Bridge Load Rating

Prepared for

Maine Department of Transportation

Bridge No. 5800

BANGOR

I – 95 NORTHBOUND

OVER

STILLWATER AVENUE

Date of Inspection: June 24, 2010

Date of Rating: July 26, 2013

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

THE Louis Berger Group, INC.
482 Congress Street, Suite 401 | Portland, ME 04101 | www.louisberger.com



Note:
This Load Rating has been updated for the FAST Act's Emergency Vehicles on 7/31/2019.

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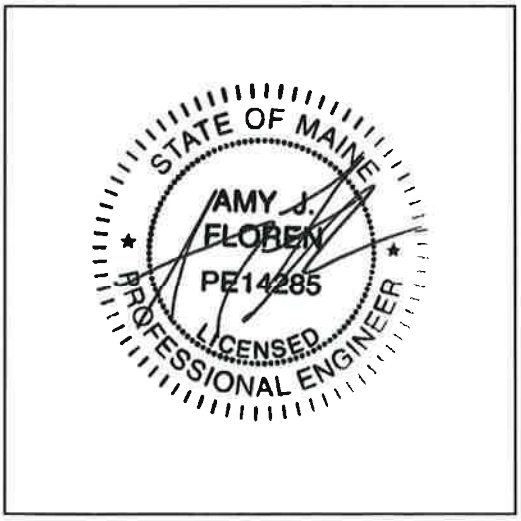
Date of Inspection: June 24, 2010

Date of Rating: July 31, 2019

Rating Factors are determined for the FAST Act's Emergency Vehicle Configurations Types EV2 and EV3 at the legal load rating level in accordance with AASHTO MBE.

Prepared By: Juan Tzoc Jr., E.I.T.
Checked By: Amy Floren, P.E.

Louis Berger U.S., Inc.
A WSP Company
106 Lafayette Street, Suite 2F, Yarmouth, ME 04096



Bridge No.:	5800	Owner:	MaineDOT
Town / City:	Bangor	Maintainer:	MaineDOT
Route Carried:	I – 95 Northbound	Year Built:	1960
Crosses:	Stillwater Avenue	Year(s) Rebuilt / Rehab:	1983, 2006

SUMMARY OF BRIDGE RATING

	VEHICLE TYPE		RF	RT (TONS)	POSTING LOAD (TONS)
	HL-93	INVENTORY	***1.06***	38.16	
		OPERATING	***1.37***	49.32	
	HL-93 Modified	INVENTORY			
		OPERATING			
	CONFIGURATION 1				
	CONFIGURATION 2				
	CONFIGURATION 3				
	CONFIGURATION 4				
	CONFIGURATION 5				
	CONFIGURATION 6				
	CONFIGURATION 7				
	CONFIGURATION 8				
(Updated 7/31/19)	TYPE EV2		1.59	45.71	
(Updated 7/31/19)	TYPE EV3		1.03	44.29	

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:	0.750
Live Load DF Routine Commercial:	0.750
Live Load DF Special Hauling:	0.750
Impact Factor:	1.33
Governing Condition Factor, Φ_c:	1.00
System Factor, Φ_s:	1.00
ADTT (one-way):	2398

***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 8 years

BREAKDOWN OF BRIDGE RATING

Town / City: Bangor
Bridge No.: 5800

Route Carried: I – 95 Northbound
Crosses: Stillwater Avenue

LOAD RATING POINTS OF INTEREST

(Updated 7/31/19)

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations								Emergency Vehicles	
	Inv. 72.0 k	Oper 72.0 k	Inv. 90.0 k	Oper 90.0 k	1 100.0 k	2 94.0 k	3 88.0 k	4 88.0 k	5 88.0 k	6 75.9 k	7 59.0 k	8 37.4 k	EV2 57.5 k	EV3 86.0 k
Span 1 Interior Beam Positive Moment 0.5L of Span	1.72	2.23											3.37*	2.21*
Span 1 Interior Beam Shear 0.0L of Span	2.27	2.94											4.49	3.03
Span 1 Interior Beam Moment at Cover Plate Transition (13.18')	2.12	2.75											4.18*	2.82*
Span 1 Exterior Beam Positive Moment 0.5L of Span	1.80	2.33											3.55*	2.32*
Span 1 Exterior Beam Shear 0.0L of Span	3.19	4.14											6.34	4.27
Span 1 Exterior Beam Moment at Cover Plate Transition (14.0')	2.29	2.97											4.52*	3.05*

BREAKDOWN OF BRIDGE RATING

Town / City: Bangor
Bridge No.: 5800

Route Carried: I – 95 Northbound
Crosses: Stillwater Avenue

LOAD RATING POINTS OF INTEREST

(Updated 7/31/19)

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations								Emergency Vehicles	
	Inv. 72.0 k	Oper 72.0 k	Inv. 90.0 k	Oper 90.0 k	1 100.0 k	2 94.0 k	3 88.0 k	4 88.0 k	5 88.0 k	6 75.9 k	7 59.0 k	8 37.4 k	EV2 57.5 k	EV3 86.0 k
Span 2 Interior Beam Positive Moment 0.5L of Span	1.50	1.94											3.16	2.08
Span 2 Interior Beam Shear 0.0L of Span	2.38	3.08											4.96	3.34
Span 2 Interior Beam Moment at Cover Plate Transition (17.14')	1.94	2.52											4.03*	2.71*
Span 2 Exterior Beam Positive Moment 0.5L of Span	1.49	1.93											3.15	2.07
Span 2 Exterior Beam Shear 0.0L of Span	3.36	4.36											7.02	4.73
Span 2 Exterior Beam Moment at Cover Plate Transition (18.28')	2.09	2.70											4.32*	2.91*

BREAKDOWN OF BRIDGE RATING

Town / City: Bangor
Bridge No.: 5800

Route Carried: I – 95 Northbound
Crosses: Stillwater Avenue

LOAD RATING POINTS OF INTEREST

(Updated 7/31/19)

	Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations								Emergency Vehicles	
		Inv. 72.0 k	Oper 72.0 k	Inv. 90.0 k	Oper 90.0 k	1 100.0 k	2 94.0 k	3 88.0 k	4 88.0 k	5 88.0 k	6 75.9 k	7 59.0 k	8 37.4 k	EV2 57.5 k	EV3 86.0 k
	Span 3 Interior Beam Positive Moment 0.5L of Span	**1.06**	**1.37**											1.61	**1.03**
(Updated 7/31/19)	**Span 3 Interior Beam Positive Moment 0.6L of Span**	1.07	1.39											**1.59**	1.07
	Span 3 Interior Beam Shear 0.0L of Span	2.88	3.74											5.03	3.41
(Updated 7/31/19)	Span 3 Exterior Beam Positive Moment 0.4L & 0.60L of Span	1.41	1.83											2.08	1.40
	Span 3 Exterior Beam Positive Moment 0.5L of Span	1.39	1.80											2.10	1.35
	Span 3 Exterior Beam Shear 0.0L of Span	4.27	5.54											7.45	5.05
	CONTROLLING RATING FACTORS	***1.06***	***1.37***											**1.59**	**1.03**

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

- Span 1 Exterior at the cover plate transition RF: 2.90 (Operating), 2.23 (Inventory)
- Span 1 Interior at the cover plate transition RF: 2.60 (Operating), 2.00 (Inventory)

DESCRIPTION OF BRIDGE

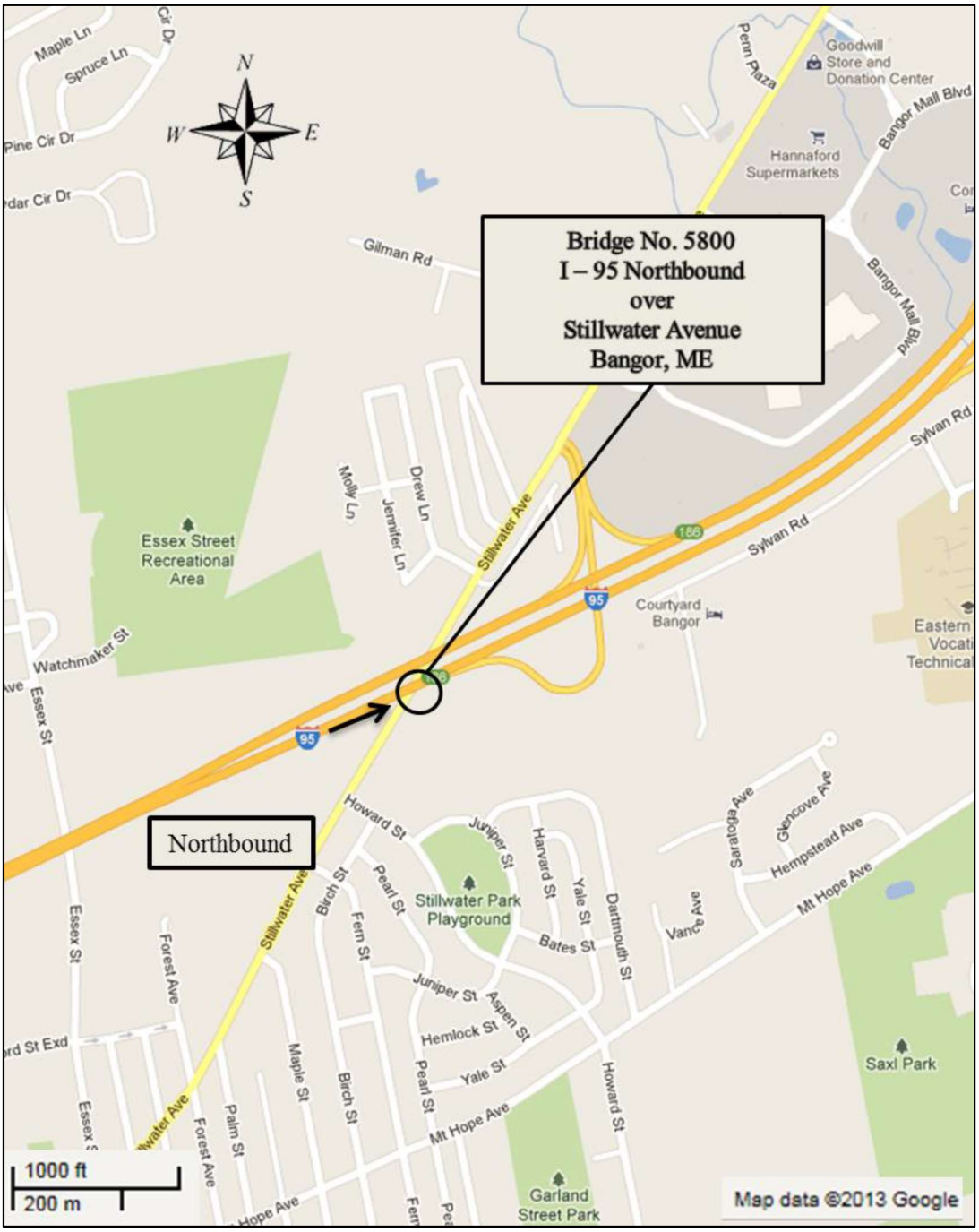
Bridge Number:	5800
Owner:	MaineDOT
Maintained By:	MaineDOT
Location:	Bangor
Route Carried:	I – 95 Northbound
Feature Intersected:	Stillwater Avenue
Latest NBI Inspection Date:	June 24, 2010
Field Verification Date:	March 25, 2013
Date of Construction:	1960
Bridge Type:	Simple span steel rolled beams with cast in place concrete deck.
Material Properties:	Steel: A373 (f_y = 32 ksi) specified for all weldable steel and all other steel assumed (f_y = 33 ksi) per MBE Table 6A.6.2.1-1. Concrete: Based on the modular ratio, n=10, and the modulus of elasticity of steel, f'_c = 2.5 ksi.
Original Design Loading:	H20-S16-44 modified for military requirements
Date(s) of Rebuild/Rehab:	1983, 2006
Description of Rebuild/Rehab:	1983: Replacement of wearing surface, railing, joints, and curbs. 2006: Unknown.
Posting:	Open, no restrictions
Superstructure:	Three spans: Span 1: Five W36x194 steel beams with bottom flange cover plates and a composite cast in place concrete deck. Span 2: Five W36x280 steel beams with bottom flange cover plates and a composite cast in place concrete deck. Span 3: Five W36x150 steel beams with a non-composite cast in place concrete deck.
Substructure:	Cast in place concrete abutments and piers (Abutment No. 1 on piles and all other substructure units on spread footings).
Bearings:	Steel plates.
Bridge Spans:	Total Span: 211.32 ft Bearing to Bearing: 74’-8”, 91’-5”, 41’-7 ½”
Bridge Skew:	56°-27’-51”
Bridge Width:	35’-0” out-to-out
Roadway Width:	31’-4” curb-to-curb
Roadway Surface:	Bituminous concrete.
Curbs:	Concrete.
Sidewalk/Walkway/Median:	None.
Utilities:	Aerial wires attached to the underside of the Span 1 exterior beams (transverse to bridge).
Bridge Railing:	Two bar aluminum rail.

Approach Railing:	Concrete endpost with bridge transition Type I/Type II (leading/trailing end) with guardrail.
Wearing Surface Condition:	Satisfactory (Field Observation)
Bridge Railing Condition:	Good (Field Observation)
Deck Condition:	Satisfactory (SI&A)
Beam Condition:	Very Good (SI&A)
Bearing Condition:	Satisfactory (Field Observation)
Abutment Condition:	Satisfactory (SI&A)
Pier Condition:	Satisfactory (SI&A)

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APPENDIX B – LOAD RATING COMPUTATIONS	
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 superstructure is listed in very good condition and the deck and substructure are listed in satisfactory condition. This information provided the basis for the bridge load rating summarized in this report.

Terminology used for the purposes of this bridge rating is shown in Figure 1 below.

Site Observations

The overall geometry of Span 1, Span 2 and Span 3 was measured. Dimensions were recorded to verify the 1959 bridge construction plans and the 1983 rehabilitation plans. The dimensions of beams, cover plates, cross frames, and haunch were verified in the field. The wearing surface, curbs and rails have been replaced based on the construction plans from 1983 and field investigation. The asphalt wearing surface is in satisfactory condition. The bridge steel superstructure is in very good condition with minor rust and paint failure on the primary beams. This does not impact the member capacity for the purpose of this load rating. Delamination and heavy spalling of concrete at pier caps and abutment seats and backwall exposes steel reinforcing bars. There are large cracks extending throughout the abutment wall. The expansion joints over Pier No. 1 and 2 are leaking causing corrosion of the bearings and beam ends.

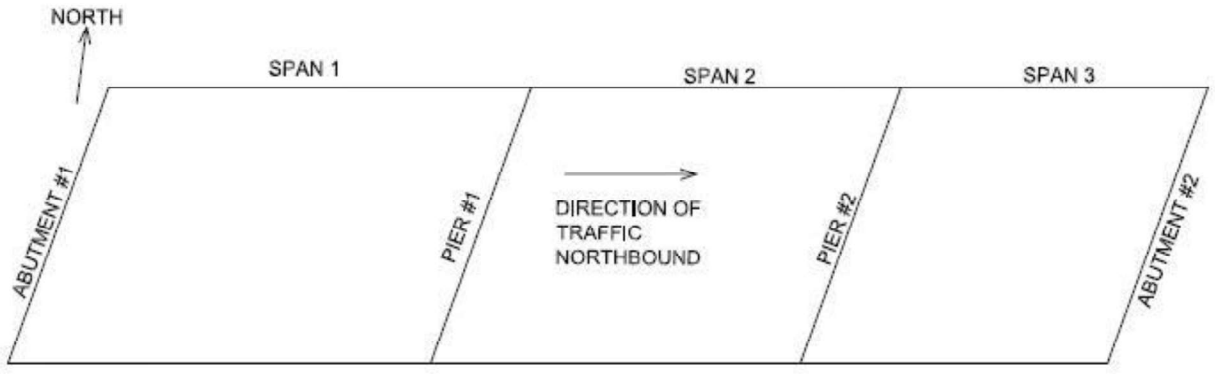


Figure 1: Bridge No. 5800 Terminology

ASSUMPTIONS, CRITERIA, AND REFERENCES

Assumptions and Criteria

This analysis for Bridge No. 5800 was performed in accordance with MBE section 6A.1.5. Where the MBE is silent, the analysis was performed per the AASHTO LRFD Manual.

Scope:

Rate both interior and exterior beams for all three simple spans; shear at the ends, moment at midspan, and at cover plate transitions.

Structural Analysis:

- Deterioration: None. Reduction of the beam capacity was not considered.
- Composite: Span 1 and Span 2 are composite with plans showing shear studs. Span 3 is non-composite.
- Unbraced Length: Fully braced; All three spans have the top flange embedded in the concrete.
- Haunch / Blocking: The rolled members are cambered. From plans, the beams are cambered to account for dead load deflection. For Spans 1 and 2, the overcamber tolerance was subtracted when calculating composite section properties.
- Stiffeners: None.
- Beam Section Properties: The rolled beam properties from the year of construction (1960 - ASD5th Edition) are compared to the default rolled sections (LRFD3) in the Merlin Dash analysis. Since the strong axis properties are within 1%, the default properties are used in the model.
- For the FAST Act's Emergency Vehicles, the rolled beam shape WF36x280 for span 2 is not available in the Merlin Dash version 10.10 section database, therefore equivalent plate girder sections are used as a substitute. (Updated 7/31/19)

Dead Load:

- Factors: γ_{DC} = 1.25 Per MBE 6A.2.2.1
  γ_{DW} = 1.50 Per MBE 6A.2.2.1
- Railing: Assumed 25 plf based on NHDOT standard detail.
- Deck: 7 inches thick (from plans).
- Blocking: Applied as distributed load. Assumed constant thickness, utilizing the maximum thickness at the bearings.
- Wearing Surface: 3 inches thick bituminous concrete based on 1983 rehabilitation plans, and field verification.
- Curb: Concrete – determined 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 and 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, Fast Act Emergency Vehicles – See loading figures on pages 6 through 9.
- Number of Lanes: Two based on current striping.

Live Load Distribution:

Per MBE 6.A.3.2 the live load distribution factors for moment and shear were computed in accordance with AASHTO *LRFD* Section 4.6.2.2. While all spans are typical cross-section (a), per AASHTO LRFD Table 4.6.2.2.1-1, the non-composite Span 3 uses the live load distribution factors for superstructure type "Open Steel Grid Deck on Steel Beams". Spans 1 and 2 are composite and use the live load distribution factors for superstructure type "Concrete Deck on Steel beams". For Spans 1 and 2 the live load distribution factors for the interior beams were determined in accordance with AASHTO LRFD Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1 for moment and shear respectively.

The live load distribution factors for the exterior beams were determined in accordance with AASHTO LRFD Section 4.6.2.2.2d and 4.6.2.2.3b for moment and shear respectively. 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 when evaluating the exterior beam. See the hand calculations for the position of the vehicle for interior beam lever rule in Span 3.

