.021272	630.4	1265.2	725.1
1	11	31.17	14054.0	0.021272	630.4	1265.2	725.1
1	12	34.00	14054.0	0.021272	630.4	1265.2	725.1
1	13	36.83	14054.0	0.021272	630.4	1265.2	725.1
1	14	39.67	11395.1	0.022778	477.8	1259.3	657.8
1	15	42.50	11395.1	0.022778	477.8	1259.3	657.8
1	16	45.33	11395.1	0.022778	477.8	1259.3	657.8
1	17	48.17	11395.1	0.022778	477.8	1259.3	657.8
1	18	51.00	11395.1	0.022778	477.8	1259.3	657.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	CONCRETE(SLAB) TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15689.7	0.029013	532.1	4594.9	1342.2
1	1	2.83	15689.7	0.029013	532.1	4594.9	1342.2
1	2	5.67	15689.7	0.029013	532.1	4594.9	1342.2
1	3	8.50	15689.7	0.029013	532.1	4594.9	1342.2
1	4	11.33	15689.7	0.029013	532.1	4594.9	1342.2
1	5	14.17	19883.5	0.027358	699.2	4008.3	1502.3
1	6	17.00	19883.5	0.027358	699.2	4008.3	1502.3
1	7	19.83	19883.5	0.027358	699.2	4008.3	1502.3

1 8	22.67	19883.5	0.027358	699.2	4008.3	1502.3
1 9	25.50	19883.5	0.027358	699.2	4008.3	1502.3
1 10	28.33	19883.5	0.027358	699.2	4008.3	1502.3
1 11	31.17	19883.5	0.027358	699.2	4008.3	1502.3
1 12	34.00	19883.5	0.027358	699.2	4008.3	1502.3
1 13	36.83	19883.5	0.027358	699.2	4008.3	1502.3
1 14	39.67	15689.7	0.029013	532.1	4594.9	1342.2
1 15	42.50	15689.7	0.029013	532.1	4594.9	1342.2
1 16	45.33	15689.7	0.029013	532.1	4594.9	1342.2
1 17	48.17	15689.7	0.029013	532.1	4594.9	1342.2
1 18	51.00	15689.7	0.029013	532.1	4594.9	1342.2

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	2.83	8.7	37.8	0.5	0.0		47.0
1	2	5.67	16.4	71.2	1.0	0.0		88.6
1	3	8.50	23.0	100.1	1.4	0.0		124.5
1	4	11.33	28.6	124.5	1.5	0.0		154.7
1	5	14.17	33.2	144.5	1.6	0.0		179.4
1	6	17.00	36.8	160.1	1.8	0.0		198.7
1	7	19.83	39.4	171.2	1.9	0.0		212.5
1	8	22.67	40.9	177.9	2.0	0.0		220.8
1	9	25.50	41.4	180.1	2.1	0.0		223.7
1	10	28.33	40.9	177.9	2.2	0.0		221.0
1	11	31.17	39.4	171.2	2.1	0.0		212.7
1	12	34.00	36.8	160.1	1.8	0.0		198.7

1 13	36.83	33.2	144.5	1.5	0.0	179.3
1 14	39.67	28.6	124.5	1.2	0.0	154.3
1 15	42.50	23.0	100.1	0.9	0.0	124.0
1 16	45.33	16.4	71.2	0.6	0.0	88.1
1 17	48.17	8.7	37.8	0.3	0.0	46.8
1 18	51.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.83	19.9	11.1	0.0	0.0	0.0	31.0
1	2	5.67	37.5	20.9	0.0	0.0	0.0	58.4
1	3	8.50	52.7	29.4	0.0	0.0	0.0	82.2
1	4	11.33	65.6	36.6	0.0	0.0	0.0	102.3
1	5	14.17	76.2	42.5	0.0	0.0	0.0	118.7
1	6	17.00	84.4	47.1	0.0	0.0	0.0	131.5
1	7	19.83	90.2	50.4	0.0	0.0	0.0	140.6
1	8	22.67	93.8	52.3	0.0	0.0	0.0	146.1
1	9	25.50	94.9	53.0	0.0	0.0	0.0	147.9
1	10	28.33	93.8	52.3	0.0	0.0	0.0	146.1
1	11	31.17	90.2	50.4	0.0	0.0	0.0	140.6
1	12	34.00	84.4	47.1	0.0	0.0	0.0	131.5
1	13	36.83	76.2	42.5	0.0	0.0	0.0	118.7
1	14	39.67	65.6	36.6	0.0	0.0	0.0	102.3
1	15	42.50	52.7	29.4	0.0	0.0	0.0	82.2
1	16	45.33	37.5	20.9	0.0	0.0	0.0	58.4
1	17	48.17	19.9	11.1	0.0	0.0	0.0	31.0
1	18	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	2.83	0.0	0.0	141.9	HL-93	0.0	HL-93
1	2	5.67	0.0	0.0	263.8	HL-93	0.0	HL-93
1	3	8.50	0.0	0.0	365.7	HL-93	0.0	HL-93
1	4	11.33	0.0	0.0	446.1	HL-93	0.0	HL-93
1	5	14.17	0.0	0.0	508.0	HL-93	0.0	HL-93
1	6	17.00	0.0	0.0	556.0	HL-93	0.0	HL-93
1	7	19.83	0.0	0.0	587.8	HL-93	0.0	HL-93
1	8	22.67	0.0	0.0	596.9	HL-93	0.0	HL-93
1	9	25.50	0.0	0.0	599.1	HL-93	0.0	HL-93
1	10	28.33	0.0	0.0	596.9	HL-93	0.0	HL-93
1	11	31.17	0.0	0.0	587.8	HL-93	0.0	HL-93
1	12	34.00	0.0	0.0	556.0	HL-93	0.0	HL-93

1 13	36.83	0.0	0.0	508.0	HL-93	0.0	HL-93
1 14	39.67	0.0	0.0	446.1	HL-93	0.0	HL-93
1 15	42.50	0.0	0.0	365.7	HL-93	0.0	HL-93
1 16	45.33	0.0	0.0	263.8	HL-93	0.0	HL-93
1 17	48.17	0.0	0.0	141.9	HL-93	0.0	HL-93
1 18	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.83	50.	0.	50.
1	2	5.67	91.	0.	91.
1	3	8.50	123.	0.	123.
1	4	11.33	148.	0.	148.
1	5	14.17	165.	0.	165.
1	6	17.00	174.	0.	174.
1	7	19.83	180.	0.	180.
1	8	22.67	188.	0.	188.
1	9	25.50	191.	0.	191.
1	10	28.33	188.	0.	188.
1	11	31.17	180.	0.	180.
1	12	34.00	174.	0.	174.
1	13	36.83	165.	0.	165.
1	14	39.67	148.	0.	148.
1	15	42.50	123.	0.	123.
1	16	45.33	91.	0.	91.
1	17	48.17	50.	0.	50.
1	18	51.00	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.83	47.0	31.0	141.9	HL-93	0.0	HL-93	220.0	78.1
1	2	5.67	88.6	58.4	263.8	HL-93	0.0	HL-93	410.8	147.0
1	3	8.50	124.5	82.2	365.7	HL-93	0.0	HL-93	572.4	206.7
1	4	11.33	154.7	102.3	446.1	HL-93	0.0	HL-93	703.1	257.0
1	5	14.17	179.4	118.7	508.0	HL-93	0.0	HL-93	806.1	298.1
1	6	17.00	198.7	131.5	556.0	HL-93	0.0	HL-93	886.2	330.2
1	7	19.83	212.5	140.6	587.8	HL-93	0.0	HL-93	940.9	353.1
1	8	22.67	220.8	146.1	596.9	HL-93	0.0	HL-93	963.8	366.9
1	9	25.50	223.7	147.9	599.1	HL-93	0.0	HL-93	970.7	371.6
1	10	28.33	221.0	146.1	596.9	HL-93	0.0	HL-93	964.0	367.1
1	11	31.17	212.7	140.6	587.8	HL-93	0.0	HL-93	941.0	353.3
1	12	34.00	198.7	131.5	556.0	HL-93	0.0	HL-93	886.2	330.2
1	13	36.83	179.3	118.7	508.0	HL-93	0.0	HL-93	805.9	298.0
1	14	39.67	154.3	102.3	446.1	HL-93	0.0	HL-93	702.7	256.6

1	15	42.50	124.0	82.2	365.7	HL-93	0.0	HL-93	571.8	206.1
1	16	45.33	88.1	58.4	263.8	HL-93	0.0	HL-93	410.3	146.6
1	17	48.17	46.8	31.0	141.9	HL-93	0.0	HL-93	219.7	77.8
1	18	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	2.83	220.0	262.5	350.9	294.1	117.1	37.6
1	2	5.67	410.8	489.9	654.8	549.3	220.5	68.4
1	3	8.50	572.4	682.1	911.5	765.2	310.0	92.4
1	4	11.33	703.1	836.9	1118.3	939.9	385.5	110.8
1	5	14.17	806.1	958.5	1280.7	1077.5	447.2	123.9
1	6	17.00	886.2	1053.0	1406.8	1184.4	495.3	130.5
1	7	19.83	940.9	1117.2	1492.5	1257.4	529.7	134.9
1	8	22.67	963.8	1142.9	1526.7	1287.9	550.4	141.0
1	9	25.50	970.7	1150.4	1536.6	1297.0	557.4	143.3
1	10	28.33	964.0	1143.1	1527.0	1288.2	550.7	141.0
1	11	31.17	941.0	1117.4	1492.8	1257.6	529.9	134.9
1	12	34.00	886.2	1053.0	1406.8	1184.4	495.3	130.5
1	13	36.83	805.9	958.3	1280.5	1077.3	446.9	123.9
1	14	39.67	702.7	836.5	1117.9	939.4	384.9	110.8
1	15	42.50	571.8	681.5	910.8	764.5	309.2	92.4
1	16	45.33	410.3	489.5	654.2	548.7	219.8	68.4
1	17	48.17	219.7	262.3	350.6	293.8	116.8	37.6
1	18	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.2	14.1	0.2	0.0	17.6
1	1	2.83	2.9	12.6	0.2	0.0	15.6
1	2	5.67	2.5	11.0	0.2	0.0	13.7
1	3	8.50	2.2	9.4	0.0	0.0	11.6
1	4	11.33	1.8	7.8	0.0	0.0	9.7
1	5	14.17	1.4	6.3	0.0	0.0	7.8
1	6	17.00	1.1	4.7	0.0	0.0	5.8
1	7	19.83	0.7	3.1	0.0	0.0	3.9
1	8	22.67	0.4	1.6	0.0	0.0	2.0
1	9	25.50	0.0	0.0	0.0	0.0	0.0
1	10	28.33	-0.4	-1.6	0.0	0.0	-1.9
1	11	31.17	-0.7	-3.1	-0.1	0.0	-4.0
1	12	34.00	-1.1	-4.7	-0.1	0.0	-5.9
1	13	36.83	-1.4	-6.3	-0.1	0.0	-7.8
1	14	39.67	-1.8	-7.8	-0.1	0.0	-9.8

1	15	42.50	-2.2	-9.4	-0.1	0.0	-11.7
1	16	45.33	-2.5	-11.0	-0.1	0.0	-13.6
1	17	48.17	-2.9	-12.6	-0.1	0.0	-15.5
1	18	51.00	-3.2	-14.1	-0.1	0.0	-17.5

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	COMPOSITE (k) UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	7.4	4.2	0.0	0.0	11.6	17.6	29.2
1	1	2.83	6.6	3.7	0.0	0.0	10.3	15.6	25.9
1	2	5.67	5.8	3.2	0.0	0.0	9.0	13.7	22.7
1	3	8.50	5.0	2.8	0.0	0.0	7.7	11.6	19.4
1	4	11.33	4.1	2.3	0.0	0.0	6.4	9.7	16.1
1	5	14.17	3.3	1.8	0.0	0.0	5.2	7.8	12.9
1	6	17.00	2.5	1.4	0.0	0.0	3.9	5.8	9.7
1	7	19.83	1.7	0.9	0.0	0.0	2.6	3.9	6.5
1	8	22.67	0.8	0.5	0.0	0.0	1.3	2.0	3.3
1	9	25.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	10	28.33	-0.8	-0.5	0.0	0.0	-1.3	-1.9	-3.2
1	11	31.17	-1.7	-0.9	0.0	0.0	-2.6	-4.0	-6.5
1	12	34.00	-2.5	-1.4	0.0	0.0	-3.9	-5.9	-9.8
1	13	36.83	-3.3	-1.8	0.0	0.0	-5.2	-7.8	-13.0
1	14	39.67	-4.1	-2.3	0.0	0.0	-6.4	-9.8	-16.2
1	15	42.50	-5.0	-2.8	0.0	0.0	-7.7	-11.7	-19.4
1	16	45.33	-5.8	-3.2	0.0	0.0	-9.0	-13.6	-22.6
1	17	48.17	-6.6	-3.7	0.0	0.0	-10.3	-15.5	-25.9
1	18	51.00	-7.4	-4.2	0.0	0.0	-11.6	-17.5	-29.1

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	52.0	HL-93	0.0	HL-93
1	1	2.83	0.0	0.0	48.1	HL-93	-1.3	HL-93
1	2	5.67	0.0	0.0	44.3	HL-93	-2.7	HL-93
1	3	8.50	0.0	0.0	40.5	HL-93	-4.9	HL-93
1	4	11.33	0.0	0.0	36.7	HL-93	-7.1	HL-93
1	5	14.17	0.0	0.0	33.1	HL-93	-9.4	HL-93
1	6	17.00	0.0	0.0	29.4	HL-93	-11.7	HL-93
1	7	19.83	0.0	0.0	25.9	HL-93	-14.1	HL-93
1	8	22.67	0.0	0.0	22.5	HL-93	-16.6	HL-93
1	9	25.50	0.0	0.0	19.2	HL-93	-19.2	HL-93
1	10	28.33	0.0	0.0	16.6	HL-93	-22.5	HL-93
1	11	31.17	0.0	0.0	14.1	HL-93	-25.9	HL-93
1	12	34.00	0.0	0.0	11.7	HL-93	-29.4	HL-93
1	13	36.83	0.0	0.0	9.4	HL-93	-33.1	HL-93

1 14	39.67	0.0	0.0	7.1	HL-93	-36.7	HL-93
1 15	42.50	0.0	0.0	4.9	HL-93	-40.5	HL-93
1 16	45.33	0.0	0.0	2.7	HL-93	-44.3	HL-93
1 17	48.17	0.0	0.0	1.3	HL-93	-48.1	HL-93
1 18	51.00	0.0	0.0	0.0	HL-93	-52.0	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	21.	0.	21.
1	1	2.83	19.	-1.	20.
1	2	5.67	17.	-2.	19.
1	3	8.50	16.	-2.	18.
1	4	11.33	14.	-3.	17.
1	5	14.17	12.	-4.	16.
1	6	17.00	10.	-5.	15.
1	7	19.83	9.	-6.	15.
1	8	22.67	9.	-7.	16.
1	9	25.50	8.	-8.	16.
1	10	28.33	7.	-9.	16.
1	11	31.17	6.	-9.	15.
1	12	34.00	5.	-10.	15.
1	13	36.83	4.	-12.	16.
1	14	39.67	3.	-14.	17.
1	15	42.50	2.	-16.	18.
1	16	45.33	2.	-17.	19.
1	17	48.17	1.	-19.	20.
1	18	51.00	0.	-21.	21.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	20.7	.31013E-01	0.32	0.2500*	.14641E-01	0.15	0.2500*
1	1	2.83	19.7	.31013E-01	0.31	0.2500*	.14641E-01	0.14	0.2500*
1	2	5.67	18.8	.31013E-01	0.29	0.2500*	.14641E-01	0.14	0.2500*
1	3	8.50	17.9	.31013E-01	0.28	0.2500*	.14641E-01	0.13	0.2500*
1	4	11.33	17.1	.31013E-01	0.26	0.2500*	.14641E-01	0.12	0.2500*
1	5	14.17	16.3	.29585E-01	0.24	0.2500*	.10903E-01	0.09	0.2500*
1	6	17.00	15.4	.29585E-01	0.23	0.2500*	.10903E-01	0.08	0.2500*
1	7	19.83	15.0	.29585E-01	0.22	0.2500*	.10903E-01	0.08	0.2500*
1	8	22.67	15.7	.29585E-01	0.23	0.2500*	.10903E-01	0.09	0.2500*
1	9	25.50	16.0	.29585E-01	0.24	0.2500*	.10903E-01	0.09	0.2500*
1	10	28.33	15.7	.29585E-01	0.23	0.2500*	.10903E-01	0.09	0.2500*
1	11	31.17	15.0	.29585E-01	0.22	0.2500*	.10903E-01	0.08	0.2500*
1	12	34.00	15.4	.29585E-01	0.23	0.2500*	.10903E-01	0.08	0.2500*
1	13	36.83	16.3	.29585E-01	0.24	0.2500*	.10903E-01	0.09	0.2500*
1	14	39.67	17.1	.31013E-01	0.26	0.2500*	.14641E-01	0.12	0.2500*
1	15	42.50	17.9	.31013E-01	0.28	0.2500*	.14641E-01	0.13	0.2500*
1	16	45.33	18.8	.31013E-01	0.29	0.2500*	.14641E-01	0.14	0.2500*
1	17	48.17	19.7	.31013E-01	0.31	0.2500*	.14641E-01	0.14	0.2500*
1	18	51.00	20.7	.31013E-01	0.32	0.2500*	.14641E-01	0.15	0.2500*

```
-----------------------------------------------------------------
  NOTE:  [1] Vr = range of shear due to live loads and impact

         [2] For non-composite construction:

             Q/I = (At * Dt) / Inc -- top flange
             Inc = moment of inertia of non-composite section
             At = area of top flange
             Dt = distance between the center of top flange and neutral axis
             Q/I = (Ab * Db) / Inc -- bottom flange
             Ab = area of bottom flange
             Db = distance between the center of bottom flange and neutral axis

             For composite construction:

             Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
             Q = statical moment about the neutral axis of the
                 composite section of the transformed compressive
                 concrete area or the area of reinforcement embedded
                 in the concrete for negative moment
             Ic = moment of inertia for composite section
             Q/I = (Ab * Db) / Ic -- bottom flange

         [3] shear flow = (Vr * Q) / (2 * I)

         * -- minimum weld size governs
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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	29.2	52.0	HL-93	0.0	HL-93	81.2	29.2
1	1	2.83	25.9	48.1	HL-93	-1.3	HL-93	74.0	24.6
1	2	5.67	22.7	44.3	HL-93	-2.7	HL-93	67.0	20.0
1	3	8.50	19.4	40.5	HL-93	-4.9	HL-93	59.8	14.5
1	4	11.33	16.1	36.7	HL-93	-7.1	HL-93	52.9	9.0
1	5	14.17	12.9	33.1	HL-93	-9.4	HL-93	46.0	3.5
1	6	17.00	9.7	29.4	HL-93	-11.7	HL-93	39.1	-2.0
1	7	19.83	6.5	25.9	HL-93	-14.1	HL-93	32.4	-7.7
1	8	22.67	3.3	22.5	HL-93	-16.6	HL-93	25.7	-13.3
1	9	25.50	0.0	19.2	HL-93	-19.2	HL-93	19.3	-19.2
1	10	28.33	-3.2	16.6	HL-93	-22.5	HL-93	13.4	-25.7
1	11	31.17	-6.5	14.1	HL-93	-25.9	HL-93	7.6	-32.4
1	12	34.00	-9.8	11.7	HL-93	-29.4	HL-93	2.0	-39.2
1	13	36.83	-13.0	9.4	HL-93	-33.1	HL-93	-3.6	-46.1
1	14	39.67	-16.2	7.1	HL-93	-36.7	HL-93	-9.1	-52.9
1	15	42.50	-19.4	4.9	HL-93	-40.5	HL-93	-14.5	-59.9
1	16	45.33	-22.6	2.7	HL-93	-44.3	HL-93	-19.9	-66.9
1	17	48.17	-25.9	1.3	HL-93	-48.1	HL-93	-24.5	-74.0
1	18	51.00	-29.1	0.0	HL-93	-52.0	HL-93	-29.1	-81.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	81.2	96.8	129.3	108.5	43.7	15.6
1	1	2.83	74.0	88.5	118.3	99.0	38.9	14.8
1	2	5.67	67.0	80.3	107.3	89.6	34.1	14.1
1	3	8.50	59.8	72.0	96.3	80.1	29.0	13.4
1	4	11.33	52.9	63.9	85.5	70.8	24.2	12.8
1	5	14.17	46.0	55.9	74.8	61.6	19.4	12.2
1	6	17.00	39.1	47.9	64.2	52.5	14.5	11.6
1	7	19.83	32.4	40.1	53.8	43.4	9.7	11.2
1	8	22.67	25.7	32.5	43.6	34.6	4.9	11.7
1	9	25.50	19.3	25.0	33.7	26.0	0.1	12.0
1	10	28.33	-25.7	-32.4	-43.5	-34.5	-4.8	11.7
1	11	31.17	-32.4	-40.2	-53.9	-43.5	-9.8	11.2
1	12	34.00	-39.2	-48.0	-64.3	-52.5	-14.6	11.6
1	13	36.83	-46.1	-56.0	-74.9	-61.7	-19.5	12.2
1	14	39.67	-52.9	-64.0	-85.6	-70.9	-24.3	12.8
1	15	42.50	-59.9	-72.0	-96.3	-80.2	-29.1	13.4
1	16	45.33	-66.9	-80.2	-107.2	-89.5	-34.0	14.1
1	17	48.17	-74.0	-88.4	-118.2	-98.9	-38.8	14.8
1	18	51.00	-81.1	-96.7	-129.2	-108.4	-43.6	15.6

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	29.16	0.00	HL-93	51.99	HL-93	ST1	24.39	129.31
						ST2	24.39	108.51
						ST4	32.58	43.74
						SE1	29.16	81.16
						SE2	29.16	96.76
2	29.08	0.00	HL-93	51.99	HL-93	ST1	24.31	129.20
						ST2	24.31	108.41
						ST4	32.45	43.62
						SE1	29.08	81.08
						SE2	29.08	96.68

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.83	-0.0175	-0.0760	0.0000	-0.0318	-0.0943	-0.0318	-0.1261

1	2	5.67	-0.0342	-0.1489	0.0000	-0.0623	-0.1849	-0.0623	-0.2472
1	3	8.50	-0.0497	-0.2161	0.0000	-0.0904	-0.2683	-0.0904	-0.3587
1	4	11.33	-0.0633	-0.2752	0.0000	-0.1151	-0.3417	-0.1151	-0.4567
1	5	14.17	-0.0748	-0.3250	0.0000	-0.1358	-0.4035	-0.1358	-0.5394
1	6	17.00	-0.0840	-0.3652	0.0000	-0.1525	-0.4534	-0.1525	-0.6059
1	7	19.83	-0.0908	-0.3946	0.0000	-0.1648	-0.4899	-0.1648	-0.6547
1	8	22.67	-0.0949	-0.4126	0.0000	-0.1723	-0.5122	-0.1723	-0.6845
1	9	25.50	-0.0963	-0.4186	0.0000	-0.1748	-0.5197	-0.1748	-0.6944
1	10	28.33	-0.0949	-0.4126	0.0000	-0.1723	-0.5122	-0.1723	-0.6844
1	11	31.17	-0.0908	-0.3946	0.0000	-0.1648	-0.4899	-0.1648	-0.6546
1	12	34.00	-0.0840	-0.3652	0.0000	-0.1525	-0.4533	-0.1525	-0.6058
1	13	36.83	-0.0748	-0.3250	0.0000	-0.1358	-0.4034	-0.1358	-0.5392
1	14	39.67	-0.0633	-0.2752	0.0000	-0.1151	-0.3415	-0.1151	-0.4566
1	15	42.50	-0.0497	-0.2161	0.0000	-0.0904	-0.2681	-0.0904	-0.3585
1	16	45.33	-0.0342	-0.1489	0.0000	-0.0623	-0.1847	-0.0623	-0.2471
1	17	48.17	-0.0175	-0.0760	0.0000	-0.0318	-0.0942	-0.0318	-0.1261
1	18	51.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.8	-0.017	0 1/16	-0.077	0 1/ 8	-0.032	0 1/16	-0.126	0 3/16
1	2	5.7	-0.034	0 1/16	-0.151	0 3/16	-0.062	0 1/16	-0.247	0 1/ 4
1	3	8.5	-0.050	0 1/16	-0.219	0 1/ 4	-0.090	0 1/ 8	-0.359	0 3/ 8
1	4	11.3	-0.063	0 1/ 8	-0.278	0 5/16	-0.115	0 1/ 8	-0.457	0 1/ 2
1	5	14.2	-0.075	0 1/ 8	-0.329	0 3/ 8	-0.136	0 3/16	-0.539	0 9/16
1	6	17.0	-0.084	0 1/ 8	-0.369	0 3/ 8	-0.153	0 3/16	-0.606	0 5/ 8
1	7	19.8	-0.091	0 1/ 8	-0.399	0 7/16	-0.165	0 3/16	-0.655	0 11/16
1	8	22.7	-0.095	0 1/ 8	-0.417	0 7/16	-0.172	0 3/16	-0.684	0 11/16
1	9	25.5	-0.096	0 1/ 8	-0.423	0 7/16	-0.175	0 3/16	-0.694	0 3/ 4
1	10	28.3	-0.095	0 1/ 8	-0.417	0 7/16	-0.172	0 3/16	-0.684	0 11/16
1	11	31.2	-0.091	0 1/ 8	-0.399	0 7/16	-0.165	0 3/16	-0.655	0 11/16
1	12	34.0	-0.084	0 1/ 8	-0.369	0 3/ 8	-0.153	0 3/16	-0.606	0 5/ 8
1	13	36.8	-0.075	0 1/ 8	-0.329	0 3/ 8	-0.136	0 3/16	-0.539	0 9/16
1	14	39.7	-0.063	0 1/ 8	-0.278	0 5/16	-0.115	0 1/ 8	-0.457	0 1/ 2
1	15	42.5	-0.050	0 1/16	-0.218	0 1/ 4	-0.090	0 1/ 8	-0.359	0 3/ 8
1	16	45.3	-0.034	0 1/16	-0.150	0 3/16	-0.062	0 1/16	-0.247	0 1/ 4
1	17	48.2	-0.017	0 1/16	-0.077	0 1/ 8	-0.032	0 1/16	-0.126	0 3/16
1	18	51.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

NUMBER OF LANE LL + I.

SPAN NO.	D FROM L SUPT (ft)	AND DIST. FOR LL DEFL.	FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.50	3	0.425	-0.289	MAX HL-93	0.76	0.00274
				0.029	MIN		
				-0.074	MAX LANE		
				0.029	MIN LANE		
		4	0.433	-0.295	MAX HL-93	0.76	0.00274
				0.030	MIN		
				-0.076	MAX LANE		
				0.030	MIN LANE		
		3	0.425	-0.182	MAX FA		
				0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	-0.29	0.29	-1.28	1.28	-1.57	1.57
1	2	5.67	-0.55	0.55	-2.42	2.42	-2.96	2.96
1	3	8.50	-0.77	0.77	-3.40	3.40	-4.17	4.17
1	4	11.33	-0.96	0.96	-4.22	4.22	-5.18	5.18
1	5	14.17	-1.04	0.83	-4.57	3.66	-5.61	4.49
1	6	17.00	-1.15	0.92	-5.06	4.05	-6.21	4.97
1	7	19.83	-1.23	0.99	-5.41	4.33	-6.64	5.32
1	8	22.67	-1.28	1.02	-5.62	4.50	-6.90	5.52
1	9	25.50	-1.30	1.04	-5.70	4.56	-6.99	5.60
1	10	28.33	-1.28	1.02	-5.63	4.51	-6.91	5.53
1	11	31.17	-1.23	0.99	-5.42	4.34	-6.65	5.32
1	12	34.00	-1.15	0.92	-5.06	4.05	-6.21	4.97
1	13	36.83	-1.04	0.83	-4.56	3.65	-5.60	4.48
1	14	39.67	-0.96	0.96	-4.21	4.21	-5.16	5.16
1	15	42.50	-0.77	0.77	-3.38	3.38	-4.15	4.15

1	16	45.33	-0.55	0.55	-2.40	2.40	-2.95	2.95
1	17	48.17	-0.29	0.29	-1.27	1.27	-1.57	1.57
1	18	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	-0.02	-0.30	0.78	-0.02	-1.87	2.35
1	2	5.67	-0.04	-0.56	1.47	-0.04	-3.52	4.43
1	3	8.50	-0.06	-0.78	2.06	-0.06	-4.95	6.23
1	4	11.33	-0.07	-0.97	2.57	-0.07	-6.15	7.74
1	5	14.17	-0.07	-1.13	2.26	-0.07	-6.74	6.75
1	6	17.00	-0.08	-1.25	2.50	-0.08	-7.46	7.47
1	7	19.83	-0.09	-1.33	2.68	-0.09	-7.98	7.99
1	8	22.67	-0.09	-1.39	2.78	-0.09	-8.29	8.30
1	9	25.50	-0.09	-1.40	2.82	-0.09	-8.40	8.41
1	10	28.33	-0.09	-1.39	2.78	-0.09	-8.30	8.31
1	11	31.17	-0.09	-1.33	2.68	-0.09	-7.98	8.00
1	12	34.00	-0.08	-1.25	2.50	-0.08	-7.46	7.47
1	13	36.83	-0.07	-1.13	2.26	-0.07	-6.73	6.74
1	14	39.67	-0.07	-0.97	2.57	-0.07	-6.14	7.73
1	15	42.50	-0.06	-0.78	2.06	-0.06	-4.93	6.21
1	16	45.33	-0.04	-0.56	1.47	-0.04	-3.50	4.42
1	17	48.17	-0.02	-0.30	0.78	-0.02	-1.86	2.35
1	18	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	-0.14	-0.37	3.20	0.00	0.00	0.00
1	2	5.67	-0.26	-0.69	5.95	0.00	0.00	0.00
1	3	8.50	-0.36	-0.95	8.25	0.00	0.00	0.00
1	4	11.33	-0.44	-1.17	10.06	0.00	0.00	0.00
1	5	14.17	-0.45	-1.52	8.72	0.00	0.00	0.00
1	6	17.00	-0.49	-1.66	9.54	0.00	0.00	0.00
1	7	19.83	-0.52	-1.76	10.09	0.00	0.00	0.00
1	8	22.67	-0.53	-1.79	10.25	0.00	0.00	0.00
1	9	25.50	-0.53	-1.79	10.28	0.00	0.00	0.00
1	10	28.33	-0.53	-1.79	10.25	0.00	0.00	0.00
1	11	31.17	-0.52	-1.76	10.09	0.00	0.00	0.00
1	12	34.00	-0.49	-1.66	9.54	0.00	0.00	0.00
1	13	36.83	-0.45	-1.52	8.72	0.00	0.00	0.00

1 14	39.67	-0.44	-1.17	10.06	0.00	0.00	0.00
1 15	42.50	-0.36	-0.95	8.25	0.00	0.00	0.00
1 16	45.33	-0.26	-0.69	5.95	0.00	0.00	0.00
1 17	48.17	-0.14	-0.37	3.20	0.00	0.00	0.00
1 18	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	2.83	0.1	0.0	0.1	C	HL	1.1	0.0	1.1	T	HL
1	2	5.67	0.2	0.0	0.2	C	HL	2.1	0.0	2.1	T	HL
1	3	8.50	0.3	0.0	0.3	C	HL	2.8	0.0	2.8	T	HL
1	4	11.33	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	5	14.17	0.5	0.0	0.5	C	HL	2.8	0.0	2.8	T	HL
1	6	17.00	0.5	0.0	0.5	C	HL	3.0	0.0	3.0	T	HL
1	7	19.83	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	8	22.67	0.6	0.0	0.6	C	HL	3.2	0.0	3.2	T	HL
1	9	25.50	0.6	0.0	0.6	C	HL	3.3	0.0	3.3	T	HL
1	10	28.33	0.6	0.0	0.6	C	HL	3.2	0.0	3.2	T	HL
1	11	31.17	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	12	34.00	0.5	0.0	0.5	C	HL	3.0	0.0	3.0	T	HL
1	13	36.83	0.5	0.0	0.5	C	HL	2.8	0.0	2.8	T	HL
1	14	39.67	0.4	0.0	0.4	C	HL	3.3	0.0	3.3	T	HL
1	15	42.50	0.3	0.0	0.3	C	HL	2.8	0.0	2.8	T	HL
1	16	45.33	0.2	0.0	0.2	C	HL	2.1	0.0	2.1	T	HL
1	17	48.17	0.1	0.0	0.1	C	HL	1.1	0.0	1.1	T	HL
1	18	51.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM TOP	TOTAL POSITIVE ,(ksi) STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM TOP	TOTAL NEGATIVE ,(ksi) STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.16	-2.24	5.55	-0.02	-1.87	2.35
1	2	5.67	33.	33.	-0.30	-4.21	10.38	-0.04	-3.52	4.43
1	3	8.50	33.	33.	-0.42	-5.90	14.48	-0.06	-4.95	6.23
1	4	11.33	33.	33.	-0.51	-7.32	17.81	-0.07	-6.15	7.74
1	5	14.17	33.	33.	-0.52	-8.26	15.47	-0.07	-6.74	6.75
1	6	17.00	33.	33.	-0.57	-9.12	17.02	-0.08	-7.46	7.47
1	7	19.83	33.	33.	-0.61	-9.74	18.08	-0.09	-7.98	7.99
1	8	22.67	33.	33.	-0.62	-10.08	18.55	-0.09	-8.29	8.30
1	9	25.50	33.	33.	-0.62	-10.19	18.69	-0.09	-8.40	8.41
1	10	28.33	33.	33.	-0.62	-10.08	18.56	-0.09	-8.30	8.31

1 11	31.17	33.	33.	-0.61	-9.74	18.08	-0.09	-7.98	8.00
1 12	34.00	33.	33.	-0.57	-9.12	17.02	-0.08	-7.46	7.47
1 13	36.83	33.	33.	-0.52	-8.25	15.46	-0.07	-6.73	6.74
1 14	39.67	33.	33.	-0.51	-7.30	17.79	-0.07	-6.14	7.73
1 15	42.50	33.	33.	-0.42	-5.89	14.46	-0.06	-4.93	6.21
1 16	45.33	33.	33.	-0.30	-4.19	10.36	-0.04	-3.50	4.42
1 17	48.17	33.	33.	-0.16	-2.23	5.55	-0.02	-1.86	2.35
1 18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.20	-2.35	6.51	-0.02	-1.87	2.35
1	2	5.67	33.	33.	-0.38	-4.42	12.16	-0.04	-3.52	4.43
1	3	8.50	33.	33.	-0.53	-6.19	16.95	-0.06	-4.95	6.23
1	4	11.33	33.	33.	-0.65	-7.67	20.82	-0.07	-6.15	7.74
1	5	14.17	33.	33.	-0.66	-8.71	18.08	-0.07	-6.74	6.75
1	6	17.00	33.	33.	-0.72	-9.62	19.88	-0.08	-7.46	7.47
1	7	19.83	33.	33.	-0.76	-10.26	21.11	-0.09	-7.98	7.99
1	8	22.67	33.	33.	-0.78	-10.61	21.62	-0.09	-8.29	8.30
1	9	25.50	33.	33.	-0.78	-10.73	21.78	-0.09	-8.40	8.41
1	10	28.33	33.	33.	-0.78	-10.62	21.63	-0.09	-8.30	8.31
1	11	31.17	33.	33.	-0.76	-10.27	21.11	-0.09	-7.98	8.00
1	12	34.00	33.	33.	-0.72	-9.62	19.88	-0.08	-7.46	7.47
1	13	36.83	33.	33.	-0.66	-8.71	18.08	-0.07	-6.73	6.74
1	14	39.67	33.	33.	-0.65	-7.65	20.81	-0.07	-6.14	7.73
1	15	42.50	33.	33.	-0.53	-6.17	16.93	-0.06	-4.93	6.21
1	16	45.33	33.	33.	-0.38	-4.40	12.15	-0.04	-3.50	4.42
1	17	48.17	33.	33.	-0.20	-2.34	6.51	-0.02	-1.86	2.35
1	18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.28	-3.03	8.67	-0.03	-2.38	3.07
1	2	5.67	33.	33.	-0.51	-5.70	16.19	-0.06	-4.49	5.77
1	3	8.50	33.	33.	-0.71	-7.98	22.55	-0.08	-6.31	8.12
1	4	11.33	33.	33.	-0.87	-9.88	27.70	-0.10	-7.84	10.09
1	5	14.17	33.	33.	-0.89	-11.26	24.06	-0.10	-8.60	8.80
1	6	17.00	33.	33.	-0.98	-12.44	26.44	-0.11	-9.52	9.74
1	7	19.83	33.	33.	-1.03	-13.26	28.07	-0.12	-10.18	10.42
1	8	22.67	33.	33.	-1.05	-13.71	28.76	-0.13	-10.58	10.83
1	9	25.50	33.	33.	-1.06	-13.86	28.96	-0.13	-10.72	10.97
1	10	28.33	33.	33.	-1.05	-13.72	28.76	-0.13	-10.59	10.83
1	11	31.17	33.	33.	-1.03	-13.27	28.08	-0.12	-10.19	10.43
1	12	34.00	33.	33.	-0.98	-12.44	26.44	-0.11	-9.52	9.74

1 13	36.83	33.	33.	-0.89	-11.25	24.05	-0.10	-8.59	8.79
1 14	39.67	33.	33.	-0.87	-9.87	27.68	-0.10	-7.83	10.08
1 15	42.50	33.	33.	-0.71	-7.96	22.53	-0.08	-6.29	8.10
1 16	45.33	33.	33.	-0.51	-5.68	16.17	-0.06	-4.47	5.76
1 17	48.17	33.	33.	-0.28	-3.02	8.66	-0.03	-2.37	3.06
1 18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.26	-2.28	7.59	-0.02	-1.63	1.99
1	2	5.67	33.	33.	-0.49	-4.28	14.16	-0.03	-3.08	3.75
1	3	8.50	33.	33.	-0.68	-6.00	19.71	-0.04	-4.33	5.28
1	4	11.33	33.	33.	-0.83	-7.42	24.16	-0.05	-5.38	6.56
1	5	14.17	33.	33.	-0.84	-8.54	20.97	-0.05	-5.88	5.71
1	6	17.00	33.	33.	-0.92	-9.43	23.02	-0.06	-6.51	6.32
1	7	19.83	33.	33.	-0.98	-10.04	24.42	-0.06	-6.96	6.76
1	8	22.67	33.	33.	-0.99	-10.36	24.96	-0.07	-7.24	7.03
1	9	25.50	33.	33.	-1.00	-10.47	25.11	-0.07	-7.33	7.12
1	10	28.33	33.	33.	-0.99	-10.37	24.96	-0.07	-7.24	7.03
1	11	31.17	33.	33.	-0.98	-10.05	24.42	-0.06	-6.97	6.77
1	12	34.00	33.	33.	-0.92	-9.43	23.02	-0.06	-6.51	6.32
1	13	36.83	33.	33.	-0.84	-8.54	20.96	-0.05	-5.88	5.71
1	14	39.67	33.	33.	-0.83	-7.41	24.15	-0.05	-5.37	6.55
1	15	42.50	33.	33.	-0.68	-5.98	19.69	-0.04	-4.31	5.26
1	16	45.33	33.	33.	-0.49	-4.27	14.15	-0.03	-3.07	3.74
1	17	48.17	33.	33.	-0.26	-2.28	7.59	-0.02	-1.63	1.99
1	18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.22	-2.88	7.39	-0.03	-2.38	3.07
1	2	5.67	33.	33.	-0.41	-5.42	13.81	-0.06	-4.49	5.77
1	3	8.50	33.	33.	-0.57	-7.60	19.25	-0.08	-6.31	8.12
1	4	11.33	33.	33.	-0.70	-9.42	23.67	-0.10	-7.84	10.09
1	5	14.17	33.	33.	-0.71	-10.65	20.57	-0.10	-8.60	8.80
1	6	17.00	33.	33.	-0.78	-11.77	22.63	-0.11	-9.52	9.74
1	7	19.83	33.	33.	-0.83	-12.56	24.04	-0.12	-10.18	10.42
1	8	22.67	33.	33.	-0.84	-13.00	24.66	-0.13	-10.58	10.83
1	9	25.50	33.	33.	-0.85	-13.14	24.85	-0.13	-10.72	10.97
1	10	28.33	33.	33.	-0.84	-13.00	24.67	-0.13	-10.59	10.83
1	11	31.17	33.	33.	-0.83	-12.57	24.04	-0.12	-10.19	10.43
1	12	34.00	33.	33.	-0.78	-11.77	22.63	-0.11	-9.52	9.74
1	13	36.83	33.	33.	-0.71	-10.65	20.56	-0.10	-8.59	8.79
1	14	39.67	33.	33.	-0.70	-9.40	23.66	-0.10	-7.83	10.08
1	15	42.50	33.	33.	-0.57	-7.58	19.23	-0.08	-6.29	8.10

1 16	45.33	33.	33.	-0.41	-5.40	13.79	-0.06	-4.47	5.76
1 17	48.17	33.	33.	-0.22	-2.87	7.38	-0.03	-2.37	3.06
1 18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	2.83	33.	33.	-0.21	-2.13	6.31	-0.02	-1.63	1.99
1	2	5.67	33.	33.	-0.38	-4.01	11.78	-0.03	-3.08	3.75
1	3	8.50	33.	33.	-0.53	-5.62	16.41	-0.04	-4.33	5.28
1	4	11.33	33.	33.	-0.65	-6.95	20.14	-0.05	-5.38	6.56
1	5	14.17	33.	33.	-0.66	-7.93	17.48	-0.05	-5.88	5.71
1	6	17.00	33.	33.	-0.73	-8.76	19.21	-0.06	-6.51	6.32
1	7	19.83	33.	33.	-0.77	-9.34	20.38	-0.06	-6.96	6.76
1	8	22.67	33.	33.	-0.78	-9.65	20.86	-0.07	-7.24	7.03
1	9	25.50	33.	33.	-0.78	-9.75	21.00	-0.07	-7.33	7.12
1	10	28.33	33.	33.	-0.78	-9.66	20.86	-0.07	-7.24	7.03
1	11	31.17	33.	33.	-0.77	-9.35	20.39	-0.06	-6.97	6.77
1	12	34.00	33.	33.	-0.73	-8.76	19.21	-0.06	-6.51	6.32
1	13	36.83	33.	33.	-0.66	-7.93	17.48	-0.05	-5.88	5.71
1	14	39.67	33.	33.	-0.65	-6.94	20.13	-0.05	-5.37	6.55
1	15	42.50	33.	33.	-0.53	-5.60	16.39	-0.04	-4.31	5.26
1	16	45.33	33.	33.	-0.38	-4.00	11.77	-0.03	-3.07	3.74
1	17	48.17	33.	33.	-0.21	-2.13	6.31	-0.02	-1.63	1.99
1	18	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP NO	JOINT TYPE	D FROM L SUPT (ft)	DL1, N=INF. MOM. (k-ft)	STRESS (ksi)	DL2, N=27.0 MOM. (k-ft)	STRESS (ksi)	POS. LL+I MOM. (k-ft)	STRESS (ksi)	NEG. LL+I MOM. (k-ft)	STRESS (ksi)	TOTAL STRESS --AND-- STRESS RANGE
1	LT	11.50	156.	-5.2	103.	-1.0	450.	-1.2	0.	0.0	-7.38
											1.17
	LB			5.2		2.6		10.1		0.0	17.96
											10.14
	RT			-4.9		-1.0		-1.3		0.0	-7.21
											1.35
	RB			3.9		2.0		7.7		0.0	13.59
											7.72
1	LT	39.50	156.	-4.9	103.	-1.0	450.	-1.3	0.	0.0	-7.20
											1.35
	LB			3.9		2.0		7.7		0.0	13.58
											7.72
	RT			-5.2		-1.0		-1.2		0.0	-7.37
											1.17
	RB			5.2		2.6		10.1		0.0	17.95
											10.14

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange

RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	28.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.5000
3	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 51.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [3]= GIRDER D. (in)	[2]/(12*[1]) [3]/(12*[1])	<=>	LIMITATION 1/25 1/30	STATUS
1	11.50	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK
				[3]=	32.90	0.05376	>	0.03300	OK
2	28.00	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK
				[3]=	32.90	0.05376	>	0.03300	OK
3	11.50	6.75	32.90	[2]=	41.18	0.06728	>	0.04000	OK
				[3]=	32.90	0.05376	>	0.03300	OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	1	2.83	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	2	5.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	3	8.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	4	11.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	5	14.17	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	6	17.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	7	19.83	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	8	22.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	9	25.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	10	28.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	11	31.17	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	12	34.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	13	36.83	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	14	39.67	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	15	42.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	16	45.33	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	17	48.17	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0
1	18	51.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46		0

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)
[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0
[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)
[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw
[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	2.83	0	1	31.42	0.550	57.13	168.97	2
1	2	5.67	0	1	31.42	0.550	57.13	168.97	2
1	3	8.50	0	1	31.42	0.550	57.13	168.97	2
1	4	11.33	0	1	31.42	0.550	57.13	168.97	2
1	5	14.17	0	1	31.42	0.550	64.85	168.97	2
1	6	17.00	0	1	31.42	0.550	64.85	168.97	2
1	7	19.83	0	1	31.42	0.550	64.85	168.97	2
1	8	22.67	0	1	31.42	0.550	64.85	168.97	2
1	9	25.50	0	1	31.42	0.550	64.85	168.97	2
1	10	28.33	0	1	31.42	0.550	64.85	168.97	2
1	11	31.17	0	1	31.42	0.550	64.85	168.97	2
1	12	34.00	0	1	31.42	0.550	64.85	168.97	2

1	13	36.83	0	1	31.42	0.550	64.85	168.97	2
1	14	39.67	0	1	31.42	0.550	57.13	168.97	2
1	15	42.50	0	1	31.42	0.550	57.13	168.97	2
1	16	45.33	0	1	31.42	0.550	57.13	168.97	2
1	17	48.17	0	1	31.42	0.550	57.13	168.97	2
1	18	51.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	1	2.83	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	2	5.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	3	8.50	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	4	11.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	5	14.17	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	6	17.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	7	19.83	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	8	22.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	9	25.50	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	10	28.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	11	31.17	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	12	34.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	13	36.83	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	14	39.67	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	15	42.50	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	16	45.33	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	17	48.17	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	18	51.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0

NOTE: [1] = 12. (Eq. 6.10.2.2-1)
[2] = D/6 (Eq. 6.10.2.2-2)

[3] = 1.1tw (Eq. 6.10.2.2-3)

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. LEFT FROM ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	7.5	0.000	5.612	3.034	0.457	1.667
			0.000	0.000	0.000	0.000	1.000
2	7.5	29.2	5.612	10.238	9.631	9.025	1.058
			0.000	0.000	0.000	0.000	1.000
3	29.2	51.0	0.000	10.238	7.499	4.761	1.000
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case
f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

If the flange transition is beyond 20% X unbraced length from the smaller end and the ratio of the smaller and larger lateral moment of inertia of the flange is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	CAT	GOV CAT
1	0	0.00	1	7.77	11.26	19.84	0	7.50	7.09	26.62	2	2
1	1	2.83	1	7.77	11.26	19.84	0	7.50	7.09	26.62	2	2
1	2	5.67	1	7.77	11.26	19.84	0	7.50	7.09	26.62	2	2
1	3	8.50	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	4	11.33	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	5	14.17	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	6	17.00	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	7	19.83	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	8	22.67	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	9	25.50	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	10	28.33	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	11	31.17	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	12	34.00	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	13	36.83	1	7.77	11.26	19.84	0	21.75	6.97	26.17	2	2
1	14	39.67	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	15	42.50	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	16	45.33	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	17	48.17	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1	18	51.00	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)

```
       Lb = unbraced length
       Lp = limiting unbraced length to achieve the nominal flexural
            of RbRhFyc under uniform bending
          = 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
       Lr = limiting unbraced length to achieve the onset of
            nominal yielding in either flange under uniform
            bending with consideration of comp. flange
            residual stress effects
          = pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

       In negative moment region, the fist line is for non-composite sections
       and the second line is for composite sections

       Flange Category
        0 = compact section
        2 = non-compact section
        3 = slender section

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 TABLE  1.2.22.7B=FLB AND LTB RESISTANCE
                  **********************

 SP IN D FROM  Co                        FLB       LTB       GOV
 NO NO L SUPT  mp  Rh    Rb    Cb        Fnc       Fnc       Fnc
         (ft)                           (Ksi)     (Ksi)     (Ksi)
-----------------------------------------------------------------

  1  0   0.00  1 1.000 1.000  1.67     33.00     33.00     33.00
  1  1   2.83  1 1.000 1.000  1.67     33.00     33.00     33.00
  1  2   5.67  1 1.000 1.000  1.67     33.00     33.00     33.00
  1  3   8.50  1 1.000 1.000  1.06     33.00     27.04     27.04
  1  4  11.33  1 1.000 1.000  1.06     33.00     27.04     27.04
  1  5  14.17  1 1.000 1.000  1.06     33.00     26.84     26.84
  1  6  17.00  1 1.000 1.000  1.06     33.00     26.84     26.84
  1  7  19.83  1 1.000 1.000  1.06     33.00     26.84     26.84
  1  8  22.67  1 1.000 1.000  1.06     33.00     26.84     26.84
  1  9  25.50  1 1.000 1.000  1.06     33.00     26.84     26.84
  1 10  28.33  1 1.000 1.000  1.06     33.00     26.84     26.84
  1 11  31.17  1 1.000 1.000  1.00     33.00     25.38     25.38
  1 12  34.00  1 1.000 1.000  1.00     33.00     25.38     25.38
  1 13  36.83  1 1.000 1.000  1.00     33.00     25.38     25.38
  1 14  39.67  1 1.000 1.000  1.00     33.00     25.57     25.57
  1 15  42.50  1 1.000 1.000  1.00     33.00     25.57     25.57
  1 16  45.33  1 1.000 1.000  1.00     33.00     25.57     25.57
  1 17  48.17  1 1.000 1.000  1.00     33.00     25.57     25.57
  1 18  51.00  1 1.000 1.000  1.00     33.00     25.57     25.57


-----------------------------------------------------------------

 Note: In the positive moment region, the result is for DL case
       In the negative moment region, the 1st line is for DL case
       and the 2nd line is for LL case
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 TABLE  1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
                  ******************************

 SP IN D FROM  Co
 NO NO L SUPT  mp     Dp       0.42Dt
         (ft)        (in)       (in)
-------------------------------------

  1  0   0.00  1       6.34      17.29
  1  1   2.83  1       6.34      17.29
  1  2   5.67  1       6.34      17.29
  1  3   8.50  1       6.34      17.29
  1  4  11.33  1       6.34      17.29
```

1	5	14.17	1	8.40	17.50
1	6	17.00	1	8.40	17.50
1	7	19.83	1	8.40	17.50
1	8	22.67	1	8.40	17.50
1	9	25.50	1	8.40	17.50
1	10	28.33	1	8.40	17.50
1	11	31.17	1	8.40	17.50
1	12	34.00	1	8.40	17.50
1	13	36.83	1	8.40	17.50
1	14	39.67	1	6.34	17.29
1	15	42.50	1	6.34	17.29
1	16	45.33	1	6.34	17.29
1	17	48.17	1	6.34	17.29
1	18	51.00	1	6.34	17.29

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	S T R E N G T H C A T E G O R Y, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	2.83	1	0	2	0		2	0
1	2	5.67	1	0	2	0		2	0
1	3	8.50	1	0	2	0		2	0
1	4	11.33	1	0	2	0		2	0
1	5	14.17	1	0	2	0		2	0
1	6	17.00	1	0	2	0		2	0
1	7	19.83	1	0	2	0		2	0
1	8	22.67	1	0	2	0		2	0
1	9	25.50	1	0	2	0		2	0
1	10	28.33	1	0	2	0		2	0
1	11	31.17	1	0	2	0		2	0
1	12	34.00	1	0	2	0		2	0
1	13	36.83	1	0	2	0		2	0
1	14	39.67	1	0	2	0		2	0
1	15	42.50	1	0	2	0		2	0
1	16	45.33	1	0	2	0		2	0
1	17	48.17	1	0	2	0		2	0
1	18	51.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		2.83	0.0	19.8	2.4	33.0	2.4	33.0	2.4	33.0	0
				0.0	-	2.4	-	2.4	33.0	-	-	0
1	2		5.67	0.0	19.8	4.4	33.0	4.4	33.0	4.4	33.0	0
				0.0	-	4.4	-	4.4	33.0	-	-	0
1	3		8.50	0.0	19.8	6.2	33.0	6.2	33.0	6.2	27.0	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	4		11.33	0.0	19.8	7.8	33.0	7.8	33.0	7.8	27.0	0
				0.0	-	7.8	-	7.8	33.0	-	-	0
1	5		14.17	0.0	19.8	8.4	33.0	8.4	33.0	8.4	26.8	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	6		17.00	0.0	19.8	9.3	33.0	9.3	33.0	9.3	26.8	0
				0.0	-	7.5	-	7.5	33.0	-	-	0
1	7		19.83	0.0	19.8	10.0	33.0	10.0	33.0	10.0	26.8	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	8		22.67	0.0	19.8	10.4	33.0	10.4	33.0	10.4	26.8	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	9		25.50	0.0	19.8	10.5	33.0	10.5	33.0	10.5	26.8	0
				0.0	-	8.4	-	8.4	33.0	-	-	0
1	10		28.33	0.0	19.8	10.4	33.0	10.4	33.0	10.4	26.8	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	11		31.17	0.0	19.8	10.0	33.0	10.0	33.0	10.0	25.4	0
				0.0	-	8.0	-	8.0	33.0	-	-	0
1	12		34.00	0.0	19.8	9.3	33.0	9.3	33.0	9.3	25.4	0
				0.0	-	7.5	-	7.5	33.0	-	-	0
1	13		36.83	0.0	19.8	8.4	33.0	8.4	33.0	8.4	25.4	0
				0.0	-	6.7	-	6.7	33.0	-	-	0
1	14		39.67	0.0	19.8	7.7	33.0	7.7	33.0	7.7	25.6	0
				0.0	-	7.7	-	7.7	33.0	-	-	0
1	15		42.50	0.0	19.8	6.2	33.0	6.2	33.0	6.2	25.6	0
				0.0	-	6.2	-	6.2	33.0	-	-	0
1	16		45.33	0.0	19.8	4.4	33.0	4.4	33.0	4.4	25.6	0
				0.0	-	4.4	-	4.4	33.0	-	-	0
1	17		48.17	0.0	19.8	2.3	33.0	2.3	33.0	2.3	25.6	0
				0.0	-	2.3	-	2.3	33.0	-	-	0
1	18		51.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	25.6	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on factored loads, and shall be taken as positive in

sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	2.83	0.000	0.072	0.072	0.072	0.072	2
					0.072		0.072	3
1	2	5.67	0.000	0.135	0.135	0.135	0.135	2
					0.135		0.135	3
1	3	8.50	0.000	0.189	0.189	0.231	0.231	4
					0.189		0.189	3
1	4	11.33	0.000	0.235	0.235	0.287	0.287	4
					0.235		0.235	3
1	5	14.17	0.000	0.255	0.255	0.313	0.313	4
					0.204		0.204	3
1	6	17.00	0.000	0.282	0.282	0.347	0.347	4
					0.226		0.226	3
1	7	19.83	0.000	0.302	0.302	0.371	0.371	4
					0.242		0.242	3
1	8	22.67	0.000	0.314	0.314	0.386	0.386	4
					0.251		0.251	3
1	9	25.50	0.000	0.318	0.318	0.391	0.391	4
					0.254		0.254	3
1	10	28.33	0.000	0.314	0.314	0.386	0.386	4
					0.251		0.251	3
1	11	31.17	0.000	0.302	0.302	0.393	0.393	4
					0.242		0.242	3
1	12	34.00	0.000	0.282	0.282	0.367	0.367	4
					0.226		0.226	3
1	13	36.83	0.000	0.255	0.255	0.331	0.331	4
					0.204		0.204	3
1	14	39.67	0.000	0.235	0.235	0.303	0.303	4
					0.235		0.235	3
1	15	42.50	0.000	0.189	0.189	0.243	0.243	4
					0.189		0.189	3
1	16	45.33	0.000	0.134	0.134	0.173	0.173	4
					0.134		0.134	3
1	17	48.17	0.000	0.071	0.071	0.092	0.092	4
					0.071		0.071	3
1	18	51.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1956.3	-	-	0.0	19.8	-	-	0--0
1	1		2.83	0	350.9	1956.3	-	-	0.0	19.8	-	-	0--0
1	2		5.67	0	654.8	1956.3	-	-	0.0	19.8	-	-	0--0
1	3		8.50	0	911.5	1956.3	-	-	0.0	19.8	-	-	0--0
1	4		11.33	0	1118.3	1956.3	-	-	0.0	19.8	-	-	0--0
1	5		14.17	0	1280.7	2326.4	-	-	0.0	19.8	-	-	0--0
1	6		17.00	0	1406.8	2326.4	-	-	0.0	19.8	-	-	0--0
1	7		19.83	0	1492.5	2326.4	-	-	0.0	19.8	-	-	0--0
1	8		22.67	0	1526.7	2326.4	-	-	0.0	19.8	-	-	0--0
1	9		25.50	0	1536.6	2326.4	-	-	0.0	19.8	-	-	0--0
1	10		28.33	0	1527.0	2326.4	-	-	0.0	19.8	-	-	0--0
1	11		31.17	0	1492.8	2326.4	-	-	0.0	19.8	-	-	0--0
1	12		34.00	0	1406.8	2326.4	-	-	0.0	19.8	-	-	0--0
1	13		36.83	0	1280.5	2326.4	-	-	0.0	19.8	-	-	0--0
1	14		39.67	0	1117.9	1956.3	-	-	0.0	19.8	-	-	0--0
1	15		42.50	0	910.8	1956.3	-	-	0.0	19.8	-	-	0--0
1	16		45.33	0	654.2	1956.3	-	-	0.0	19.8	-	-	0--0
1	17		48.17	0	350.6	1956.3	-	-	0.0	19.8	-	-	0--0
1	18		51.00	0	0.0	1956.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP	IN	D	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		2.83	0	0.179				0.179	1

1	2	5.67	0	0.335	0.335	1
1	3	8.50	0	0.466	0.466	1
1	4	11.33	0	0.572	0.572	1
1	5	14.17	0	0.550	0.550	1
1	6	17.00	0	0.605	0.605	1
1	7	19.83	0	0.642	0.642	1
1	8	22.67	0	0.656	0.656	1
1	9	25.50	0	0.661	0.661	1
1	10	28.33	0	0.656	0.656	1
1	11	31.17	0	0.642	0.642	1
1	12	34.00	0	0.605	0.605	1
1	13	36.83	0	0.550	0.550	1
1	14	39.67	0	0.571	0.571	1
1	15	42.50	0	0.466	0.466	1
1	16	45.33	0	0.334	0.334	1
1	17	48.17	0	0.179	0.179	1
1	18	51.00	0	0.000	0.000	1