The live load distribution factors are summarized below. Spans 1 and 2 are corrected for skew = 56°-27'-51".

Element	Moment	Shear
Interior Beams – Span 1	0.544	0.979
Exterior Beams – Span 1	0.466	0.704
Interior Beams – Span 2	0.539	0.969
Exterior Beams – Span 2	0.471	0.696
Interior Beams – Span 3	0.750	0.833
Exterior Beams – Span 3	0.560	0.560

Condition Factor: (*Rating Guide* Section 3.5.1)

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

System Factor: (*Rating Guide* Section 3.5.2)

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

Material Properties:

Steel: The 1959 bridge construction plans specify the material properties for all weldable steel (Spans 1 and 2) as ASTM A373 (f_y = 32 ksi). All other steel assumed to have a yield strength, f_y, of 33 ksi based on MBE Table 6A.6.2.1-1 and the year of construction. Since the top flange of the beams is embedded in the concrete deck, it is considered to be continuously braced. Lateral torsional buckling will not control the beam capacity.

Concrete: The deck is assumed to be normal weight concrete. The material properties are based on the modular ratio n = 10 specified on the 1959 bridge construction plans and the modulus of elasticity of steel, to derive the concrete compressive strength, f'c = 2.5 ksi.

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

Although the serviceability ratings were low, no serviceability related issues were observed. 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' 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 to determine the finite fatigue life per MBE 7.2.5. The cover plate weld of the Span 1 exterior beam controls the finite fatigue life of the structure.

References

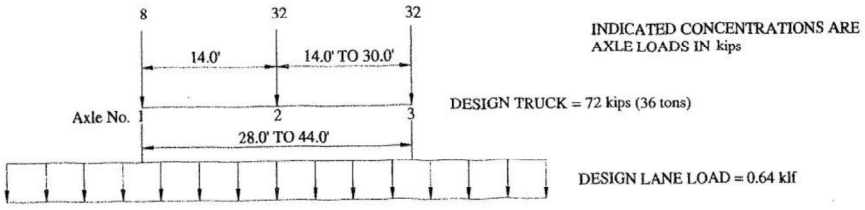
MBE	AASHTO Manual for Bridge Evaluation, 2nd Edition, with 2011 and 2013 Interims.
AASHTO LRFD	AASHTO *LRFD Bridge Design Specifications*, 5th Edition, 2010, with 2010 Interim Revisions.
BDG	MaineDOT *Bridge Design Guide*, 2003, with revisions through August 2008.
Rating Guide	MaineDOT *Draft Load Rating Guide*, November 2011.

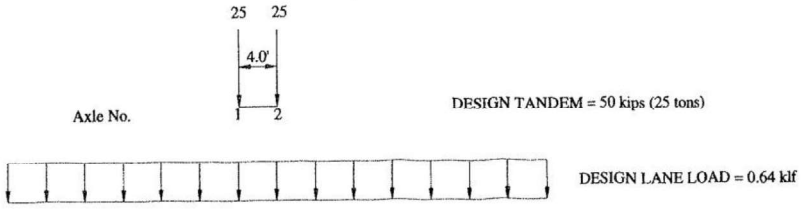
SI&A	MaineDOT Structure Inventory and Appraisal Sheet, June 24, 2010.
LRFD3	Manual of Steel Construction: Load and Resistance Factor Design, American Institute of Steel Construction, Inc., Third Edition, 2001.

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%)

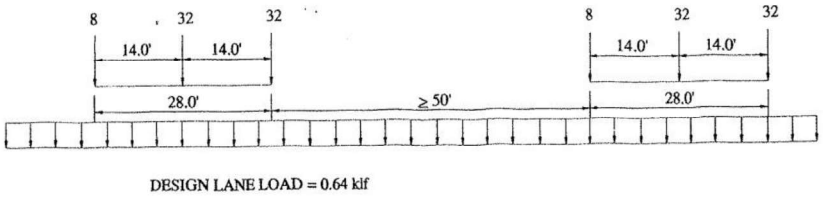
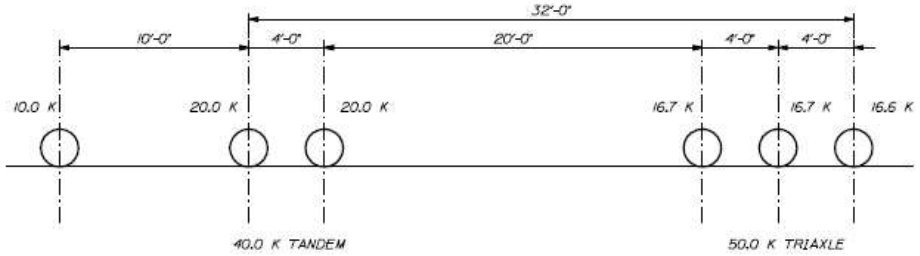


Figure C6A-1—LRFD Design Live Load (HL-93)

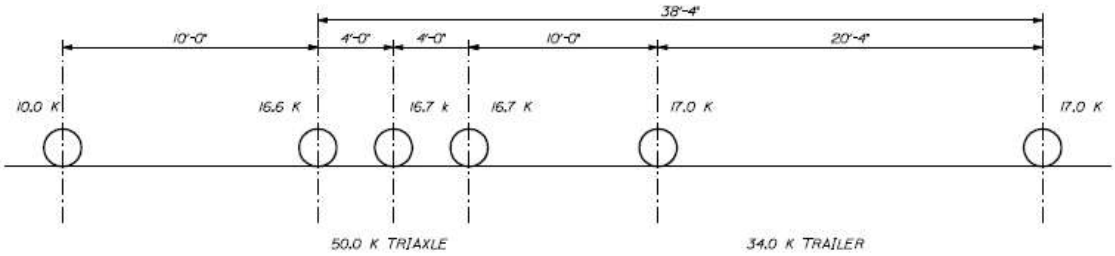
LOADINGS USED FOR BRIDGE RATINGS, CONT'D

Maine Legal Load Configurations:



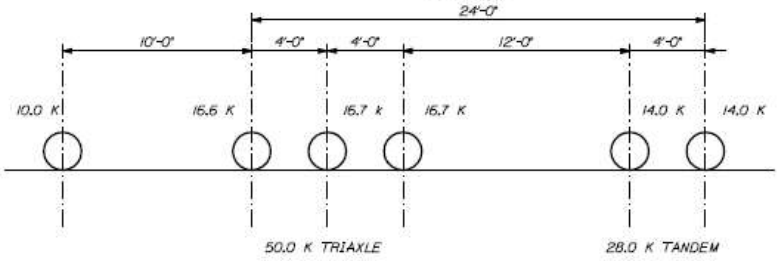
CONFIGURATON 1

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



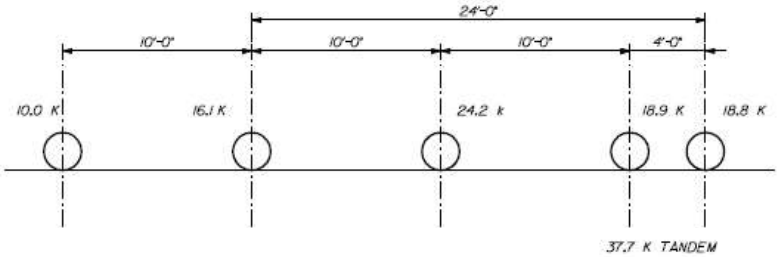
CONFIGURATON 2

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



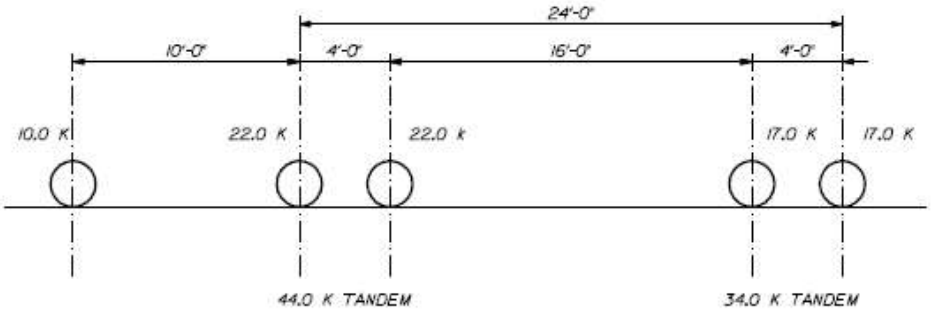
CONFIGURATON 4

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

REVISION 03-07-11

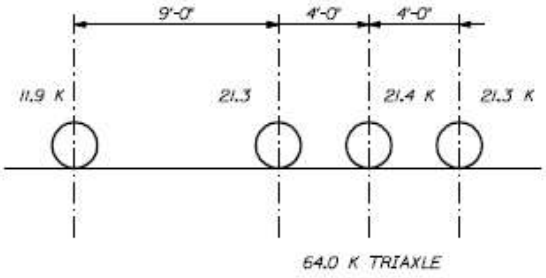
LOADINGS USED FOR BRIDGE RATINGS, CONT'D

Maine Legal Load Configurations:



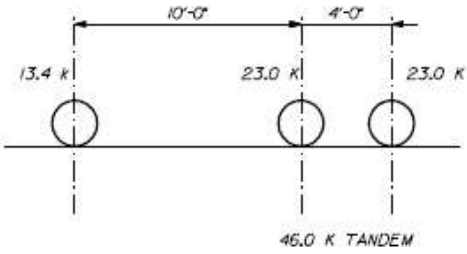
CONFIGURATON 5 — FIVE AXLE TANDEM AXLE TRUCK OR TRACTOR SEMI-TRAILER

88.0 K GVW



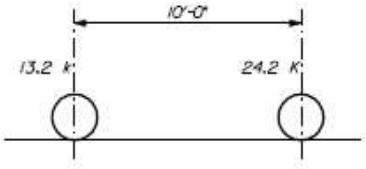
CONFIGURATON 6 — FOUR AXLE TRUCK

75.9 K GVW



CONFIGURATON 7 — THREE AXLE TRUCK

59.0 K GVW

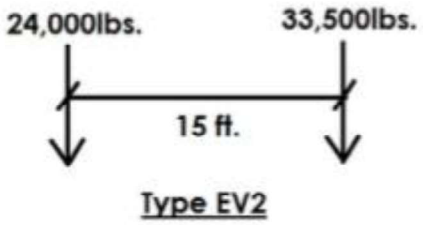


CONFIGURATON 8 — TWO AXLE TRUCK

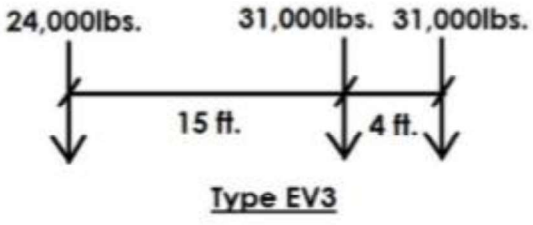
37.4 K GVW

LOADINGS USED FOR BRIDGE RATINGS, CONT'D

FAST Act Legal Load Configurations: (Updated 7/31/19)



Type EV2



Type EV3

PHOTOGRAPHS

Photo No. 1

Location:
Stillwater Avenue

Description:
Span 1
South Elevation



Photo No. 2

Location:
Stillwater Avenue

Description:
Span 2
South Elevation



Photo No. 3

Location:
Stillwater Avenue

Description:
Span 3
South Elevation



Photo No. 4

Location:
Stillwater Avenue

Description:
Abutment No. 1
Elevation



Photo No. 5

Location:
Stillwater Avenue

Description:
Pier No. 1
Elevation



Photo No. 6

Location:
Pier No. 1

Description:
Pier Cap
Deterioration of
Pier No. 1



Photo No. 7
Location: Underside of Bridge
Description: Typical Diaphragms Span 2



Photo No. 8
Location: Cover Plate
Description: Typical Beam Cover Plate Transition



Photo No. 9

Location:
Bridge Joint

Description:
Overhang
Alignment at Pier



Photo No. 10

Location:
Abutment No. 1

Description:
Abutment
Backwall and Seat
Heavy Spalling



Photo No. 11

Location:
Bridge Deck

Description:
Bridge Rail and Concrete Curb

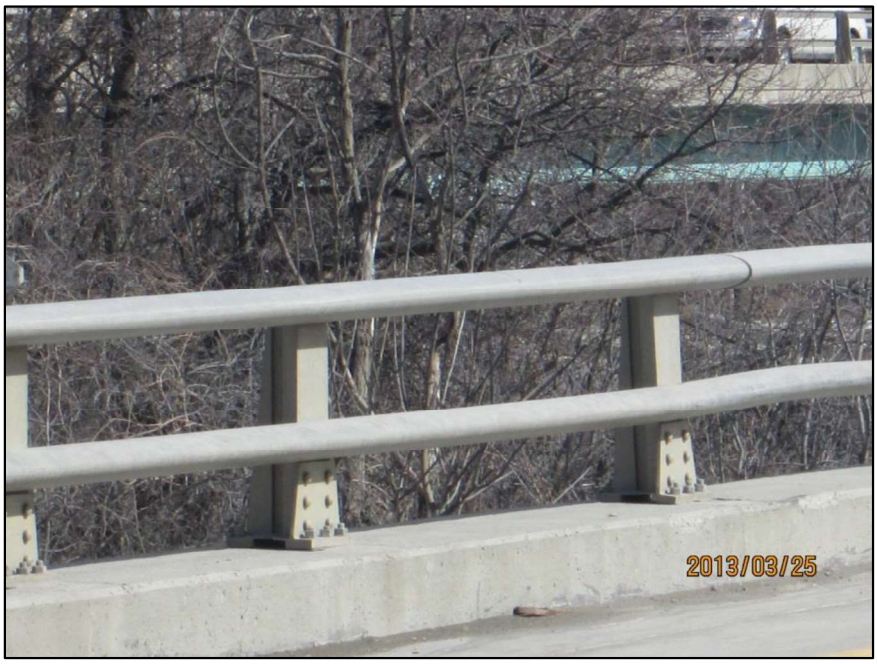


Photo No. 12

Location:
Bridge Deck

Description:
Bridge Guardrail Transition



Photo No. 13

Location:
Pier No. 2

Description:
Bridge Expansion
Bearing
Span 2



Photo No. 14

Location:
Pier No. 2

Description:
Bridge Fixed
Bearing
Span 3



Photo No. 15
Location: Abutment No. 2
Description: Bridge Rail Transition (Trailing End)



Photo No. 16
Location: Bridge Deck
Description: Wearing Surface



APPENDIX A

BACKGROUND INFORMATION

(See CD)

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

Structure Inventory and Appraisal Sheet

Bridge Key: 5800 Agency ID: 5800 SR: 74.8 SD/FO: ND

IDENTIFICATION

State 1:	23 Maine	Struc Num 8:	5800
Facility Carried 7:	I95 NORTHBOUND	Location 9:	0.8 MI N JCT RTE 15
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:	19020 Bangor	Kilometer Post 11:	297.8 km
Feature Intersected 6:	STILLWATER AVENUE		
Latitude 16:	44d 49' 24"	Longitude 17:	068d 45' 30"
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: 3

Main Span Material/Design 43A/B:

3 Steel 02 Stringer/Girder

Deck Type 107:	1 Concrete-Cast-in-Place
Wearing Surface 108A:	6 Bituminous
Membrane 108B:	2 Preformed Fabric
Deck Protection 108C:	None

AGE AND SERVICE

Year Built 27:	1960	Year Reconstructed 106:	2006		
Type of Service on 42A:	1 Highway				
Type of Service under 42B:	1 Highway				
Lanes on 28A:	2	Lanes Under 28B:	2	Detour Length 19:	4.4 km
ADT 29:	23,980	Truck ADT 109:	10 %	Year of ADT 30:	2011

GEOMETRIC DATA

Length Max Span 48:	27.74 m	Structure Length 49:	67.06 m
Curb/Sdwlk Width L 50A:	0.15 m	Curb/Sidewalk Width R 50B:	0.15 m
Width Curb to Curb 51:	9.20 m	Width Out to Out 52:	10.67 m
Approach Roadway Width 32:(w/ shoulders)	12.19 m	Median 33:	0 No median

Deck Area: 715.36 m²

Skew 34: 56.00 ° Structure Flared 35: 0 No flare

Vertical Clearance 10: 99.99 m Horiz. Clearance 47: 9.14 m

Minimum Vertical Clearance Over Bridge 53:	99.90 m
Minimum Vertical Underclearance Reference 54A:	H Hwy beneath struct
Minimum Vertical Underclearance 54B:	04.42 m
Minimum Lateral Underclearance Reference R 55A:	H Hwy beneath struct
Minimum Lateral Underclearance R 55:	03.70 m
Minimum Lateral Underclearance L 56:	03.70 m

INSPECTION

Frequency 91:	24 months	Inspection Date 90:	6/24/2010	Next Inspection:	06/24/2012
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:	06/24/2010	Next Elem. Insp. Due:	06/24/2012

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:	11 Urban 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: 8 Very Good Sub 60: 6 Satisfactory

Culvert 62: N N/A (NBI) Channel/Channel Protection 61: 6 Bank Slumping

LOAD RATING AND POSTING

Inventory Rating Method 65: 2 AS Allowable Stress Operating Rating Method 63:2 AS Allowable Stress

Inventory Rating 66:	MS18.2	Operating Rating 64:	MS33.3
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:	1 Meets Standards	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:	4 Tolerable

Underclearance, Vertical and Horizontal 69: 6 Equal Minimum

Waterway Adequacy 71:	N Not applicable	Approach Alignment 72:	8 Equal Desirable Crit
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:	33,572
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

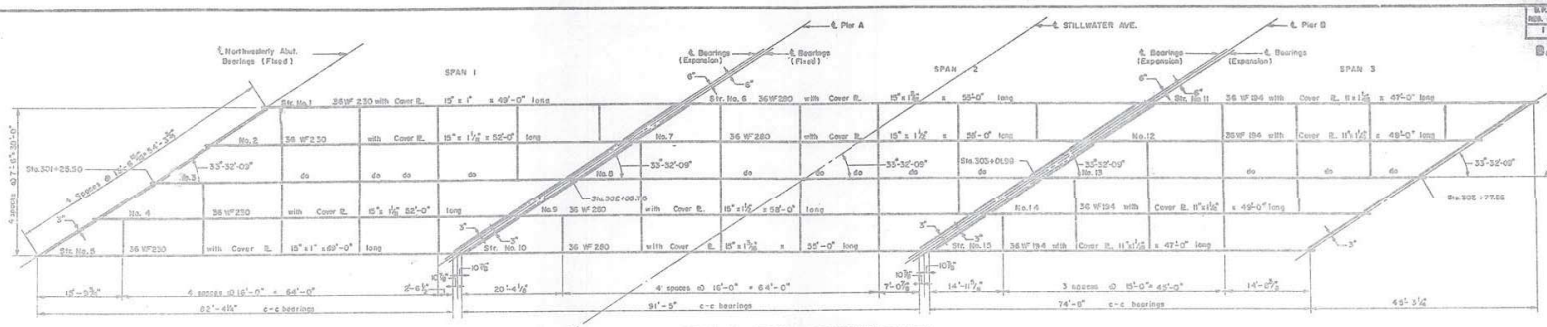
Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	14/2	P Conc Deck/AC Ovly	sq.m.	715	0 %	0	0 %	0	100 %	715	0 %	0	0 %	0
1	107/2	Paint Stl Opn Girder	m.	335	80 %	268	12 %	40	6 %	20	2 %	7	0 %	0
1	205/2	R/Conc Column	ea.	6	0 %	0	82 %	5	18 %	1	0 %	0	0 %	0
1	215/2	R/Conc Abutment	m.	37	0 %	0	43 %	16	55 %	20	2 %	1	0 %	0
1	218/2	Undefined Wall Elem.	m.	27	19 %	5	53 %	14	27 %	7	1 %	0	0 %	0