```
NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
      [2] = (PHI)f * Fnc for comp. flange of composite
            sections in positive flexure (Eq. 6.10.7.2.1-1)
          = (PHI)f * Rh * Fyt for tension flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
      [3] = 0.6*Fyt for composite sections in positive flexure
            or tension flange for composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
          = 0.6*Fyc for comp. flange of composite sections in
            negative flexure and non-composite sections (Eq. 6.10.1.6-1)
      [4] = (PHI)f * Fnt for non-compact tension flange
            of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
          = (PHI)f * Fnc for comp. flange of composite sections
            in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

      The governing number is listed as below.
        1 = (Mu+1/3flSxt) / [1]
        2 = fbu / [2]
        3 = fl / [3]
        4 = (fbu + 1/3fl) / [4]
```

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	2.83	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	5.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	8.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	11.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	14.17	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	17.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	19.83	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	22.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	25.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	28.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	31.17	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	34.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	36.83	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	39.67	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	42.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	45.33	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	17	48.17	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	18	51.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

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     NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
                 = 5 for unstiffened beams and girders
                 = 5 for stiffened girders when (d0/D) > 3, or
                                                (d0/D) > (260/(D/tw))**2

           [2] = 1.12 * sqrt(Ek / Fyw)

           [3] = 1.40 * sqrt(Ek / Fyw)

           [4] C = ratio of nominal shear resistance and plastic shear force, Vp

              For D/tw < [2], C = 1.0

              For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                          ..... AASHTO LRFD Eq.6.10.9.3.2-6

              For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                          ..... AASHTO LRFD Eq.6.10.9.3.2-7


           ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
              stiffened
              if (d0/D) < 3 without long. stiffener, or
              if (d0/D) > 3 with both transv. and long. stiffeners.
              For handling requirement, (d0/D) > (260/(D/tw))**2

                   Vu = CVp

              where D = clear, unsupported distance between flange components

                   d0 = distance between transverse stiffeners

           ** This Article (6.10.9.2) indicates that a designer cannot
              count on post-buckling shear resistance from tension-
              field action for an unstiffened girder.
```

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	STRENGTH CATEGORY; SEE NOTE CATEGORY	OVERALL WEB
1	0	0.00	0	129.3	<	330.8	0	0
1	1	2.83	0	118.3	<	330.8	0	0
1	2	5.67	0	107.3	<	330.8	0	0
1	3	8.50	0	96.3	<	330.8	0	0
1	4	11.33	0	85.5	<	330.8	0	0
1	5	14.17	0	74.8	<	330.8	0	0
1	6	17.00	0	64.2	<	330.8	0	0
1	7	19.83	0	53.8	<	330.8	0	0
1	8	22.67	0	43.6	<	330.8	0	0
1	9	25.50	0	33.7	<	330.8	0	0
1	10	28.33	0	43.5	<	330.8	0	0
1	11	31.17	0	53.9	<	330.8	0	0
1	12	34.00	0	64.3	<	330.8	0	0
1	13	36.83	0	74.9	<	330.8	0	0
1	14	39.67	0	85.6	<	330.8	0	0
1	15	42.50	0	96.3	<	330.8	0	0
1	16	45.33	0	107.2	<	330.8	0	0
1	17	48.17	0	118.2	<	330.8	0	0

```
 1 18  51.00       0            129.2   <      330.8       0          0


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 NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

          For compact section  (composite sections in positive flexure)

            w/o long. stiff.

            D/tw <= 150 (Eq. 6.10.2.1.1-1)

            w / long. stiff.

            D/tw <= 300 (Eq. 6.10.2.1.2-1)

            2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ......... AASHTO Eq.6.10.6.2.2-1

          For compact or non-compact sections specified in Art. 6.10.8

            (composite sections in negative flexure and
             non-composite sections)

            2Dc/tw .LE. 5.7*SQRT(E/Fyc) ........... AASHTO Eq.6.10.6.2.3-1


       [2]: SHEAR CAPACITY ... web crippling control

          Vn = CVp .......... AASHTO Eq.6.10.9.2-1

          Vn = VpRc1 ......... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

          Vn = VpRc2 ......... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

          Vp = 0.58*Fy*D*tw ...... AASHTO Eq.6.10.9.3.2-2

        * For the detailed description of shear capacity,
          please refer to AASHTO Art.6.10.9.2 and .3

        * STRENGTH CATEGORY; 0 = compact section
                             2 = braced non-compact section
                             3 = slender section
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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
                ***************************

          YIELD LFD/LRFD UNSTIFFENED  REQUIREMENT OF    MAX. ALLOWABLE
SP IN D FROM STRESS MAXIMUM    SHEAR   ----------------- TRANS. STIFFENERS
NO NO L SUPT   Fy    SHEAR   CAPACITY  TRANS. STIFFENERS     SPACING
       (ft)  (ksi)    (k)      (k)        1=YES , 0=NO       (ft-in)
-----------------------------------------------------------------------

 1  0   0.00   33.   129.31   330.76            0
 1  1   2.83   33.   118.26   330.76            0
 1  2   5.67   33.   107.30   330.76            0
 1  3   8.50   33.    96.27   330.76            0
 1  4  11.33   33.    85.51   330.76            0
 1  5  14.17   33.    74.85   330.76            0
 1  6  17.00   33.    64.23   330.76            0
 1  7  19.83   33.    53.79   330.76            0
                                                Page 37
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1	8	22.67	33.	43.61	330.76	0
1	9	25.50	33.	33.68	330.76	0
1	10	28.33	33.	43.51	330.76	0
1	11	31.17	33.	53.87	330.76	0
1	12	34.00	33.	64.31	330.76	0
1	13	36.83	33.	74.93	330.76	0
1	14	39.67	33.	85.59	330.76	0
1	15	42.50	33.	96.35	330.76	0
1	16	45.33	33.	107.20	330.76	0
1	17	48.17	33.	118.16	330.76	0
1	18	51.00	33.	129.20	330.76	0

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D NO	L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	BOT. FLANGE ff+fl/2 (ksi)	[2]	COMP. FLG fc (ksi)	fcrw (ksi)	FLAG [3]	TBC
1	0	0.00	0.0	-		31.4	0.0	31.4	-	-	00-
1	1	2.83	2.4	-		31.4	6.5	31.4	-	-	00-
1	2	5.67	4.4	-		31.4	12.2	31.4	-	-	00-
1	3	8.50	6.2	-		31.4	17.0	31.4	-	-	00-
1	4	11.33	7.7	-		31.4	20.8	31.4	-	-	00-
1	5	14.17	8.7	-		31.4	18.1	31.4	-	-	00-
1	6	17.00	9.6	-		31.4	19.9	31.4	-	-	00-
1	7	19.83	10.3	-		31.4	21.1	31.4	-	-	00-
1	8	22.67	10.6	-		31.4	21.6	31.4	-	-	00-
1	9	25.50	10.7	-		31.4	21.8	31.4	-	-	00-
1	10	28.33	10.6	-		31.4	21.6	31.4	-	-	00-
1	11	31.17	10.3	-		31.4	21.1	31.4	-	-	00-
1	12	34.00	9.6	-		31.4	19.9	31.4	-	-	00-
1	13	36.83	8.7	-		31.4	18.1	31.4	-	-	00-
1	14	39.67	7.7	-		31.4	20.8	31.4	-	-	00-
1	15	42.50	6.2	-		31.4	16.9	31.4	-	-	00-
1	16	45.33	4.4	-		31.4	12.1	31.4	-	-	00-
1	17	48.17	2.3	-		31.4	6.5	31.4	-	-	00-
1	18	51.00	0.0	-		31.4	0.0	31.4	-	-	00-

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NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

Flag check - 0 = OK; 1 = NG
T = Top Flange; B = Bottom Flange; C = Comp. Flange

The values of fl shall be determined based on factored loads, and shall be taken as positive in

sign in all resistance equations (Art. 6.10.1.6)

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TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.83	TR	-0.2	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	2	5.67	TR	-0.4	A B B^C^C D E E^	TR	3.1	A B B^C^C D E
1	3	8.50	TR	-0.5	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	4	11.33	TR	-0.6	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1		11.50	SCG					
1	5	14.17	TR	-0.7	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1	6	17.00	TR	-0.8	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	7	19.83	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	8	22.67	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9	25.50	TR	-0.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	10	28.33	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	11	31.17	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	12	34.00	TR	-0.8	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	13	36.83	TR	-0.7	A B B^C^C D E E^	TR	4.3	A B B^C^C D E
1		39.50	SCG					
1	14	39.67	TR	-0.6	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	15	42.50	TR	-0.5	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	16	45.33	TR	-0.4	A B B^C^C D E E^	TR	3.1	A B B^C^C D E
1	17	48.17	TR	-0.2	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	18	51.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

TOP OF TOP FLANGE BOTTOM OF BOTTOM FLANGE

SP NO	IN NO	D FROM L SUPT (ft)	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	2.83	TR	-0.1	A B B^C^C D E E^	TR	0.8	A B B^C^C D E E^
1	2	5.67	TR	-0.2	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	3	8.50	TR	-0.2	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	4	11.33	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1		11.50	SCG					
1	5	14.17	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	6	17.00	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	7	19.83	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	8	22.67	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	9	25.50	TR	-0.4	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	10	28.33	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	11	31.17	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	12	34.00	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	13	36.83	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1		39.50	SCG					
1	14	39.67	TR	-0.3	A B B^C^C D E E^	TR	2.5	A B B^C^C D E
1	15	42.50	TR	-0.2	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^
1	16	45.33	TR	-0.2	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	17	48.17	TR	-0.1	A B B^C^C D E E^	TR	0.8	A B B^C^C D E E^
1	18	51.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.75 load factor.
Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	AASHTO E6.10.10.1.2 Vsr [3]	AASHTO LRFD Eqn. 6.10.10.2-1 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	31.1	.29013E-01	0.9	TR 3.09	3	10.28
1	1	2.83	29.6	.29013E-01	0.9	TR 3.09	3	10.81
1	2	5.67	28.1	.29013E-01	0.8	TR 3.09	3	11.36
1	3	8.50	26.8	.29013E-01	0.8	TR 3.09	3	11.93
1	4	11.33	25.6	.29013E-01	0.7	TR 3.09	3	12.50
1	5	14.17	24.5	.27358E-01	0.7	TR 3.09	3	13.86
1	6	17.00	23.2	.27358E-01	0.6	TR 3.09	3	14.64
1	7	19.83	22.5	.27358E-01	0.6	TR 3.09	3	15.10
1	8	22.67	23.5	.27358E-01	0.6	TR 3.09	3	14.45
1	9	25.50	23.9	.27358E-01	0.7	TR 3.09	3	14.17
1	10	28.33	23.5	.27358E-01	0.6	TR 3.09	3	14.45
1	11	31.17	22.5	.27358E-01	0.6	TR 3.09	3	15.10
1	12	34.00	23.2	.27358E-01	0.6	TR 3.09	3	14.64
1	13	36.83	24.5	.27358E-01	0.7	TR 3.09	3	13.86
1	14	39.67	25.6	.29013E-01	0.7	TR 3.09	3	12.50
1	15	42.50	26.8	.29013E-01	0.8	TR 3.09	3	11.93
1	16	45.33	28.1	.29013E-01	0.8	TR 3.09	3	11.36

```
 1 17  48.17  29.6 .29013E-01    0.9      TR    3.09        3          10.81
 1 18  51.00  31.1 .29013E-01    0.9      TR    3.09        3          10.28


 ---------------------------------------------------------------------------
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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2

        [2] Q/I : Q= statical moment about the neutral axis of the composite
                     section of the transformed compressive concrete area or
                     the area of reinforcement embedded in the concrete for
                     negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                  I= moment of inertia of the transformed composite girder in
                     positive moment regions or the moment of inertia provided
                     by the steel beam including or excluding the area of
                     reinforcement embedded in the concrete in negative moment
                     regions, in inches to the fourth power.

        [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                  of the slab and girder at the point in the span under
                  consideration.
                = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
            Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
            Ffat = radial fatigue shear range per unit length
                 = Ffat2 = Frc/w for skews exceeding 20 degrees
            Frc = 25.0 kips for both exterior and interior girders
            w = the effective length of the deck
              = 48 inches except at end supports where w is taken
                as 24 inches

        [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                 connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

        [5] No. of shear connectors per transverse section. This value is from
            input data type 12032.

        [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

            ** The connector spacing shown here is calculated based upon
               AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
           *** If ADTT Single Lane is less than 960, use Fatigue II Table.
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 TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
                   *******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-2     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ---------------------------------------------------------------------------

  1  0   0.00  15.6 .29013E-01    0.5      TR    1.36        3           9.06
  1  1   2.83  14.8 .29013E-01    0.4      TR    1.36        3           9.53
  1  2   5.67  14.1 .29013E-01    0.4      TR    1.36        3          10.02
  1  3   8.50  13.4 .29013E-01    0.4      TR    1.36        3          10.52
  1  4  11.33  12.8 .29013E-01    0.4      TR    1.36        3          11.02
  1  5  14.17  12.2 .27358E-01    0.3      TR    1.36        3          12.22
  1  6  17.00  11.6 .27358E-01    0.3      TR    1.36        3          12.91
  1  7  19.83  11.2 .27358E-01    0.3      TR    1.36        3          13.31
```

1	8	22.67	11.7	.27358E-01	0.3	TR	1.36	3	12.74
1	9	25.50	12.0	.27358E-01	0.3	TR	1.36	3	12.49
1	10	28.33	11.7	.27358E-01	0.3	TR	1.36	3	12.74
1	11	31.17	11.2	.27358E-01	0.3	TR	1.36	3	13.31
1	12	34.00	11.6	.27358E-01	0.3	TR	1.36	3	12.91
1	13	36.83	12.2	.27358E-01	0.3	TR	1.36	3	12.22
1	14	39.67	12.8	.29013E-01	0.4	TR	1.36	3	11.02
1	15	42.50	13.4	.29013E-01	0.4	TR	1.36	3	10.52
1	16	45.33	14.1	.29013E-01	0.4	TR	1.36	3	10.02
1	17	48.17	14.8	.29013E-01	0.4	TR	1.36	3	9.53
1	18	51.00	15.6	.29013E-01	0.5	TR	1.36	3	9.06

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NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); AASHTO LRFD Eqn.6.10.10.1.2-2

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

* Default ALPHA value based on 7/8" diameter and input road type

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2 where ADTT Single Lane= 1140 & No. of Cycles= 31207500

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

```
                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
       (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
-------------------------------------------------------------------------

 1  9  25.50   1 1296.9  1204.9           93         >  63
 1 18  51.00   1 1296.9  1204.9           93         >  63


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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
- N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
- N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
- P = total nominal horizontal shear force
- Qr = factored shear resistance of one shear connector = (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
- Qn = nominal shear resistance
- (Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
- f'c = specified 28-day compressive strength of the concrete
- bs = effective width of the slab
- bfc = width of compression flange
- bft = width of tension flange
- ts = slab thickness
- Fyw = specified minimum yield strength of the web
- Fyt = specified minimum yield strength of the tension flange
- Fyc = specified minimum yield strength of the compression flange
- D = web depth
- tft = thickness of tension flange
- tfc = thickness of compression flange
- tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
- bs = effective width of the concrete deck
- ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.

If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

1

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	25.500	10.000
1	25.500	51.000	10.000

1

```
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TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	T C R
1	0.0	51.0	C

NOTE: T: TENSION AREA, C: COMPRESSION AREA,
R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
STRESSES

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
****

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
OK	1.2.22.10=CONSTRUCBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	1956.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	2.83	1956.3	47.0	31.0	141.9	0.0	9.68	HL-93	7.47	0
1	2	5.67	1956.3	88.6	58.4	263.8	0.0	4.95	HL-93	3.82	0
1	3	8.50	1956.3	124.5	82.2	365.7	0.0	3.41	HL-93	2.63	0
1	4	11.33	1956.3	154.7	102.3	446.1	0.0	2.69	HL-93	2.07	0
1	5	14.17	2326.4	179.4	118.7	508.0	0.0	2.82	HL-93	2.18	0
1	6	17.00	2326.4	198.7	131.5	556.0	0.0	2.52	HL-93	1.95	0
1	7	19.83	2326.4	212.5	140.6	587.8	0.0	2.35	HL-93	1.81	0
1	8	22.67	2326.4	220.8	146.1	596.9	0.0	2.29	HL-93	1.77	0
1	9	25.50	2326.4	223.7	147.9	599.1	0.0	2.27	HL-93	1.75	0
1	10	28.33	2326.4	221.0	146.1	596.9	0.0	2.29	HL-93	1.77	0
1	11	31.17	2326.4	212.7	140.6	587.8	0.0	2.35	HL-93	1.81	0
1	12	34.00	2326.4	198.7	131.5	556.0	0.0	2.52	HL-93	1.95	0
1	13	36.83	2326.4	179.3	118.7	508.0	0.0	2.82	HL-93	2.18	0
1	14	39.67	1956.3	154.3	102.3	446.1	0.0	2.69	HL-93	2.07	0
1	15	42.50	1956.3	124.0	82.2	365.7	0.0	3.41	HL-93	2.63	0
1	16	45.33	1956.3	88.1	58.4	263.8	0.0	4.95	HL-93	3.82	0
1	17	48.17	1956.3	46.8	31.0	141.9	0.0	9.68	HL-93	7.47	0
1	18	51.00	1956.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT		UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT		MAX. NEG. --- STEEL--- TOP;ksi;BOT		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	2.83	31.4	-1.87	2.35	-0.37	3.20	0.00	0.00	9.06	HL-93	6.97
1	2	5.67	31.4	-3.52	4.43	-0.69	5.95	0.00	0.00	4.52	HL-93	3.48
1	3	8.50	31.4	-4.95	6.23	-0.95	8.25	0.00	0.00	3.05	HL-93	2.34
1	4	11.33	31.4	-6.15	7.74	-1.17	10.06	0.00	0.00	2.35	HL-93	1.80
1	5	14.17	31.4	-6.74	6.75	-1.52	8.72	0.00	0.00	2.82	HL-93	2.17
1	6	17.00	31.4	-7.46	7.47	-1.66	9.54	0.00	0.00	2.50	HL-93	1.92
1	7	19.83	31.4	-7.98	7.99	-1.76	10.09	0.00	0.00	2.32	HL-93	1.78
1	8	22.67	31.4	-8.29	8.30	-1.79	10.25	0.00	0.00	2.25	HL-93	1.73
1	9	25.50	31.4	-8.40	8.41	-1.79	10.28	0.00	0.00	2.23	HL-93	1.72
1	10	28.33	31.4	-8.30	8.31	-1.79	10.25	0.00	0.00	2.25	HL-93	1.73
1	11	31.17	31.4	-7.98	8.00	-1.76	10.09	0.00	0.00	2.31	HL-93	1.78
1	12	34.00	31.4	-7.46	7.47	-1.66	9.54	0.00	0.00	2.50	HL-93	1.92
1	13	36.83	31.4	-6.73	6.74	-1.52	8.72	0.00	0.00	2.82	HL-93	2.17
1	14	39.67	31.4	-6.14	7.73	-1.17	10.06	0.00	0.00	2.35	HL-93	1.81
1	15	42.50	31.4	-4.93	6.21	-0.95	8.25	0.00	0.00	3.05	HL-93	2.34
1	16	45.33	31.4	-3.50	4.42	-0.69	5.95	0.00	0.00	4.53	HL-93	3.48
1	17	48.17	31.4	-1.86	2.35	-0.37	3.20	0.00	0.00	9.06	HL-93	6.97
1	18	51.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS, THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING
UNIVERSITY OF MARYLAND

MERLIN V 10.4
COMPOSITE
LRF -- 2010
RATING

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE

1	51.00	25.50	STRENGTH L.S.	2.27	1.75	HL-93
1	51.00	25.50	SERVICEBILITY	2.23	1.72	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.231
Inventory= 1.716

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	51.00	25.50	STRENGTH L.S.	3.22	570.62
1	51.00	25.50	SERVICEBILITY	3.00	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 3.003

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	17.6	11.6	52.0	0.0	4.17	HL-93	3.21
1	1	2.83	330.8	15.6	10.3	48.1	-1.3	4.57	HL-93	3.52
1	2	5.67	330.8	13.7	9.0	44.3	-2.7	5.04	HL-93	3.89
1	3	8.50	330.8	11.6	7.7	40.5	-4.9	5.59	HL-93	4.31
1	4	11.33	330.8	9.7	6.4	36.7	-7.1	6.24	HL-93	4.81
1	5	14.17	330.8	7.8	5.2	33.1	-9.4	7.03	HL-93	5.42
1	6	17.00	330.8	5.8	3.9	29.4	-11.7	8.01	HL-93	6.18
1	7	19.83	330.8	3.9	2.6	25.9	-14.1	9.23	HL-93	7.12
1	8	22.67	330.8	2.0	1.3	22.5	-16.6	9.99	HL-93	8.30
1	9	25.50	330.8	0.0	0.0	19.2	-19.2	9.99	HL-93	9.83
1	10	28.33	330.8	-1.9	-1.3	16.6	-22.5	9.99	HL-93	8.30
1	11	31.17	330.8	-4.0	-2.6	14.1	-25.9	9.22	HL-93	7.11
1	12	34.00	330.8	-5.9	-3.9	11.7	-29.4	8.01	HL-93	6.18
1	13	36.83	330.8	-7.8	-5.2	9.4	-33.1	7.03	HL-93	5.42
1	14	39.67	330.8	-9.8	-6.4	7.1	-36.7	6.24	HL-93	4.81
1	15	42.50	330.8	-11.7	-7.7	4.9	-40.5	5.59	HL-93	4.31
1	16	45.33	330.8	-13.6	-9.0	2.7	-44.3	5.04	HL-93	3.89
1	17	48.17	330.8	-15.5	-10.3	1.3	-48.1	4.57	HL-93	3.53
1	18	51.00	330.8	-17.5	-11.6	0.0	-52.0	4.17	HL-93	3.22

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	0.00	STRENGTH L.S.	4.17	3.21	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.166
Inventory= 3.214

TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	51.00	0.00	STRENGTH L.S.	5.18	56.41

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
Legal Load Rating= 5.184

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TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
****

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FOR LL DEFL.	LL + I. FACTOR	DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	25.50	3	0.425	-0.29	HL-93	2.65
		4	0.433	-0.29		2.60

NOTE: [1] "-" indicates downward deflection

[2] RF * (live load defl.) = span length/800, therefore

RF = L/(800 * LL Defl.), if = 99, No rating

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TABLE  0.0.1.1 PROJECT DATA
               ************


                  DESCRIPTION                                   DATE
                  -----------                                   ----
MaineDOT Bridge No. 5969 I-95 NB Span 4 Exterior Fatigue  8/19/2013


CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
---------------     ------    --------  -BY- -BY- ------------
CKE410A4  5969      Exterior  DGB/TDM   TDM/KSW


TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS
              **********************

 OUTPUT    SPAN    CONSTRUCTION        ANALYSIS CODE              PROGRAM
  LEVEL  INTERVAL  ------------  --------------------------        FLOW
  (0,1)  (MAX=20)  1= COMPOSITE   CODE   YEAR  UNIT DESIGN        CONTROL
                   2= NONCOMP.     ID          TYPE OPTION
 ------  --------  ------------ -------------------------- -------------

    1       16           1       AASHTO  2010     0     2            3
 ----------------------------------------------------------------------

 * output level    : 0 = basic output
                    1 = detailed output

 * span interval   : maximum = 20
                    default = 10

 * structural type : 1 = composite (default)
                    2 = noncomposite
                    3 = reinforced concrete
                    4 = prestressed concrete

 * type of unit    : 0 = English (default)
                    1 = Metric
                    2 = Metric input English output
                    3 = English input Metric output

 * design option   : 0 = WSD (default)
                    1 = LFD
                    2 = LRFD


 * program flow    : 0 = DL analysis only
                    1 = DL + LL analysis
                    2 = code check
                    3 = rating
                    4 = design
                    5 = design + code check
                    6 = design + recycle + code check
                    7 = DL stage only
                    8 = DL stage + LL

 * EFFECTIVE FLANGE WITH OPTION =    0
    0 - DEFAULT (2008)
    1 - "PRIOR TO 2007" WIDTH IS USED
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TABLE 0.0.3.1 STRUCTURAL DETAILS
              *****************


         BEAM     WIDTH            EDGE OF              COMPOSITE  STEEL
       POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
```

OF GIRDERS	1=INT. 2=EXT.	BARRIER (ft)	-------- (ft)	------- (ft)	DEPTH (in)	WIDTH. (in)	REGION (%)	FACTOR >= 1.0
6	2	40.00	1.90	1.58	2.27	19.50	0.00	1.00

* WIDTH BETWEEN CURBS OR BARRIERS (ROAD WIDTH) is used for the determination of traffic lanes

* The section properties with composite percentage at negative moment region is calculated by using the linear interpolation between the noncomposite section (N=Inf.) and 100% composite for the analysis at negative moment region.

* DETAIL FACTOR is used for the steel dead load only

TABLE 0.0.3.2 SPAN LENGTHS --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
51.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 0.0.3.4 BEAM SPACING --- in feet
****

SPAN-1	SPAN-2	SPAN-3	SPAN-4	SPAN-5	SPAN-6	SPAN-7	SPAN-8	SPAN-9	SPAN-10
7.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	NOMINAL ID.	WEIGHT (in)	WEB DEPTH (lb/ft)	WEB THICK. (in)	STANDARD SECTN PLATE GIRDER	ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in) TOP PLATE WIDTH	THICK.	BOT. PLATE WIDTH	THICK.	REINFORCED CONCRETE SECTION AREA (in**2)	Ix (in**4)
1	W	33	118			0.00	0.0000	0.00	0.0000		
2	W	33	118			0.00	0.00001	0.00	0.5000		

NOTE: [1] maximum allowable section number is 70

[2] For design option (flow 4, 5 or 6) this card need not be input

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TABLE 0.0.5.1 DEFINITION OF MEMBERS
****

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	11.50				33.	33.	33.
2	2	2	28.00				33.	33.	33.
3	1	1	11.50				33.	33.	33.

NOTE: [1] maximum allowable member number is 70.

[2] For design process (flow 4, 5 or 6) this card need not be input

[3] For hybrid section, yield stress defined here will override DATA TYPE 13012 for code checking

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)
****

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE 1,2,3 OR 4 ADTT	ADTTSL	SIDEWALK LIVE LOAD (k/ft)
HL-93	1	1	1341	1140	0.00

HL-93 VEHICLE X FACTOR OF 1.00

NOTE: * Road types 1, 2, 3 and 4 are used for fatigue check.

* Road type 1 is Rural Interstate. 2 is Urban Interstate. 3 is Other Rural. 4 is Other Urban. truck on the bridge distributed to the girders as designated in AASHTO LRFD Art.4.6.2.2 for one traffic lane loading.

For Fatigue, Fraction of Truck, p, is based on the Road Types.

Ref. AASHTO LRFD Table C3.6.1.4.2.1.

* Default road type = 1

* Sidewalk live loading is assumed taken by exterior girder only

* HL-93 is for both truck(s) + lane and tandem(s) + lane loading, as per 3.6.1.3.1.

* ADTT used in this calculation is 1341

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TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
****

IMPACT FACTOR TO OVERRIDE THE AASHTO FORMULA

OPTIONAL CALCULATION OF FACTOR LOADING TYPES

OPTIONAL DISTRIBUTION FACTORS

SP NO	IMP FCTR STR/SER (%)	DF M- ST/SE	DF M+ FA	DF M- FA	IMP F FA (%)	A D M G C NO=0 ;YES= 1	DF M+ ST/SE	LOADING TYPES A D M G C
1	33.00	0	0.00	0.37	0.00	15.00	1 1 1 1 1	0.57	2 2 2 2 2
1	0.00	0	0.00	0.39	0.00	0.00	0 0 0 0 0	0.55	3 3 3 3 3

NOTE ** : distribution factor - fraction of a wheel load for WSD/LFD
or fraction of an axle load for LRFD
0 = The special distribution factor defined is not applied to the indicated loading type.

1 = The special distribution factor defined is applied to the indicated loading type of calculation of all moment, shear and deflection.

2 = The special distribution factor defined is only applied to the loading type for calculating moment.

3 = The special distribution factor defined is only applied to the loading type for calculating shear.

4 = The special distribution factor defined is only applied to the loading type for calculating deflection.

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TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION POUR NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	FINAL (in)	POUR DAY	MODULAR RATIO N1=3n N1	N2=n N2	SLAB LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1	Deck+haunch	0.0	6.8	1	27.0	9.0	0.55	0.00	51.00

AASHTO Art.10.38.1.3 or LRFD Art.6.10.1.1.1b

The ratio of the moduli of elasticity of steel (29000 ksi) to those of normal weight concrete (W=145 pcf) of various design strength shall be as follows:

fc' = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days in pounds per square inch.

n = ratio of modulus of elasticity of steel to that of concrete.

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
****

LOAD IDENTIFICATION			UNIFROM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	TYPE	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	2	Diaphragm	0.000	0.00	0.00	0.14	7.50
2	2	Diaphragm	0.000	0.00	0.00	0.14	29.25
3	1	Railing	0.087	0.00	51.00	0.00	0.00
4	1	Curb	0.076	0.00	51.00	0.00	0.00
5	0	Pavement	0.292	0.00	51.00	0.00	0.00

NOTE: LOAD TYPE, 0 = (Default) Loads for noncomposite construction or Superimposed Loads for composite construction (In LRFD, it is for DW load)

1 = Superimposed Loads (In LRFD, it is for DC2 load)

2 = Noncomposite Loads,(In LRFD, it is for DC1 load) where N = modulus ratio = Es/Ec

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TABLE 0.0.12.3 SHEAR CONNECTOR AND SLAB REINFORCEMENT DATA
****

SHEAR CONNECTOR			Qn VALUE	Zr VALUE		SLAB REINFORCEMENT		CONCRETE	
NO. PER TRAN. SEC (in)	DIA.	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	AASHTO ART. 6.10.10.4 (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR BAR AREA YIELD PER FOOT STRESS OF SLAB Fy (ksi)	(in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
3	0.750	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00

NOTE: Qr = nominal resistance of the shear connector
= (phi)sc x Qn
... see AASHTO LRFD Eqs.6.10.10.4.1-1,4.3-1 or 4.3-2

Zr = shear fatigue resistance of an individual shear connector
... see AASHTO LRFD Eqs.6.10.10.2-1 & -2

fc' = unit ultimate compressive strength of concrete as determined by cylinder test at the age of 28 days
= 4 ksi (default)

fc = allowable compressive strength of concrete
= 0.85fc' (default)

* default number of shear connector per trans. section = 3

* If the shear connectors and slab reinforcements are supplied in the negative moment region, the contribution of rebar on the section properties in the negative moment region (for N = 3n & N = n) will be considered.

* If Zr left blank, Road type input in Data 06012 and 7/8"-diameter studs are assumed

* default rebar yield stress = 60 ksi

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TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
****

LOCATION DISTANCE FROM (ft)	LOCATION DISTANCE TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	7.50	33.0	33.0	7.50
7.50	29.25	33.0	33.0	21.75
29.25	51.00	33.0	33.0	21.75

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

[3] The spacing of lateral bracing is also assumed to be the diaphragm spacing which is used for the calculation of wind effect (code check only).

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TABLE 1.2.1.1=PROGRAM ASSUMPTIONS
****

NO.	DESCRIPTIONS
1	Small deflection theory
2	Material is elastic
3	Beam length is much greater than lateral dimensions
4	Torsional effects are neglected
5	Shear deformations are neglected
6	Two kinematic degree-of-freedom are assumed 'at each joint (vertical deflection and bending rotation)
7	Concentrated joint loads
8	Uniform member loads
9	Transformed sections are used for composite sections .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b
10	Sections symmetrical about vertical, principal axis
11	Unshored construction
12	Hinged bridge ends

FACTORS USED BY LRFD

```
 13    GAMMA for Load DC maximum     = 1.25

 14    GAMMA for Load DC minimum     = 0.90

 15    GAMMA for Load DW minimum     = 1.50

 16    GAMMA for Load DW minimum     = 0.65

 17    GAMMA for LL Load Strength I  = 1.75

 18    GAMMA for LL Load Strength II = 1.35

 19    GAMMA for LL Load Service I   = 1.00

 20    GAMMA for LL Load Service II  = 1.30

 21    GAMMA for LL Load Fatigue     = 0.75

 22    ETA for Service Limit State   = 1.00

 23    ETA for Strength Limit State  = 1.00

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TABLE 1.2.2.1=LOADING INFORMATION
             *******************

      AVERAGE DEAD LOAD INTENSITIES
      *****************************

   SPAN     SLAB    +   STEEL   =  TOTAL
    NO.    (K/FT)       (K/FT)    (K/FT)
   --------------------------------------

     1      0.554       0.1274     0.6814


   --------------------------------------




SUPERIMPOSED DEAD LOADS
***********************


   LOAD       INTENSITY      DIST       DIST
                             FROM        TO
           (K or K/Ft)       (Ft)       (Ft)
   ----------------------------------------

     1   DW       0.29        0.00      51.00

     1   DC2     0.087       0.000     51.000

     2   DC2     0.076       0.000     51.000


   ----------------------------------------
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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
              ******************************
```

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	1238.4	45.9	185.8	39.0	224.74

NOTE: [1] Concrete unit weight assumed to be 150. lb/ft**3

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH
****

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.88	70.00

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TABLE 1.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.567	0.567	0.660	0.660	FOR STRENGTH POSITIVE MOMENT
	0.550				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.390				FOR FATIGUE POSITIVE SHEAR

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TABLE 1.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP.
1	0	0.00	5900.0	32.9	16.45	358.7	358.7
1	1	3.19	5900.0	32.9	16.45	358.7	358.7
1	2	6.38	5900.0	32.9	16.45	358.7	358.7
1	3	9.56	5900.0	32.9	16.45	358.7	358.7
1	4	12.75	7121.5	32.9	14.85	479.7	383.8
1	5	15.94	7121.5	32.9	14.85	479.7	383.8
1	6	19.12	7121.5	32.9	14.85	479.7	383.8
1	7	22.31	7121.5	32.9	14.85	479.7	383.8
1	8	25.50	7121.5	32.9	14.85	479.7	383.8
1	9	28.69	7121.5	32.9	14.85	479.7	383.8
1	10	31.87	7121.5	32.9	14.85	479.7	383.8
1	11	35.06	7121.5	32.9	14.85	479.7	383.8
1	12	38.25	7121.5	32.9	14.85	479.7	383.8

1	13	41.44	5900.0	32.9	16.45	358.7	358.7
1	14	44.62	5900.0	32.9	16.45	358.7	358.7
1	15	47.81	5900.0	32.9	16.45	358.7	358.7
1	16	51.00	5900.0	32.9	16.45	358.7	358.7

NOTE: For rolled section, the 5th column is the depth d (inch)

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	11395.1	0.022778	477.8	1259.3	657.8
1	1	3.19	11395.1	0.022778	477.8	1259.3	657.8
1	2	6.38	11395.1	0.022778	477.8	1259.3	657.8
1	3	9.56	11395.1	0.022778	477.8	1259.3	657.8
1	4	12.75	14054.0	0.021272	630.4	1265.2	725.1
1	5	15.94	14054.0	0.021272	630.4	1265.2	725.1
1	6	19.12	14054.0	0.021272	630.4	1265.2	725.1
1	7	22.31	14054.0	0.021272	630.4	1265.2	725.1
1	8	25.50	14054.0	0.021272	630.4	1265.2	725.1
1	9	28.69	14054.0	0.021272	630.4	1265.2	725.1
1	10	31.87	14054.0	0.021272	630.4	1265.2	725.1
1	11	35.06	14054.0	0.021272	630.4	1265.2	725.1
1	12	38.25	14054.0	0.021272	630.4	1265.2	725.1
1	13	41.44	11395.1	0.022778	477.8	1259.3	657.8
1	14	44.62	11395.1	0.022778	477.8	1259.3	657.8
1	15	47.81	11395.1	0.022778	477.8	1259.3	657.8
1	16	51.00	11395.1	0.022778	477.8	1259.3	657.8

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 9.00
****

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix Q=1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	15689.7	0.029013	532.1	4594.9	1342.2
1	1	3.19	15689.7	0.029013	532.1	4594.9	1342.2
1	2	6.38	15689.7	0.029013	532.1	4594.9	1342.2
1	3	9.56	15689.7	0.029013	532.1	4594.9	1342.2
1	4	12.75	19883.5	0.027358	699.2	4008.3	1502.3
1	5	15.94	19883.5	0.027358	699.2	4008.3	1502.3
1	6	19.12	19883.5	0.027358	699.2	4008.3	1502.3
1	7	22.31	19883.5	0.027358	699.2	4008.3	1502.3
1	8	25.50	19883.5	0.027358	699.2	4008.3	1502.3
1	9	28.69	19883.5	0.027358	699.2	4008.3	1502.3
1	10	31.87	19883.5	0.027358	699.2	4008.3	1502.3
1	11	35.06	19883.5	0.027358	699.2	4008.3	1502.3

1 12	38.25	19883.5	0.027358	699.2	4008.3	1502.3
1 13	41.44	15689.7	0.029013	532.1	4594.9	1342.2
1 14	44.62	15689.7	0.029013	532.1	4594.9	1342.2
1 15	47.81	15689.7	0.029013	532.1	4594.9	1342.2
1 16	51.00	15689.7	0.029013	532.1	4594.9	1342.2

Please read NOTE on the following page

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NOTE [1] If the section modulus for the top flange indicates overflows (***), the neutral axis may be very closed to the top of the top flange.

[2] The section properties shown in this table are used for the calculation of stresses.

[3] AASHTO Art.10.38.1.6 or LRFD Art.6.10.1.1 --- Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete 'on the tension side of the neutral axis shall not be considered in calculating resulting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflections and for determining stiffness factors used in calculating moments and shears

[4] AASHTO LRFD 6.6.1.2.1 & C6.10.10.1.2 --- Q/I value shall be using short-term composite section for positeve & negative flxure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	3.19	9.7	42.2	0.6	0.0	52.5
1	2	6.38	18.1	78.8	1.2	0.0	98.1
1	3	9.56	25.2	109.8	1.5	0.0	136.5
1	4	12.75	31.1	135.1	1.6	0.0	167.8
1	5	15.94	35.6	154.8	1.7	0.0	192.1
1	6	19.12	38.8	168.9	1.8	0.0	209.5
1	7	22.31	40.8	177.3	2.0	0.0	220.1
1	8	25.50	41.4	180.1	2.1	0.0	223.7
1	9	28.69	40.8	177.3	2.2	0.0	220.3
1	10	31.87	38.8	168.9	2.0	0.0	209.7
1	11	35.06	35.6	154.8	1.7	0.0	192.0
1	12	38.25	31.1	135.1	1.3	0.0	167.5
1	13	41.44	25.2	109.8	1.0	0.0	136.0
1	14	44.62	18.1	78.8	0.7	0.0	97.6
1	15	47.81	9.7	42.2	0.3	0.0	52.3
1	16	51.00	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.2=COMPOSITE DEAD LOAD MOMENTS FOR N = 27.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	DC2	CONCENTRATED LOADS (k-ft) DW	DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.19	22.3	12.4	0.0	0.0	0.0	34.7
1	2	6.38	41.5	23.2	0.0	0.0	0.0	64.7
1	3	9.56	57.9	32.3	0.0	0.0	0.0	90.1
1	4	12.75	71.2	39.7	0.0	0.0	0.0	110.9
1	5	15.94	81.6	45.5	0.0	0.0	0.0	127.1
1	6	19.12	89.0	49.7	0.0	0.0	0.0	138.7
1	7	22.31	93.5	52.2	0.0	0.0	0.0	145.6
1	8	25.50	94.9	53.0	0.0	0.0	0.0	147.9
1	9	28.69	93.5	52.2	0.0	0.0	0.0	145.6
1	10	31.87	89.0	49.7	0.0	0.0	0.0	138.7
1	11	35.06	81.6	45.5	0.0	0.0	0.0	127.1
1	12	38.25	71.2	39.7	0.0	0.0	0.0	110.9
1	13	41.44	57.9	32.3	0.0	0.0	0.0	90.1
1	14	44.62	41.5	23.2	0.0	0.0	0.0	64.7
1	15	47.81	22.3	12.4	0.0	0.0	0.0	34.7
1	16	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 9.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	3.19	0.0	0.0	158.2	HL-93	0.0	HL-93
1	2	6.38	0.0	0.0	291.1	HL-93	0.0	HL-93
1	3	9.56	0.0	0.0	398.7	HL-93	0.0	HL-93
1	4	12.75	0.0	0.0	478.9	HL-93	0.0	HL-93
1	5	15.94	0.0	0.0	539.6	HL-93	0.0	HL-93
1	6	19.12	0.0	0.0	581.4	HL-93	0.0	HL-93
1	7	22.31	0.0	0.0	596.3	HL-93	0.0	HL-93
1	8	25.50	0.0	0.0	599.1	HL-93	0.0	HL-93
1	9	28.69	0.0	0.0	596.3	HL-93	0.0	HL-93
1	10	31.87	0.0	0.0	581.4	HL-93	0.0	HL-93
1	11	35.06	0.0	0.0	539.6	HL-93	0.0	HL-93
1	12	38.25	0.0	0.0	478.9	HL-93	0.0	HL-93
1	13	41.44	0.0	0.0	398.7	HL-93	0.0	HL-93
1	14	44.62	0.0	0.0	291.1	HL-93	0.0	HL-93
1	15	47.81	0.0	0.0	158.2	HL-93	0.0	HL-93
1	16	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 1.2.5.3A=FATIGUE LIVE LOAD MOMENT RANGE FOR N = 9.0 (k-ft) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	3.19	56.	0.	56.
1	2	6.38	100.	0.	100.
1	3	9.56	133.	0.	133.
1	4	12.75	157.	0.	157.
1	5	15.94	172.	0.	172.
1	6	19.12	178.	0.	178.
1	7	22.31	187.	0.	187.
1	8	25.50	191.	0.	191.
1	9	28.69	187.	0.	187.
1	10	31.87	178.	0.	178.
1	11	35.06	172.	0.	172.
1	12	38.25	157.	0.	157.
1	13	41.44	133.	0.	133.
1	14	44.62	100.	0.	100.
1	15	47.81	56.	0.	56.
1	16	51.00	0.	0.	0.

TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD NON COMP. N=Infin. (k-ft)	COMP. N=27.0 (k-ft)	LL+I:N= 9.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	NEGATIVE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	3.19	52.5	34.7	158.2	HL-93	0.0	HL-93	245.4	87.2
1	2	6.38	98.1	64.7	291.1	HL-93	0.0	HL-93	454.0	162.8
1	3	9.56	136.5	90.1	398.7	HL-93	0.0	HL-93	625.3	226.6
1	4	12.75	167.8	110.9	478.9	HL-93	0.0	HL-93	757.6	278.7
1	5	15.94	192.1	127.1	539.6	HL-93	0.0	HL-93	858.8	319.2
1	6	19.12	209.5	138.7	581.4	HL-93	0.0	HL-93	929.7	348.2
1	7	22.31	220.1	145.6	596.3	HL-93	0.0	HL-93	962.0	365.7
1	8	25.50	223.7	147.9	599.1	HL-93	0.0	HL-93	970.7	371.6
1	9	28.69	220.3	145.6	596.3	HL-93	0.0	HL-93	962.3	365.9
1	10	31.87	209.7	138.7	581.4	HL-93	0.0	HL-93	929.8	348.4
1	11	35.06	192.0	127.1	539.6	HL-93	0.0	HL-93	858.8	319.2
1	12	38.25	167.5	110.9	478.9	HL-93	0.0	HL-93	757.3	278.4
1	13	41.44	136.0	90.1	398.7	HL-93	0.0	HL-93	624.9	226.1
1	14	44.62	97.6	64.7	291.1	HL-93	0.0	HL-93	453.4	162.3
1	15	47.81	52.3	34.7	158.2	HL-93	0.0	HL-93	245.2	86.9
1	16	51.00	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

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****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	3.19	245.4	292.9	391.4	328.2	130.8	41.8
1	2	6.38	454.0	541.3	723.4	606.9	244.2	74.9
1	3	9.56	625.3	745.0	995.5	836.0	339.9	100.1
1	4	12.75	757.6	901.3	1204.2	1012.7	418.1	117.9
1	5	15.94	858.8	1020.7	1363.7	1147.9	478.9	128.8
1	6	19.12	929.7	1104.1	1475.1	1242.5	522.4	133.8
1	7	22.31	962.0	1140.9	1524.1	1285.5	548.5	140.3
1	8	25.50	970.7	1150.4	1536.6	1297.0	557.4	143.3
1	9	28.69	962.3	1141.2	1524.4	1285.8	548.9	140.3
1	10	31.87	929.8	1104.3	1475.2	1242.7	522.6	133.8
1	11	35.06	858.8	1020.6	1363.7	1147.8	478.8	128.8
1	12	38.25	757.3	901.0	1203.9	1012.3	417.6	117.9
1	13	41.44	624.9	744.5	994.9	835.4	339.2	100.1
1	14	44.62	453.4	540.8	722.8	606.3	243.5	74.9
1	15	47.81	245.2	292.6	391.1	327.8	130.4	41.8
1	16	51.00	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 1.2.6.1=NONCOMPOSITE DEAD LOAD SHEAR FOR N=INFINITY (UNFACTORED)
****

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL NONCOMPOSITE DEAD LOAD (kip)
1	0	0.00	3.2	14.1	0.2	0.0	17.6
1	1	3.19	2.8	12.4	0.2	0.0	15.4
1	2	6.38	2.4	10.6	0.2	0.0	13.2
1	3	9.56	2.0	8.8	0.0	0.0	10.9
1	4	12.75	1.6	7.1	0.0	0.0	8.7
1	5	15.94	1.2	5.3	0.0	0.0	6.6
1	6	19.12	0.8	3.5	0.0	0.0	4.4
1	7	22.31	0.4	1.8	0.0	0.0	2.2
1	8	25.50	0.0	0.0	0.0	0.0	0.0
1	9	28.69	-0.4	-1.8	0.0	0.0	-2.1
1	10	31.87	-0.8	-3.5	-0.1	0.0	-4.4
1	11	35.06	-1.2	-5.3	-0.1	0.0	-6.6
1	12	38.25	-1.6	-7.1	-0.1	0.0	-8.8
1	13	41.44	-2.0	-8.8	-0.1	0.0	-11.0
1	14	44.62	-2.4	-10.6	-0.1	0.0	-13.1
1	15	47.81	-2.8	-12.4	-0.1	0.0	-15.3
1	16	51.00	-3.2	-14.1	-0.1	0.0	-17.5

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TABLE 1.2.6.2=NONCOMPOSITE AND COMPOSITE DEAD LOAD SHEAR SUMMARY (UNFACTORED)
****

COMPOSITE (k) TOTAL TOTAL

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM LOADS DW	UNIFORM LOADS DC2	CONCENTRATED LOADS DW	CONCENTRATED LOADS DC2	COMPOSITE (k)	NONCOMPOSITE (k)	DEAD LOADS (k)
1	0	0.00	7.4	4.2	0.0	0.0	11.6	17.6	29.2
1	1	3.19	6.5	3.6	0.0	0.0	10.2	15.4	25.5
1	2	6.38	5.6	3.1	0.0	0.0	8.7	13.2	21.9
1	3	9.56	4.7	2.6	0.0	0.0	7.3	10.9	18.2
1	4	12.75	3.7	2.1	0.0	0.0	5.8	8.7	14.5
1	5	15.94	2.8	1.6	0.0	0.0	4.4	6.6	10.9
1	6	19.12	1.9	1.0	0.0	0.0	2.9	4.4	7.3
1	7	22.31	0.9	0.5	0.0	0.0	1.5	2.2	3.7
1	8	25.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	9	28.69	-0.9	-0.5	0.0	0.0	-1.5	-2.1	-3.6
1	10	31.87	-1.9	-1.0	0.0	0.0	-2.9	-4.4	-7.3
1	11	35.06	-2.8	-1.6	0.0	0.0	-4.4	-6.6	-11.0
1	12	38.25	-3.7	-2.1	0.0	0.0	-5.8	-8.8	-14.6
1	13	41.44	-4.7	-2.6	0.0	0.0	-7.3	-11.0	-18.2
1	14	44.62	-5.6	-3.1	0.0	0.0	-8.7	-13.1	-21.8
1	15	47.81	-6.5	-3.6	0.0	0.0	-10.2	-15.3	-25.5
1	16	51.00	-7.4	-4.2	0.0	0.0	-11.6	-17.5	-29.1

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	52.0	HL-93	0.0	HL-93
1	1	3.19	0.0	0.0	47.6	HL-93	-1.5	HL-93
1	2	6.38	0.0	0.0	43.3	HL-93	-3.3	HL-93
1	3	9.56	0.0	0.0	39.1	HL-93	-5.7	HL-93
1	4	12.75	0.0	0.0	34.9	HL-93	-8.3	HL-93
1	5	15.94	0.0	0.0	30.8	HL-93	-10.9	HL-93
1	6	19.12	0.0	0.0	26.7	HL-93	-13.5	HL-93
1	7	22.31	0.0	0.0	22.9	HL-93	-16.3	HL-93
1	8	25.50	0.0	0.0	19.2	HL-93	-19.2	HL-93
1	9	28.69	0.0	0.0	16.3	HL-93	-22.9	HL-93
1	10	31.87	0.0	0.0	13.5	HL-93	-26.7	HL-93
1	11	35.06	0.0	0.0	10.9	HL-93	-30.8	HL-93
1	12	38.25	0.0	0.0	8.3	HL-93	-34.9	HL-93
1	13	41.44	0.0	0.0	5.7	HL-93	-39.1	HL-93
1	14	44.62	0.0	0.0	3.3	HL-93	-43.3	HL-93
1	15	47.81	0.0	0.0	1.5	HL-93	-47.6	HL-93
1	16	51.00	0.0	0.0	0.0	HL-93	-52.0	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 9.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	TRUCK ONLY NEG	TRUCK ONLY RANGE
1	0	0.00	21.	0.	21.
1	1	3.19	19.	-1.	20.

1	2	6.38	17.	-2.	19.
1	3	9.56	15.	-3.	18.
1	4	12.75	13.	-4.	17.
1	5	15.94	11.	-5.	16.
1	6	19.12	9.	-6.	15.
1	7	22.31	9.	-7.	16.
1	8	25.50	8.	-8.	16.
1	9	28.69	7.	-9.	16.
1	10	31.87	6.	-9.	15.
1	11	35.06	5.	-11.	16.
1	12	38.25	4.	-13.	17.
1	13	41.44	3.	-15.	18.
1	14	44.62	2.	-17.	19.
1	15	47.81	1.	-19.	20.
1	16	51.00	0.	-21.	21.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	20.7	.31013E-01	0.32	0.2500*	.14641E-01	0.15	0.2500*
1	1		3.19	19.6	.31013E-01	0.30	0.2500*	.14641E-01	0.14	0.2500*
1	2		6.38	18.5	.31013E-01	0.29	0.2500*	.14641E-01	0.14	0.2500*
1	3		9.56	17.6	.31013E-01	0.27	0.2500*	.14641E-01	0.13	0.2500*
1	4		12.75	16.7	.29585E-01	0.25	0.2500*	.10903E-01	0.09	0.2500*
1	5		15.94	15.8	.29585E-01	0.23	0.2500*	.10903E-01	0.09	0.2500*
1	6		19.12	15.0	.29585E-01	0.22	0.2500*	.10903E-01	0.08	0.2500*
1	7		22.31	15.6	.29585E-01	0.23	0.2500*	.10903E-01	0.08	0.2500*
1	8		25.50	16.0	.29585E-01	0.24	0.2500*	.10903E-01	0.09	0.2500*
1	9		28.69	15.6	.29585E-01	0.23	0.2500*	.10903E-01	0.08	0.2500*
1	10		31.87	15.0	.29585E-01	0.22	0.2500*	.10903E-01	0.08	0.2500*
1	11		35.06	15.8	.29585E-01	0.23	0.2500*	.10903E-01	0.09	0.2500*
1	12		38.25	16.7	.29585E-01	0.25	0.2500*	.10903E-01	0.09	0.2500*
1	13		41.44	17.6	.31013E-01	0.27	0.2500*	.14641E-01	0.13	0.2500*
1	14		44.62	18.5	.31013E-01	0.29	0.2500*	.14641E-01	0.14	0.2500*
1	15		47.81	19.6	.31013E-01	0.30	0.2500*	.14641E-01	0.14	0.2500*
1	16		51.00	20.7	.31013E-01	0.32	0.2500*	.14641E-01	0.15	0.2500*

NOTE: [1] Vr = range of shear due to live loads and impact

[2] For non-composite construction:

Q/I = (At * Dt) / Inc -- top flange
Inc = moment of inertia of non-composite section
At = area of top flange
Dt = distance between the center of top flange and neutral axis
Q/I = (Ab * Db) / Inc -- bottom flange
Ab = area of bottom flange
Db = distance between the center of bottom flange and neutral axis

For composite construction:

Q/I = (Q/Ic) + (At * Dt) / Ic -- top flange
Q = statical moment about the neutral axis of the
composite section of the transformed compressive
concrete area or the area of reinforcement embedded
in the concrete for negative moment
Ic = moment of inertia for composite section
Q/I = (Ab * Db) / Ic -- bottom flange

[3] shear flow = (Vr * Q) / (2 * I)

* -- minimum weld size governs

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TABLE 1.2.6.4=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , LOAD TYPE = HL - 93 POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips) POS.	NEG.
1	0	0.00	29.2	52.0	HL-93	0.0	HL-93	81.2	29.2
1	1	3.19	25.5	47.6	HL-93	-1.5	HL-93	73.2	24.0
1	2	6.38	21.9	43.3	HL-93	-3.3	HL-93	65.2	18.6
1	3	9.56	18.2	39.1	HL-93	-5.7	HL-93	57.2	12.4
1	4	12.75	14.5	34.9	HL-93	-8.3	HL-93	49.4	6.3
1	5	15.94	10.9	30.8	HL-93	-10.9	HL-93	41.7	0.0
1	6	19.12	7.3	26.7	HL-93	-13.5	HL-93	34.0	-6.3
1	7	22.31	3.7	22.9	HL-93	-16.3	HL-93	26.6	-12.6
1	8	25.50	0.0	19.2	HL-93	-19.2	HL-93	19.3	-19.2
1	9	28.69	-3.6	16.3	HL-93	-22.9	HL-93	12.7	-26.5
1	10	31.87	-7.3	13.5	HL-93	-26.7	HL-93	6.2	-34.1
1	11	35.06	-11.0	10.9	HL-93	-30.8	HL-93	-0.1	-41.8
1	12	38.25	-14.6	8.3	HL-93	-34.9	HL-93	-6.3	-49.5
1	13	41.44	-18.2	5.7	HL-93	-39.1	HL-93	-12.5	-57.3
1	14	44.62	-21.8	3.3	HL-93	-43.3	HL-93	-18.6	-65.1
1	15	47.81	-25.5	1.5	HL-93	-47.6	HL-93	-24.0	-73.1
1	16	51.00	-29.1	0.0	HL-93	-52.0	HL-93	-29.1	-81.1

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TABLE 1.2.6.5=SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	81.2	96.8	129.3	108.5	43.7	15.6
1	1	3.19	73.2	87.4	116.9	97.8	38.3	14.7
1	2	6.38	65.2	78.2	104.6	87.3	32.9	13.9
1	3	9.56	57.2	68.9	92.2	76.6	27.2	13.2
1	4	12.75	49.4	59.9	80.2	66.2	21.8	12.5
1	5	15.94	41.7	50.9	68.2	55.9	16.4	11.8
1	6	19.12	34.0	42.1	56.4	45.7	10.9	11.3
1	7	22.31	26.6	33.4	44.9	35.7	5.5	11.7
1	8	25.50	19.3	25.0	33.7	26.0	0.1	12.0
1	9	28.69	-26.5	-33.3	-44.8	-35.6	-5.4	11.7
1	10	31.87	-34.1	-42.1	-56.5	-45.8	-11.0	11.3
1	11	35.06	-41.8	-51.0	-68.3	-56.0	-16.5	11.8
1	12	38.25	-49.5	-60.0	-80.2	-66.3	-21.9	12.5
1	13	41.44	-57.3	-69.0	-92.3	-76.7	-27.3	13.2
1	14	44.62	-65.1	-78.1	-104.5	-87.2	-32.8	13.9
1	15	47.81	-73.1	-87.4	-116.8	-97.7	-38.2	14.7
1	16	51.00	-81.1	-96.7	-129.2	-108.4	-43.6	15.6

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	29.16	0.00	HL-93	51.99	HL-93	ST1	24.39	129.31
						ST2	24.39	108.51
						ST4	32.58	43.74
						SE1	29.16	81.16
						SE2	29.16	96.76
2	29.08	0.00	HL-93	51.99	HL-93	ST1	24.31	129.20
						ST2	24.31	108.41
						ST4	32.45	43.62
						SE1	29.08	81.08
						SE2	29.08	96.68

NOTE: [1] " - " Indicates Uplift

ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.