Structure Inventory and Appraisal Sheet

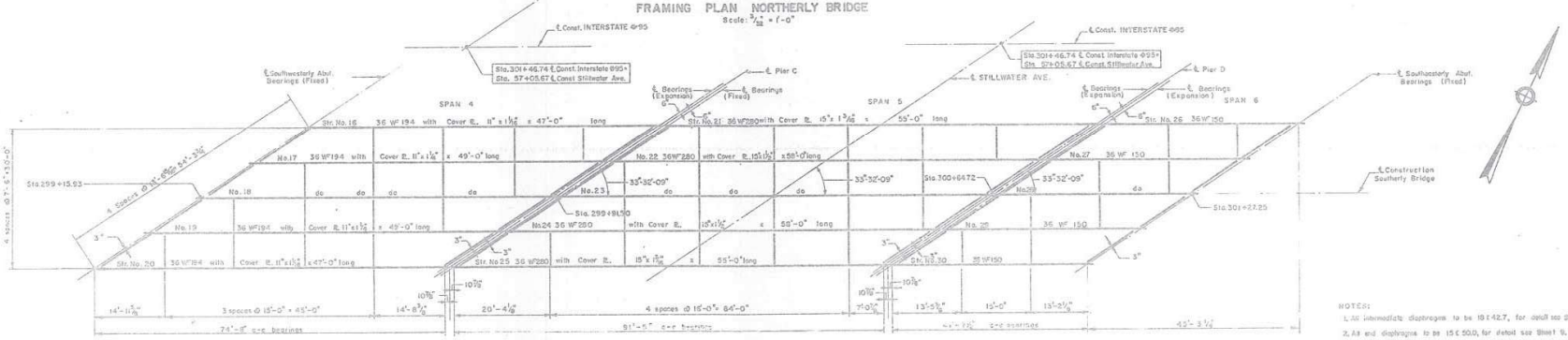
Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	234/2	R/Conc Cap	m.	37	36 %	13	52 %	19	11 %	4	1 %	0	0 %	0
1	300/2	Strip Seal Exp Joint	m.	19	85 %	16	15 %	3	0 %	0	0 %	0	0 %	0
1	302/2	Compressn Joint Seal	m.	19	85 %	16	15 %	3	0 %	0	0 %	0	0 %	0
1	311/2	Moveable Bearing	ea.	30	97 %	29	3 %	1	0 %	0	0 %	0	0 %	0
1	330/2	Metal Rail Uncoated	m.	134	85 %	114	10 %	13	4 %	5	1 %	1	0 %	0
1	383/2	Wear.Surf- AC+Membr.	sq.m.	617	75 %	463	10 %	62	15 %	93	0 %	0	0 %	0
1	388/2	Paint	sq.m.	1,561	82 %	1,280	11 %	172	5 %	78	2 %	31	0 %	0
1	389/2	Reinfor conc dk/slab	sq.m.	715	33 %	236	33 %	236	34 %	243	0 %	0	0 %	0

W.P.R. Proj. No.	STATE	PROJECT NO.	COUNTY NO.	TOTAL SHTS.
1	MAINE	I-95-6(63)	10	20

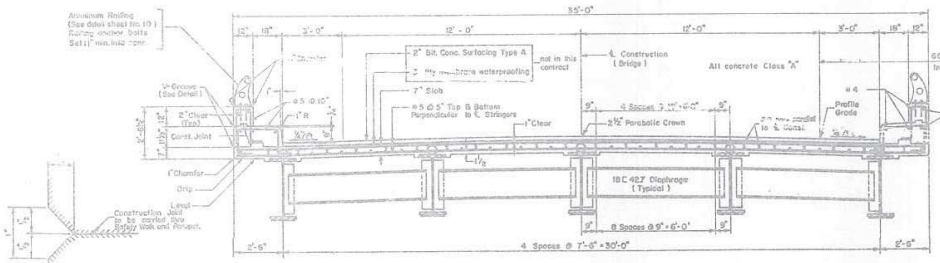
BANGOR INTERSTATE



FRAMING PLAN NORTHERLY BRIDGE
Scale: $\frac{3}{32}$" = 1'-0"



FRAMING PLAN SOUTHERLY BRIDGE
Scale: $\frac{3}{32}$" = 1'-0"



TYPICAL CROSS SECTION
Scale: $\frac{3}{16}$" = 1'-0"

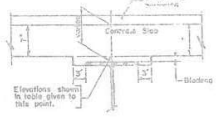
V-GROOVE DETAIL
Not to Scale

ELEVATIONS OF BOTTOM OF ROADWAY SLAB

Point No.	℄ Brg.	¼ Face	¼ Point	⅜ Point	℄ Brg.	Point No.	℄ Brg.	¼ Point	⅜ Point	℄ Brg.
1	150.61	151.071	151.181	151.311	151.29	16	145.48	145.875	146.251	146.63
2	150.85	151.196	151.373	151.441	151.43	17	145.48	145.887	146.236	146.475
3	150.91	151.196	151.386	151.443	151.45	18	145.37	145.785	146.150	146.58
4	150.74	151.033	151.289	151.31	151.301	19	145.09	145.511	145.861	145.121
5	150.457	150.752	150.951	151.041	151.05	20	144.701	145.150	145.466	145.742
6	151.10	151.542	151.743	151.926	151.92	21	146.66	147.146	147.594	147.74
7	151.44	151.769	151.969	151.984	151.787	22	146.66	147.127	147.551	147.795
8	151.46	151.758	151.909	151.959	151.84	23	146.59	147.050	147.451	147.726
9	151.32	151.602	151.771	151.882	151.72	24	146.34	146.810	147.242	147.65
10	151.06	151.350	151.516	151.662	151.49	25	145.98	146.484	146.882	147.14
11	151.65	151.764	151.830	151.917	151.72	26	147.66	148.02	148.15	148.26
12	151.79	151.921	152.105	151.987	151.961	27	147.94	148.06	148.12	148.32
13	151.64	151.587	152.074	152.063	151.88	28	147.98	148.02	148.16	148.27
14	151.73	151.584	151.973	151.984	151.89	29	147.63	147.87	147.84	147.98
15	151.50	151.453	151.741	151.793	151.66	30	147.32	147.47	147.61	147.75

*Low station end of beam.

Note: After the structural steel is erected and before forms are built or concrete is placed, elevations on the top flange of the beams are to be obtained at the points indicated in the table. The differences between these elevations and those shown in the table give the actual blocking distance from top of the beam to the bottom of the slab.



DETAIL OF HAUNCH
Scale: 1"=1'-0"

NOTES:
1. All intermediate diaphragms to be 18 C 42.7, for detail see Sheet 9.
2. All end diaphragms to be 15 C 50.0, for detail see Sheet 9.
3. For Cover Plate and Shear Connector details see Sheet 9.
4. All beams to be cambered to the extent that the bridge deck under full dead load will follow as nearly as practicable the design profile, and in no case with less camber than is likely to remain permanent. The proposed camber diagram shall be submitted to the Engineer for approval.
5. Parapets and safety walks to have transverse construction joints 20'± on centers. Joints are not to be located under railing posts. Contact surfaces of joints to be painted with a bituminous material. Exposed surfaces are to chamfered ¾". Reinforcing steel to be carried thru joints.
6. No paint shall be applied to the top flanges of beams to which shear connectors are to be welded.
7. The concrete deck for each span shall be placed in one continuous operation.

THE CLARKESON ENGINEERING CO., INC.

DESIGN J.T.S R.F.	CHECK D.S.	BRIDGE NO. SURVEY
DRAWN E.K.	APPROVED W.A.M. B.C.M.	PLOT

STATE HIGHWAY COMMISSION

INTERSTATE #95
OVER
STILLWATER AVENUE
IN THE CITY OF
BANGOR
PENOBSCOT COUNTY

FRAMING PLAN & TYPICAL CROSS SECTION

SHEET 8 OF 10 AUGUSTA, MAINE

78-17

0 1 2 3 4 5 INCHES

APPENDIX B

LOAD RATING COMPUTATIONS
(See CD)

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

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

- Moment of Inertia Calculation
  - Span 1
  - Span 2

- Mathcad Calculations
  - Rigid Body Rotation Calculation
  - Finite Fatigue Life Evaluation

- Merlin-Dash Output:
  - HL-93 Rating — Interior Beam – Span 1 CP and Fatigue
  - HL-93 Rating — Interior Beam –Span 1 Midspan
  - HL-93 Rating — Exterior Beam – Span 1
  - HL-93 Rating — Interior Beam – Span 2
  - HL-93 Rating — Exterior Beam – Span 2
  - HL-93 Rating — Interior Beam – Span 3
  - HL-93 Rating — Exterior Beam – Span 3
  - EV2 Rating — Interior Beam – Span 1 CP and Fatigue (Updated 7/31/19)
  - EV2 Rating — Interior Beam –Span 1 Midspan (Updated 7/31/19)
  - EV2 Rating — Exterior Beam – Span 1 (Updated 7/31/19)
  - EV2 Rating — Interior Beam – Span 2 (Updated 7/31/19)
  - EV2 Rating — Exterior Beam – Span 2 (Updated 7/31/19)
  - EV2 Rating — Interior Beam – Span 3 (Updated 7/31/19)
  - EV2 Rating — Exterior Beam – Span 3 (Updated 7/31/19)
  - EV3 Rating — Interior Beam – Span 1 CP and Fatigue (Updated 7/31/19)
  - EV3 Rating — Interior Beam –Span 1 Midspan (Updated 7/31/19)
  - EV3 Rating — Exterior Beam – Span 1 (Updated 7/31/19)
  - EV3 Rating — Interior Beam – Span 2 (Updated 7/31/19)
  - EV3 Rating — Exterior Beam – Span 2 (Updated 7/31/19)
  - EV3 Rating — Interior Beam – Span 3 (Updated 7/31/19)

- EV3 Rating Exterior Beam – Span 3 (Updated 7/31/19)

- Hand Calculations
  - Post-processed EV Load Ratings (Updated 7/31/19)

THE Louis Berger Group, INC.
482 Congress Street, Suite 401
Portland, ME 04101

BY IAS/TDM DATE 04/30/13 SHEET 1 OF 5
CHKD BY KSW DATE 07/10/13 PROJECT CKE410A4
SUBJECT # 5800 - I-95 Northbound - Span 1 (4)

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 - 5800, Span 1 (4)

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} =	74.67'	
curb-to-curb width of bridge =	b_{pvmt} =	31.33'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	35.00'	
number of girders =	N_g =	5	
girder spacing =	S_g =	7.500'	
maximum top flange width =	b_{f_max} =	12.12"	WF 36x194
maximum top flange thickness =	t_{f_max} =	1.26"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~7.00"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	7.00"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	9.00"	per plans
width of overhang =	$b_{overhang}$ =	2.500 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} =	2.50"	
width of haunch =	b_{haunch} =	18.12"	
weight of single girder haunch =	W_{haunch} =	**0.031 klf**	(bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.500 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	6.250 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	**0.656 klf**	Average Thickness = 0.58'
deck weight for exterior girder =	W_{deck_ext} =	**0.547 klf**	Average Thickness = 0.58'

MERLIN Dead Loads - 5800, Span 1 (4) (CONT'D)

References

Wearing Surface:

type of wearing surface =		*Asphalt*	
thickness of wearing surface =	t_{pvmt} =	*3.00"*	(average)
thickness of waterproofing membrane =	t_{wm} =	*0.250"*	
total weight of wearing surface =	W_{ws} =	**1.188 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} =	*1.91 ft²*	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	*0.00 ft²*	(granite curb only, if used)
weight of curbs =	W_{curbs} =	**0.573 klf**	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	*2.50"*	Field measured
width of soffit =	b_{soffit} =	*7.90"*	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	**0.021 klf**	

MERLIN Dead Loads - 5800, Span 1 (4) (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$ =	2-bar Alum.	0.025 klf	Similar to NHDOT standard rail
$Rail_{Right}$ =	2-bar Alum.	0.025 klf	Similar to NHDOT standard rail
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.050 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 intermediate diaphragm.

# of Attachment Plate 1 =	*2*	
Attachment Plate 1 Height =	*30.00"*	
Attachment Plate 1 Width =	*6.00"*	
Attachment Plate 1 Thickness =	*0.50"*	
Attachment Plate 1 Weight =	0.051 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 =	*43 plf*	
Length of Horizontal Member =	*7.00'*	
Horizontal Member Weight =	0.299 kip	
Interior Girder Diaphragm Weight =	*0.35 kips*	
Exterior Girder Diaphragm Weight =	*0.17 kips*	1/2 Interior Girder Diaphragm Weight
Interior Girder End Diaphragm Weight =	*0.76 kips*	Hand calculations
Exterior Girder End Diaphragm Weight =	*0.38 kips*	Hand calculations

MERLIN Dead Loads - 5800, Span 1 (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	

MERLIN Dead Loads - 5800, Span 1 (4) (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = **0.688 klf** = $W_{deck_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.599 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) = **0.350 kip** per location

Diaphragm (exterior girder) = **0.175 kip** per location

End Diaphragm (interior girder) = **0.760 kip** per location

End Diaphragm (exterior girder) = **0.380 kip** per location

MERLIN loads - Composite (DC2):

wearing surface = **0.238 klf** = $W_{ws} \div N_g$

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

railing = **0.010 klf** = $W_{rails} \div N_g$

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

curb = **0.115 klf** = $W_{curbs} \div N_g$

BRIDGE RATING

The Manual for Bridge Evaluation - Second Edition with 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

Live Load Distribution: Span 1 (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	0.544	0.979	0.328	0.691
Exterior Stringer	0.466	0.704	0.366	0.553

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		23980	From SI&A
$\%_{trucks}$		10%	From SI&A
ADTT = $\%_{trucks}$*ADT		2398	
	Single Lane	2038	Merlin Dash 06012
Condition Factor	ϕ_c =	1	Merlin Dash 06012
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	56.46	degrees	
Minimum Shoulder Width	$W_{shoulder}$	3.667	Ft.	
Girder/Stringer Spacing	S	7.5	Ft.	
Girder/Stringer Span Length	L	74.667	Ft.	
Deck Overhang	d_{ovhg}	2.5	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.833	Ft.	
Roadway Overhang	d_e	0.667	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	7	in.	
Deck Concrete Strength	f'c	2.5	ksi	**Based on plans, n=10 and E_B**
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	5		
Number of Design Lanes	N_L	2		**LRFD 3.6.1.1.1**
Average Depth of beam	d	37.21	in.	**See Hand Calculation**
Average Area of Beam	A	65.12	in^2	**See Hand Calculation**
Avg. Moment of Inertia for Beam	I	14417.99	in^4	**See Hand Calculation**
Top Flange Thickness	t_{tf}	1.26	in	**AISC Historic Shape Database**
Overcamber Allowance		0.875	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	2881	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	10.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	24.26	in.	
Longitudinal Stiffness Parameter	K_g	527328		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.06		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.14		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.85		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.26		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		0.83		**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	7.5	OK
Slab	7	OK
Length	74.6667	OK
No. Beams	5	OK
K_g	527328	OK

g_1 =	0.473	One lane loaded
g_m =	0.653	Two or more lanes loaded
g_{1_fat} =	0.328	One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.544**	Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

g_1 =	0.660	One lane loaded
g_m =	0.779	Two or more lanes loaded
g_{1_fat} =	0.691	One lane loaded without multiple presence for fatigue analysis
Control g_c =	**0.979**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:
Roadway Overhang *0.67* OK

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.551		Two or more lanes loaded
	g_{RB1} =	0.528		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.560		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	**g_c =**	**0.466**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.519		Two or more lanes loaded
	g_{RB1} =	0.528		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.560		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	g_{1_fat} =	0.553		One lane loaded without multiple presence for fatigue analysis
Control	**g_c =**	**0.704**		Corrected for skew

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	36.48	12.12	57.11	18.24	1041.69	0.00	0.00	12103.40	12103.40
				$\bar{y}$ (in)=	18.24				

Moment of Inertia Calculation for Rolled Shape with Cover Plate Exterior Beam - Midspan									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	36.48	12.12	57.11	19.30	1102.37	3.19	580.76	12103.40	12684.16
Plate (bottom)	1.06	11.00	11.69	0.53	6.21	-15.58	2837.83	1.10	2838.93
Total	37.54		68.80		1108.57				15523.09
				$\bar{y}$ (in)=	16.11				

Moment of Inertia Calculation for Rolled Shape with Cover Plate Interior Beam - Midspan									
Section	**Depth (in)**	**Width (in)**	**A (in^2)**	**y (in)**	**Ay (in^3)**	**d**	**Ad^2**	**I_o**	**I**
Beam	36.48	12.12	57.11	19.49	1113.07	3.66	765.30	12103.40	12868.70
Plate (bottom)	1.25	11.00	13.75	0.63	8.59	-15.20	3178.62	1.79	3180.41
Total	37.73		70.86		1121.67				16049.10
				$\bar{y}$ (in)=	15.83				

Diaphragm Spacing	
Angle	33.5358
Girder Spa	7.5
Tan (Angle	0.66279
	11.3159
	14.9688
	3.65287
	14.6979
	11.0138

Length of Bridge 74.667 ft
Length of Exterior Coverplate 45 ft Does not Include Transition Length
Length of Interior Coverplate 47 ft Does not Include Transition Length

Find Weighted Average Properties of Interior and Exterior Beams

Average Depth	37.21	in	See Hand Calculation
Average Area	65.12	in^2	See Hand Calculation
Average I_o	14417.99	in^4	See Hand Calculation
Average $\bar{y}$	16.82	in	See Hand Calculation
e_g =	24.26	in	See Hand Calculation

Interior Girder		Exterior Girder	
0.453	0.453		
3.20	3.653	0.453	0.453
15.0	18.653	14.516	14.969
15.0	33.653	15.0	29.969
15.0	48.653	15.0	44.969
15.0	63.653	15.0	59.969
10.561	74.214	14.245	74.214
0.453	74.667	0.453	74.667

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Symmetrical Deck, Equal Girder Spacing

MERLIN Dead Loads - 5800, Span 2 (5)

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} =	91.42'	
curb-to-curb width of bridge =	b_{pvmt} =	31.33'	
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	35.00'	
number of girders =	N_g =	5	
girder spacing =	S_g =	7.500'	
maximum top flange width =	b_{f_max} =	16.60"	WF 36 x 280
maximum top flange thickness =	t_{f_max} =	1.57"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~7.00"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	7.00"	(Constant Deck Thickness)
~~cross slope =~~	~~c_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	9.00"	per plans
width of overhang =	$b_{overhang}$ =	2.500 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} =	2.50"	
width of haunch =	b_{haunch} =	22.60"	
weight of single girder haunch =	W_{haunch} =	**0.032 klf**	(bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.500 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	6.250 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	**0.656 klf**	Average Thickness = 0.58'
deck weight for exterior girder =	W_{deck_ext} =	**0.547 klf**	Average Thickness = 0.58'

MERLIN Dead Loads - 5800, Span 2 (5) (CONT'D)

References

Wearing Surface:

type of wearing surface =		*Asphalt*	
thickness of wearing surface =	t_{pvmt} =	*3.00"*	(average)
thickness of waterproofing membrane =	t_{wm} =	*0.250"*	
total weight of wearing surface =	W_{ws} =	**1.188 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} =	*1.91 ft²*	(concrete portion)
cross-sectional area of granite curb =	A_{curbs} =	*0.00 ft²*	(granite curb only, if used)
weight of curbs =	W_{curbs} =	**0.573 klf**	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	1.56"	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	**0.000 klf**	

MERLIN Dead Loads - 5800, Span 2 (5) (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$ =	2-bar Alum.	0.025 klf	Similar to NHDOT standard
$Rail_{Right}$ =	2-bar Alum.	0.025 klf	Similar to NHDOT standard
Barrier Mounted Rail Left =	NONE	0.000 klf	
Barrier Mounted Rail Right =	NONE	0.000 klf	
total weight of railing =	W_{rails} =	0.050 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 intermediate diaphragm.