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TABLE 1.2.8.1=COMP AND NONCOMP DL DEFL FOR INFINITY AND N = 27.0 (UNFACTORED)
****

SP NO	IN D FROM L SUPT (ft)	NONCOMPOSITE DL BEAM (in)	SLAB (in)	COMPOSITE DL CONCENTRATED (in)	UNIFORM (in)	T O T A L NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1 0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 1	3.19	-0.0196	-0.0853	0.0000	-0.0357	-0.1059	-0.0357	-0.1417
1 2	6.38	-0.0383	-0.1663	0.0000	-0.0696	-0.2065	-0.0696	-0.2762
1 3	9.56	-0.0550	-0.2393	0.0000	-0.1001	-0.2971	-0.1001	-0.3973
1 4	12.75	-0.0693	-0.3012	0.0000	-0.1259	-0.3740	-0.1259	-0.4999
1 5	15.94	-0.0808	-0.3513	0.0000	-0.1468	-0.4362	-0.1468	-0.5830
1 6	19.12	-0.0893	-0.3883	0.0000	-0.1622	-0.4821	-0.1622	-0.6442
1 7	22.31	-0.0945	-0.4110	0.0000	-0.1716	-0.5102	-0.1716	-0.6818
1 8	25.50	-0.0963	-0.4186	0.0000	-0.1748	-0.5197	-0.1748	-0.6944
1 9	28.69	-0.0945	-0.4110	0.0000	-0.1716	-0.5102	-0.1716	-0.6818
1 10	31.87	-0.0893	-0.3883	0.0000	-0.1622	-0.4820	-0.1622	-0.6442
1 11	35.06	-0.0808	-0.3513	0.0000	-0.1468	-0.4361	-0.1468	-0.5828
1 12	38.25	-0.0693	-0.3012	0.0000	-0.1259	-0.3738	-0.1259	-0.4998
1 13	41.44	-0.0550	-0.2393	0.0000	-0.1001	-0.2969	-0.1001	-0.3971
1 14	44.62	-0.0383	-0.1663	0.0000	-0.0696	-0.2064	-0.0696	-0.2760
1 15	47.81	-0.0196	-0.0853	0.0000	-0.0357	-0.1058	-0.0357	-0.1416
1 16	51.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " Indicates downward deflections

NOTE: The total noncomposite DL deflection is the sum of the deflections due to beam, slab, arbitrary DL uniform load and arbitrary DL concentrated load.

NOTE: Due to space limit only beam deflections and slab deflections are printed out.

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TABLE 1.2.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN D NO	FROM L SUPT (ft)	NONCOMPOSITE DEAD LOADS (in) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	COMPOSITE DL (in) DEFL.	CAMBER SIZE	TOTAL (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	3.2	-0.020	0 1/16	-0.086	0 1/ 8	-0.036	0 1/16	-0.142	0 3/16
1	2	6.4	-0.038	0 1/16	-0.168	0 3/16	-0.070	0 1/ 8	-0.276	0 5/16
1	3	9.6	-0.055	0 1/16	-0.242	0 1/ 4	-0.100	0 1/ 8	-0.397	0 7/16
1	4	12.7	-0.069	0 1/ 8	-0.305	0 5/16	-0.126	0 3/16	-0.500	0 1/ 2
1	5	15.9	-0.081	0 1/ 8	-0.355	0 3/ 8	-0.147	0 3/16	-0.583	0 5/ 8
1	6	19.1	-0.089	0 1/ 8	-0.393	0 7/16	-0.162	0 3/16	-0.644	0 11/16
1	7	22.3	-0.095	0 1/ 8	-0.416	0 7/16	-0.172	0 3/16	-0.682	0 11/16
1	8	25.5	-0.096	0 1/ 8	-0.423	0 7/16	-0.175	0 3/16	-0.694	0 3/ 4
1	9	28.7	-0.095	0 1/ 8	-0.416	0 7/16	-0.172	0 3/16	-0.682	0 11/16
1	10	31.9	-0.089	0 1/ 8	-0.393	0 7/16	-0.162	0 3/16	-0.644	0 11/16
1	11	35.1	-0.081	0 1/ 8	-0.355	0 3/ 8	-0.147	0 3/16	-0.583	0 5/ 8
1	12	38.2	-0.069	0 1/ 8	-0.305	0 5/16	-0.126	0 3/16	-0.500	0 1/ 2
1	13	41.4	-0.055	0 1/16	-0.242	0 1/ 4	-0.100	0 1/ 8	-0.397	0 7/16
1	14	44.6	-0.038	0 1/16	-0.168	0 3/16	-0.070	0 1/ 8	-0.276	0 5/16
1	15	47.8	-0.020	0 1/16	-0.086	0 1/ 8	-0.036	0 1/16	-0.142	0 3/16
1	16	51.0	0.000	--	0.000	--	0.000	--	0.000	--

NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2

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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
****
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE AND DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	25.50	3 0.425	-0.289	MAX HL-93	0.76	0.00274
			0.029	MIN		
			-0.074	MAX LANE		
			0.029	MIN LANE		
		4 0.433	-0.295	MAX HL-93	0.76	0.00274
			0.030	MIN		
			-0.076	MAX LANE		
			0.030	MIN LANE		
		3 0.425	-0.182	MAX FA		
			0.000	MIN FA		

NOTE: [1] " - " indicates downward deflection

[2] The distribution factor for LL+I deflection is defined as

DF = (NL/Ng)*(RF)lane ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders
(RF)lane = reduction factor from AASHTO LRFD Art. 3.6.1.1.2

[3] This table is based upon the optional criteria specified in AASHTO LRFD Art. 3.6.1.3.2

[4] The number of traffic lanes is determined according to AASHTO LRFD Art.3.6.1.1.1.
The 1st line is for the most probable number of lanes and the 2nd line is for the next probable number of lanes.

[5] Max rotations at left (1st line) & right (2nd line) supports of the span without averaging, factor and impact

[6] If ADTT is between 100 and 1000, multi-presence factor of 0.95 is applied. If ADTT is below 100, factor is 0.9 (AASHTO LRFD C3.6.1.1.2).

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	STEEL DEAD LOAD STEEL BEAM TOP (ksi)	STEEL DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.32	0.32	-1.43	1.43	-1.76	1.76
1	2	6.38	-0.61	0.61	-2.68	2.68	-3.28	3.28
1	3	9.56	-0.84	0.84	-3.72	3.72	-4.57	4.57
1	4	12.75	-0.97	0.78	-4.27	3.42	-5.24	4.20
1	5	15.94	-1.11	0.89	-4.89	3.92	-6.01	4.81
1	6	19.12	-1.21	0.97	-5.34	4.27	-6.55	5.24
1	7	22.31	-1.27	1.02	-5.60	4.49	-6.88	5.51
1	8	25.50	-1.30	1.04	-5.70	4.56	-6.99	5.60
1	9	28.69	-1.27	1.02	-5.61	4.49	-6.89	5.51
1	10	31.87	-1.21	0.97	-5.34	4.27	-6.56	5.25
1	11	35.06	-1.11	0.89	-4.89	3.91	-6.00	4.80
1	12	38.25	-0.97	0.78	-4.26	3.41	-5.24	4.19
1	13	41.44	-0.84	0.84	-3.71	3.71	-4.55	4.55
1	14	44.62	-0.61	0.61	-2.66	2.66	-3.27	3.27
1	15	47.81	-0.32	0.32	-1.42	1.42	-1.75	1.75
1	16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.2=COMPOSITE DL STRESS FOR N = 27.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD,(ksi) CONCRETE TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM TOP	SUPERIMPOSED DEAD LOAD,(ksi) STEEL BEAM BOT	TOTAL DEAD LOAD,(ksi) CONCRETE TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM TOP	TOTAL DEAD LOAD,(ksi) STEEL BEAM BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.02	-0.33	0.87	-0.02	-2.09	2.63
1	2	6.38	-0.04	-0.62	1.63	-0.04	-3.90	4.91
1	3	9.56	-0.06	-0.86	2.26	-0.06	-5.42	6.83
1	4	12.75	-0.07	-1.05	2.11	-0.07	-6.30	6.31
1	5	15.94	-0.08	-1.21	2.42	-0.08	-7.21	7.23
1	6	19.12	-0.09	-1.32	2.64	-0.09	-7.87	7.88
1	7	22.31	-0.09	-1.38	2.77	-0.09	-8.26	8.28
1	8	25.50	-0.09	-1.40	2.82	-0.09	-8.40	8.41
1	9	28.69	-0.09	-1.38	2.77	-0.09	-8.27	8.28
1	10	31.87	-0.09	-1.32	2.64	-0.09	-7.87	7.89
1	11	35.06	-0.08	-1.21	2.42	-0.08	-7.21	7.22
1	12	38.25	-0.07	-1.05	2.11	-0.07	-6.29	6.30

1 13	41.44	-0.06	-0.86	2.26	-0.06	-5.41	6.81
1 14	44.62	-0.04	-0.62	1.63	-0.04	-3.88	4.89
1 15	47.81	-0.02	-0.33	0.87	-0.02	-2.08	2.62
1 16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3=LIVE LOAD STRESSES FOR N = 9.0 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR. TOP	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR. TOP	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	-0.16	-0.41	3.57	0.00	0.00	0.00
1	2	6.38	-0.29	-0.76	6.57	0.00	0.00	0.00
1	3	9.56	-0.40	-1.04	8.99	0.00	0.00	0.00
1	4	12.75	-0.43	-1.43	8.22	0.00	0.00	0.00
1	5	15.94	-0.48	-1.62	9.26	0.00	0.00	0.00
1	6	19.12	-0.52	-1.74	9.98	0.00	0.00	0.00
1	7	22.31	-0.53	-1.79	10.24	0.00	0.00	0.00
1	8	25.50	-0.53	-1.79	10.28	0.00	0.00	0.00
1	9	28.69	-0.53	-1.79	10.24	0.00	0.00	0.00
1	10	31.87	-0.52	-1.74	9.98	0.00	0.00	0.00
1	11	35.06	-0.48	-1.62	9.26	0.00	0.00	0.00
1	12	38.25	-0.43	-1.43	8.22	0.00	0.00	0.00
1	13	41.44	-0.40	-1.04	8.99	0.00	0.00	0.00
1	14	44.62	-0.29	-0.76	6.57	0.00	0.00	0.00
1	15	47.81	-0.16	-0.41	3.57	0.00	0.00	0.00
1	16	51.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.3A=FATIGUE LIVE LOAD STRESS RANGE FOR N = 9.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	ONLY	STRESS RANGE	GOVERNING LOAD TP	TYPE	BOT OF BOT FLANGE TRUCK	ONLY	STRESS RANGE	GOVERNING LOAD TP	TYPE
1	0	0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	3.19	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	2	6.38	0.3	0.0	0.3	C	HL	2.3	0.0	2.3	T	HL
1	3	9.56	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	4	12.75	0.5	0.0	0.5	C	HL	2.7	0.0	2.7	T	HL
1	5	15.94	0.5	0.0	0.5	C	HL	2.9	0.0	2.9	T	HL
1	6	19.12	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	7	22.31	0.6	0.0	0.6	C	HL	3.2	0.0	3.2	T	HL
1	8	25.50	0.6	0.0	0.6	C	HL	3.3	0.0	3.3	T	HL
1	9	28.69	0.6	0.0	0.6	C	HL	3.2	0.0	3.2	T	HL
1	10	31.87	0.5	0.0	0.5	C	HL	3.1	0.0	3.1	T	HL
1	11	35.06	0.5	0.0	0.5	C	HL	2.9	0.0	2.9	T	HL
1	12	38.25	0.5	0.0	0.5	C	HL	2.7	0.0	2.7	T	HL
1	13	41.44	0.3	0.0	0.3	C	HL	3.0	0.0	3.0	T	HL
1	14	44.62	0.3	0.0	0.3	C	HL	2.3	0.0	2.3	T	HL
1	15	47.81	0.1	0.0	0.1	C	HL	1.3	0.0	1.3	T	HL
1	16	51.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

NOTE: LOAD TYPE: HL = Fatigue Truck or Tandem
STRESS TYPE: R = Reversal, C = Compression, and T = Tension

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TABLE 1.2.9.5A=SERVICE I TOTAL (DC+DW+LL+I) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.18	-2.50	6.20	-0.02	-2.09	2.63
1	2	6.38	33.	33.	-0.33	-4.66	11.47	-0.04	-3.90	4.91
1	3	9.56	33.	33.	-0.46	-6.47	15.82	-0.06	-5.42	6.83
1	4	12.75	33.	33.	-0.49	-7.73	14.53	-0.07	-6.30	6.31
1	5	15.94	33.	33.	-0.56	-8.83	16.49	-0.08	-7.21	7.23
1	6	19.12	33.	33.	-0.60	-9.61	17.86	-0.09	-7.87	7.88
1	7	22.31	33.	33.	-0.62	-10.05	18.51	-0.09	-8.26	8.28
1	8	25.50	33.	33.	-0.62	-10.19	18.69	-0.09	-8.40	8.41
1	9	28.69	33.	33.	-0.62	-10.05	18.52	-0.09	-8.27	8.28
1	10	31.87	33.	33.	-0.60	-9.61	17.87	-0.09	-7.87	7.89
1	11	35.06	33.	33.	-0.56	-8.83	16.49	-0.08	-7.21	7.22
1	12	38.25	33.	33.	-0.49	-7.72	14.52	-0.07	-6.29	6.30
1	13	41.44	33.	33.	-0.46	-6.45	15.81	-0.06	-5.41	6.81
1	14	44.62	33.	33.	-0.33	-4.64	11.46	-0.04	-3.88	4.89
1	15	47.81	33.	33.	-0.18	-2.49	6.19	-0.02	-2.08	2.62
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO L	FROM SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.23	-2.62	7.27	-0.02	-2.09	2.63
1	2	6.38	33.	33.	-0.42	-4.89	13.44	-0.04	-3.90	4.91
1	3	9.56	33.	33.	-0.58	-6.78	18.52	-0.06	-5.42	6.83
1	4	12.75	33.	33.	-0.62	-8.16	16.99	-0.07	-6.30	6.31
1	5	15.94	33.	33.	-0.70	-9.31	19.27	-0.08	-7.21	7.23
1	6	19.12	33.	33.	-0.76	-10.13	20.86	-0.09	-7.87	7.88
1	7	22.31	33.	33.	-0.78	-10.58	21.58	-0.09	-8.26	8.28
1	8	25.50	33.	33.	-0.78	-10.73	21.78	-0.09	-8.40	8.41
1	9	28.69	33.	33.	-0.78	-10.59	21.59	-0.09	-8.27	8.28
1	10	31.87	33.	33.	-0.76	-10.13	20.86	-0.09	-7.87	7.89
1	11	35.06	33.	33.	-0.70	-9.31	19.26	-0.08	-7.21	7.22
1	12	38.25	33.	33.	-0.62	-8.15	16.99	-0.07	-6.29	6.30
1	13	41.44	33.	33.	-0.58	-6.76	18.50	-0.06	-5.41	6.81
1	14	44.62	33.	33.	-0.42	-4.87	13.43	-0.04	-3.88	4.89
1	15	47.81	33.	33.	-0.23	-2.62	7.26	-0.02	-2.08	2.62
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L (ft)	SUPT	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.31	-3.39	9.67	-0.03	-2.66	3.42
1	2	6.38	33.	33.	-0.57	-6.30	17.89	-0.06	-4.97	6.40
1	3	9.56	33.	33.	-0.78	-8.74	24.64	-0.09	-6.92	8.90
1	4	12.75	33.	33.	-0.84	-10.55	22.61	-0.10	-8.04	8.22
1	5	15.94	33.	33.	-0.95	-12.04	25.63	-0.11	-9.21	9.42
1	6	19.12	33.	33.	-1.02	-13.09	27.74	-0.12	-10.04	10.28
1	7	22.31	33.	33.	-1.05	-13.67	28.70	-0.13	-10.55	10.79
1	8	25.50	33.	33.	-1.06	-13.86	28.96	-0.13	-10.72	10.97
1	9	28.69	33.	33.	-1.05	-13.68	28.71	-0.13	-10.56	10.80
1	10	31.87	33.	33.	-1.02	-13.10	27.74	-0.12	-10.05	10.28
1	11	35.06	33.	33.	-0.95	-12.03	25.63	-0.11	-9.21	9.42
1	12	38.25	33.	33.	-0.84	-10.54	22.60	-0.10	-8.03	8.22
1	13	41.44	33.	33.	-0.78	-8.72	24.62	-0.09	-6.90	8.88
1	14	44.62	33.	33.	-0.57	-6.28	17.86	-0.06	-4.95	6.37
1	15	47.81	33.	33.	-0.31	-3.37	9.66	-0.03	-2.65	3.41
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY
****

SP NO	IN D NO	FROM L (ft)	SUPT	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.19	33.	33.	-0.29	-2.55	8.47	-0.02	-1.83	2.23
1	2	6.38	33.	33.	-0.54	-4.74	15.65	-0.03	-3.41	4.16
1	3	9.56	33.	33.	-0.74	-6.57	21.52	-0.05	-4.74	5.78
1	4	12.75	33.	33.	-0.79	-8.01	19.72	-0.05	-5.50	5.34
1	5	15.94	33.	33.	-0.90	-9.12	22.32	-0.06	-6.30	6.12
1	6	19.12	33.	33.	-0.97	-9.92	24.13	-0.06	-6.87	6.67
1	7	22.31	33.	33.	-0.99	-10.34	24.92	-0.07	-7.21	7.00
1	8	25.50	33.	33.	-1.00	-10.47	25.11	-0.07	-7.33	7.12
1	9	28.69	33.	33.	-0.99	-10.34	24.92	-0.07	-7.22	7.01
1	10	31.87	33.	33.	-0.97	-9.92	24.14	-0.06	-6.87	6.67
1	11	35.06	33.	33.	-0.90	-9.12	22.32	-0.06	-6.30	6.11
1	12	38.25	33.	33.	-0.79	-8.00	19.72	-0.05	-5.49	5.33
1	13	41.44	33.	33.	-0.74	-6.55	21.51	-0.05	-4.73	5.77
1	14	44.62	33.	33.	-0.54	-4.73	15.63	-0.03	-3.39	4.14
1	15	47.81	33.	33.	-0.29	-2.54	8.46	-0.02	-1.82	2.22
1	16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN D FROM NO L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	3.19	33.	33.	-0.25	-3.22	8.24	-0.03	-2.66	3.42
1 2	6.38	33.	33.	-0.45	-6.00	15.26	-0.06	-4.97	6.40
1 3	9.56	33.	33.	-0.62	-8.32	21.04	-0.09	-6.92	8.90
1 4	12.75	33.	33.	-0.67	-9.98	19.32	-0.10	-8.04	8.22
1 5	15.94	33.	33.	-0.76	-11.39	21.92	-0.11	-9.21	9.42
1 6	19.12	33.	33.	-0.82	-12.39	23.75	-0.12	-10.04	10.28
1 7	22.31	33.	33.	-0.84	-12.96	24.61	-0.13	-10.55	10.79
1 8	25.50	33.	33.	-0.85	-13.14	24.85	-0.13	-10.72	10.97
1 9	28.69	33.	33.	-0.84	-12.97	24.62	-0.13	-10.56	10.80
1 10	31.87	33.	33.	-0.82	-12.40	23.75	-0.12	-10.05	10.28
1 11	35.06	33.	33.	-0.76	-11.39	21.92	-0.11	-9.21	9.42
1 12	38.25	33.	33.	-0.67	-9.96	19.31	-0.10	-8.03	8.22
1 13	41.44	33.	33.	-0.62	-8.31	21.02	-0.09	-6.90	8.88
1 14	44.62	33.	33.	-0.45	-5.98	15.24	-0.06	-4.95	6.37
1 15	47.81	33.	33.	-0.25	-3.21	8.23	-0.03	-2.65	3.41
1 16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.9.5F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY
****

SP NO	IN D FROM NO L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	BOT	TOTAL POSITIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT	TOTAL NEGATIVE ,(ksi) CONCR. TOP	STEEL BEAM TOP	STEEL BEAM BOT
1 0	0.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00
1 1	3.19	33.	33.	-0.23	-2.38	7.04	-0.02	-1.83	2.23
1 2	6.38	33.	33.	-0.42	-4.44	13.02	-0.03	-3.41	4.16
1 3	9.56	33.	33.	-0.58	-6.15	17.92	-0.05	-4.74	5.78
1 4	12.75	33.	33.	-0.62	-7.43	16.44	-0.05	-5.50	5.34
1 5	15.94	33.	33.	-0.70	-8.48	18.62	-0.06	-6.30	6.12
1 6	19.12	33.	33.	-0.76	-9.22	20.14	-0.06	-6.87	6.67
1 7	22.31	33.	33.	-0.78	-9.62	20.82	-0.07	-7.21	7.00
1 8	25.50	33.	33.	-0.78	-9.75	21.00	-0.07	-7.33	7.12
1 9	28.69	33.	33.	-0.78	-9.63	20.83	-0.07	-7.22	7.01
1 10	31.87	33.	33.	-0.76	-9.22	20.15	-0.06	-6.87	6.67
1 11	35.06	33.	33.	-0.70	-8.48	18.62	-0.06	-6.30	6.11
1 12	38.25	33.	33.	-0.62	-7.43	16.43	-0.05	-5.49	5.33
1 13	41.44	33.	33.	-0.58	-6.14	17.91	-0.05	-4.73	5.77
1 14	44.62	33.	33.	-0.42	-4.42	13.00	-0.03	-3.39	4.14
1 15	47.81	33.	33.	-0.23	-2.38	7.03	-0.02	-1.82	2.22
1 16	51.00	33.	33.	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 1.2.10.1=STRESSES AT SECTION-CHANGE POINTS (UNFACTORED)
****

SP	JOINT D FROM	DL1, N=INF.	DL2, N=27.0	POS. LL+I	NEG. LL+I	TOTAL STRESS --AND--

NO	TYPE	L SUPT (ft)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	MOM. STRESS (k-ft)	(ksi)	STRESS RANGE
1	LT	11.50	155.	-5.2	103.	-1.0	447.	-1.2	0.	0.0	-7.35
											1.17
	LB			5.2		2.6		10.1		0.0	17.87
											10.09
	RT			-4.9		-1.0		-1.3		0.0	-7.18
											1.34
	RB			3.9		2.0		7.7		0.0	13.53
											7.68
1	LT	39.50	155.	-4.8	103.	-1.0	447.	-1.3	0.	0.0	-7.16
											1.34
	LB			3.9		2.0		7.7		0.0	13.52
											7.68
	RT			-5.2		-1.0		-1.2		0.0	-7.34
											1.17
	RB			5.2		2.6		10.1		0.0	17.86
											10.09

NOTE: LT = left-hand-side point, top of top flange
LB = left-hand-side point, bottom of bottom flange
RT = right-hand-side point, top of top flange
RB = right-hand-side point, bottom of bottom flange

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TABLE 1.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000
2	28.00	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	10.00	0.5000
3	11.50	W	11.5	0.7400	31.4	0.5500	11.5	0.7400	11.50	0.0000	11.50	0.0000

NOTE: PG = plate Girder ,W = standard W-section with/without cover plates

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TABLE 1.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 51.0

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH (in)		[2]=OVERALL D. [2]/(12*[1])	<=>	LIMITATION 1/25	STATUS
				[3]= GIRDER D. [3]/(12*[1])			1/30	
1	11.50	6.75	32.90	[2]=	41.18	0.06728	> 0.04000	OK
				[3]=	32.90	0.05376	> 0.03300	OK
2	28.00	6.75	32.90	[2]=	41.18	0.06728	> 0.04000	OK
				[3]=	32.90	0.05376	> 0.03300	OK
3	11.50	6.75	32.90	[2]=	41.18	0.06728	> 0.04000	OK

[3]= 32.90 0.05376 > 0.03300 OK

NOTE : [1] -- Span length or Average of two adjacent span lengths

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

* For composite girders, the minimum overall depth of girder (concrete slab plus haunch & girder) preferably should not be less than 0.040*Length of span (or 0.032*L if continuous spans). and the depth of steel girder alone preferably should not be less than 0.033*Length of span (or 0.027*L if continuous)

[3] -- same as NOTE [2]

** -- cover plates not taken into account

** -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)
****

SP NO	IN D NO	FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	[3]	[4]	2Dc/tw [5]	[6]	Web Cat
1	0	0.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	1	3.19	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	2	6.38	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	3	9.56	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	4	12.75	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	5	15.94	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	6	19.12	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	7	22.31	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	8	25.50	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	9	28.69	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	10	31.87	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	11	35.06	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	12	38.25	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	13	41.44	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	14	44.62	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	15	47.81	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0
1	16	51.00	0	1	31.42	0.550	57.1	150.0	0.00	111.46			0

NOTE:

[1] 0 - No long. stiffeners
1 - long. stiffeners

[2] D/tw limit (Eq. 6.10.2.1-1 or Eq. 6.10.2.1.2-1)

[3] For composite sections in positive flexure, use Article D6.3.2 to calculate Dcp and 2*Dcp/tw
Note: If the plastic N.A. is not in the web, Dcp = 0

[4] 2Dcp/tw limit: 3.76*SQRT(E/Fyc) (Eq.6.10.6.2.2-1)

[5] Use Article D6.3.1 to calculate Dc and 2.*Dc/tw

[6] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D FROM L SUPT	Lo ng [1]	Co mp	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0	0.00	0	1	31.42	0.550	57.13	168.97	2
1	1	3.19	0	1	31.42	0.550	57.13	168.97	2
1	2	6.38	0	1	31.42	0.550	57.13	168.97	2
1	3	9.56	0	1	31.42	0.550	57.13	168.97	2
1	4	12.75	0	1	31.42	0.550	64.85	168.97	2
1	5	15.94	0	1	31.42	0.550	64.85	168.97	2
1	6	19.12	0	1	31.42	0.550	64.85	168.97	2
1	7	22.31	0	1	31.42	0.550	64.85	168.97	2
1	8	25.50	0	1	31.42	0.550	64.85	168.97	2
1	9	28.69	0	1	31.42	0.550	64.85	168.97	2
1	10	31.87	0	1	31.42	0.550	64.85	168.97	2
1	11	35.06	0	1	31.42	0.550	64.85	168.97	2
1	12	38.25	0	1	31.42	0.550	64.85	168.97	2
1	13	41.44	0	1	31.42	0.550	57.13	168.97	2
1	14	44.62	0	1	31.42	0.550	57.13	168.97	2
1	15	47.81	0	1	31.42	0.550	57.13	168.97	2
1	16	51.00	0	1	31.42	0.550	57.13	168.97	2

NOTE:
[1] 0 - No long. stiffeners
1 - long. stiffeners
[2] For non-composite sections, calculate Dc and 2*Dc/tw
[3] 2Dc/tw limit: 5.7*SQRT(E/Fyc) (Eq.6.10.6.2.3-1)