# of Attachment Plate 1 =	*2*	
Attachment Plate 1 Height =	*30.00"*	
Attachment Plate 1 Width =	*6.00"*	
Attachment Plate 1 Thickness =	*0.50"*	
Attachment Plate 1 Weight =	0.051 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 =	*43 plf*	
Length of Horizontal Member =	*7.00'*	
Horizontal Member Weight =	0.299 kip	
Interior Girder Diaphragm Weight =	*0.35 kips*	
Exterior Girder Diaphragm Weight =	*0.17 kips*	1/2 Interior Girder Diaphragm Weight
Interior Girder End Diaphragm Weight =	*0.76 kips*	Hand calculations
Exterior Girder End Diaphragm Weight =	*0.38 kips*	Hand calculations

MERLIN Dead Loads - 5800, Span 2 (5) (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

MERLIN Dead Loads - 5800, Span 2 (5) (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = **0.688 klf** = $W_{deck_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.579 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) = **0.350 kip** per location

Diaphragm (exterior girder) = **0.175 kip** per location

End Diaphragm (interior girder) = **0.760 kip** per location

End Diaphragm (exterior girder) = **0.380 kip** per location

MERLIN loads - Composite (DC2):

wearing surface = **0.238 klf** = $W_{ws} \div N_g$

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

railing = **0.010 klf** = $W_{rails} \div N_g$

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

curb = **0.115 klf** = $W_{curbs} \div N_g$

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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	0.539	0.969	0.319	0.684
Exterior Stringer	0.471	0.696	0.370	0.547

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	23980	From SI&A
$\%_{trucks}$	10%	From SI&A
ADTT = $\%_{trucks}$*ADT	2398	

Merlin Dash 06012

Condition Factor	ϕ_c =	1
System Factor	ϕ_s =	1
	$\phi_c*\phi_s$ =	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	56.4642	degrees	
Minimum Shoulder Width	$W_{shoulder}$	3.667	Ft.	
Girder/Stringer Spacing	S	7.5	Ft.	
Girder/Stringer Span Length	L	91.4167	Ft.	
Deck Overhang	d_{ovhg}	2.5	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.83333	Ft.	
Roadway Overhang	d_e	0.66667	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	7	in.	
Deck Concrete Strength	f'c	2.5	ksi	**Based on plans, n=10 and E_B**
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	5		
Number of Design Lanes	N_L	2		**LRFD 3.6.1.1.1**
Average Depth of beam	d	37.33	in.	**See Hand Calculation**
Average Area of Beam	A	94.72	in^2	**See Hand Calculation**
Avg. Moment of Inertia for Beam	I	22371.78	in^4	**See Hand Calculation**
Top Flange Thickness	t_{tf}	1.57	in	**AISC Historic Shape Database**
Overcamber Allowance		1.125	in	**From NHBDM**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	2881	ksi	**LRFD 5.4.2.4-1**
Modular Ratio	n	10.0		**LRFD 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	24.03	in.	
Longitudinal Stiffness Parameter	K_g	770509		**LRFD 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		1.07		**LRFD Table 4.6.2.2.2b-1**
Longitudinal Stiffness Constant (Skew Correction for Moment)		1.20		**LRFD Table 4.6.2.2.2e-1**
Longitudinal Stiffness Constant (Skew Correction for Shear)		0.81		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Shear		1.24		**LRFD Table 4.6.2.2.3c-1**
Skew Correction for Moment		0.84		**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	7.5	OK
Slab	7	OK
Length	91.4167	OK
No. Beams	5	OK
K_g	770509	OK

	g_1 =	0.455	One lane loaded
	g_m =	0.640	Two or more lanes loaded
	g_{1_fat} =	0.319	One lane loaded without multiple presence for fatigue analysis
Control	**g_c =**	**0.539**	Corrected for skew

Shear LRFD Table 4.6.2.2.3a-1

	g_1 =	0.660	One lane loaded
	g_m =	0.779	Two or more lanes loaded
	g_{1_fat} =	0.684	One lane loaded without multiple presence for fatigue analysis
Control	**g_c =**	**0.969**	Corrected for skew

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:
Roadway Overhang *0.67* OK

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.540		Two or more lanes loaded
	g_{RB1} =	0.528		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.560		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	g_{1_fat} =	0.370		One lane loaded without multiple presence for fatigue analysis
Control	**g_c =**	**0.471**		Corrected for skew

Shear LRFD Table 4.6.2.2.3b-1

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	0.519		Two or more lanes loaded
	g_{RB1} =	0.528		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calculation)
	g_{RB2} =	0.560		Rigid Body Rotation C4.6.2.2.2d-2 (See MathCad Calculation)
	g_{1_fat} =	0.547		One lane loaded without multiple presence for fatigue analysis
Control	**g_c =**	**0.696**		Corrected for skew

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Portland, ME 04101

BY IAS/TDM DATE: 04/30/13 SHEET 1 OF 1
CHKD BY KSW DATE: 07/10/13 PROJECT CKE410A4
SUBJECT # 5800 - I-95 Northbound - Span 2 (5)

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	36.50	16.60	82.32	18.25	1502.34	0.00	0.00	18819.30	18819.30
				$\bar{y}$ (in)=	18.25				

Moment of Inertia Calculation for Rolled Shape with Cover Plate Exterior Beam - Midspan									
Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	36.50	16.60	82.32	19.44	1600.10	3.35	925.00	18819.30	19744.30
Plate (bottom)	1.1875	15.00	17.81	0.59	10.58	-15.49	4274.84	2.09	4276.94
Total	37.69		100.13		1610.67				24021.23
				$\bar{y}$ (in)=	16.09				

Moment of Inertia Calculation for Rolled Shape with Cover Plate Interior Beam - Midspan									
Section	Depth (in)	Width (in)	A (in^2)	y (in)	Ay (in^3)	d	Ad^2	I_o	I
Beam	36.50	16.60	82.32	19.75	1625.82	4.08	1369.27	18819.30	20188.57
Plate (bottom)	1.50	15.00	22.50	0.75	16.88	-14.92	5009.70	4.22	5013.92
Total	38.00		104.82		1642.70				25202.49
				$\bar{y}$ (in)=	15.67				

Diaphragm Spacing	
Angle	33.5358
Girder Spa	7.5
Tan (Angle	0.66279
	11.3159
	20.3438
	13.712
	7.07292
	13.7047

Length of Bridge 91.417 ft
Length of Exterior Coverplate 53 ft Does not Include Transition Length
Length of Interior Coverplate 56 ft Does not Include Transition Length

Find Weighted Average Properties of Interior and Exterior Beams

Average Depth	37.33	in	See Hand Calculation
Average Area	94.72	in^2	See Hand Calculation
Average I_o	22371.78	in^4	See Hand Calculation
Average $\bar{y}$	16.80	in	See Hand Calculation

Interior Girder		Exterior Girder	
0.453	0.453		
13.259	13.712	0.453	0.453
16.000	29.712	19.891	20.344
16.000	45.712	16.0	36.344
16.000	61.712	16.0	52.344
16.000	77.712	16.0	68.344
13.252	90.964	16.0	84.344
0.453	91.417	6.620	90.964
		0.453	91.417

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 - No. 5800 Span 3 (6)

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} =	41.63'	Per Plans
curb-to-curb width of bridge =	b_{pvmt} =	31.33'	Per Rehabilitation Plans
clear width of sidewalk =	b_{sw} =	0.00'	(No sidewalk on this structure)
out-to-out width of bridge deck =	b_{deck} =	35.00'	Per Plans
number of girders =	N_g =	5	
girder spacing =	S_g =	7.500'	
maximum top flange width =	b_{f_max} =	11.97"	WF 36x150
maximum top flange thickness =	t_{f_max} =	0.940"	
~~thickness of concrete deck @ fascia =~~	~~t_{deck_f} =~~	~~7.00"~~	
thickness of concrete deck @ CL deck =	t_{deck_CL} =	7.00"	(Constant Deck Thickness)
~~cross slope =~~	~~e_s =~~	~~0.0%~~	(Enter value if deck thickness varies)
curb reveal =	h_{reveal} =	9.00"	From Plans - Could Not Measure in Field
width of overhang =	$b_{overhang}$ =	2.500 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} =	2.50"	
width of haunch =	b_{haunch} =	17.97"	
weight of single girder haunch =	W_{haunch} =	**0.035 klf**	bhaunch*thaunch - bf_max*tf_max)*wc

Concrete Deck Loads:

tributary width for interior girder =	TW_{int} =	7.500 ft	= S_g
tributary width for exterior girder =	TW_{ext} =	6.250 ft	= $S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	**0.656 klf**	Average Thickness = 0.58'
deck weight for exterior girder =	W_{deck_ext} =	**0.547 klf**	Average Thickness = 0.58'

MERLIN Dead Loads - No. 5800 Span 3 (6) (CONT'D)

References

Wearing Surface:

type of wearing surface =		*Asphalt*	
thickness of wearing surface =	t_{pvmt} =	*3.00"*	
thickness of waterproofing membrane =	t_{wm} =	*0.250"*	
total weight of wearing surface =	W_{ws} =	**1.188 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} =	*1.91 ft²*	See Hand Calculation
cross-sectional area of granite curb =	A_{curbs} =	*0.00 ft²*	No Granite Curb
weight of curbs =	W_{curbs} =	**0.573 klf**	

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	2.50"	
width of soffit =	b_{soffit} =	7.90"	= $b_{overhang}$ - (b_{haunch}/2)
soffit weight =	W_{soffit} =	**0.021 klf**	

MERLIN Dead Loads - No. 5800 Span 3 (6) (CONT'D)

References

Railings:

Current Standard MaineDOT Bridge Rail?	*No*		
$Rail_{Left}$=	Type IIIA	0.025 klf	Similar to NHDOT standard
$Rail_{Right}$=	Type IIIA	0.025 klf	Similar to NHDOT standard
$Rail_{Left}$=	Other	0.000 klf	
$Rail_{Left}$=	Other	0.000 klf	
total weight of railing =	W_{rails} =	0.050 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 intermediate diaphragm.

# of Attachment Plate 1 =	*2*	
Attachment Plate 1 Height =	*30.00"*	Field Measured
Attachment Plate 1 Width =	*6.00"*	Per Plans
Attachment Plate 1 Thickness =	*0.500"*	Per Plans
Attachment Plate 1 Weight =	0.051 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 =	*43.0 plf*	
Length of Horizontal Member =	*7.00'*	
Horizontal Member Weight =	0.301 kip	
Interior Girder Diaphragm Weight =	*0.35 kips*	
Exterior Girder Diaphragm Weight =	*0.18 kips*	1/2 Interior Girder Diaphragm Weight
Interior Girder End Diaphragm Weight =	*0.76 kips*	Hand calculations
Exterior Girder End Diaphragm Weight =	*0.38 kips*	Hand calculations

MERLIN Dead Loads - No. 5800 Span 3 (6) (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

MERLIN Dead Loads - No. 5800 Span 3 (6) (CONT'D)

References

MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) =	**0.691 klf**	= $W_{deck_int} + W_{haunch}$
deck + haunch + soffit (exterior girder) =	**0.603 klf**	= $W_{deck_ext} + W_{haunch} + W_{soffit}$

MERLIN loads - Additional non-composite loads (DC1):

utility loads (interior girder) =	**0.000 klf**	= W_{util_int}
utility loads (exterior girder) =	**0.000 klf**	= W_{util_ext}
Diaphragm (interior girder) =	**0.352 kip** per location	
Diaphragm (exterior girder) =	**0.176 kip** per location	
End Diaphragm (interior girder) =	**0.760 kip** per location	
End Diaphragm (exterior girder) =	**0.380 kip** per location	

wearing surface (interior girder) =	**0.341 klf**	= $W_{ws}*T_{wint}/b_{pvmt}*(1.5 / 1.25)$
wearing surface (exterior girder) =	**0.201 klf**	= $W_{ws}*(T_{wext} - d_{curb})/b_{pvmt}*(1.5 / 1.25)$
sidewalk (exterior girder) =	**0.000 klf**	= $W_{sw_conc}/2$
railing (exterior girder)=	**0.025 klf**	= $W_{rails}/2$
snow fence (exterior girder)=	**0.000 klf**	= $W_{fences}/2$
curb (exterior girder) =	**0.287 klf**	= $W_{curb}/2$

BRIDGE RATING

The Manual for Bridge Evaluation - Second Edition with 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

Live Load Distribution: Span 3 (6)

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:

Element	Moment	Shear
Interior Stringer	0.750	0.833
Exterior Stringer	0.560	0.560

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	23980	From SI&A
$\%_{trucks}$	10%	From SI&A
ADTT = $\%_{trucks}$*ADT	2398	

Merlin Dash 06012

Condition Factor	ϕ_c =	1
System Factor	ϕ_s =	1
	$\phi_c*\phi_s$ =	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	0	degrees	No skew correction for non-composite
Minimum Shoulder Width	$W_{shoulder}$	3.66667	Ft.	
Girder/Stringer Spacing	S	7.5	Ft.	
Girder/Stringer Span Length	L	41.625	Ft.	
Deck Overhang	d_{ovhg}	2.5	Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.83333	Ft.	
Roadway Overhang	d_e	0.66667	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	7	in.	
Deck Concrete Strength	f'c	2.5	ksi	**Based upon Modular Ratio and E_B**
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	5		
Number of Design Lanes	N_L	2		**LRFD 3.6.1.1.1**
Total Depth of beam	d	35.84	in.	**From AISC Historic Shape Database**
Area of Beam	A	44.16	in^2	**From AISC Historic Shape Database**
Moment of Inertia for Beam	I	9012.1	in^4	**From AISC Historic Shape Database**
Top Flange Thickness	t_{tf}	0.94	in	**From AISC Historic Shape Database**
Modulus of Elasticity for Beam	E_B	29000	ksi	
Modulus of Elasticity for Deck	E_D	2881	ksi	**LRFD 5.4.2.4-1**

Skew Correction for Shear	1.00
Skew Correction for Moment	1.00

Interior Beams **REFERENCE**

Moment

Reinforced Concrete Deck LRFD Table 4.6.2.2.2b-1
(Non-Composite section similar to the open steel grid deck on steel beams)

Check Range of Applicability:

One Lane Loaded	Spacing	7.5	NG
Two Lanes Loaded	Spacing	7.5	OK

	g_1 =	N/A	One lane loaded
	g_m =	0.750	Two or more lanes loaded (S/10)
Control	g_c =	**0.750**	No skew correction for Non-composite

Shear LRFD Table 4.6.2.2.3a-1
(Non-Composite section similar to the open steel grid deck on steel beams)

	g_1 =	0.720	Lever Rule	One lane loaded (m=1.2)	Hand calculation
	g_m =	0.833	Lever Rule	Two lanes loaded (m=1.0)	Hand calculation
Control	g_c =	**0.833**		No skew correction for Non-composite	

Exterior Beams **REFERENCE**

Moment LRFD Table 4.6.2.2.2d-1
(Non-Composite section similar to the open steel grid deck on steel beams)

Reinforced Concrete Deck

Check Range of Applicability:
Roadway Overhang *0.67* OK

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	N/A		Two or more lanes loaded
	g_{RB} =	0.560		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calc)
Control	**g_c =**	**0.560**		No skew correction for Non-composite

Shear LRFD Table 4.6.2.2.3b-1
(Non-Composite section similar to the open steel grid deck on steel beams)

	g_1 =	0.507	Lever Rule	One lane loaded (m=1.2) without accounting for lane line.
	g_{1l} =	0.360	Lever Rule	One lane loaded (m=1.2) accounting for lane line.
	g_m =	N/A	Lever Rule	Two or more lanes loaded
	g_{RB} =	0.560		Rigid Body Rotation C4.6.2.2.2d-1 (See MathCad Calc)
Control	**g_c =**	**0.560**		No skew correction for Non-composite

Diaphragm Spacing	
Angle	33.5358
Girder Spa	7.5 ft
Tan (Angle)	0.66279
	11.316
	13.448
	2.132
	13.177
	9.493

Interior Girder			
0.453	0.453	Exterior Girder	
1.6795	2.132	0.453	0.453
15.0	17.132	12.995	13.448
15.0	32.132	15.0	28.448
9.040	41.172	12.725	41.172
0.453	41.625	0.453	41.625

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BY: IAS/TDM DATE: 6/26/2013 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/10/2013 SUBJECT: #5800 - I - 95 NB

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

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.

$B_{shoulder} := 3ft + 8in$ — Width of Shoulder

$d_{shoulder} := B_{shoulder} - 2ft = 1.667\ ft$ — Defaults to 0' when not accounting for Lane Striping Lines, or shoulder 2' or less. When shoulder greater than 2', subtract 2 ft so that wheel is on white stripe.

$N_L := 2$ $N_{L.Striped} := 2$ — Number of Design/Striped Lanes

$N_b := 5$ — Number of Beams

$S := 7 \cdot ft + 6 \cdot in$ $S = 7.5\ ft$ — Girder Spacing

$d_{e.act} := 8 \cdot in$ $d_e := d_{e.act} - d_{shoulder}$ $d_e = -1\ ft$ — Actual distance from the centerline of the exterior web to the "face of curb" accounting for shoulder

$X_{ext} := \frac{(N_b - 1)}{2} \cdot S$ $X_{ext} = 15\ ft$ — Horizontal Distance from the center of gravity of the pattern of girders to the exterior girder.

$e := \begin{pmatrix} X_{ext} + d_e - 5 \cdot ft \\ X_{ext} + d_e - 5 \cdot ft - B_{lane} \end{pmatrix}$ $e = \begin{pmatrix} 9 \\ -3 \end{pmatrix} ft$ — 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.

$R_1 := \left[\frac{1}{N_b} + \frac{X_{ext} \cdot e_1}{2 \cdot (2 \cdot S)^2 + 2 \cdot (1 \cdot S)^2} \right] \cdot m_{p_1}$ — Distribution Factor for 1 lane loaded - Equation C4.6.2.2.2d-1

$R_2 := \left[\frac{2}{N_b} + \frac{X_{ext} \cdot (e_1 + e_2)}{2 \cdot (2 \cdot S)^2 + 2 \cdot (1 \cdot S)^2} \right] \cdot m_{p_2}$ — Distribution Factor for 2 lanes loaded - Equation C4.6.2.2.2d-1

$R = \begin{pmatrix} 0.528 \\ 0.56 \end{pmatrix}$

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BY: IAS/TDM DATE: 6/26/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/10/13 SUBJECT: No. 5800 - I-95 NB Bridge

References

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

Infinite Fatigue Check of Interior Girder Span 1

$t_{bf} := 1.260 \cdot in$ Thickness of the bottom flange for W36 x194 from AISC Historic Shape Database V14.0.

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

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

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

$\Delta f_{LL_IM} := 2.7 \cdot ksi$ Unfactored fatigue live load stress range at 13.18ft, from Merlin Dash Table 1.2.9.3A.

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

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

$R_s := R_{sa} \cdot R_{st}$ $R_s = 1$ 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.

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

$\Delta f_{eff} := R_s \cdot \gamma_{LL} \cdot \Delta f_{LL_IM}$ $\Delta f_{eff} = 2.025 \cdot ksi$ 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.

$\Delta f_{LL_IM.max} = 4.05 \cdot ksi$

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

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

The Louis Berger Group, Inc. - 482 Congress Street, Suite 401 Portland, ME 04101

BY: IAS/TDM DATE: 6/26/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/10/13 SUBJECT: No.5800 - I-95 NB Bridge

Use the following only for cases with Finite Fatigue Life.

Determine Evaluation Finite Fatigue Life

$Year_{built} := 1960$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $Age = 53$ — Age of bridge in 2013.

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

$n := 1.0$ — Cycles per truck passage, n, AASHTO LRFD Table 6.6.1.2.5-2 for simple span L>40'.

$ADTT := 2398$ — Average daily truck traffic from excel spreadsheet.

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

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

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $ADTT_{SL} = 2038$ — Average daily truck traffic in a single lane assuming 0 for the 53 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\ 3}} \qquad Y = 101$$

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

$Y_{remaining} := Y - Age$ $Y_{remaining} = 48$ — Remaining Fatigue Life

The Louis Berger Group, Inc. - 482 Congress Street, Suite 401 Portland, ME 04101

BY: IAS/TDM DATE: 6/26/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/10/13 SUBJECT: No. 5800 - I-95 NB Bridge

References

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

Infinite Fatigue Check of Exterior Girder Span 1

$t_{bf} := \mathbf{1.260}\cdot in$ Thickness of the bottom flange for W36 x194 from AISC Historic Shape Database V14.0.

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'.

$\Delta f_{LL_IM} := \mathbf{3.2}\cdot ksi$ Unfactored fatigue live load stress range at 14.00ft, 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.

$R_s := R_{sa}\cdot R_{st}$ $R_s = 1$ 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.

$\Delta f_{eff} := R_s\cdot\gamma_{LL}\cdot\Delta f_{LL_IM}$ $\Delta f_{eff} = 2.4\cdot ksi$ 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.

$\Delta f_{LL_IM.max} = 4.8\cdot ksi$

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

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

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BY: IAS/TDM DATE: 6/26/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/10/13 SUBJECT: No.5800 - I-95 Northbound

Use the following only for cases with Finite Fatigue Life.

Determine Evaluation Finite Fatigue Life

$Year_{built} := 1960$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $Age = 53$ — Age of bridge in 2013.

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

$n := 1.0$ — Cycles per truck passage, n, AASHTO LRFD Table 6.6.1.2.5-2 for simple span L>40'.

$ADTT := 2398$ — Average daily truck traffic from excel spreadsheet.

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

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

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $ADTT_{SL} = 2038$ — Average daily truck traffic in a single lane assuming 0 for the 53 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\ 3}} \qquad Y = 61$$

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

$Y_{remaining} := Y - Age$ $Y_{remaining} = 8$ — Remaining Fatigue Life

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BY: IAS/TDM DATE: 7/01/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/11/13 SUBJECT: No. 5800 - I-95 NB Bridge

References

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

Infinite Fatigue Check of Interior Girder Span 2

$t_{bf} := 1.57 \cdot in$ Thickness of the bottom flange for W36 x280 from AISC Historic Shape Database V14.0.

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

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

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

$\Delta f_{LL_IM} := 2.6 \cdot ksi$ Unfactored fatigue live load stress range at 17.14ft, from Merlin Dash Table 1.2.9.3A.

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

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

$R_s := R_{sa} \cdot R_{st}$ $R_s = 1$ 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.

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

$\Delta f_{eff} := R_s \cdot \gamma_{LL} \cdot \Delta f_{LL_IM}$ $\Delta f_{eff} = 1.95 \cdot ksi$ 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.

$\Delta f_{LL_IM.max} = 3.9 \cdot ksi$

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

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

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BY: IAS/TDM DATE: 7/01/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/11/13 SUBJECT: No.5800 - I-95 NB Bridge

Use the following only for cases with Finite Fatigue Life.

Determine Evaluation Finite Fatigue Life

$Year_{built} := 1960$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $Age = 53$ — 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 for simple span L>40'.

$ADTT := 2398$ — 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%.

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $ADTT_{SL} = 2038$ — Average daily truck traffic in a single lane assuming 0 for the 53 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\ 3}} \qquad Y = 113$$

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

$Y_{remaining} := Y - Age$ $Y_{remaining} = 60$ — Remaining Fatigue Life

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BY: IAS/TDM DATE: 7/1/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/11/13 SUBJECT: No. 5800 - I-95 NB Bridge

References

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

Infinite Fatigue Check of Exterior Girder Span 2

$t_{bf} := 1.57 \cdot in$ Thickness of the bottom flange for W36 x280 from AISC Historic Shape Database V14.0.

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

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

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

$\Delta f_{LL_IM} := 3.1 \cdot ksi$ Unfactored fatigue live load stress range at 18.28ft, from Merlin Dash Table 1.2.9.3A.

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

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

$R_s := R_{sa} \cdot R_{st}$ $R_s = 1$ 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.

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

$\Delta f_{eff} := R_s \cdot \gamma_{LL} \cdot \Delta f_{LL_IM}$ $\Delta f_{eff} = 2.325 \cdot ksi$ 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.

$\Delta f_{LL_IM.max} = 4.65 \cdot ksi$

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

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

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BY: IAS/TDM DATE: 7/1/13 PROJECT: Maine Load Rating Steel Bridges

CHK BY: KSW DATE: 7/11/13 SUBJECT: No.5800 - I-95 NB Bridge

Use the following only for cases with Finite Fatigue Life.

Determine Evaluation Finite Fatigue Life

$Year_{built} := 1960$ — Year of bridge construction

$Age := 2013 - Year_{built}$ $Age = 53$ — Age of bridge in 2013.

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

$n := 1.0$ — Cycles per truck passage, n, AASHTO LRFD Table 6.6.1.2.5-2 for simple span L>40'.

$ADTT := 2398$ — Average daily truck traffic from excel spreadsheet.

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

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

$ADTT_{SL} := g \cdot ADTT \cdot \rho$ $ADTT_{SL} = 2038$ — Average daily truck traffic in a single lane assuming 0 for the 53 year old bridge with assumed growth rate.

$$Y := \frac{R_R \cdot A}{365 \cdot n \cdot ADTT_{SL} \cdot \Delta f_{eff}^{\ 3}} \qquad Y = 67$$

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

$Y_{remaining} := Y - Age$ $Y_{remaining} = 14$ — Remaining Fatigue Life

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                  PROVISIONS OF THIS AGREEMENT HAVE BEEN BREACHED OR
                  HAVE PROVEN INEFFECTIVE.
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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5800 Span 1 Interior Beam CP & Fatigue 7/15/2013	

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 1(4)	IAS/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	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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UNIVERSITY OF MARYLAND LRF -- 2010
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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

BEAM NUMBER	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)	DEPTH (ft)	WIDTH. (in)	(in)	REGION (%)	FACTOR >= 1.0
5	1	31.33	2.50	1.83	1.62	18.12	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
74.67	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
7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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	36		194		0.00	0.0000	0.00	0.0000		
2	W	36		194		0.00	0.0000	11.00	1.2500		

NOTE: [1] maximum allowable section number is 70

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

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	13.83				32.	32.	32.
2	2	2	47.00				32.	32.	32.
3	1	1	13.83				32.	32.	32.

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	2	2398	2038	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 2398

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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 (%)	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	0.33	0.00	15.00	1 1 1 1 1	0.54	2 2 2 2 2
1	0.00	0	0.00	0.69	0.00	0.00	0 0 0 0 0	0.98	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	7.0	0	30.0	10.0	0.69	0.00	74.67

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	0	wearing surf	0.238	0.00	74.67	0.00	0.00
2	1	rail	0.010	0.00	74.67	0.00	0.00
3	1	curb	0.115	0.00	74.67	0.00	0.00
4	2	end diaphrag	0.000	0.00	0.00	0.76	0.45
5	2	int diaphrag	0.000	0.00	0.00	0.17	3.65
6	2	int diaphrag	0.000	0.00	0.00	0.35	18.65
7	2	int diaphrag	0.000	0.00	0.00	0.35	33.65
8	2	int diaphrag	0.000	0.00	0.00	0.35	48.65
9	2	int diaphrag	0.000	0.00	0.00	0.17	63.65
10	2	end diaphrag	0.000	0.00	0.00	0.76	74.21

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 YIELD Fy (ksi)	BAR AREA PER FOOT OF SLAB (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	2.50	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		SPACING
		STRESS		OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	3.65	33.0	33.0	3.20
3.65	63.65	33.0	33.0	15.00
63.65	74.21	33.0	33.0	10.56

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

TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.688		0.2234		0.9114

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.24	0.00	74.67
1	DC2	0.010	0.000	74.667
2	DC2	0.115	0.000	74.667

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

CONCRETE DECK				STEEL	SUPERSTRUCTURE TOTAL WEIGHT
	VOLUME		TOTAL	TOTAL	
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	WEIGHT (kip)	WEIGHT (kip)	(kip)
150.00	1524.5	56.5	228.7	83.4	312.07

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	7.50	90.00

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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	0.544	0.544	0.664	0.664	FOR STRENGTH POSITIVE MOMENT
	0.979				FOR STRENGTH POSITIVE SHEAR
1	0.328				FOR FATIGUE POSITIVE MOMENT
	0.691				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)	STEEL TOP.
1	0	0.00	12100.0	36.5	18.25	663.0	663.0

1	1	4.39	12100.0	36.5	18.25	663.0	663.0
1	2	8.78	12100.0	36.5	18.25	663.0	663.0
1	3	13.18	12100.0	36.5	18.25	663.0	663.0
1	4	17.57	16070.6	36.5	15.83	1015.1	733.2
1	5	21.96	16070.6	36.5	15.83	1015.1	733.2
1	6	26.35	16070.6	36.5	15.83	1015.1	733.2
1	7	30.75	16070.6	36.5	15.83	1015.1	733.2
1	8	35.14	16070.6	36.5	15.83	1015.1	733.2
1	9	39.53	16070.6	36.5	15.83	1015.1	733.2
1	10	43.92	16070.6	36.5	15.83	1015.1	733.2
1	11	48.31	16070.6	36.5	15.83	1015.1	733.2
1	12	52.71	16070.6	36.5	15.83	1015.1	733.2
1	13	57.10	16070.6	36.5	15.83	1015.1	733.2
1	14	61.49	12100.0	36.5	18.25	663.0	663.0
1	15	65.88	12100.0	36.5	18.25	663.0	663.0
1	16	70.27	12100.0	36.5	18.25	663.0	663.0
1	17	74.67	12100.0	36.5	18.25	663.0	663.0

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 = 30.00

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	19760.3	0.017683	814.0	1616.7	1008.7
1	1	4.39	19760.3	0.017683	814.0	1616.7	1008.7
1	2	8.78	19760.3	0.017683	814.0	1616.7	1008.7
1	3	13.18	19760.3	0.017683	814.0	1616.7	1008.7
1	4	17.57	27039.6	0.015941	1239.5	1696.8	1160.4
1	5	21.96	27039.6	0.015941	1239.5	1696.8	1160.4
1	6	26.35	27039.6	0.015941	1239.5	1696.8	1160.4
1	7	30.75	27039.6	0.015941	1239.5	1696.8	1160.4
1	8	35.14	27039.6	0.015941	1239.5	1696.8	1160.4
1	9	39.53	27039.6	0.015941	1239.5	1696.8	1160.4
1	10	43.92	27039.6	0.015941	1239.5	1696.8	1160.4
1	11	48.31	27039.6	0.015941	1239.5	1696.8	1160.4
1	12	52.71	27039.6	0.015941	1239.5	1696.8	1160.4
1	13	57.10	27039.6	0.015941	1239.5	1696.8	1160.4
1	14	61.49	19760.3	0.017683	814.0	1616.7	1008.7
1	15	65.88	19760.3	0.017683	814.0	1616.7	1008.7
1	16	70.27	19760.3	0.017683	814.0	1616.7	1008.7
1	17	74.67	19760.3	0.017683	814.0	1616.7	1008.7

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 10.00

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	27052.0	0.024957	903.7	4121.1	1941.9
1	1	4.39	27052.0	0.024957	903.7	4121.1	1941.9
1	2	8.78	27052.0	0.024957	903.7	4121.1	1941.9
1	3	13.18	27052.0	0.024957	903.7	4121.1	1941.9
1	4	17.57	38611.6	0.022794	1375.2	3991.7	2266.1
1	5	21.96	38611.6	0.022794	1375.2	3991.7	2266.1
1	6	26.35	38611.6	0.022794	1375.2	3991.7	2266.1
1	7	30.75	38611.6	0.022794	1375.2	3991.7	2266.1
1	8	35.14	38611.6	0.022794	1375.2	3991.7	2266.1
1	9	39.53	38611.6	0.022794	1375.2	3991.7	2266.1
1	10	43.92	38611.6	0.022794	1375.2	3991.7	2266.1
1	11	48.31	38611.6	0.022794	1375.2	3991.7	2266.1
1	12	52.71	38611.6	0.022794	1375.2	3991.7	2266.1
1	13	57.10	38611.6	0.022794	1375.2	3991.7	2266.1
1	14	61.49	27052.0	0.024957	903.7	4121.1	1941.9
1	15	65.88	27052.0	0.024957	903.7	4121.1	1941.9
1	16	70.27	27052.0	0.024957	903.7	4121.1	1941.9
1	17	74.67	27052.0	0.024957	903.7	4121.1	1941.9

Please read NOTE on the following page

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 L FROM 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	4.39	34.5	106.2	3.6	0.0		144.2
1	2	8.78	64.6	199.1	6.2	0.0		269.9
1	3	13.18	90.5	278.7	8.8	0.0		378.0

1	4	17.57	112.1	345.1	11.4	0.0	468.6
1	5	21.96	129.3	398.2	12.9	0.0	540.3
1	6	26.35	142.2	438.0	13.9	0.0	594.2
1	7	30.75	150.8	464.5	15.0	0.0	630.4
1	8	35.14	155.2	477.8	15.6	0.0	648.5
1	9	39.53	155.2	477.8	15.1	0.0	648.1
1	10	43.92	150.8	464.5	14.6	0.0	630.0
1	11	48.31	142.2	438.0	14.2	0.0	594.4
1	12	52.71	129.3	398.2	12.3	0.0	539.8
1	13	57.10	112.1	345.1	10.3	0.0	467.4
1	14	61.49	90.5	278.7	8.3	0.0	377.5
1	15	65.88	64.6	199.1	5.9	0.0	269.6
1	16	70.27	34.5	106.2	3.1	0.0	143.8
1	17	74.67	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 = 30.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	(k-ft) DC2	CONCENTRATED LOADS (k-ft) DW	(k-ft) 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	4.39	36.7	19.3	0.0	0.0	0.0	56.0
1	2	8.78	68.9	36.2	0.0	0.0	0.0	105.0
1	3	13.18	96.4	50.6	0.0	0.0	0.0	147.1
1	4	17.57	119.4	62.7	0.0	0.0	0.0	182.1
1	5	21.96	137.7	72.3	0.0	0.0	0.0	210.1
1	6	26.35	151.5	79.6	0.0	0.0	0.0	231.1
1	7	30.75	160.7	84.4	0.0	0.0	0.0	245.1
1	8	35.14	165.3	86.8	0.0	0.0	0.0	252.1
1	9	39.53	165.3	86.8	0.0	0.0	0.0	252.1
1	10	43.92	160.7	84.4	0.0	0.0	0.0	245.1
1	11	48.31	151.5	79.6	0.0	0.0	0.0	231.1
1	12	52.71	137.7	72.3	0.0	0.0	0.0	210.1
1	13	57.10	119.4	62.7	0.0	0.0	0.0	182.1
1	14	61.49	96.4	50.6	0.0	0.0	0.0	147.1
1	15	65.88	68.9	36.2	0.0	0.0	0.0	105.0
1	16	70.27	36.7	19.3	0.0	0.0	0.0	56.0
1	17	74.67	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 = 10.00 (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	4.39	0.0	0.0	240.5	HL-93	0.0	HL-93
1	2	8.78	0.0	0.0	447.3	HL-93	0.0	HL-93
1	3	13.18	0.0	0.0	620.5	HL-93	0.0	HL-93
1	4	17.57	0.0	0.0	758.2	HL-93	0.0	HL-93
1	5	21.96	0.0	0.0	865.6	HL-93	0.0	HL-93