Web Category
0 = compact section
2 = non-compact section
3 = slender section

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TABLE 1.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	1	3.19	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	2	6.38	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	3	9.56	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	4	12.75	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	5	15.94	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	6	19.12	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	7	22.31	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	8	25.50	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	9	28.69	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	10	31.87	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	11	35.06	7.77	12.	11.5	5.2	0.740	0.605	
			7.77	12.	11.5	5.2	0.740	0.605	1.00 0
1	12	38.25	7.77	12.	11.5	5.2	0.740	0.605	

```
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 13  41.44   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 14  44.62   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 15  47.81   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0
  1 16  51.00   7.77 12.  11.5   5.2  0.740  0.605
                7.77 12.  11.5   5.2  0.740  0.605    1.00    0


 ------------------------------------------------------------

   NOTE: [1] = 12.  (Eq. 6.10.2.2-1)
         [2] = D/6  (Eq. 6.10.2.2-2)
         [3] = 1.1tw  (Eq. 6.10.2.2-3)

         For each nodal point, the 1st line checked criteria for top
         flange and the 2nd line checked criteria for bottom flange
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 TABLE  1.2.22.7.0=CB VALUES FOR LATERAL BRACING
                   ****************************

         DIST.   FROM
 LATERAL LEFT    SUPT
 BRACING ----(FT)----   fo      f2     fmid     f1
   NO    FROM      TO  (ksi)   (ksi)   (ksi)   (ksi)     Cb
-------------------------------------------------------------

    1      0.0   7.5   0.000   5.603   3.039   0.475   1.663
                       0.000   0.000   0.000   0.000   1.000
    2      7.5  29.2   5.603  10.244   9.634   9.025   1.058
                       0.000   0.000   0.000   0.000   1.000
    3     29.2  51.0   0.000  10.244   7.249   4.254   1.000
                       0.000   0.000   0.000   0.000   1.000

-------------------------------------------------------------
 Note: The 1st line is for DL case and the 2nd line is for LL case
       f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

       If the flange transition is beyond 20% X unbraced length
       from the smaller end and the ratio of the smaller
       and larger lateral moment of inertia of the flange
       is smaller than 50%, then Cb=1.0.
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 TABLE  1.2.22.7A=FLB AND LTB CATEGORIES
                  **********************

 SP IN D FROM  Co --------- FLB --------- ------------ LTB -------------
 NO NO L SUPT  mp LMDAf LMDApf LMDArf       Lb       Lp       Lr         GOV
         (ft)                          CAT  (ft)     (ft)     (ft)    CAT CAT
-------------------------------------------------------------------------------

  1  0   0.00  1  7.77  11.26  19.84   0     7.50     7.09    26.62   2   2
  1  1   3.19  1  7.77  11.26  19.84   0     7.50     7.09    26.62   2   2
  1  2   6.38  1  7.77  11.26  19.84   0     7.50     7.09    26.62   2   2
  1  3   9.56  1  7.77  11.26  19.84   0    21.75     7.09    26.62   2   2
  1  4  12.75  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1  5  15.94  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1  6  19.12  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1  7  22.31  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1  8  25.50  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1  9  28.69  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1 10  31.87  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1 11  35.06  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1 12  38.25  1  7.77  11.26  19.84   0    21.75     6.97    26.17   2   2
  1 13  41.44  1  7.77  11.26  19.84   0    21.75     7.09    26.62   2   2
```

1 14	44.62	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1 15	47.81	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2
1 16	51.00	1	7.77	11.26	19.84	0	21.75	7.09	26.62	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural
of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of
nominal yielding in either flange under uniform
bending with consideration of comp. flange
residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

In negative moment region, the fist line is for non-composite sections
and the second line is for composite sections

Flange Category
0 = compact section
2 = non-compact section
3 = slender section

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE
****

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0	0.00	1	1.000	1.000	1.66	33.00	33.00	33.00
1	1	3.19	1	1.000	1.000	1.66	33.00	33.00	33.00
1	2	6.38	1	1.000	1.000	1.66	33.00	33.00	33.00
1	3	9.56	1	1.000	1.000	1.06	33.00	27.05	27.05
1	4	12.75	1	1.000	1.000	1.06	33.00	26.85	26.85
1	5	15.94	1	1.000	1.000	1.06	33.00	26.85	26.85
1	6	19.12	1	1.000	1.000	1.06	33.00	26.85	26.85
1	7	22.31	1	1.000	1.000	1.06	33.00	26.85	26.85
1	8	25.50	1	1.000	1.000	1.06	33.00	26.85	26.85
1	9	28.69	1	1.000	1.000	1.06	33.00	26.85	26.85
1	10	31.87	1	1.000	1.000	1.00	33.00	25.38	25.38
1	11	35.06	1	1.000	1.000	1.00	33.00	25.38	25.38
1	12	38.25	1	1.000	1.000	1.00	33.00	25.38	25.38
1	13	41.44	1	1.000	1.000	1.00	33.00	25.57	25.57
1	14	44.62	1	1.000	1.000	1.00	33.00	25.57	25.57
1	15	47.81	1	1.000	1.000	1.00	33.00	25.57	25.57
1	16	51.00	1	1.000	1.000	1.00	33.00	25.57	25.57

Note: In the positive moment region, the result is for DL case
In the negative moment region, the 1st line is for DL case
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFOMATION FOR DUCTILITY CHECK
****

SP NO	IN NO	D FROM L SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0	0.00	1	6.34	17.29
1	1	3.19	1	6.34	17.29
1	2	6.38	1	6.34	17.29
1	3	9.56	1	6.34	17.29
1	4	12.75	1	8.40	17.50
1	5	15.94	1	8.40	17.50
1	6	19.12	1	8.40	17.50
1	7	22.31	1	8.40	17.50
1	8	25.50	1	8.40	17.50
1	9	28.69	1	8.40	17.50
1	10	31.87	1	8.40	17.50
1	11	35.06	1	8.40	17.50
1	12	38.25	1	8.40	17.50
1	13	41.44	1	6.34	17.29
1	14	44.62	1	6.34	17.29
1	15	47.81	1	6.34	17.29
1	16	51.00	1	6.34	17.29

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

SP NO	IN NO	D FROM L SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.	Comp.
1	0	0.00	1	0	2	0		2	0
1	1	3.19	1	0	2	0		2	0
1	2	6.38	1	0	2	0		2	0
1	3	9.56	1	0	2	0		2	0
1	4	12.75	1	0	2	0		2	0
1	5	15.94	1	0	2	0		2	0
1	6	19.12	1	0	2	0		2	0
1	7	22.31	1	0	2	0		2	0
1	8	25.50	1	0	2	0		2	0
1	9	28.69	1	0	2	0		2	0
1	10	31.87	1	0	2	0		2	0
1	11	35.06	1	0	2	0		2	0
1	12	38.25	1	0	2	0		2	0
1	13	41.44	1	0	2	0		2	0
1	14	44.62	1	0	2	0		2	0
1	15	47.81	1	0	2	0		2	0
1	16	51.00	1	0	2	0		2	0

Please read NOTE on the following page

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NOTE: [1] For non-conposite sections, check Eq.6.10.6.2.3-1
[2] For non-composite FLB/LTB, use Article 6.10.8.2 to decide section category
[3] For composite sections in positive flexure, use Article 6.10.6.2.2 to decide section category
For composite sections in negative flexure, use Eq. 6.10.6.2.3-1
[4] For composite FLB or LTB, use Article 6.10.8.2 to compute

section category

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

FOR N = INF. : Category No. is the maximum of [1] and [2]
FOR N = n : Category No. is [3] for composite sections in positive flexure
Category No. is the maximum of [3] and [4] for composite sections in negative flexure

Non-compact for Fy of the flanges > 70 ksi (483 MPa)

1

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TABLE 1.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP	IN	D	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	33.0	0
				0.0	-	0.0	-	0.0	33.0	-	-	0
1	1		3.19	0.0	19.8	2.6	33.0	2.6	33.0	2.6	33.0	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	2		6.38	0.0	19.8	4.9	33.0	4.9	33.0	4.9	33.0	0
				0.0	-	4.9	-	4.9	33.0	-	-	0
1	3		9.56	0.0	19.8	6.8	33.0	6.8	33.0	6.8	27.0	0
				0.0	-	6.8	-	6.8	33.0	-	-	0
1	4		12.75	0.0	19.8	7.9	33.0	7.9	33.0	7.9	26.8	0
				0.0	-	6.3	-	6.3	33.0	-	-	0
1	5		15.94	0.0	19.8	9.0	33.0	9.0	33.0	9.0	26.8	0
				0.0	-	7.2	-	7.2	33.0	-	-	0
1	6		19.12	0.0	19.8	9.8	33.0	9.8	33.0	9.8	26.8	0
				0.0	-	7.9	-	7.9	33.0	-	-	0
1	7		22.31	0.0	19.8	10.3	33.0	10.3	33.0	10.3	26.8	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	8		25.50	0.0	19.8	10.5	33.0	10.5	33.0	10.5	26.8	0
				0.0	-	8.4	-	8.4	33.0	-	-	0
1	9		28.69	0.0	19.8	10.3	33.0	10.3	33.0	10.3	26.8	0
				0.0	-	8.3	-	8.3	33.0	-	-	0
1	10		31.87	0.0	19.8	9.8	33.0	9.8	33.0	9.8	25.4	0
				0.0	-	7.9	-	7.9	33.0	-	-	0
1	11		35.06	0.0	19.8	9.0	33.0	9.0	33.0	9.0	25.4	0
				0.0	-	7.2	-	7.2	33.0	-	-	0
1	12		38.25	0.0	19.8	7.9	33.0	7.9	33.0	7.9	25.4	0
				0.0	-	6.3	-	6.3	33.0	-	-	0
1	13		41.44	0.0	19.8	6.8	33.0	6.8	33.0	6.8	25.6	0
				0.0	-	6.8	-	6.8	33.0	-	-	0
1	14		44.62	0.0	19.8	4.9	33.0	4.9	33.0	4.9	25.6	0
				0.0	-	4.9	-	4.9	33.0	-	-	0
1	15		47.81	0.0	19.8	2.6	33.0	2.6	33.0	2.6	25.6	0
				0.0	-	2.6	-	2.6	33.0	-	-	0
1	16		51.00	0.0	19.8	0.0	33.0	0.0	33.0	0.0	25.6	0
				0.0	-	0.0	-	0.0	33.0	-	-	0

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

"-" is N.A.
Under FLAG Column, 0 = OK; 1= NG

For each nodal point, the 1st line checked criteria for top flange and the 2nd line checked criteria for bottom flange

The values of fbu and fl shall be determined based on
factored loads, and shall be taken as positive in
sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table,
the users can use the maximum value within the
unbraced length to do their own check

1

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TABLE 1.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP	IN	D FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	3.19	0.000	0.080	0.080	0.080	0.080	2
					0.080		0.080	3
1	2	6.38	0.000	0.149	0.149	0.149	0.149	2
					0.149		0.149	3
1	3	9.56	0.000	0.208	0.208	0.253	0.253	4
					0.208		0.208	3
1	4	12.75	0.000	0.238	0.238	0.293	0.293	4
					0.191		0.191	3
1	5	15.94	0.000	0.273	0.273	0.336	0.336	4
					0.218		0.218	3
1	6	19.12	0.000	0.298	0.298	0.366	0.366	4
					0.238		0.238	3
1	7	22.31	0.000	0.313	0.313	0.384	0.384	4
					0.250		0.250	3
1	8	25.50	0.000	0.318	0.318	0.391	0.391	4
					0.254		0.254	3
1	9	28.69	0.000	0.313	0.313	0.385	0.385	4
					0.251		0.251	3
1	10	31.87	0.000	0.298	0.298	0.387	0.387	4
					0.238		0.238	3
1	11	35.06	0.000	0.273	0.273	0.355	0.355	4
					0.218		0.218	3
1	12	38.25	0.000	0.238	0.238	0.309	0.309	4
					0.190		0.190	3
1	13	41.44	0.000	0.207	0.207	0.267	0.267	4
					0.207		0.207	3
1	14	44.62	0.000	0.148	0.148	0.192	0.192	4
					0.148		0.148	3
1	15	47.81	0.000	0.079	0.079	0.103	0.103	4
					0.079		0.079	3
1	16	51.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

NOTE: [1] = (PHI)f * Fcrw (Eq. 6.10.3.2.1-3)
[2] = (PHI)f * Rh * Fyc (Eq. 6.10.3.2.1-1) or
= (PHI)f * Rh * Fyt (Eq. 6.10.3.2.2-1)
[3] = (PHI)f * Fnc (Eq. 6.10.3.2.1-2)

The governing number is listed as below.
1 = fl / 0.6Fyt
2 = fbu / [1]
3 = fbu + fl / [2]
4 = (fbu + 1/3fl) / [3]

1

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TABLE 1.2.22.14=STRENGTH LIMIT STATE CHECK

****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	1956.3	-	-	0.0	19.8	-	-	0--0
1	1		3.19	0	391.4	1956.3	-	-	0.0	19.8	-	-	0--0
1	2		6.38	0	723.4	1956.3	-	-	0.0	19.8	-	-	0--0
1	3		9.56	0	995.5	1956.3	-	-	0.0	19.8	-	-	0--0
1	4		12.75	0	1204.2	2326.4	-	-	0.0	19.8	-	-	0--0
1	5		15.94	0	1363.7	2326.4	-	-	0.0	19.8	-	-	0--0
1	6		19.12	0	1475.1	2326.4	-	-	0.0	19.8	-	-	0--0
1	7		22.31	0	1524.1	2326.4	-	-	0.0	19.8	-	-	0--0
1	8		25.50	0	1536.6	2326.4	-	-	0.0	19.8	-	-	0--0
1	9		28.69	0	1524.4	2326.4	-	-	0.0	19.8	-	-	0--0
1	10		31.87	0	1475.2	2326.4	-	-	0.0	19.8	-	-	0--0
1	11		35.06	0	1363.7	2326.4	-	-	0.0	19.8	-	-	0--0
1	12		38.25	0	1203.9	2326.4	-	-	0.0	19.8	-	-	0--0
1	13		41.44	0	994.9	1956.3	-	-	0.0	19.8	-	-	0--0
1	14		44.62	0	722.8	1956.3	-	-	0.0	19.8	-	-	0--0
1	15		47.81	0	391.1	1956.3	-	-	0.0	19.8	-	-	0--0
1	16		51.00	0	0.0	1956.3	-	-	0.0	19.8	-	-	0--0

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility

For negative moment region or non-compact sections in positive moment region, the 1st line is for top flange and the 2nd line is for bottom flange

The values of fbu, Mu and fl shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations (Art. 6.10.1.6)

The value of fbu is the actual stress in this table, the users can use the maximum value within the unbraced length to do their own check

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TABLE 1.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY

****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0		0.00	0	0.000				0.000	1
1	1		3.19	0	0.200				0.200	1
1	2		6.38	0	0.370				0.370	1
1	3		9.56	0	0.509				0.509	1
1	4		12.75	0	0.518				0.518	1

1	5	15.94	0	0.586	0.586	1
1	6	19.12	0	0.634	0.634	1
1	7	22.31	0	0.655	0.655	1
1	8	25.50	0	0.661	0.661	1
1	9	28.69	0	0.655	0.655	1
1	10	31.87	0	0.634	0.634	1
1	11	35.06	0	0.586	0.586	1
1	12	38.25	0	0.517	0.517	1
1	13	41.44	0	0.509	0.509	1
1	14	44.62	0	0.369	0.369	1
1	15	47.81	0	0.200	0.200	1
1	16	51.00	0	0.000	0.000	1

NOTE: [1] = (PHI)f * Mn (Eq. 6.10.7.1.1-1)
[2] = (PHI)f * Fnc for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)
= (PHI)f * Rh * Fyt for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)
[3] = 0.6*Fyt for composite sections in positive flexure or tension flange for composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
= 0.6*Fyc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)
[4] = (PHI)f * Fnt for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)
= (PHI)f * Fnc for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.1-1)

The governing number is listed as below.
1 = (Mu+1/3flSxt) / [1]
2 = fbu / [2]
3 = fl / [3]
4 = (fbu + 1/3fl) / [4]

1

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
****

SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	[3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	1	3.19	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	2	6.38	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	3	9.56	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	4	12.75	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	5	15.94	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	6	19.12	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	7	22.31	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	8	25.50	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	9	28.69	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	10	31.87	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	11	35.06	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	12	38.25	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	13	41.44	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	14	44.62	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	15	47.81	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8
1	16	51.00	33.0	5.0	74.2	92.8	57.1	1.000	330.8	330.8

Please read NOTE on the following page

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```
NOTE: [1] k= buckling coefficient = 5+5/((d0/D)**2)
           = 5 for unstiffened beams and girders
           = 5 for stiffened girders when (d0/D) > 3, or
                                          (d0/D) > (260/(D/tw))**2

      [2] = 1.12 * sqrt(Ek / Fyw)

      [3] = 1.40 * sqrt(Ek / Fyw)

      [4] C = ratio of nominal shear resistance and plastic shear force, Vp

         For D/tw < [2], C = 1.0

         For [2] <= D/tw <= [3], C = 1.12 * sqrt(Ek / Fyw) / (D/tw)
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-6

         For D/tw > [3], C = 1.57 * (Ek / Fyw) / (D/tw)^2
                                      ..... AASHTO LRFD Eq.6.10.9.3.2-7


      ** AASHTO LRFD Art.6.10.9.1 --- For interior web panels considered
         stiffened
         if (d0/D) < 3 without long. stiffener, or
         if (d0/D) > 3 with both transv. and long. stiffeners.
         For handling requirement, (d0/D) > (260/(D/tw))**2

              Vu = CVp

         where D = clear, unsupported distance between flange components

              d0 = distance between transverse stiffeners

      ** This Article (6.10.9.2) indicates that a designer cannot
         count on post-buckling shear resistance from tension-
         field action for an unstiffened girder.
```

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

S T R E N G T H C A T E G O R Y; SEE NOTE

SP NO	IN NO	D FROM L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	129.3	<	330.8	0	0
1	1	3.19	0	116.9	<	330.8	0	0
1	2	6.38	0	104.6	<	330.8	0	0
1	3	9.56	0	92.2	<	330.8	0	0
1	4	12.75	0	80.2	<	330.8	0	0
1	5	15.94	0	68.2	<	330.8	0	0
1	6	19.12	0	56.4	<	330.8	0	0
1	7	22.31	0	44.9	<	330.8	0	0
1	8	25.50	0	33.7	<	330.8	0	0
1	9	28.69	0	44.8	<	330.8	0	0
1	10	31.87	0	56.5	<	330.8	0	0
1	11	35.06	0	68.3	<	330.8	0	0
1	12	38.25	0	80.2	<	330.8	0	0
1	13	41.44	0	92.3	<	330.8	0	0
1	14	44.62	0	104.5	<	330.8	0	0
1	15	47.81	0	116.8	<	330.8	0	0
1	16	51.00	0	129.2	<	330.8	0	0

Please read NOTE on the following page

NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

D/tw <= 150 (Eq. 6.10.2.1.1-1)

w / long. stiff.

D/tw <= 300 (Eq. 6.10.2.1.2-1)

2Dcp/tw .LE. 3.76*sqrt(E/Fyc) ..... AASHTO Eq.6.10.6.2.2-1

For compact or non-compact sections specified in Art. 6.10.8

(composite sections in negative flexure and
non-composite sections)

2Dc/tw .LE. 5.7*SQRT(E/Fyc) ..... AASHTO Eq.6.10.6.2.3-1

[2]: SHEAR CAPACITY ... web crippling control

Vn = CVp ..... AASHTO Eq.6.10.9.2-1

Vn = VpRc1 ..... AASHTO Eq.6.10.9.3.2-2

Rc1 = C+[0.87(1-C)]/sqrt(1+(d0/D)**2) for 2Dtw/(bfctfc+bfttft) <=2.5

Vn = VpRc2 ..... AASHTO Eq.6.10.9.3.2-8

Rc2 = C+[0.87(1-C)]/[sqrt(1+(d0/D)**2)+(d0/D)] for 2Dtw/(bfctfc+bfttft) > 2.5

Vp = 0.58*Fy*D*tw ..... AASHTO Eq.6.10.9.3.2-2

* For the detailed description of shear capacity,
please refer to AASHTO Art.6.10.9.2 and .3

* STRENGTH CATEGORY; 0 = compact section
2 = braced non-compact section
3 = slender section

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	129.31	330.76	0	
1	1	3.19	33.	116.88	330.76	0	
1	2	6.38	33.	104.58	330.76	0	
1	3	9.56	33.	92.22	330.76	0	
1	4	12.75	33.	80.17	330.76	0	
1	5	15.94	33.	68.21	330.76	0	
1	6	19.12	33.	56.38	330.76	0	
1	7	22.31	33.	44.87	330.76	0	
1	8	25.50	33.	33.68	330.76	0	
1	9	28.69	33.	44.77	330.76	0	
1	10	31.87	33.	56.46	330.76	0	
1	11	35.06	33.	68.29	330.76	0	
1	12	38.25	33.	80.25	330.76	0	
1	13	41.44	33.	92.30	330.76	0	
1	14	44.62	33.	104.48	330.76	0	
1	15	47.81	33.	116.78	330.76	0	
1	16	51.00	33.	129.20	330.76	0	

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     TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
                     ************************

     SP IN D FROM    TOP FLANGE      BOT. FLANGE    COMP. FLG
                  ff   ff+fl/2  [1] ff+fl/2  [2]    fc   fcrw FLAG
     NO NO L SUPT                                          [3]  TBC
             (ft) (ksi)  (ksi)  (ksi)  (ksi)  (ksi) (ksi)
     ---------------------------------------------------------------

      1  0   0.00   0.0    -     31.4     0.0  31.4   -     -   00-
      1  1   3.19   2.6    -     31.4     7.3  31.4   -     -   00-
      1  2   6.38   4.9    -     31.4    13.4  31.4   -     -   00-
      1  3   9.56   6.8    -     31.4    18.5  31.4   -     -   00-
      1  4  12.75   8.2    -     31.4    17.0  31.4   -     -   00-
      1  5  15.94   9.3    -     31.4    19.3  31.4   -     -   00-
      1  6  19.12  10.1    -     31.4    20.9  31.4   -     -   00-
      1  7  22.31  10.6    -     31.4    21.6  31.4   -     -   00-
      1  8  25.50  10.7    -     31.4    21.8  31.4   -     -   00-
      1  9  28.69  10.6    -     31.4    21.6  31.4   -     -   00-
      1 10  31.87  10.1    -     31.4    20.9  31.4   -     -   00-
      1 11  35.06   9.3    -     31.4    19.3  31.4   -     -   00-
      1 12  38.25   8.2    -     31.4    17.0  31.4   -     -   00-
      1 13  41.44   6.8    -     31.4    18.5  31.4   -     -   00-
      1 14  44.62   4.9    -     31.4    13.4  31.4   -     -   00-
      1 15  47.81   2.6    -     31.4     7.3  31.4   -     -   00-
      1 16  51.00   0.0    -     31.4     0.0  31.4   -     -   00-


     ---------------------------------------------------------------
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      NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
                = 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
            [3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
                  but not to exceed the smaller of RhFyc and Fyw/0.7
                  k = bending-buckling coefficient
                    = 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
                  where:
                  Dc = depth of the web in compression in the elastic
                       range. For composite sections, Dc shall be
                       determined as specified in Article D6.3.1

            "-" is N.A.

            Flag check - 0 = OK; 1 = NG
            T = Top Flange; B = Bottom Flange; C = Comp. Flange

            The values of fl shall be determined based on
            factored loads, and shall be taken as positive in
            sign in all resistance equations (Art. 6.10.1.6)
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 TABLE 1.2.22.23.1=FATIGUE I STRESS RANGE FOR INFINITE LIFE (FACTORED)
                   ****************************************

     (1) Main (Longitudinal) Load Carrying Members
```

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.19	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	2	6.38	TR	-0.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	3	9.56	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1		11.50	SCG					
1	4	12.75	TR	-0.7	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	5	15.94	TR	-0.8	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	6	19.12	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	7	22.31	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	8	25.50	TR	-0.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	9	28.69	TR	-0.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	10	31.87	TR	-0.8	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	11	35.06	TR	-0.8	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	12	38.25	TR	-0.7	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1		39.50	SCG					
1	13	41.44	TR	-0.5	A B B^C^C D E E^	TR	4.5	A B B^C^C D
1	14	44.62	TR	-0.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	15	47.81	TR	-0.2	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	16	51.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

NOTE: Negative sign means live load stresses all in compression or the permanent load compressive stress more than twice the max. live load tensile stress.

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.5 load factor. Design for Infinite Life

NOTE: ITEM ; INT = Span interval point
SCG = Section-change point
POC = Dead load point of contraflexure

I:P = Point where INT coincides with POC
I:C = Point where INT coincides with SCG
S:P = Point where SCG coincides with POC

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TABLE 1.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = I --- Rural Interstate

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	3.19	TR	-0.1	A B B^C^C D E E^	TR	0.9	A B B^C^C D E E^
1	2	6.38	TR	-0.2	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	3	9.56	TR	-0.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1		11.50	SCG					
1	4	12.75	TR	-0.4	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	5	15.94	TR	-0.4	A B B^C^C D E E^	TR	2.2	A B B^C^C D E E^
1	6	19.12	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1	7	22.31	TR	-0.4	A B B^C^C D E E^	TR	2.4	A B B^C^C D E

```
 1  8  25.50    TR     -0.4 A B B^C^C D E E^  TR      2.5 A B B^C^C D E
 1  9  28.69    TR     -0.4 A B B^C^C D E E^  TR      2.4 A B B^C^C D E
 1 10  31.87    TR     -0.4 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1 11  35.06    TR     -0.4 A B B^C^C D E E^  TR      2.2 A B B^C^C D E E^
 1 12  38.25    TR     -0.4 A B B^C^C D E E^  TR      2.0 A B B^C^C D E E^
 1     39.50  SCG
 1 13  41.44    TR     -0.3 A B B^C^C D E E^  TR      2.3 A B B^C^C D E E^
 1 14  44.62    TR     -0.2 A B B^C^C D E E^  TR      1.7 A B B^C^C D E E^
 1 15  47.81    TR     -0.1 A B B^C^C D E E^  TR      0.9 A B B^C^C D E E^
 1 16  51.00    TR      0.0 A B B^C^C D E E^  TR      0.0 A B B^C^C D E E^


 ------------------------------------------------------------------------------
 NOTE: Negative sign means live load stresses all in compression or the permanent
       load compressive stress more than twice the max. live load tensile stress.
 ------------------------------------------------------------------------------
 NOTE: TR = Truck loading;  LRFD Fatigue II Limit State with 0.75 load factor.
       Design for Finite Life w/ ADTT Single Lane= 1140 & No. of Cycles= 31207500
     * If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

 NOTE: ITEM ; INT = Span interval point
              SCG = Section-change point
              POC = Dead load point of contraflexure

              I:P = Point where INT coincides with POC
              I:C = Point where INT coincides with SCG
              S:P = Point where SCG coincides with POC

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 TABLE 1.2.22.24.1=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE I (FACTORED)
                   *******************************************

                  AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN.  MAX. ALLOW.
 SP IN D FROM ----------------------  6.10.10.2-1     PER TRANS.   SHEAR CONN.
 NO NO L SUPT   Vf    Q/I      Vsr  ----------------   SECTION    ---PITCH----
        (ft)   [1]    [2]      [3]    [4],Zr <--Input [5], input     (in),[6]
 ------------------------------------------------------------------------------

  1  0   0.00  31.1 .29013E-01    0.9      TR    3.09        3          10.28
  1  1   3.19  29.4 .29013E-01    0.9      TR    3.09        3          10.88
  1  2   6.38  27.8 .29013E-01    0.8      TR    3.09        3          11.50
  1  3   9.56  26.3 .29013E-01    0.8      TR    3.09        3          12.14
  1  4  12.75  25.0 .27358E-01    0.7      TR    3.09        3          13.56
  1  5  15.94  23.7 .27358E-01    0.6      TR    3.09        3          14.32
  1  6  19.12  22.6 .27358E-01    0.6      TR    3.09        3          15.04
  1  7  22.31  23.4 .27358E-01    0.6      TR    3.09        3          14.53
  1  8  25.50  23.9 .27358E-01    0.7      TR    3.09        3          14.17
  1  9  28.69  23.4 .27358E-01    0.6      TR    3.09        3          14.53
  1 10  31.87  22.6 .27358E-01    0.6      TR    3.09        3          15.04
  1 11  35.06  23.7 .27358E-01    0.6      TR    3.09        3          14.32
  1 12  38.25  25.0 .27358E-01    0.7      TR    3.09        3          13.56
  1 13  41.44  26.3 .29013E-01    0.8      TR    3.09        3          12.14
  1 14  44.62  27.8 .29013E-01    0.8      TR    3.09        3          11.50
  1 15  47.81  29.4 .29013E-01    0.9      TR    3.09        3          10.88
  1 16  51.00  31.1 .29013E-01    0.9      TR    3.09        3          10.28


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  NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                 section, the range of shear shall be taken as the difference
                 in the minimum and maximum shear envelopes (excluding dead
                 loads); AASHTO LRFD Eqn.6.10.10.1.2-2
```

[2] Q/I : Q= statical moment about the neutral axis of the composite section of the transformed compressive concrete area or the area of reinforcement embedded in the concrete for negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

I= moment of inertia of the transformed composite girder in positive moment regions or the moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, in inches to the fourth power.