1	6	26.35	0.0	0.0	945.9	HL-93	0.0	HL-93
1	7	30.75	0.0	0.0	993.5	HL-93	0.0	HL-93
1	8	35.14	0.0	0.0	1010.4	HL-93	0.0	HL-93
1	9	39.53	0.0	0.0	1010.4	HL-93	0.0	HL-93
1	10	43.92	0.0	0.0	993.5	HL-93	0.0	HL-93
1	11	48.31	0.0	0.0	945.9	HL-93	0.0	HL-93
1	12	52.71	0.0	0.0	865.6	HL-93	0.0	HL-93
1	13	57.10	0.0	0.0	758.2	HL-93	0.0	HL-93
1	14	61.49	0.0	0.0	620.5	HL-93	0.0	HL-93
1	15	65.88	0.0	0.0	447.3	HL-93	0.0	HL-93
1	16	70.27	0.0	0.0	240.5	HL-93	0.0	HL-93
1	17	74.67	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 = 10.0 (k-ft) (UNFACTORED)

SP NO	IN NO	D L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0		0.00	0.	0.	0.
1	1		4.39	83.	0.	83.
1	2		8.78	152.	0.	152.
1	3		13.18	206.	0.	206.
1	4		17.57	251.	0.	251.
1	5		21.96	285.	0.	285.
1	6		26.35	308.	0.	308.
1	7		30.75	314.	0.	314.
1	8		35.14	306.	0.	306.
1	9		39.53	306.	0.	306.
1	10		43.92	314.	0.	314.
1	11		48.31	308.	0.	308.
1	12		52.71	285.	0.	285.
1	13		57.10	251.	0.	251.
1	14		61.49	206.	0.	206.
1	15		65.88	152.	0.	152.
1	16		70.27	83.	0.	83.
1	17		74.67	0.	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN NO	D L	FROM SUPT (ft)	DEAD NON COMP. N=Infin. (k-ft)	LOAD COMP. N=30.0 (k-ft)	LL+I:N=10.0 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		4.39	144.2	56.0	240.5	HL-93	0.0	HL-93	440.7	200.3
1	2		8.78	269.9	105.0	447.3	HL-93	0.0	HL-93	822.3	375.0
1	3		13.18	378.0	147.1	620.5	HL-93	0.0	HL-93	1145.6	525.1
1	4		17.57	468.6	182.1	758.2	HL-93	0.0	HL-93	1408.9	650.6
1	5		21.96	540.3	210.1	865.6	HL-93	0.0	HL-93	1616.0	750.4
1	6		26.35	594.2	231.1	945.9	HL-93	0.0	HL-93	1771.1	825.2
1	7		30.75	630.4	245.1	993.5	HL-93	0.0	HL-93	1869.0	875.5
1	8		35.14	648.5	252.1	1010.4	HL-93	0.0	HL-93	1911.0	900.6
1	9		39.53	648.1	252.1	1010.4	HL-93	0.0	HL-93	1910.5	900.2

1	10	43.92	630.0	245.1	993.5	HL-93	0.0	HL-93	1868.7	875.1
1	11	48.31	594.4	231.1	945.9	HL-93	0.0	HL-93	1771.4	825.5
1	12	52.71	539.8	210.1	865.6	HL-93	0.0	HL-93	1615.4	749.8
1	13	57.10	467.4	182.1	758.2	HL-93	0.0	HL-93	1407.7	649.5
1	14	61.49	377.5	147.1	620.5	HL-93	0.0	HL-93	1145.1	524.6
1	15	65.88	269.6	105.0	447.3	HL-93	0.0	HL-93	822.0	374.7
1	16	70.27	143.8	56.0	240.5	HL-93	0.0	HL-93	440.3	199.8
1	17	74.67	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	4.39	440.7	512.9	680.4	584.2	300.4	62.4
1	2	8.78	822.3	956.5	1268.7	1089.8	562.5	113.6
1	3	13.18	1145.6	1331.8	1766.4	1518.2	787.6	154.8
1	4	17.57	1408.9	1636.3	2170.1	1866.8	975.9	188.4
1	5	21.96	1616.0	1875.7	2487.2	2141.0	1125.6	213.6
1	6	26.35	1771.1	2054.9	2724.8	2346.4	1237.9	230.9
1	7	30.75	1869.0	2167.1	2873.2	2475.8	1313.2	235.4
1	8	35.14	1911.0	2214.1	2935.2	2531.1	1350.9	229.3
1	9	39.53	1910.5	2213.6	2934.6	2530.5	1350.2	229.3
1	10	43.92	1868.7	2166.7	2872.8	2475.3	1312.7	235.4
1	11	48.31	1771.4	2055.1	2725.1	2346.7	1238.2	230.9
1	12	52.71	1615.4	1875.1	2486.5	2140.3	1124.7	213.6
1	13	57.10	1407.7	1635.2	2168.6	1865.3	974.2	188.4
1	14	61.49	1145.1	1331.2	1765.7	1517.5	786.8	154.8
1	15	65.88	822.0	956.2	1268.3	1089.4	562.0	113.6
1	16	70.27	440.3	512.4	679.8	583.6	299.7	62.4
1	17	74.67	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	8.3	25.7	1.5	0.0	35.6
1	1	4.39	7.4	22.7	0.6	0.0	30.6
1	2	8.78	6.4	19.6	0.6	0.0	26.6
1	3	13.18	5.4	16.6	0.6	0.0	22.6
1	4	17.57	4.4	13.6	0.6	0.0	18.6
1	5	21.96	3.4	10.6	0.2	0.0	14.3
1	6	26.35	2.5	7.6	0.2	0.0	10.3
1	7	30.75	1.5	4.5	0.2	0.0	6.2
1	8	35.14	0.5	1.5	-0.1	0.0	1.9
1	9	39.53	-0.5	-1.5	-0.1	0.0	-2.1
1	10	43.92	-1.5	-4.5	-0.1	0.0	-6.1
1	11	48.31	-2.5	-7.6	-0.1	0.0	-10.1

1	12	52.71	-3.4	-10.6	-0.5	0.0	-14.5
1	13	57.10	-4.4	-13.6	-0.5	0.0	-18.5
1	14	61.49	-5.4	-16.6	-0.5	0.0	-22.5
1	15	65.88	-6.4	-19.6	-0.6	0.0	-26.7
1	16	70.27	-7.4	-22.7	-0.6	0.0	-30.7
1	17	74.67	-8.3	-25.7	-1.4	0.0	-35.4

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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	4.7	0.0	0.0	13.6	35.6	49.1
1	1	4.39	7.8	4.1	0.0	0.0	12.0	30.6	42.6
1	2	8.78	6.8	3.6	0.0	0.0	10.4	26.6	37.0
1	3	13.18	5.7	3.0	0.0	0.0	8.8	22.6	31.4
1	4	17.57	4.7	2.5	0.0	0.0	7.2	18.6	25.8
1	5	21.96	3.7	1.9	0.0	0.0	5.6	14.3	19.8
1	6	26.35	2.6	1.4	0.0	0.0	4.0	10.3	14.2
1	7	30.75	1.6	0.8	0.0	0.0	2.4	6.2	8.6
1	8	35.14	0.5	0.3	0.0	0.0	0.8	1.9	2.7
1	9	39.53	-0.5	-0.3	0.0	0.0	-0.8	-2.1	-2.9
1	10	43.92	-1.6	-0.8	0.0	0.0	-2.4	-6.1	-8.5
1	11	48.31	-2.6	-1.4	0.0	0.0	-4.0	-10.1	-14.1
1	12	52.71	-3.7	-1.9	0.0	0.0	-5.6	-14.5	-20.0
1	13	57.10	-4.7	-2.5	0.0	0.0	-7.2	-18.5	-25.6
1	14	61.49	-5.7	-3.0	0.0	0.0	-8.8	-22.5	-31.2
1	15	65.88	-6.8	-3.6	0.0	0.0	-10.4	-26.7	-37.0
1	16	70.27	-7.8	-4.1	0.0	0.0	-12.0	-30.7	-42.6
1	17	74.67	-8.9	-4.7	0.0	0.0	-13.6	-35.4	-49.0

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	105.4	HL-93	0.0	HL-93
1	1	4.39	0.0	0.0	97.2	HL-93	-2.7	HL-93
1	2	8.78	0.0	0.0	89.2	HL-93	-6.2	HL-93
1	3	13.18	0.0	0.0	81.3	HL-93	-10.5	HL-93
1	4	17.57	0.0	0.0	73.6	HL-93	-14.9	HL-93
1	5	21.96	0.0	0.0	66.1	HL-93	-19.4	HL-93
1	6	26.35	0.0	0.0	58.7	HL-93	-24.5	HL-93
1	7	30.75	0.0	0.0	51.5	HL-93	-30.8	HL-93
1	8	35.14	0.0	0.0	44.5	HL-93	-37.6	HL-93
1	9	39.53	0.0	0.0	37.6	HL-93	-44.5	HL-93
1	10	43.92	0.0	0.0	30.8	HL-93	-51.5	HL-93
1	11	48.31	0.0	0.0	24.5	HL-93	-58.7	HL-93
1	12	52.71	0.0	0.0	19.4	HL-93	-66.1	HL-93

1	13	57.10	0.0	0.0	14.9	HL-93	-73.6	HL-93
1	14	61.49	0.0	0.0	10.5	HL-93	-81.3	HL-93
1	15	65.88	0.0	0.0	6.2	HL-93	-89.2	HL-93
1	16	70.27	0.0	0.0	2.7	HL-93	-97.2	HL-93
1	17	74.67	0.0	0.0	0.0	HL-93	-105.4	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 10.0 (kips) (UNFACTORED)
****

SP NO	IN D L NO	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	43.	0.	43.
1	1	4.39	40.	-1.	41.
1	2	8.78	37.	-3.	39.
1	3	13.18	33.	-4.	38.
1	4	17.57	30.	-6.	36.
1	5	21.96	26.	-8.	35.
1	6	26.35	23.	-10.	33.
1	7	30.75	20.	-12.	32.
1	8	35.14	17.	-14.	31.
1	9	39.53	14.	-17.	31.
1	10	43.92	12.	-20.	32.
1	11	48.31	10.	-23.	33.
1	12	52.71	8.	-26.	35.
1	13	57.10	6.	-30.	36.
1	14	61.49	4.	-33.	38.
1	15	65.88	3.	-37.	39.
1	16	70.27	1.	-40.	41.
1	17	74.67	0.	-43.	43.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN D L NO	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	43.2	.27984E-01	0.60	0.3125*.15400E-01		0.33	0.3125*
1	1	4.39	41.4	.27984E-01	0.58	0.3125*.15400E-01		0.32	0.3125*
1	2	8.78	39.5	.27984E-01	0.55	0.3125*.15400E-01		0.30	0.3125*
1	3	13.18	37.6	.27984E-01	0.53	0.3125*.15400E-01		0.29	0.3125*
1	4	17.57	36.1	.26097E-01	0.47	0.3125*.96076E-02		0.17	0.3125*
1	5	21.96	34.6	.26097E-01	0.45	0.3125*.96076E-02		0.17	0.3125*
1	6	26.35	32.9	.26097E-01	0.43	0.3125*.96076E-02		0.16	0.3125*
1	7	30.75	31.5	.26097E-01	0.41	0.3125*.96076E-02		0.15	0.3125*
1	8	35.14	30.5	.26097E-01	0.40	0.3125*.96076E-02		0.15	0.3125*
1	9	39.53	30.5	.26097E-01	0.40	0.3125*.96076E-02		0.15	0.3125*
1	10	43.92	31.5	.26097E-01	0.41	0.3125*.96076E-02		0.15	0.3125*
1	11	48.31	32.9	.26097E-01	0.43	0.3125*.96076E-02		0.16	0.3125*
1	12	52.71	34.6	.26097E-01	0.45	0.3125*.96076E-02		0.17	0.3125*
1	13	57.10	36.1	.26097E-01	0.47	0.3125*.96076E-02		0.17	0.3125*
1	14	61.49	37.6	.27984E-01	0.53	0.3125*.15400E-01		0.29	0.3125*
1	15	65.88	39.5	.27984E-01	0.55	0.3125*.15400E-01		0.30	0.3125*
1	16	70.27	41.4	.27984E-01	0.58	0.3125*.15400E-01		0.32	0.3125*
1	17	74.67	43.2	.27984E-01	0.60	0.3125*.15400E-01		0.33	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

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	49.1	105.4	HL-93	0.0	HL-93	154.5	49.1
1	1	4.39	42.6	97.2	HL-93	-2.7	HL-93	139.8	39.9
1	2	8.78	37.0	89.2	HL-93	-6.2	HL-93	126.2	30.7
1	3	13.18	31.4	81.3	HL-93	-10.5	HL-93	112.7	20.9
1	4	17.57	25.8	73.6	HL-93	-14.9	HL-93	99.4	10.9
1	5	21.96	19.8	66.1	HL-93	-19.4	HL-93	85.9	0.4
1	6	26.35	14.2	58.7	HL-93	-24.5	HL-93	73.0	-10.3
1	7	30.75	8.6	51.5	HL-93	-30.8	HL-93	60.1	-22.2
1	8	35.14	2.7	44.5	HL-93	-37.6	HL-93	47.1	-34.9
1	9	39.53	-2.9	37.6	HL-93	-44.5	HL-93	34.7	-47.4
1	10	43.92	-8.5	30.8	HL-93	-51.5	HL-93	22.3	-60.0
1	11	48.31	-14.1	24.5	HL-93	-58.7	HL-93	10.4	-72.8
1	12	52.71	-20.0	19.4	HL-93	-66.1	HL-93	-0.6	-86.1
1	13	57.10	-25.6	14.9	HL-93	-73.6	HL-93	-10.8	-99.3
1	14	61.49	-31.2	10.5	HL-93	-81.3	HL-93	-20.8	-112.6
1	15	65.88	-37.0	6.2	HL-93	-89.2	HL-93	-30.8	-126.2
1	16	70.27	-42.6	2.7	HL-93	-97.2	HL-93	-39.9	-139.8
1	17	74.67	-49.0	0.0	HL-93	-105.4	HL-93	-49.0	-154.4

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	154.5	186.1	248.1	205.9	73.7	32.4
1	1	4.39	139.8	169.0	225.3	186.4	63.9	31.0
1	2	8.78	126.2	152.9	204.0	168.3	55.5	29.6
1	3	13.18	112.7	137.1	183.0	150.5	47.1	28.2
1	4	17.57	99.4	121.5	162.3	132.8	38.7	27.0
1	5	21.96	85.9	105.8	141.4	114.9	29.8	25.9
1	6	26.35	73.0	90.6	121.2	97.7	21.4	24.7
1	7	30.75	60.1	75.6	101.3	80.7	13.0	23.7
1	8	35.14	47.1	60.5	81.3	63.5	4.0	22.9
1	9	39.53	-47.4	-60.7	-81.6	-63.8	-4.4	22.9
1	10	43.92	-60.0	-75.5	-101.2	-80.6	-12.8	23.7
1	11	48.31	-72.8	-90.4	-121.0	-97.6	-21.1	24.7
1	12	52.71	-86.1	-106.0	-141.6	-115.2	-30.1	25.9
1	13	57.10	-99.3	-121.4	-162.1	-132.6	-38.5	27.0
1	14	61.49	-112.6	-137.0	-182.8	-150.3	-46.9	28.2
1	15	65.88	-126.2	-153.0	-204.1	-168.4	-55.5	29.6
1	16	70.27	-139.8	-169.0	-225.4	-186.5	-63.9	31.0
1	17	74.67	-154.4	-186.0	-247.9	-205.7	-73.5	32.4

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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	49.11	0.00	HL-93	105.41	HL-93	ST1	41.98	248.07
						ST2	41.98	205.91
						ST4	60.33	73.66
						SE1	49.11	154.52
						SE2	49.11	186.14
2	48.97	0.00	HL-93	105.41	HL-93	ST1	41.85	247.90
						ST2	41.85	205.73
						ST4	60.13	73.45
						SE1	48.97	154.38
						SE2	48.97	186.00

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 = 30.0 (UNFACTORED)
****

SP NO	IN D NO	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	4.39	-0.0663	-0.2041	0.0000	-0.0644	-0.2769	-0.0644	-0.3413
1	2	8.78	-0.1293	-0.3982	0.0000	-0.1256	-0.5403	-0.1256	-0.6659
1	3	13.18	-0.1862	-0.5736	0.0000	-0.1808	-0.7781	-0.1808	-0.9589
1	4	17.57	-0.2355	-0.7251	0.0000	-0.2283	-0.9837	-0.2283	-1.2121
1	5	21.96	-0.2767	-0.8520	0.0000	-0.2681	-1.1559	-0.2681	-1.4240

1	6	26.35	-0.3087	-0.9505	0.0000	-0.2990	-1.2895	-0.2990	-1.5885
1	7	30.75	-0.3305	-1.0178	0.0000	-0.3201	-1.3807	-0.3201	-1.7007
1	8	35.14	-0.3416	-1.0519	0.0000	-0.3308	-1.4269	-0.3308	-1.7576
1	9	39.53	-0.3416	-1.0519	0.0000	-0.3308	-1.4268	-0.3308	-1.7576
1	10	43.92	-0.3305	-1.0178	0.0000	-0.3201	-1.3805	-0.3201	-1.7006
1	11	48.31	-0.3087	-0.9505	0.0000	-0.2990	-1.2892	-0.2990	-1.5882
1	12	52.71	-0.2767	-0.8520	0.0000	-0.2681	-1.1556	-0.2681	-1.4237
1	13	57.10	-0.2355	-0.7252	0.0000	-0.2283	-0.9834	-0.2283	-1.2117
1	14	61.49	-0.1863	-0.5736	0.0000	-0.1808	-0.7778	-0.1808	-0.9586
1	15	65.88	-0.1293	-0.3983	0.0000	-0.1256	-0.5400	-0.1256	-0.6657
1	16	70.27	-0.0663	-0.2041	0.0000	-0.0644	-0.2768	-0.0644	-0.3412
1	17	74.67	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 STEEL DEFL. (in)	CAMBER SIZE	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	4.4	-0.066	0 1/ 8	-0.211	0 1/ 4	-0.064	0 1/ 8	-0.341	0 3/ 8
1	2	8.8	-0.129	0 3/16	-0.411	0 7/16	-0.126	0 3/16	-0.666	0 11/16
1	3	13.2	-0.186	0 3/16	-0.592	0 5/ 8	-0.181	0 3/16	-0.959	1 0/16
1	4	17.6	-0.235	0 1/ 4	-0.748	0 3/ 4	-0.228	0 1/ 4	-1.212	1 1/ 4
1	5	22.0	-0.277	0 5/16	-0.879	0 15/16	-0.268	0 5/16	-1.424	1 7/16
1	6	26.4	-0.309	0 5/16	-0.981	1 0/16	-0.299	0 5/16	-1.588	1 5/ 8
1	7	30.7	-0.330	0 3/ 8	-1.050	1 1/16	-0.320	0 3/ 8	-1.701	1 3/ 4
1	8	35.1	-0.342	0 3/ 8	-1.085	1 1/ 8	-0.331	0 3/ 8	-1.758	1 13/16
1	9	39.5	-0.342	0 3/ 8	-1.085	1 1/ 8	-0.331	0 3/ 8	-1.758	1 13/16
1	10	43.9	-0.330	0 3/ 8	-1.050	1 1/16	-0.320	0 3/ 8	-1.701	1 3/ 4
1	11	48.3	-0.309	0 5/16	-0.981	1 0/16	-0.299	0 5/16	-1.588	1 5/ 8
1	12	52.7	-0.277	0 5/16	-0.879	0 15/16	-0.268	0 5/16	-1.424	1 7/16
1	13	57.1	-0.235	0 1/ 4	-0.748	0 3/ 4	-0.228	0 1/ 4	-1.212	1 1/ 4
1	14	61.5	-0.186	0 3/16	-0.592	0 5/ 8	-0.181	0 3/16	-0.959	1 0/16
1	15	65.9	-0.129	0 3/16	-0.411	0 7/16	-0.126	0 3/16	-0.666	0 11/16
1	16	70.3	-0.066	0 1/ 8	-0.210	0 1/ 4	-0.064	0 1/ 8	-0.341	0 3/ 8
1	17	74.7	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	37.33	2 0.400	-0.480	MAX HL-93	1.12	0.00330

```
                                  5800 Int G S1 CP and Fatigue.RES
                                     0.030 MIN
                                    -0.165 MAX LANE
                                     0.030 MIN LANE

                      3    0.510     -0.612 MAX HL-93     1.12            0.00330
                                     0.039 MIN
                                    -0.210 MAX LANE
                                     0.039 MIN LANE

                      2    0.400     -0.390 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   4.39   -0.62    0.62    -1.99    1.99    -2.61    2.61
  1  2   8.78   -1.17    1.17    -3.72    3.72    -4.89    4.89
  1  3  13.18   -1.64    1.64    -5.20    5.20    -6.84    6.84
  1  4  17.57   -1.83    1.32    -5.83    4.21    -7.67    5.54
  1  5  21.96   -2.12    1.53    -6.73    4.86    -8.84    6.39
  1  6  26.35   -2.33    1.68    -7.40    5.34    -9.72    7.02
  1  7  30.75   -2.47    1.78    -7.85    5.67   -10.32    7.45
  1  8  35.14   -2.54    1.83    -8.07    5.83   -10.61    7.67
  1  9  39.53   -2.54    1.83    -8.07    5.83   -10.61    7.66
  1 10  43.92   -2.47    1.78    -7.84    5.66   -10.31    7.45
  1 11  48.31   -2.33    1.68    -7.40    5.35    -9.73    7.03
  1 12  52.71   -2.12    1.53    -6.72    4.85    -8.83    6.38
  1 13  57.10   -1.83    1.32    -5.82    4.20    -7.65    5.53
  1 14  61.49   -1.64    1.64    -5.19    5.19    -6.83    6.83
  1 15  65.88   -1.17    1.17    -3.71    3.71    -4.88    4.88
  1 16  70.27   -0.62    0.62    -1.98    1.98    -2.60    2.60
  1 17  74.67    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 = 30.0 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	4.39	-0.02	-0.42	0.83	-0.02	-3.03	3.44
1	2	8.78	-0.04	-0.78	1.55	-0.04	-5.67	6.43
1	3	13.18	-0.06	-1.09	2.17	-0.06	-7.93	9.01
1	4	17.57	-0.06	-1.29	1.76	-0.06	-8.96	7.30
1	5	21.96	-0.07	-1.49	2.03	-0.07	-10.33	8.42
1	6	26.35	-0.08	-1.63	2.24	-0.08	-11.36	9.26
1	7	30.75	-0.08	-1.73	2.37	-0.08	-12.05	9.82
1	8	35.14	-0.09	-1.78	2.44	-0.09	-12.40	10.11
1	9	39.53	-0.09	-1.78	2.44	-0.09	-12.39	10.10
1	10	43.92	-0.08	-1.73	2.37	-0.08	-12.04	9.82
1	11	48.31	-0.08	-1.63	2.24	-0.08	-11.36	9.26
1	12	52.71	-0.07	-1.49	2.03	-0.07	-10.32	8.41
1	13	57.10	-0.06	-1.29	1.76	-0.06	-8.94	7.29
1	14	61.49	-0.06	-1.09	2.17	-0.06	-7.92	9.00
1	15	65.88	-0.04	-0.78	1.55	-0.04	-5.66	6.43
1	16	70.27	-0.02	-0.42	0.83	-0.02	-3.02	3.43
1	17	74.67	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 = 10.0 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(MPa) CONCR.	STEEL TOP	STEEL BOT	MAXIMUM NEGATIVE,(ksi) CONCR.	STEEL TOP	STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	-0.15	-0.70	3.19	0.00	0.00	0.00
1	2	8.78	-0.28	-1.30	5.94	0.00	0.00	0.00
1	3	13.18	-0.38	-1.81	8.24	0.00	0.00	0.00
1	4	17.57	-0.40	-2.28	6.62	0.00	0.00	0.00
1	5	21.96	-0.46	-2.60	7.55	0.00	0.00	0.00
1	6	26.35	-0.50	-2.84	8.25	0.00	0.00	0.00
1	7	30.75	-0.53	-2.99	8.67	0.00	0.00	0.00
1	8	35.14	-0.54	-3.04	8.82	0.00	0.00	0.00
1	9	39.53	-0.54	-3.04	8.82	0.00	0.00	0.00
1	10	43.92	-0.53	-2.99	8.67	0.00	0.00	0.00
1	11	48.31	-0.50	-2.84	8.25	0.00	0.00	0.00
1	12	52.71	-0.46	-2.60	7.55	0.00	0.00	0.00
1	13	57.10	-0.40	-2.28	6.62	0.00	0.00	0.00
1	14	61.49	-0.38	-1.81	8.24	0.00	0.00	0.00
1	15	65.88	-0.28	-1.30	5.94	0.00	0.00	0.00
1	16	70.27	-0.15	-0.70	3.19	0.00	0.00	0.00
1	17	74.67	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 = 10.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE STRESS RANGE TRUCK	TOP OF TOP FLANGE STRESS RANGE ONLY	GOVERNING LOAD TP TYPE	BOT OF BOT FLANGE STRESS RANGE TRUCK	BOT OF BOT FLANGE STRESS RANGE ONLY	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	4.39	0.2	0.0	0.2 C HL	1.1	0.0	1.1 T HL
1	2	8.78	0.4	0.0	0.4 C HL	2.0	0.0	2.0 T HL
1	3	13.18	0.6	0.0	0.6 C HL	2.7	0.0	2.7 T HL
1	4	17.57	0.8	0.0	0.8 C HL	2.2	0.0	2.2 T HL
1	5	21.96	0.9	0.0	0.9 C HL	2.5	0.0	2.5 T HL
1	6	26.35	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	7	30.75	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	8	35.14	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	9	39.53	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	10	43.92	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	11	48.31	0.9	0.0	0.9 C HL	2.7	0.0	2.7 T HL
1	12	52.71	0.9	0.0	0.9 C HL	2.5	0.0	2.5 T HL
1	13	57.10	0.8	0.0	0.8 C HL	2.2	0.0	2.2 T HL
1	14	61.49	0.6	0.0	0.6 C HL	2.7	0.0	2.7 T HL
1	15	65.88	0.4	0.0	0.4 C HL	2.0	0.0	2.0 T HL
1	16	70.27	0.2	0.0	0.2 C HL	1.1	0.0	1.1 T HL
1	17	74.67	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.17	-3.73	6.63	-0.02	-3.03	3.44
1	2	8.78	32.	32.	-0.32	-6.97	12.37	-0.04	-5.67	6.43
1	3	13.18	32.	32.	-0.44	-9.74	17.25	-0.06	-7.93	9.01
1	4	17.57	32.	32.	-0.46	-11.24	13.92	-0.06	-8.96	7.30
1	5	21.96	32.	32.	-0.53	-12.93	15.97	-0.07	-10.33	8.42
1	6	26.35	32.	32.	-0.58	-14.20	17.51	-0.08	-11.36	9.26
1	7	30.75	32.	32.	-0.61	-15.04	18.49	-0.08	-12.05	9.82
1	8	35.14	32.	32.	-0.62	-15.43	18.92	-0.09	-12.40	10.11
1	9	39.53	32.	32.	-0.62	-15.43	18.92	-0.09	-12.39	10.10
1	10	43.92	32.	32.	-0.61	-15.03	18.49	-0.08	-12.04	9.82
1	11	48.31	32.	32.	-0.58	-14.21	17.52	-0.08	-11.36	9.26
1	12	52.71	32.	32.	-0.53	-12.92	15.97	-0.07	-10.32	8.41
1	13	57.10	32.	32.	-0.46	-11.22	13.90	-0.06	-8.94	7.29
1	14	61.49	32.	32.	-0.44	-9.73	17.24	-0.06	-7.92	9.00
1	15	65.88	32.	32.	-0.32	-6.96	12.37	-0.04	-5.66	6.43
1	16	70.27	32.	32.	-0.17	-3.72	6.62	-0.02	-3.02	3.43
1	17	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.22	-3.94	7.59	-0.02	-3.03	3.44
1	2	8.78	32.	32.	-0.40	-7.36	14.16	-0.04	-5.67	6.43
1	3	13.18	32.	32.	-0.56	-10.28	19.72	-0.06	-7.93	9.01
1	4	17.57	32.	32.	-0.58	-11.92	15.90	-0.06	-8.96	7.30
1	5	21.96	32.	32.	-0.67	-13.71	18.24	-0.07	-10.33	8.42
1	6	26.35	32.	32.	-0.73	-15.06	19.99	-0.08	-11.36	9.26
1	7	30.75	32.	32.	-0.77	-15.93	21.10	-0.08	-12.05	9.82
1	8	35.14	32.	32.	-0.78	-16.35	21.57	-0.09	-12.40	10.11
1	9	39.53	32.	32.	-0.78	-16.34	21.56	-0.09	-12.39	10.10
1	10	43.92	32.	32.	-0.77	-15.93	21.09	-0.08	-12.04	9.82
1	11	48.31	32.	32.	-0.73	-15.06	19.99	-0.08	-11.36	9.26
1	12	52.71	32.	32.	-0.67	-13.70	18.23	-0.07	-10.32	8.41
1	13	57.10	32.	32.	-0.58	-11.90	15.89	-0.06	-8.94	7.29
1	14	61.49	32.	32.	-0.56	-10.27	19.71	-0.06	-7.92	9.00
1	15	65.88	32.	32.	-0.40	-7.35	14.15	-0.04	-5.66	6.43
1	16	70.27	32.	32.	-0.22	-3.93	7.58	-0.02	-3.02	3.43
1	17	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.29	-5.08	10.02	-0.03	-3.85	4.43
1	2	8.78	32.	32.	-0.54	-9.49	18.69	-0.06	-7.21	8.30
1	3	13.18	32.	32.	-0.75	-13.26	26.04	-0.08	-10.10	11.62
1	4	17.57	32.	32.	-0.79	-15.40	20.99	-0.09	-11.41	9.42
1	5	21.96	32.	32.	-0.90	-17.71	24.08	-0.10	-13.15	10.86
1	6	26.35	32.	32.	-0.99	-19.44	26.39	-0.11	-14.47	11.94
1	7	30.75	32.	32.	-1.04	-20.57	27.84	-0.12	-15.35	12.67
1	8	35.14	32.	32.	-1.06	-21.10	28.46	-0.12	-15.79	13.03
1	9	39.53	32.	32.	-1.06	-21.09	28.46	-0.12	-15.78	13.03
1	10	43.92	32.	32.	-1.04	-20.57	27.84	-0.12	-15.34	12.66
1	11	48.31	32.	32.	-0.99	-19.45	26.39	-0.11	-14.47	11.95
1	12	52.71	32.	32.	-0.90	-17.70	24.07	-0.10	-13.14	10.85
1	13	57.10	32.	32.	-0.79	-15.37	20.98	-0.09	-11.38	9.40
1	14	61.49	32.	32.	-0.75	-13.25	26.03	-0.08	-10.08	11.61
1	15	65.88	32.	32.	-0.54	-9.48	18.68	-0.06	-7.20	8.29
1	16	70.27	32.	32.	-0.29	-5.07	10.01	-0.03	-3.84	4.42
1	17	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.28	-3.88	8.55	-0.02	-2.66	2.96
1	2	8.78	32.	32.	-0.51	-7.25	15.93	-0.03	-4.97	5.54
1	3	13.18	32.	32.	-0.71	-10.12	22.17	-0.04	-6.96	7.75
1	4	17.57	32.	32.	-0.75	-11.84	17.86	-0.05	-7.85	6.28
1	5	21.96	32.	32.	-0.86	-13.61	20.46	-0.05	-9.05	7.25
1	6	26.35	32.	32.	-0.94	-14.93	22.41	-0.06	-9.95	7.97
1	7	30.75	32.	32.	-0.98	-15.79	23.63	-0.06	-10.56	8.45
1	8	35.14	32.	32.	-1.00	-16.18	24.12	-0.06	-10.87	8.70
1	9	39.53	32.	32.	-1.00	-16.17	24.12	-0.06	-10.86	8.69
1	10	43.92	32.	32.	-0.98	-15.78	23.62	-0.06	-10.56	8.45
1	11	48.31	32.	32.	-0.94	-14.93	22.42	-0.06	-9.96	7.97
1	12	52.71	32.	32.	-0.86	-13.60	20.46	-0.05	-9.04	7.24
1	13	57.10	32.	32.	-0.75	-11.82	17.85	-0.05	-7.83	6.27
1	14	61.49	32.	32.	-0.71	-10.11	22.17	-0.04	-6.95	7.75
1	15	65.88	32.	32.	-0.51	-7.25	15.93	-0.03	-4.97	5.53
1	16	70.27	32.	32.	-0.28	-3.87	8.54	-0.02	-2.65	2.95
1	17	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.23	-4.80	8.74	-0.03	-3.85	4.43
1	2	8.78	32.	32.	-0.43	-8.97	16.32	-0.06	-7.21	8.30
1	3	13.18	32.	32.	-0.60	-12.54	22.74	-0.08	-10.10	11.62
1	4	17.57	32.	32.	-0.63	-14.48	18.35	-0.09	-11.41	9.42
1	5	21.96	32.	32.	-0.72	-16.67	21.06	-0.10	-13.15	10.86
1	6	26.35	32.	32.	-0.79	-18.30	23.09	-0.11	-14.47	11.94
1	7	30.75	32.	32.	-0.83	-19.38	24.37	-0.12	-15.35	12.67
1	8	35.14	32.	32.	-0.85	-19.89	24.94	-0.12	-15.79	13.03
1	9	39.53	32.	32.	-0.85	-19.88	24.93	-0.12	-15.78	13.03
1	10	43.92	32.	32.	-0.83	-19.37	24.37	-0.12	-15.34	12.66
1	11	48.31	32.	32.	-0.79	-18.31	23.09	-0.11	-14.47	11.95
1	12	52.71	32.	32.	-0.72	-16.66	21.05	-0.10	-13.14	10.85
1	13	57.10	32.	32.	-0.63	-14.46	18.33	-0.09	-11.38	9.40
1	14	61.49	32.	32.	-0.60	-12.52	22.73	-0.08	-10.08	11.61
1	15	65.88	32.	32.	-0.43	-8.96	16.31	-0.06	-7.20	8.29
1	16	70.27	32.	32.	-0.23	-4.79	8.73	-0.03	-3.84	4.42
1	17	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.39	32.	32.	-0.22	-3.60	7.27	-0.02	-2.66	2.96
1	2	8.78	32.	32.	-0.40	-6.73	13.56	-0.03	-4.97	5.54
1	3	13.18	32.	32.	-0.56	-9.40	18.88	-0.04	-6.96	7.75
1	4	17.57	32.	32.	-0.59	-10.93	15.21	-0.05	-7.85	6.28
1	5	21.96	32.	32.	-0.67	-12.57	17.44	-0.05	-9.05	7.25
1	6	26.35	32.	32.	-0.73	-13.79	19.11	-0.06	-9.95	7.97
1	7	30.75	32.	32.	-0.77	-14.59	20.16	-0.06	-10.56	8.45
1	8	35.14	32.	32.	-0.79	-14.97	20.60	-0.06	-10.87	8.70
1	9	39.53	32.	32.	-0.79	-14.96	20.59	-0.06	-10.86	8.69
1	10	43.92	32.	32.	-0.77	-14.59	20.15	-0.06	-10.56	8.45
1	11	48.31	32.	32.	-0.73	-13.80	19.11	-0.06	-9.96	7.97
1	12	52.71	32.	32.	-0.67	-12.56	17.44	-0.05	-9.04	7.24
1	13	57.10	32.	32.	-0.59	-10.91	15.20	-0.05	-7.83	6.27
1	14	61.49	32.	32.	-0.56	-9.39	18.87	-0.04	-6.95	7.75
1	15	65.88	32.	32.	-0.40	-6.72	13.55	-0.03	-4.97	5.53
1	16	70.27	32.	32.	-0.22	-3.59	7.26	-0.02	-2.65	2.95
1	17	74.67	32.	32.	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=30.0 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	13.83	392.	-7.1	152.	-1.1	641.	-1.9	0.	0.0	-10.08
	LB			7.1		2.2		8.5		0.0	1.87 17.84
	RT			-6.4		-1.1		-1.9		0.0	8.51 -9.41
	RB			4.6		1.5		5.6		0.0	1.93 11.70
1	LT	60.83	391.	-6.4	152.	-1.1	641.	-1.9	0.	0.0	5.59 -9.40
	LB			4.6		1.5		5.6		0.0	1.93 11.69
	RT			-7.1		-1.1		-1.9		0.0	5.60 -10.08
	RB			7.1		2.2		8.5		0.0	1.87 17.84
											8.52

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	STEEL TYPE	TOP FLANGE WIDTH	THICK	WEB DEPTH	THICK	BOT FLANGE WIDTH	THICK	COVER PLATE -- (in) TOP	BOT

	(ft)		(in)		(in)		(in)		WIDTH	THICK	WIDTH	THICK
1	13.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	0.0000
2	47.00	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	11.00	1.2500
3	13.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	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) = 74.7

NO	MEMBER 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	13.83	7.00	36.50	[2]= 43.87	0.04896	>	0.04000	OK
				[3]= 36.50	0.04074	>	0.03300	OK
2	47.00	7.00	36.50	[2]= 43.87	0.04896	>	0.04000	OK
				[3]= 36.50	0.04074	>	0.03300	OK
3	13.83	7.00	36.50	[2]= 43.87	0.04896	>	0.04000	OK
				[3]= 36.50	0.04074	>	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	33.98	0.764	44.5	150.0	0.00	113.19			0
1	1	4.39	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	2	8.78	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0

1	3	13.18	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	4	17.57	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	5	21.96	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	6	26.35	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	7	30.75	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	8	35.14	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	9	39.53	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	10	43.92	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	11	48.31	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	12	52.71	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	13	57.10	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	14	61.49	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	15	65.88	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	16	70.27	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	17	74.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19	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	33.98	0.764	44.48	171.59	2