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.
= SQRT(Vfat^2 + Ffat^2) (Eq. 6.10.10.1.2-2)
Vfat = Vf*Q/I (Eq. 6.10.10.1.2-3)
Ffat = radial fatigue shear range per unit length
= Ffat2 = Frc/w for skews exceeding 20 degrees
Frc = 25.0 kips for both exterior and interior girders
w = the effective length of the deck
= 48 inches except at end supports where w is taken as 24 inches

[4] Zr = allowable range of horizontal shear, in Kips, on an individual connector, AASHTO LRFD Eqn.6.10.10.2-1 for Fatigue I

[5] No. of shear connectors per transverse section. This value is from input data type 12032.

[6] Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

** The connector spacing shown here is calculated based upon AASHTO Fatigue Combinations I where ADTT Single Lane= 1140
*** If ADTT Single Lane is less than 960, use Fatigue II Table.

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TABLE 1.2.22.24.2=SHEAR CONNECTOR (FATIGUE CRITERIA) FATIGUE II (FACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	Vf [1]	Q/I [2]	Vsr [3]	AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. 6.10.10.2-2 [4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	15.6	.29013E-01	0.5	TR 1.36	3	9.06
1	1	3.19	14.7	.29013E-01	0.4	TR 1.36	3	9.59
1	2	6.38	13.9	.29013E-01	0.4	TR 1.36	3	10.14
1	3	9.56	13.2	.29013E-01	0.4	TR 1.36	3	10.70
1	4	12.75	12.5	.27358E-01	0.3	TR 1.36	3	11.96
1	5	15.94	11.8	.27358E-01	0.3	TR 1.36	3	12.63
1	6	19.12	11.3	.27358E-01	0.3	TR 1.36	3	13.26
1	7	22.31	11.7	.27358E-01	0.3	TR 1.36	3	12.81
1	8	25.50	12.0	.27358E-01	0.3	TR 1.36	3	12.49
1	9	28.69	11.7	.27358E-01	0.3	TR 1.36	3	12.81
1	10	31.87	11.3	.27358E-01	0.3	TR 1.36	3	13.26
1	11	35.06	11.8	.27358E-01	0.3	TR 1.36	3	12.63
1	12	38.25	12.5	.27358E-01	0.3	TR 1.36	3	11.96
1	13	41.44	13.2	.29013E-01	0.4	TR 1.36	3	10.70
1	14	44.62	13.9	.29013E-01	0.4	TR 1.36	3	10.14
1	15	47.81	14.7	.29013E-01	0.4	TR 1.36	3	9.59
1	16	51.00	15.6	.29013E-01	0.5	TR 1.36	3	9.06

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 NOTE: [1] Vf = range of shear due to live loads and impact in Kips; at any
                section, the range of shear shall be taken as the difference
                in the minimum and maximum shear envelopes (excluding dead
                loads); AASHTO LRFD Eqn.6.10.10.1.2-2

       [2] Q/I : Q= statical moment about the neutral axis of the composite
                   section of the transformed compressive concrete area or
                   the area of reinforcement embedded in the concrete for
                   negative moment, in cubic inches; AASHTO LRFD Eqn.6.10.10.1.2-3

                 I= moment of inertia of the transformed composite girder in
                   positive moment regions or the moment of inertia provided
                   by the steel beam including or excluding the area of
                   reinforcement embedded in the concrete in negative moment
                   regions, in inches to the fourth power.

       [3] Vsr = range of horizontal shear, in Kips per inches, at the junction
                 of the slab and girder at the point in the span under
                 consideration.
               = SQRT(Vfat^2 + Ffat^2)   (Eq. 6.10.10.1.2-2)
           Vfat = Vf*Q/I   (Eq. 6.10.10.1.2-3)
           Ffat = radial fatigue shear range per unit length
                = Ffat2 = Frc/w for skews exceeding 20 degrees
           Frc = 25.0 kips for both exterior and interior girders
           w = the effective length of the deck
             = 48 inches except at end supports where w is taken
               as 24 inches

       [4] Zr = allowable range of horizontal shear, in Kips, on an individual
                connector, AASHTO LRFD Eqn.6.10.10.2-2 for Fatigue II

           * Default ALPHA value based on 7/8" diameter and input road type


       [5] No. of shear connectors per transverse section. This value is from
           input data type 12032.

       [6]  Maximum allowable pitch = 24 inches, AASHTO LRFD ART. 6.10.10.1.2

           ** The connector spacing shown here is calculated based upon
              AASHTO Fatigue Combination II. For II, see AASHTO 6.10.10.1.2
              where ADTT Single Lane= 1140 & No. of Cycles=   31207500
          *** If ADTT Single Lane greater than or equal to 960, refer
              to Fatigue I Table.
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```
 TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
                 ************************************

                 MOMENT
                 REGION P2p,1n P1p   P2n    NO OF SHEAR     SEE NOTE  STATUS
     SP IN D FROM ------ ----- ----- ------  CONNECTOR      -------- --------
     NO NO L SUPT 1=POS.  (k)   (k)   (k)   N(PER ZONE) <=>  N1  N2  BLANK=OK
            (ft)  0=NEG.  LRFD 6.10.10.4.2  FATIG. CRI.               CHECK=**
     ---------------------------------------------------------------------------

      1  8  25.50   1 1296.9  1204.9           93         >  63
      1 16  51.00   1 1296.9  1204.9           93         >  63


     ---------------------------------------------------------------------------
      Please read NOTE on the Following Page

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Strength Limit State (Article 6.10.10.4)

[1] The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment, or between each adjacent point of 0.0 moment and centerline of an interior support shall be checked to ensure that adequate connectors are provided for Strength Limit State.

[2] The number of shear connectors required equal or exceed the number given by the formula :

N1 (N2) = P / Qr - - - - - - (Eq. 6.10.10.4.1-2)

Where
N1 = Number of connectors between points of maximum positive moment and each adjacent point of 0.0 moment
N2 = number of connectors between each adjacent point of 0.0 moment and the centerline of an interior support
P = total nominal horizontal shear force
Qr = factored shear resistance of one shear connector
= (Phi)sc Qn - - - - - - (Eq. 6.10.10.4.1-1)
Qn = nominal shear resistance
(Phi)sc = resistance factor for shear connectors

[3] The total horizontal shear force between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser either:

P1p = 0.85 f'c bs ts - - - - - - (Eq. 6.10.10.4.2-2)

or

P2p,1n = Fyw D tw + Fyt bft tft + Fyc bfc tfc (Eq. 6.10.10.4.2-3,7)

Where
f'c = specified 28-day compressive strength of the concrete
bs = effective width of the slab
bfc = width of compression flange
bft = width of tension flange
ts = slab thickness
Fyw = specified minimum yield strength of the web
Fyt = specified minimum yield strength of the tension flange
Fyc = specified minimum yield strength of the compression flange
D = web depth
tft = thickness of tension flange
tfc = thickness of compression flange
tw = web thickness

[4] For continuous-span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

P2n = 0.45 f'c bs ts- - - - - - (Eq. 6.10.10.4.2-8)

Where
bs = effective width of the concrete deck
ts = thickness of a concrete deck

[5] If ADTT Single Lane is greater than 960, the pitch from Fatigue I will be used.
If ADTT Single Lane is less than 960, the pitch should use Fatigue II.

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TABLE 1.2.22.24B=RECOMMANDED SHEAR CONNECTOR REQUIRED PITCH
****

SPAN NO.	CURRENT SPAN FROM (ft)	TO (ft)	REQUIRED PITCH (in)
1	0.000	25.500	10.000
1	25.500	51.000	10.000

```
                ----------------------------------
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 TABLE 1.2.22.24C=TENSION-COMPRESSION-REVERSAL AREAS
                 *********************************

                 SPAN     CURRENT SPAN    T
                  NO.     FROM      TO    C
                          (ft)     (ft)   R
                ---------------------------

                    1       0.0      51.0 C

                ---------------------------

                NOTE: T: TENSION AREA, C: COMPRESSION AREA,
                R: REVERSAL AREA BASED ON TOTAL TOP FLANGE
                STRESSES
1
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     TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY
                    ************************

 STATUS                 TABLE NO.
-------------------------------------------------------------------------------

  OK     1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION

  OK     1.2.22.4=DEPTH RATIOS

  OK     1.2.22.10=CONSTRUCBILITY CHECK

  OK     1.2.22.14=STRENGTH LIMIT STATE CHECK

  OK     1.2.22.17=TRANSVERSE STIFFENER SPACING

  OK     1.2.22.21=SERVICE LIMIT STATE CHECK


         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
1
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 TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
                 **********************************

                            UNFACTORED   UNFACTORED L+I
                            DEAD LOAD    --------------  OPERAT.       INVENT. C
 SP IN D FROM  MOMENT        MOMENT       MAX. MOMENT    RATING  LIVE  RATING  A
 NO NO L SUPT  CAP.                       --------------- FACTOR  LOAD  FACTOR  T
        (ft)  (k-ft)       D1; k-ft  D2     POS.     NEG.  [3]   TYPE
            or STRESS          ksi        MAX. STRESS
-------------------------------------------------------------------------------

  1  0   0.00   1956.3      0.0      0.0      0.0      0.0  9.99  HL-93  9.99  0
```

1	1	3.19	1956.3	52.5	34.7	158.2	0.0	8.62	HL-93	6.65	0
1	2	6.38	1956.3	98.1	64.7	291.1	0.0	4.43	HL-93	3.42	0
1	3	9.56	1956.3	136.5	90.1	398.7	0.0	3.08	HL-93	2.38	0
1	4	12.75	2326.4	167.8	110.9	478.9	0.0	3.03	HL-93	2.34	0
1	5	15.94	2326.4	192.1	127.1	539.6	0.0	2.62	HL-93	2.02	0
1	6	19.12	2326.4	209.5	138.7	581.4	0.0	2.38	HL-93	1.84	0
1	7	22.31	2326.4	220.1	145.6	596.3	0.0	2.29	HL-93	1.77	0
1	8	25.50	2326.4	223.7	147.9	599.1	0.0	2.27	HL-93	1.75	0
1	9	28.69	2326.4	220.3	145.6	596.3	0.0	2.29	HL-93	1.77	0
1	10	31.87	2326.4	209.7	138.7	581.4	0.0	2.38	HL-93	1.84	0
1	11	35.06	2326.4	192.0	127.1	539.6	0.0	2.62	HL-93	2.02	0
1	12	38.25	2326.4	167.5	110.9	478.9	0.0	3.03	HL-93	2.34	0
1	13	41.44	1956.3	136.0	90.1	398.7	0.0	3.08	HL-93	2.38	0
1	14	44.62	1956.3	97.6	64.7	291.1	0.0	4.43	HL-93	3.42	0
1	15	47.81	1956.3	52.3	34.7	158.2	0.0	8.62	HL-93	6.65	0
1	16	51.00	1956.3	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0

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NOTE [1]: GENERAL LOAD-RATING PROCEDURES ARE IN AASHTO LRFR 6.4

---> BASED ON LRFD , STRENGTH LIMIT STATE CRITERIA

[2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: LRFD SECTION IS RATED BY
RF=(Mu-ENS1*(GDC*D1+GDW*D2)))/ENS1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-ENS1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/ENS1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

[5]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT		UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT		MAX. NEG. --- STEEL--- TOP;ksi;BOT		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	3.19	31.4	-2.09	2.63	-0.41	3.57	0.00	0.00	8.05	HL-93	6.19
1	2	6.38	31.4	-3.90	4.91	-0.76	6.57	0.00	0.00	4.03	HL-93	3.10
1	3	9.56	31.4	-5.42	6.83	-1.04	8.99	0.00	0.00	2.73	HL-93	2.10
1	4	12.75	31.4	-6.30	6.31	-1.43	8.22	0.00	0.00	3.05	HL-93	2.34
1	5	15.94	31.4	-7.21	7.23	-1.62	9.26	0.00	0.00	2.60	HL-93	2.00
1	6	19.12	31.4	-7.87	7.88	-1.74	9.98	0.00	0.00	2.35	HL-93	1.81
1	7	22.31	31.4	-8.26	8.28	-1.79	10.24	0.00	0.00	2.25	HL-93	1.73
1	8	25.50	31.4	-8.40	8.41	-1.79	10.28	0.00	0.00	2.23	HL-93	1.72
1	9	28.69	31.4	-8.27	8.28	-1.79	10.24	0.00	0.00	2.25	HL-93	1.73

1 10	31.87	31.4	-7.87	7.89	-1.74	9.98	0.00	0.00	2.35	HL-93	1.81
1 11	35.06	31.4	-7.21	7.22	-1.62	9.26	0.00	0.00	2.60	HL-93	2.00
1 12	38.25	31.4	-6.29	6.30	-1.43	8.22	0.00	0.00	3.05	HL-93	2.34
1 13	41.44	31.4	-5.41	6.81	-1.04	8.99	0.00	0.00	2.73	HL-93	2.10
1 14	44.62	31.4	-3.88	4.89	-0.76	6.57	0.00	0.00	4.03	HL-93	3.10
1 15	47.81	31.4	-2.08	2.62	-0.41	3.57	0.00	0.00	8.05	HL-93	6.19
1 16	51.00	31.4	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

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NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA

[2]: AASHTO 5.5.2.2A; SERVICEABILITY STRENGTH.GE. [D+RF(L+I)]

[3]: RATING FACTOR 9.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

RATING FACTOR 0.0 INDICATES THAT EITHER ALLOWABLE STRESS
IS TOO LOW OR DEAD LOAD STRESS IS TOO HIGH

[4]: FOR CONTINUOUS BEAM WITH COMPACT SECTIONS,
THE STRESSES ARE CALCULATED AFTER REDISTRIBUTION

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TABLE 1.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	25.50	STRENGTH L.S.	2.27	1.75	HL-93
1	51.00	25.50	SERVICEBILITY	2.23	1.72	HL-93

NOTE: Control Rating Factor for Current Bridge:
SERVICEBILITY
Operating= 2.231
Inventory= 1.716

TABLE 1.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	51.00	25.50	STRENGTH L.S.	3.22	570.62
1	51.00	25.50	SERVICEBILITY	3.00	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1341
Legal Load Rating= 3.003

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TABLE 1.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I D2	MAX. SHEAR POS.	MAX. SHEAR NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	330.8	17.6	11.6	52.0	0.0	4.17	HL-93	3.21
1	1	3.19	330.8	15.4	10.2	47.6	-1.5	4.62	HL-93	3.57
1	2	6.38	330.8	13.2	8.7	43.3	-3.3	5.16	HL-93	3.98
1	3	9.56	330.8	10.9	7.3	39.1	-5.7	5.82	HL-93	4.49
1	4	12.75	330.8	8.7	5.8	34.9	-8.3	6.61	HL-93	5.10
1	5	15.94	330.8	6.6	4.4	30.8	-10.9	7.61	HL-93	5.87
1	6	19.12	330.8	4.4	2.9	26.7	-13.5	8.90	HL-93	6.86
1	7	22.31	330.8	2.2	1.5	22.9	-16.3	9.99	HL-93	8.14
1	8	25.50	330.8	0.0	0.0	19.2	-19.2	9.99	HL-93	9.83
1	9	28.69	330.8	-2.1	-1.5	16.3	-22.9	9.99	HL-93	8.14
1	10	31.87	330.8	-4.4	-2.9	13.5	-26.7	8.89	HL-93	6.86
1	11	35.06	330.8	-6.6	-4.4	10.9	-30.8	7.61	HL-93	5.87
1	12	38.25	330.8	-8.8	-5.8	8.3	-34.9	6.61	HL-93	5.10
1	13	41.44	330.8	-11.0	-7.3	5.7	-39.1	5.82	HL-93	4.49
1	14	44.62	330.8	-13.1	-8.7	3.3	-43.3	5.17	HL-93	3.99
1	15	47.81	330.8	-15.3	-10.2	1.5	-47.6	4.63	HL-93	3.57
1	16	51.00	330.8	-17.5	-11.6	0.0	-52.0	4.17	HL-93	3.22

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	51.00	0.00	STRENGTH L.S.	4.17	3.21	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.166
Inventory= 3.214

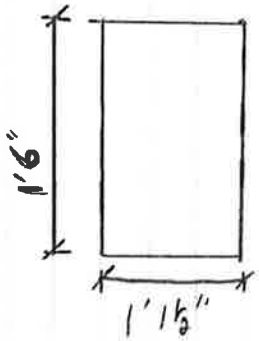
TABLE 1.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	51.00	0.00	STRENGTH L.S.	5.18	56.41

```
 ------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.667 & ADTT = 1341
                Legal Load Rating= 5.184
1
              BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER       MERLIN V 10.4
                DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING          COMPOSITE
                          UNIVERSITY OF MARYLAND                    LRF -- 2010
                                                                    RATING
 FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE  87\5969\Rating\5969

 TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION
                ***************************

                  NUMBER OF LANE   LL + I.
     SPAN  D FROM -----AND------ DEFLECTION GOVERN. RATING FACTOR FOR LL DEFL.
      NO.  L SUPT  DIST. FACTOR  ----------  LOAD   --------------------------
            (ft)   FOR LL DEFL.     (inch)   TYPE      RF = L/(800 * LL DEFL.)
     ---------------------------------------------------------------------------

       1    25.50    3   0.425       -0.29    HL-93            2.65

                     4   0.433       -0.29                     2.60

     ---------------------------------------------------------------------------
     NOTE: [1] "-" indicates downward deflection

           [2] RF * (live load defl.) = span length/800, therefore

               RF = L/(800 * LL Defl.), if = 99, No rating
```

Determine railing weight including Parapet

Parapet



$V = 1.5' * 1.125' * 1' = 1.688\ ft^3$

$weight/ft = 1.688\ ft^3 * 150\ lbs/ft^3 = 253.2\ lbs/ft$

Rail

$3\frac{1}{2}$" O.D. = 1.498 lb/ft

4" O.D. = 1.720 lb/ft

Post (see CAD Drawing)

recessed area = $0.2327\ ft^2 + 0.2848\ ft^2 = 0.5177\ ft^2$

Pipe area = $0.0688\ ft^2 + 0.0873\ ft^2 = 0.1561\ ft^2$

Total area = $0.9692\ ft^2$

5" thick area = $0.9692 - 0.1561 - 0.5177 = 0.2954\ ft^2$

Volume of post = $0.5177\ ft^2 * \frac{0.5"}{12} + 0.2954\ ft^2 * \frac{5"}{12} = 0.145\ ft^3$

Weight of Aluminum = 175 lb/ft3 Table 3.5.1-1

$0.145\ ft^3 * 175 = 25.4$ lbs/post

rail

Rail weight con't

rail spacing = Varies.

From Plans (and photos), 7+6+6+6 = 25
7+8+6+6 = 27
52 post

$$\frac{52 \times 25.4 \text{ lbs}}{2 \times (19.7' - 8\tfrac{1}{2})} = 3.34 \text{ plf}$$

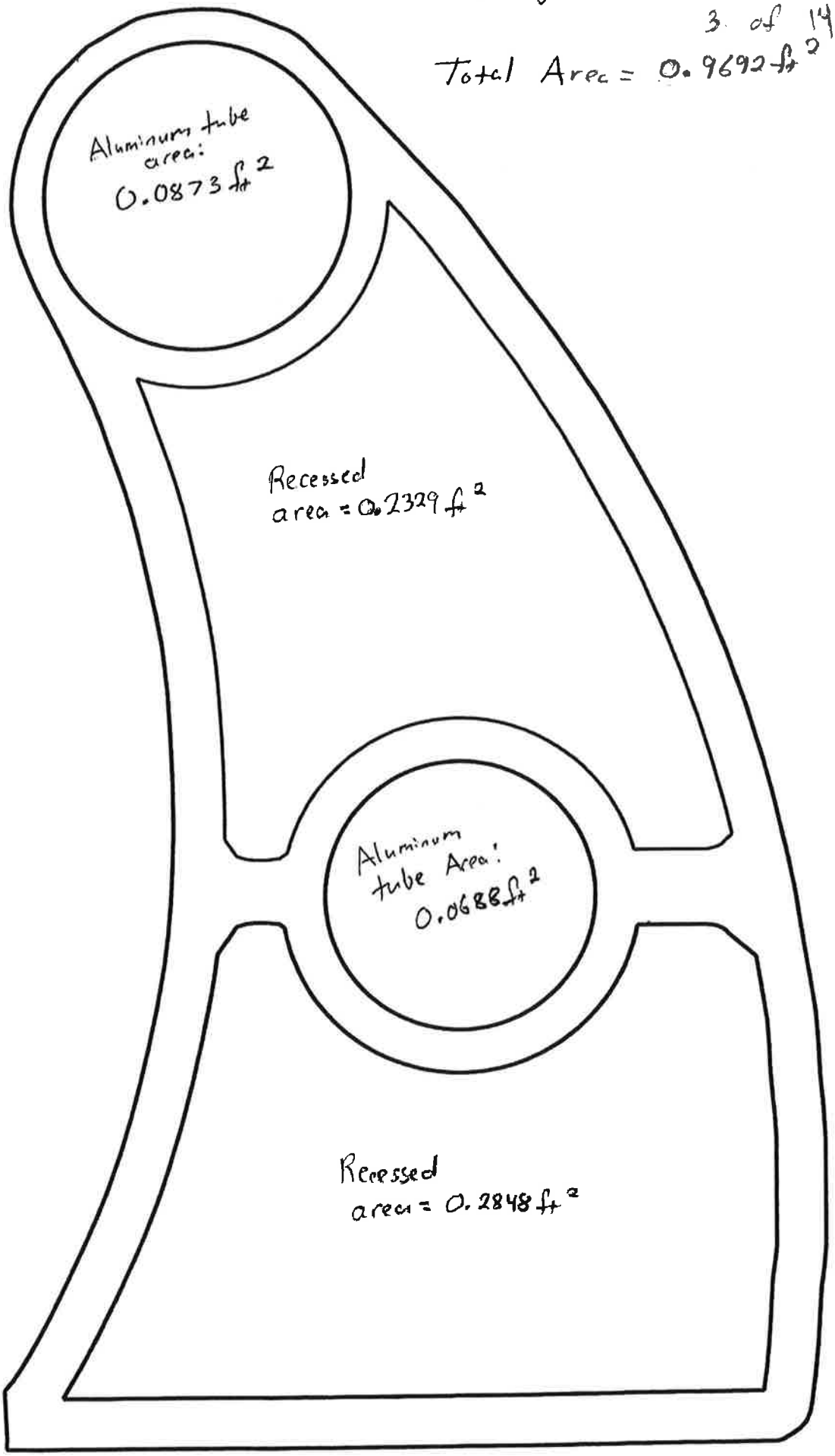
Total weight = 3.34 lbs/ft + 1.418 + 1.720 + 253.2 = 259.8 lbs/ft

$$\boxed{= 0.260 \text{ k/ft}}$$

Bridge No. 5969.

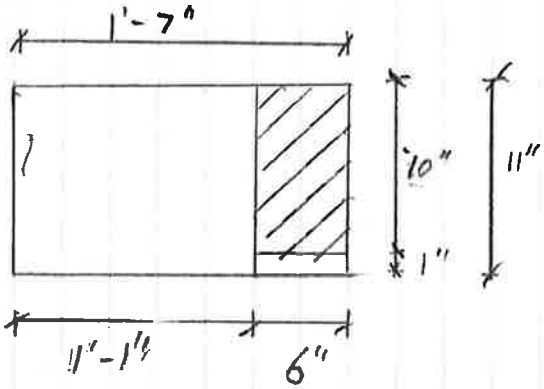
3 of 14

Total Area = 0.9692 ft²



...\Aluminum Bridge Railing.dgn 3/14/2013 3:21:03 PM

Determine Curb lateral



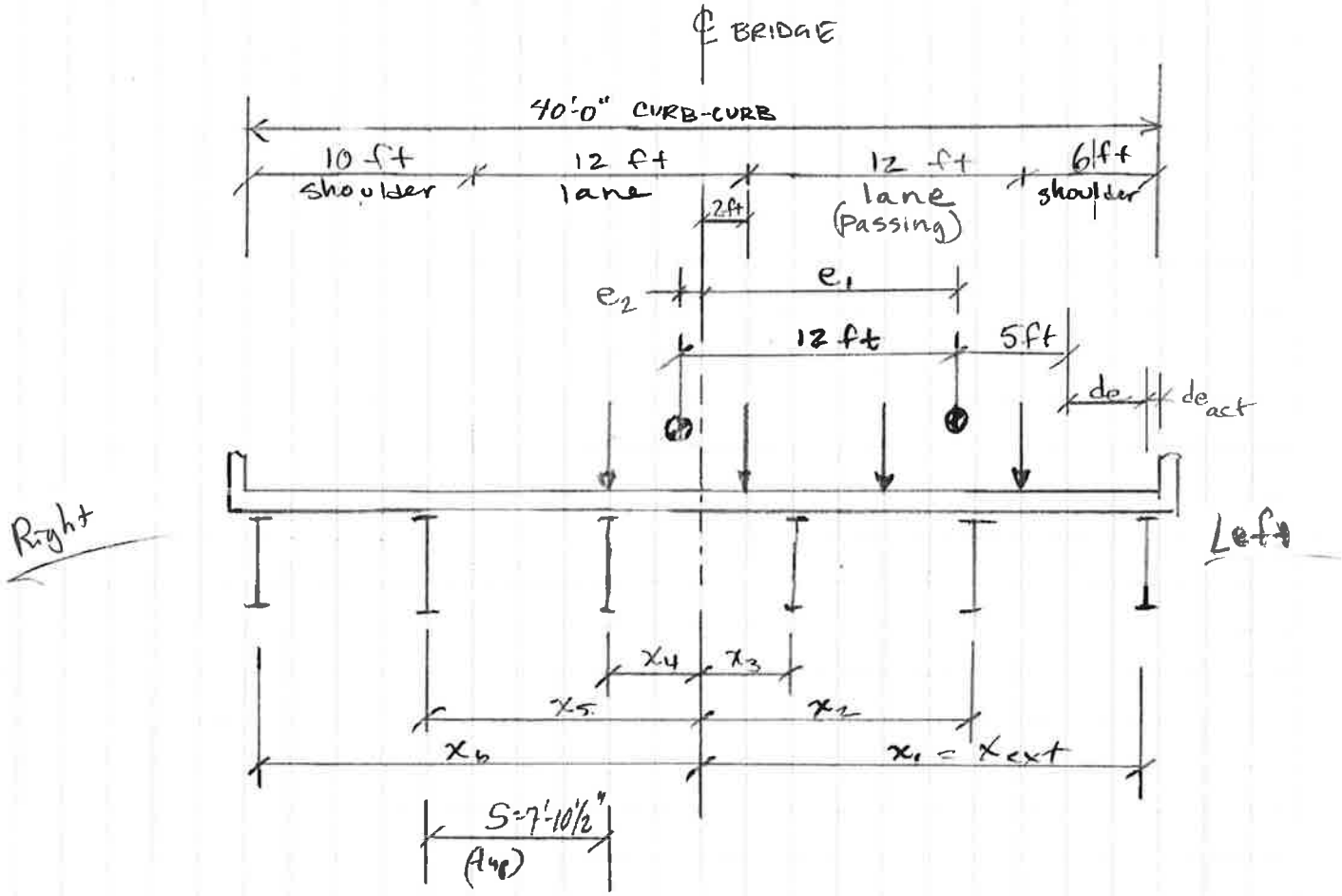
Concrete

$$\frac{(1'-7'')(11'')}{12^2} - \text{granite} = \frac{209}{144} - 0.42 = 1.035\ ft^2$$

Granite

$$\frac{10'' \times 6''}{144} = 0.42\ ft^2$$

BY TDM DATE 6/13/13 THE Louis Berger Group, INC. SHEET NO. 5 OF 14
CHKD. BY KSW DATE 6/27/13 PROJECT CKE 410A4
SUBJECT 5969 - RIGID BODY DIAGRAM - 6 GIRDERS



$$d_{e_{act}} = \frac{40}{2} - 2.5\left(7'\text{-}10\tfrac{1}{2}''\right) = 0.3125 \text{ ft} = 3.75 \text{ inches}$$

Determine which beams to rate

The four simple spans each are different lengths and have different skews.

The beams in span 1 are all 33 WF 130. Three beams in span 2 are 33WF118 while the other three beams are 33 WF 130. The beams in spans 3 & 4 are all 33 WF 118. All 4 spans have coverplates of different widths, thicknesses and lengths.

Fortunately, the girder spacing, deck thickness, and overhang are consistent.

The shoulder widths on the bridge from the plans are:

Right: $\left(\frac{40}{2} + 2\right) - 12 = 10$ ft Right shoulder (travel lane)

Left: $\left(\frac{40}{2} - 2\right) - 12 = 6$ ft Left shoulder (Passing Lane)

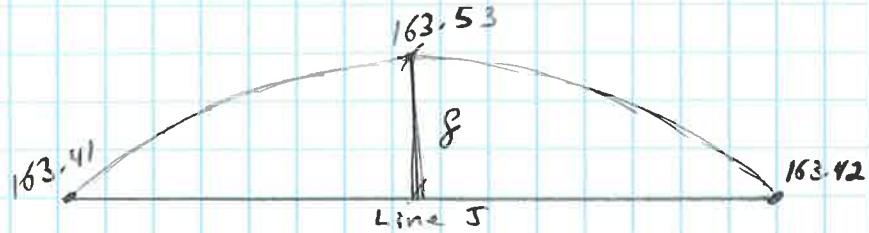
Therefore, Exterior beams on left side will see more live load.

For Spans 1, 3, and 4 - Rate the exterior beams on left side and an interior beam from each span.

For Span 2, rate all six beams to determine controlling values.

Determine Girder Haunch

5969 Span 1



Camber = 2"
nominal blocking = 2 3/4"
t_f = 0.86

Camber tolerance for rolled beam with span L = 61ft is 0.75in

$$\bar{x} = \frac{163.41 + 163.42}{2} = 163.415$$

$$\delta = 163.53 - 163.415 = 0.115' = 1.38''$$

$$2.75'' + 2'' + 1.38'' = 2.13'' - 0.75 + 0.86 = \underline{\underline{2.24''}}$$ for LL eg and Merlin

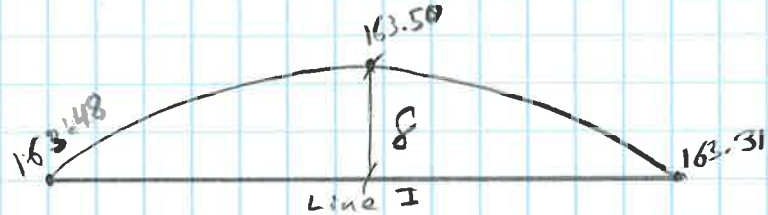
camber → ← extra camber; tol; t_f

However, for DL use 2 3/4" since thicker at ends

$$2.75 + 0.86 = \underline{\underline{3.61''}}$$ for DL

5969 Span 2

Max Camber = 1 3/4"
nominal blocking = 2 3/4"
t_f = 0.86"



Camber tolerance for rolled beam with avg. span L = 55.57 ⇒ 0.625in

$$\bar{x} = \frac{163.48 + 163.31}{2} = 163.4$$

$$\delta = 163.5 - 163.4 = 0.10' = 1.26''$$

$$2.75 - 1.75 + 1.26'' = 2.26'' - 0.625 + 0.86'' = \underline{\underline{2.495''}}$$ for LL eg & Merlin

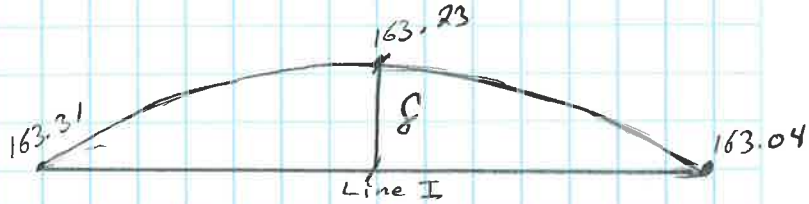
camber; extra camber; tol

However, for DL use 2 3/4" since thicker at ends

$$2.75'' + 0.86 = \boxed{3.61'' \text{ for DL}}$$

5969 Span 3

Camber = 1½"
Nominal = 2¾"
t_f = 0.74"



overcamber tolerance = 0.5 in
for Span L = 48 ft

$$\bar{x} = \frac{163.31 + 163.04}{2} = 163.175$$

$$\delta = 163.23 - 163.175 = 0.055' = 0.66''$$

$$2\tfrac{3}{4}'' - 1\tfrac{1}{2}'' + 0.66'' = 1.91''$$

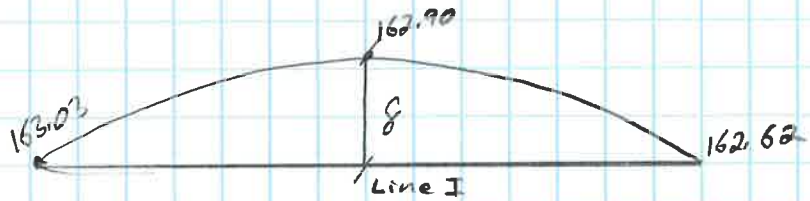
$$1.91'' + 0.74'' - 0.5'' = \boxed{2.15'' \text{ for LL}}$$

For DL use 2¾" since it is thicker at ends

$$2.75'' + 0.74'' = \boxed{3.49'' \text{ for dead Load}}$$

5969 Span 4

Camber = 1½"
Nominal = 2¾"
t_f = 0.74"



overcamber tolerance = 0.625
for span L = 51 ft

$$\bar{x} = \frac{162.62 + 163.03}{2} = 162.825'$$

$$\delta = 162.90 - 162.825 = 0.075' = 0.9''$$

$$2\tfrac{3}{4}'' - 1.5'' + 0.9'' = 2.15''$$

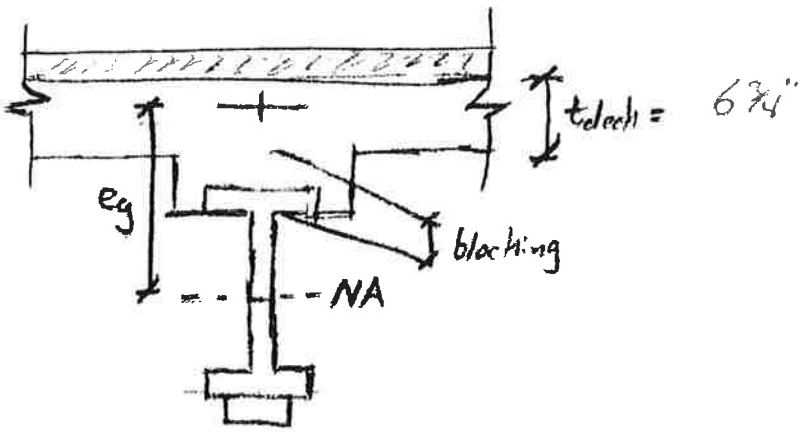
$$2.15'' + 0.74'' - 0.625'' = \boxed{2.265'' \text{ for LL}}$$

For DL use 2¾" since it is thicker @ ends

$$2.75'' + 0.74'' = \boxed{3.49'' \text{ for DL}}$$

Determine c.g. distance e.g. – 5969 Span 1

The following diagram and formula were used in all calculations of e.g.



c.g. distance, e.g. =

$$\frac{t_{deck}}{2} + (\text{blocking}_{LL} - t_f) + (D_{Avg} - \bar{y}_{avg})$$

$$\left(\frac{6\frac{3}{4}''}{2}\right) + (2.24 - 0.86) + (33.77 - 14.48) = 24.04''$$

$$K_g = n + (I_{total} + A_{total} * e_g^2)$$

Find Average Values of Interior and Exterior Beams

5969 Span 1

See Moment of Inertia Calculation for values used

Average Depth $= \frac{2\times(22' * 33.10 + 39' * 34.10) + (19' * 33.10 + 42' * 34.10') \times 4}{61 * 6}$

$\boxed{= 33.77''}$

Average Area $= \frac{(22' * 38.26 + 39' * 48.26) \times 2 + (19' * 38.26 + 42' * 51.26) \times 4}{61 \times 6}$

$\boxed{= 46.36 in^2}$

Average I_o $= \frac{(22' * 6699 + 39' * 9204.49) \times 2 + (19' \times 6699 + 42' \times 9520.79) \times 4}{61 \times 6}$

$\boxed{= 8485.58 in^4}$

Average $\bar{y}$ $= \frac{(22' \times 16.55 + 39' \times 14.02) \times 2 + (19' \times 16.55 + 42' \times 13.23) \times 4}{61' \times 6}$

$\boxed{= 14.48''}$

5969 Span 2, 3, and 4

See moment of Inertia Calculation for values used and calculation. Calculation similar to the one seen for span 1

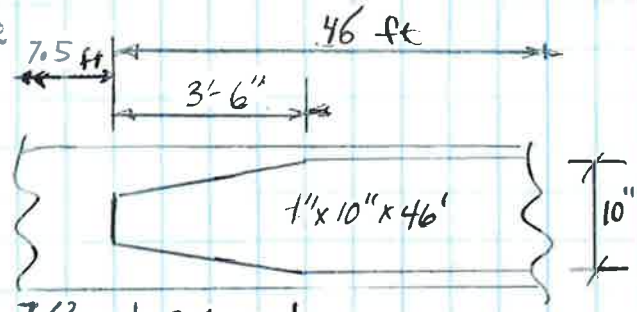
DETERMINE POINTS OF ANALYSIS IN MERLIN

SPAN 1

EXTERIOR: L = 61 ft

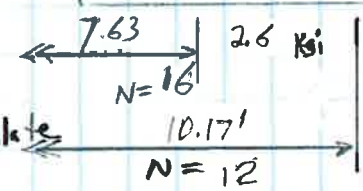
coverplate

$\frac{61-46}{2} = 7.5'$



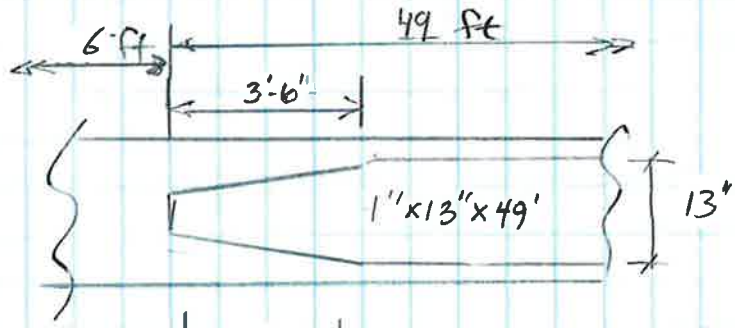
Fatigue Evaluation Category E' Detail

Strength Rating @ Coverplate



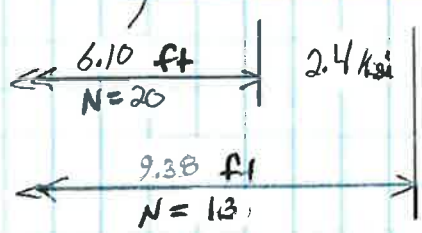
INTERIOR:

$\frac{61-49}{2} = 6'$



Fatigue Evaluation @ Category E' Detail

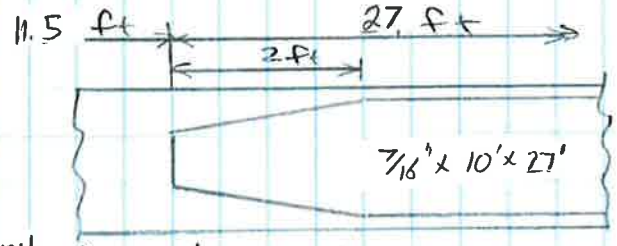
Strength Rating at Cover plate

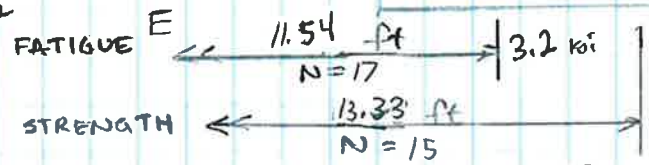


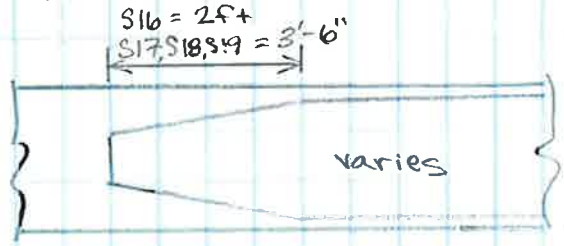
SPAN 2

EXTERIOR: S15
L = 50.0 ft

$\frac{50-27}{2} = 11.5$ ft



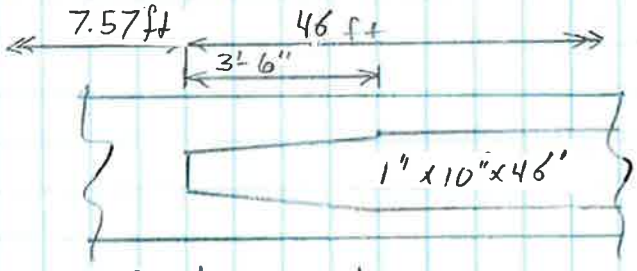


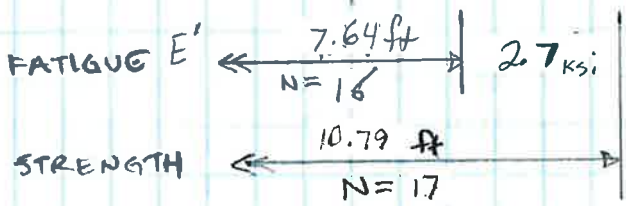


	L	N_{fat}	Δf_{fat}		$N_{strength}$
S16	52.23	20	3.1 ksi	E	17
S17	54.45	17	2.7 ksi	E	17
S18	56.68	15	2.9 ksi	E'	16
S19	58.91	20	2.5 ksi	E'	20

See excel tabulation "5969_Analysis_Points.xlsx"

EXTERIOR: S20
L = 61.14 ft





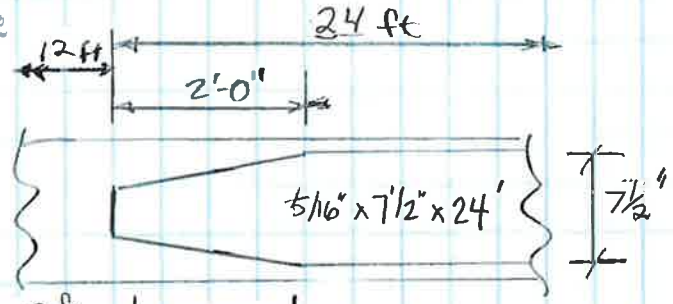
DETERMINE POINTS OF ANALYSIS IN MERLIN

SPAN 3

EXTERIOR: L = 48 ft

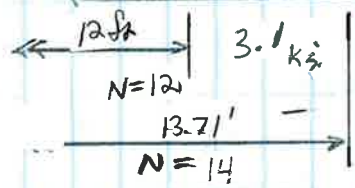
coverplate

$\frac{48-24}{2} = 12\text{ft}$



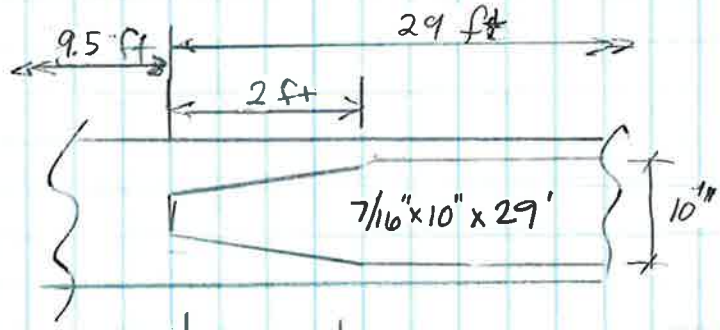
Fatigue Evaluation Category E Detail

Strength Rating @ coverplate



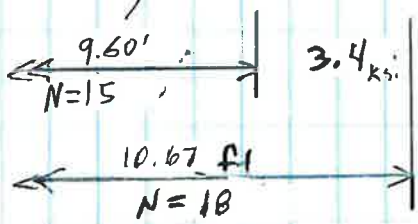
INTERIOR:

$\frac{48-29}{2} = 9.5\text{ft}$



Fatigue Evaluation Category E Detail

Strength Rating @ coverplate



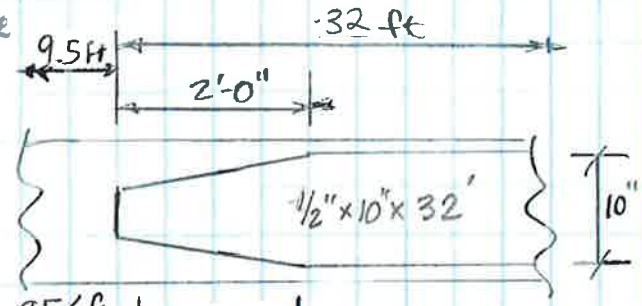
BY DGB/TDM DATE 8/19/13 THE Louis Berger Group, INC. SHEET NO. 14 OF 14
CHKD. BY KSW DATE 8/22/13 PROJECT CKE 410A4
SUBJECT 5969

DETERMINE POINTS OF ANALYSIS IN MERLIN

SPAN 4. EXTERIOR: L = 51 ft

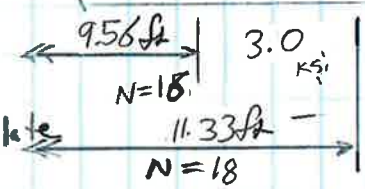
coverplate

$\frac{51-32}{2} = 9.5'$



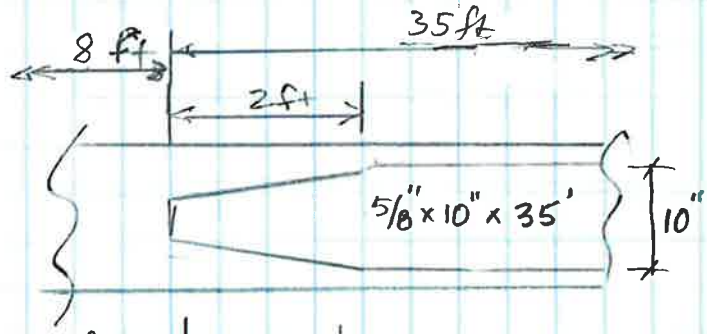
Fatigue Evaluation Category E Detail

Strength Rating @ Coverplate



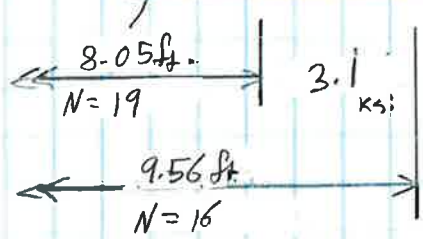
INTERIOR:

$\frac{51-35}{2} = 8 ft$



Fatigue Evaluation @ Category E Detail

Strength Rating at Cover plate



1962

Compare the rolled member properties from the year of construction to those used by default in the analysis software Merlin-Dash.

Edition	Type	Designation			A	d	t_w	b_f	t_f	W	I_x	S_x	r_x	I_y	S_y	r_y
ASD6	WF	WF 33x11.5	33 x	11 1/2	34.71	32.86	0.55	11.48	0.74	118.00	5886.9	358.3	13.02	170.3	29.7	2.22
LRFD3	W	W33x118			34.70	32.90	0.55	11.50	0.74	118.00	5900	359	13	187	32.6	2.32
					-0.03%	0.12%	-0.73%	0.14%	0.27%	0.00%	0.22%	0.19%	-0.15%	8.93%	8.90%	4.31%
ASD6	WF	WF 33x11.5	33 x	11 1/2	38.26	33.10	0.58	11.51	0.86	130.00	6699	404.8	13.23	201.4	35	2.29
LRFD3	W	W33x130			38.30	33.10	0.58	11.50	0.86	130.00	6710	406	13.2	218	37.9	2.39
					0.10%	0.00%	0.00%	-0.09%	0.00%	0.00%	0.16%	0.30%	-0.23%	7.61%	7.65%	4.18%

Weight is the same.
Since top flange is fully braced, strong -axis values important.
They are less than 1% different, therefore use the default values in Merlin-Dash.

Determine N for evaluation points in MerlinDash.

Span 1 Transition = 3.5 ft

61	1	2	3	4	5	6
20	3.05	6.10	9.15	12.20	15.25	18.30
19	3.21	6.42	9.63	12.84	16.05	19.26
18	3.39	6.78	10.17	13.56	16.94	20.33
17	3.59	7.18	10.76	14.35	17.94	21.53
16	3.81	7.63	11.44	15.25	19.06	22.88
15	4.07	8.13	12.20	16.27	20.33	24.40
14	4.36	8.71	13.07	17.43	21.79	26.14
13	4.69	9.38	14.08	18.77	23.46	28.15
12	5.08	10.17	15.25	20.33	25.42	30.50
11	5.55	11.09	16.64	22.18	27.73	33.27

Exterior:

Cover plate transition begins at:	7.5	therefore use N=16 for Fatigue
Full width cover plate begins at:	11	therefore use N=12 for Strength

Interior:

Cover plate transition begins at:	6	therefore use N=20 for Fatigue and Mid M
Full width cover plate begins at:	9.5	therefore use N=13 for Strength at CP

Span 2

Beam S15 27 ft, CP Length 2 ft, CP Transition

50	1	2	3	4	5	6
20	2.50	5.00	7.50	10.00	12.50	15.00
19	2.63	5.26	7.89	10.53	13.16	15.79
18	2.78	5.56	8.33	11.11	13.89	16.67
17	2.94	5.88	8.82	11.76	14.71	17.65
16	3.13	6.25	9.38	12.50	15.63	18.75
15	3.33	6.67	10.00	13.33	16.67	20.00
14	3.57	7.14	10.71	14.29	17.86	21.43
13	3.85	7.69	11.54	15.38	19.23	23.08
12	4.17	8.33	12.50	16.67	20.83	25.00
11	4.55	9.09	13.64	18.18	22.73	27.27

Cover plate transition begins at: 11.5 therefore use N=13 for Fatigue
Full width cover plate begins at: 13.5 therefore use N=15 for Strength at CP
therefore use N=20 for Mid M

Beam S16 37 ft, CP Length 2 ft, CP Transition

52.23	1	2	3	4	5
20	2.61	5.22	7.83	10.45	13.06
19	2.75	5.50	8.25	11.00	13.74
18	2.90	5.80	8.71	11.61	14.51
17	3.07	6.14	9.22	12.29	15.36
16	3.26	6.53	9.79	13.06	16.32
15	3.48	6.96	10.45	13.93	17.41
14	3.73	7.46	11.19	14.92	18.65
13	4.02	8.04	12.05	16.07	20.09
12	4.35	8.71	13.06	17.41	21.76
11	4.75	9.50	14.24	18.99	23.74

Cover plate transition begins at: 7.615 therefore use N=20 for Fatigue and Mid M
Full width cover plate begins at: 9.615 therefore use N=17 for Strength at CP

Beam S17 42 ft, CP Length 3.5 ft, CP Transition

54.45	1	2	3	4	5
20	2.72	5.45	8.17	10.89	13.61
19	2.87	5.73	8.60	11.46	14.33
18	3.03	6.05	9.08	12.10	15.13
17	3.20	6.41	9.61	12.81	16.01
16	3.40	6.81	10.21	13.61	17.02
15	3.63	7.26	10.89	14.52	18.15
14	3.89	7.78	11.67	15.56	19.45
13	4.19	8.38	12.57	16.75	20.94
12	4.54	9.08	13.61	18.15	22.69
11	4.95	9.90	14.85	19.80	24.75

Cover plate transition begins at: 6.225 therefore use N=17 for Fatigue
Full width cover plate begins at: 9.725 use N=17 & 20 for Strength @ CP & Mid M, respectively

Span 2

Beam S18 42 ft, CP Length 3.5 ft, CP Transition

56.68	1	2	3	4	5	6
20	2.83	5.67	8.50	11.34	14.17	17.00
19	2.98	5.97	8.95	11.93	14.92	17.90
18	3.15	6.30	9.45	12.60	15.74	18.89
17	3.33	6.67	10.00	13.34	16.67	20.00
16	3.54	7.09	10.63	14.17	17.71	21.26
15	3.78	7.56	11.34	15.11	18.89	22.67
14	4.05	8.10	12.15	16.19	20.24	24.29
13	4.36	8.72	13.08	17.44	21.80	26.16
12	4.72	9.45	14.17	18.89	23.62	28.34
11	5.15	10.31	15.46	20.61	25.76	30.92

Cover plate transition begins at: 7.34 therefore use N=15 for Fatigue
Full width cover plate begins at: 10.84 therefore use N=16 for Strength

Beam S19 48 ft, CP Length 3.5 ft, CP Transition

58.91	1	2	3	4	5	6
20	2.95	5.89	8.84	11.78	14.73	17.67
19	3.10	6.20	9.30	12.40	15.50	18.60
18	3.27	6.55	9.82	13.09	16.36	19.64
17	3.47	6.93	10.40	13.86	17.33	20.79
16	3.68	7.36	11.05	14.73	18.41	22.09
15	3.93	7.85	11.78	15.71	19.64	23.56
14	4.21	8.42	12.62	16.83	21.04	25.25
13	4.53	9.06	13.59	18.13	22.66	27.19
12	4.91	9.82	14.73	19.64	24.55	29.46
11	5.36	10.71	16.07	21.42	26.78	32.13

Cover plate transition begins at: 5.455 therefore use N=20 for Fatigue
Full width cover plate begins at: 8.955 therefore use N=20 for Strength

Beam S20 46 ft, CP Length 3.5 ft, CP Transition

61.14	1	2	3	4	5	6	7
20	3.06	6.11	9.17	12.23	15.29	18.34	21.40
19	3.22	6.44	9.65	12.87	16.09	19.31	22.53
18	3.40	6.79	10.19	13.59	16.98	20.38	23.78
17	3.60	7.19	10.79	14.39	17.98	21.58	25.18
16	3.82	7.64	11.46	15.29	19.11	22.93	26.75
15	4.08	8.15	12.23	16.30	20.38	24.46	28.53
14	4.37	8.73	13.10	17.47	21.84	26.20	30.57
13	4.70	9.41	14.11	18.81	23.52	28.22	32.92
12	5.10	10.19	15.29	20.38	25.48	30.57	35.67
11	5.56	11.12	16.67	22.23	27.79	33.35	38.91

Cover plate transition begins at: 7.57 therefore use N=16 for Fatigue and Mid M
Full width cover plate begins at: 11.07 therefore use N=17 for Strength at CP

Determine N for evaluation points in MerlinDash.

Span 3 Transition = 2 ft

48	1	2	3	4	5	6	7
20	2.40	4.80	7.20	9.60	12.00	14.40	16.80
19	2.53	5.05	7.58	10.11	12.63	15.16	17.68
18	2.67	5.33	8.00	10.67	13.33	16.00	18.67
17	2.82	5.65	8.47	11.29	14.12	16.94	19.76
16	3.00	6.00	9.00	12.00	15.00	18.00	21.00
15	3.20	6.40	9.60	12.80	16.00	19.20	22.40
14	3.43	6.86	10.29	13.71	17.14	20.57	24.00
13	3.69	7.38	11.08	14.77	18.46	22.15	25.85
12	4.00	8.00	12.00	16.00	20.00	24.00	28.00
11	4.36	8.73	13.09	17.45	21.82	26.18	30.55

Exterior:

Cover plate transition begins at:	12	therefore use N=12 for Fatigue
Full width cover plate begins at:	14	therefore use N=14 for Strength

Interior:

Cover plate transition begins at:	9.5	therefore use N=15 for Fatigue
Full width cover plate begins at:	11.5	therefore use N=18 for Strength

Determine N for evaluation points in MerlinDash.

Span 4 Transition = 2 ft

51	1	2	3	4	5	6
20	2.55	5.10	7.65	10.20	12.75	15.30
19	2.68	5.37	8.05	10.74	13.42	16.11
18	2.83	5.67	8.50	11.33	14.17	17.00
17	3.00	6.00	9.00	12.00	15.00	18.00
16	3.19	6.38	9.56	12.75	15.94	19.13
15	3.40	6.80	10.20	13.60	17.00	20.40
14	3.64	7.29	10.93	14.57	18.21	21.86
13	3.92	7.85	11.77	15.69	19.62	23.54
12	4.25	8.50	12.75	17.00	21.25	25.50
11	4.64	9.27	13.91	18.55	23.18	27.82

Exterior:

Cover plate transition begins at:	9.5	therefore use N=16 for Fatigue
Full width cover plate begins at:	11.5	therefore use N=18 for Strength

Interior:

Cover plate transition begins at:	8	therefore use N=19 for Fatigue
Full width cover plate begins at:	10	therefore use N=16 for Strength

	Spacing		Distance from Left End		Check
S15	12.31000		12.31		
	18.84		31.15		50.00
					18.85
S16	17.45		17.45		
	18.84		36.29		52.23
					15.94
S17	3.68		3.68		
	18.84		22.52		54.45
	18.84		41.36		13.09
S18	8.76		8.76		
	18.84		27.60		56.68
	18.84		46.44		10.24
S19	13.83		13.83		
	18.84		32.67		58.91
	18.84		51.51		7.40
S20	18.84		18.84		
	18.84		37.68		61.14
	18.84		56.52		4.62

APPENDIX C

SECTION LOSSES SKETCHES

(See CD)

(Not APPLICABLE)