1	1	4.39	0	1	33.98	0.764	44.48	171.59	2
1	2	8.78	0	1	33.98	0.764	44.48	171.59	2
1	3	13.18	0	1	33.98	0.764	44.48	171.59	2
1	4	17.57	0	1	33.98	0.764	54.16	171.59	2
1	5	21.96	0	1	33.98	0.764	54.16	171.59	2
1	6	26.35	0	1	33.98	0.764	54.16	171.59	2
1	7	30.75	0	1	33.98	0.764	54.16	171.59	2
1	8	35.14	0	1	33.98	0.764	54.16	171.59	2
1	9	39.53	0	1	33.98	0.764	54.16	171.59	2
1	10	43.92	0	1	33.98	0.764	54.16	171.59	2
1	11	48.31	0	1	33.98	0.764	54.16	171.59	2
1	12	52.71	0	1	33.98	0.764	54.16	171.59	2
1	13	57.10	0	1	33.98	0.764	54.16	171.59	2
1	14	61.49	0	1	33.98	0.764	44.48	171.59	2
1	15	65.88	0	1	33.98	0.764	44.48	171.59	2
1	16	70.27	0	1	33.98	0.764	44.48	171.59	2
1	17	74.67	0	1	33.98	0.764	44.48	171.59	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 SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1 0	0.00	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 1	4.39	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 2	8.78	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 3	13.18	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 4	17.57	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 5	21.96	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 6	26.35	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 7	30.75	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 8	35.14	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 9	39.53	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 10	43.92	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 11	48.31	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 12	52.71	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 13	57.10	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 14	61.49	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 15	65.88	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 16	70.27	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 17	74.67	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	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 SUPT FROM (FT) FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb

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 (ft)	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	GOV CAT	GOV CAT
1	0	0.00	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	1	4.39	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	2	8.78	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	3	13.18	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	4	17.57	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	5	21.96	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	6	26.35	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	7	30.75	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	8	35.14	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	9	39.53	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	10	43.92	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	11	48.31	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	12	52.71	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	13	57.10	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0	
1	14	61.49	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	15	65.88	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	16	70.27	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0	
1	17	74.67	1	4.81	11.44	20.15	0	0.00	7.73	29.04	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	32.00	32.00	32.00
1	1		4.39	1	1.000	1.000	1.00	32.00	32.00	32.00
1	2		8.78	1	1.000	1.000	1.00	32.00	32.00	32.00
1	3		13.18	1	1.000	1.000	1.00	32.00	32.00	32.00
1	4		17.57	1	1.000	1.000	1.00	32.00	32.00	32.00
1	5		21.96	1	1.000	1.000	1.00	32.00	32.00	32.00
1	6		26.35	1	1.000	1.000	1.00	32.00	32.00	32.00
1	7		30.75	1	1.000	1.000	1.00	32.00	32.00	32.00
1	8		35.14	1	1.000	1.000	1.00	32.00	32.00	32.00
1	9		39.53	1	1.000	1.000	1.00	32.00	32.00	32.00
1	10		43.92	1	1.000	1.000	1.00	32.00	32.00	32.00
1	11		48.31	1	1.000	1.000	1.00	32.00	32.00	32.00
1	12		52.71	1	1.000	1.000	1.00	32.00	32.00	32.00
1	13		57.10	1	1.000	1.000	1.00	32.00	32.00	32.00
1	14		61.49	1	1.000	1.000	1.00	32.00	32.00	32.00
1	15		65.88	1	1.000	1.000	1.00	32.00	32.00	32.00
1	16		70.27	1	1.000	1.000	1.00	32.00	32.00	32.00
1	17		74.67	1	1.000	1.000	1.00	32.00	32.00	32.00

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	7.97	18.42
1	1		4.39	1	7.97	18.42
1	2		8.78	1	7.97	18.42
1	3		13.18	1	7.97	18.42
1	4		17.57	1	8.54	18.95
1	5		21.96	1	8.54	18.95
1	6		26.35	1	8.54	18.95
1	7		30.75	1	8.54	18.95
1	8		35.14	1	8.54	18.95
1	9		39.53	1	8.54	18.95
1	10		43.92	1	8.54	18.95
1	11		48.31	1	8.54	18.95
1	12		52.71	1	8.54	18.95
1	13		57.10	1	8.54	18.95
1	14		61.49	1	7.97	18.42
1	15		65.88	1	7.97	18.42
1	16		70.27	1	7.97	18.42
1	17		74.67	1	7.97	18.42

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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.

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	0
1	1	4.39	1	0	0	0	0	0	0
1	2	8.78	1	0	0	0	0	0	0
1	3	13.18	1	0	0	0	0	0	0
1	4	17.57	1	0	0	0	0	0	0
1	5	21.96	1	0	0	0	0	0	0
1	6	26.35	1	0	0	0	0	0	0
1	7	30.75	1	0	0	0	0	0	0
1	8	35.14	1	0	0	0	0	0	0
1	9	39.53	1	0	0	0	0	0	0
1	10	43.92	1	0	0	0	0	0	0
1	11	48.31	1	0	0	0	0	0	0
1	12	52.71	1	0	0	0	0	0	0
1	13	57.10	1	0	0	0	0	0	0
1	14	61.49	1	0	0	0	0	0	0
1	15	65.88	1	0	0	0	0	0	0
1	16	70.27	1	0	0	0	0	0	0
1	17	74.67	1	0	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]	fbu+fl (ksi)	[2]	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	0
1	1		4.39	0.0	19.2	3.9	32.0	3.9	32.0	3.9	32.0	0
				0.0	-	3.9	-	3.9	32.0	-	-	0
1	2		8.78	0.0	19.2	7.3	32.0	7.3	32.0	7.3	32.0	0
				0.0	-	7.3	-	7.3	32.0	-	-	0

1	3	13.18	0.0	19.2	10.3	32.0	10.3	32.0	10.3	32.0	0
			0.0	-	10.3	-	10.3	32.0	-	-	0
1	4	17.57	0.0	19.2	11.5	32.0	11.5	32.0	11.5	32.0	0
			0.0	-	8.3	-	8.3	32.0	-	-	0
1	5	21.96	0.0	19.2	13.3	32.0	13.3	32.0	13.3	32.0	0
			0.0	-	9.6	-	9.6	32.0	-	-	0
1	6	26.35	0.0	19.2	14.6	32.0	14.6	32.0	14.6	32.0	0
			0.0	-	10.5	-	10.5	32.0	-	-	0
1	7	30.75	0.0	19.2	15.5	32.0	15.5	32.0	15.5	32.0	0
			0.0	-	11.2	-	11.2	32.0	-	-	0
1	8	35.14	0.0	19.2	15.9	32.0	15.9	32.0	15.9	32.0	0
			0.0	-	11.5	-	11.5	32.0	-	-	0
1	9	39.53	0.0	19.2	15.9	32.0	15.9	32.0	15.9	32.0	0
			0.0	-	11.5	-	11.5	32.0	-	-	0
1	10	43.92	0.0	19.2	15.5	32.0	15.5	32.0	15.5	32.0	0
			0.0	-	11.2	-	11.2	32.0	-	-	0
1	11	48.31	0.0	19.2	14.6	32.0	14.6	32.0	14.6	32.0	0
			0.0	-	10.5	-	10.5	32.0	-	-	0
1	12	52.71	0.0	19.2	13.3	32.0	13.3	32.0	13.3	32.0	0
			0.0	-	9.6	-	9.6	32.0	-	-	0
1	13	57.10	0.0	19.2	11.5	32.0	11.5	32.0	11.5	32.0	0
			0.0	-	8.3	-	8.3	32.0	-	-	0
1	14	61.49	0.0	19.2	10.2	32.0	10.2	32.0	10.2	32.0	0
			0.0	-	10.2	-	10.2	32.0	-	-	0
1	15	65.88	0.0	19.2	7.3	32.0	7.3	32.0	7.3	32.0	0
			0.0	-	7.3	-	7.3	32.0	-	-	0
1	16	70.27	0.0	19.2	3.9	32.0	3.9	32.0	3.9	32.0	0
			0.0	-	3.9	-	3.9	32.0	-	-	0
1	17	74.67	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
			0.0	-	0.0	-	0.0	32.0	-	-	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		4.39	0.000	0.122	0.122	0.122	0.122	2
						0.122		0.122	3
1	2		8.78	0.000	0.229	0.229	0.229	0.229	2
						0.229		0.229	3
1	3		13.18	0.000	0.321	0.321	0.321	0.321	2
						0.321		0.321	3
1	4		17.57	0.000	0.359	0.359	0.359	0.359	2
						0.260		0.260	3
1	5		21.96	0.000	0.415	0.415	0.415	0.415	2
						0.299		0.299	3
1	6		26.35	0.000	0.456	0.456	0.456	0.456	2

					0.329		0.329	3
1	7	30.75	0.000	0.484	0.484	0.484	0.484	2
					0.349		0.349	3
1	8	35.14	0.000	0.498	0.498	0.498	0.498	2
					0.359		0.359	3
1	9	39.53	0.000	0.497	0.497	0.497	0.497	2
					0.359		0.359	3
1	10	43.92	0.000	0.483	0.483	0.483	0.483	2
					0.349		0.349	3
1	11	48.31	0.000	0.456	0.456	0.456	0.456	2
					0.329		0.329	3
1	12	52.71	0.000	0.414	0.414	0.414	0.414	2
					0.299		0.299	3
1	13	57.10	0.000	0.359	0.359	0.359	0.359	2
					0.259		0.259	3
1	14	61.49	0.000	0.320	0.320	0.320	0.320	2
					0.320		0.320	3
1	15	65.88	0.000	0.229	0.229	0.229	0.229	2
					0.229		0.229	3
1	16	70.27	0.000	0.122	0.122	0.122	0.122	2
					0.122		0.122	3
1	17	74.67	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	2985.7	-	-	0.0	19.2	-	-	0--0
1	1		4.39	0	680.4	2985.7	-	-	0.0	19.2	-	-	0--0
1	2		8.78	0	1268.7	2985.7	-	-	0.0	19.2	-	-	0--0
1	3		13.18	0	1766.4	2985.7	-	-	0.0	19.2	-	-	0--0
1	4		17.57	0	2170.1	4215.3	-	-	0.0	19.2	-	-	0--0
1	5		21.96	0	2487.2	4215.3	-	-	0.0	19.2	-	-	0--0
1	6		26.35	0	2724.8	4215.3	-	-	0.0	19.2	-	-	0--0
1	7		30.75	0	2873.2	4215.3	-	-	0.0	19.2	-	-	0--0
1	8		35.14	0	2935.2	4215.3	-	-	0.0	19.2	-	-	0--0
1	9		39.53	0	2934.6	4215.3	-	-	0.0	19.2	-	-	0--0
1	10		43.92	0	2872.8	4215.3	-	-	0.0	19.2	-	-	0--0
1	11		48.31	0	2725.1	4215.3	-	-	0.0	19.2	-	-	0--0
1	12		52.71	0	2486.5	4215.3	-	-	0.0	19.2	-	-	0--0
1	13		57.10	0	2168.6	4215.3	-	-	0.0	19.2	-	-	0--0
1	14		61.49	0	1765.7	2985.7	-	-	0.0	19.2	-	-	0--0
1	15		65.88	0	1268.3	2985.7	-	-	0.0	19.2	-	-	0--0
1	16		70.27	0	679.8	2985.7	-	-	0.0	19.2	-	-	0--0
1	17		74.67	0	0.0	2985.7	-	-	0.0	19.2	-	-	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	4.39 0	0.228			0.228		1
1	2	8.78 0	0.425			0.425		1
1	3	13.18 0	0.592			0.592		1
1	4	17.57 0	0.515			0.515		1
1	5	21.96 0	0.590			0.590		1
1	6	26.35 0	0.646			0.646		1
1	7	30.75 0	0.682			0.682		1
1	8	35.14 0	0.696			0.696		1
1	9	39.53 0	0.696			0.696		1
1	10	43.92 0	0.682			0.682		1
1	11	48.31 0	0.646			0.646		1
1	12	52.71 0	0.590			0.590		1
1	13	57.10 0	0.514			0.514		1
1	14	61.49 0	0.591			0.591		1
1	15	65.88 0	0.425			0.425		1
1	16	70.27 0	0.228			0.228		1
1	17	74.67 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]	Art.6.10.9.2 [3]	D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2- Vn = C x Vp
1	0	0.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	1	4.39	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	2	8.78	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	3	13.18	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	4	17.57	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	5	21.96	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	6	26.35	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	7	30.75	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	8	35.14	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	9	39.53	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	10	43.92	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	11	48.31	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	12	52.71	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	13	57.10	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	14	61.49	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	15	65.88	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	16	70.27	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	17	74.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9

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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 CAPACITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	248.1	<	481.9	0	0
1	1	4.39	0	225.3	<	481.9	0	0
1	2	8.78	0	204.0	<	481.9	0	0
1	3	13.18	0	183.0	<	481.9	0	0
1	4	17.57	0	162.3	<	481.9	0	0
1	5	21.96	0	141.4	<	481.9	0	0
1	6	26.35	0	121.2	<	481.9	0	0
1	7	30.75	0	101.3	<	481.9	0	0
1	8	35.14	0	81.3	<	481.9	0	0
1	9	39.53	0	81.6	<	481.9	0	0
1	10	43.92	0	101.2	<	481.9	0	0
1	11	48.31	0	121.0	<	481.9	0	0
1	12	52.71	0	141.6	<	481.9	0	0
1	13	57.10	0	162.1	<	481.9	0	0
1	14	61.49	0	182.8	<	481.9	0	0
1	15	65.88	0	204.1	<	481.9	0	0
1	16	70.27	0	225.4	<	481.9	0	0
1	17	74.67	0	247.9	<	481.9	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 STIFFENERS SPACING (ft-in)
1	0	0.00	32.	248.07	481.86	0	
1	1	4.39	32.	225.32	481.86	0	
1	2	8.78	32.	204.02	481.86	0	
1	3	13.18	32.	183.00	481.86	0	
1	4	17.57	32.	162.27	481.86	0	
1	5	21.96	32.	141.38	481.86	0	
1	6	26.35	32.	121.22	481.86	0	
1	7	30.75	32.	101.33	481.86	0	
1	8	35.14	32.	81.29	481.86	0	
1	9	39.53	32.	81.56	481.86	0	
1	10	43.92	32.	101.16	481.86	0	
1	11	48.31	32.	121.04	481.86	0	
1	12	52.71	32.	141.65	481.86	0	
1	13	57.10	32.	162.10	481.86	0	
1	14	61.49	32.	182.83	481.86	0	
1	15	65.88	32.	204.07	481.86	0	
1	16	70.27	32.	225.37	481.86	0	
1	17	74.67	32.	247.90	481.86	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)	FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-		30.4	0.0	30.4	-	-	00-
1	1	4.39	3.9	-		30.4	7.6	30.4	-	-	00-
1	2	8.78	7.4	-		30.4	14.2	30.4	-	-	00-
1	3	13.18	10.3	-		30.4	19.7	30.4	-	-	00-
1	4	17.57	11.9	-		30.4	15.9	30.4	-	-	00-
1	5	21.96	13.7	-		30.4	18.2	30.4	-	-	00-
1	6	26.35	15.1	-		30.4	20.0	30.4	-	-	00-
1	7	30.75	15.9	-		30.4	21.1	30.4	-	-	00-
1	8	35.14	16.3	-		30.4	21.6	30.4	-	-	00-

1	9	39.53	16.3	-	30.4	21.6	30.4	-	-	00-
1	10	43.92	15.9	-	30.4	21.1	30.4	-	-	00-
1	11	48.31	15.1	-	30.4	20.0	30.4	-	-	00-
1	12	52.71	13.7	-	30.4	18.2	30.4	-	-	00-
1	13	57.10	11.9	-	30.4	15.9	30.4	-	-	00-
1	14	61.49	10.3	-	30.4	19.7	30.4	-	-	00-
1	15	65.88	7.4	-	30.4	14.2	30.4	-	-	00-
1	16	70.27	3.9	-	30.4	7.6	30.4	-	-	00-
1	17	74.67	0.0	-	30.4	0.0	30.4	-	-	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 = II -- Urban Interstate

SP NO	IN NO	D L (ft)	FROM SUPT	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	4.39		TR	-0.4	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	2	8.78		TR	-0.7	A B B^C^C D E E^	TR	3.0	A B B^C^C D E
1	3	13.18		TR	-0.9	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1		13.83	SCG						
1	4	17.57		TR	-1.1	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1	5	21.96		TR	-1.3	A B B^C^C D E E^	TR	3.7	A B B^C^C D E
1	6	26.35		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	7	30.75		TR	-1.4	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	8	35.14		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	9	39.53		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	10	43.92		TR	-1.4	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	11	48.31		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	12	52.71		TR	-1.3	A B B^C^C D E E^	TR	3.7	A B B^C^C D E
1	13	57.10		TR	-1.1	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1		60.83	SCG						

| | | |

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	64.9	.24957E-01	1.6	TR	3.09	3	5.73
1	1	4.39	62.0	.24957E-01	1.5	TR	3.09	3	5.99
1	2	8.78	59.2	.24957E-01	1.5	TR	3.09	3	6.28
1	3	13.18	56.5	.24957E-01	1.4	TR	3.09	3	6.59
1	4	17.57	54.1	.22794E-01	1.2	TR	3.09	3	7.53
1	5	21.96	51.8	.22794E-01	1.2	TR	3.09	3	7.85
1	6	26.35	49.4	.22794E-01	1.1	TR	3.09	3	8.24
1	7	30.75	47.3	.22794E-01	1.1	TR	3.09	3	8.60
1	8	35.14	45.8	.22794E-01	1.0	TR	3.09	3	8.89
1	9	39.53	45.8	.22794E-01	1.0	TR	3.09	3	8.89
1	10	43.92	47.3	.22794E-01	1.1	TR	3.09	3	8.60
1	11	48.31	49.4	.22794E-01	1.1	TR	3.09	3	8.24
1	12	52.71	51.8	.22794E-01	1.2	TR	3.09	3	7.85
1	13	57.10	54.1	.22794E-01	1.2	TR	3.09	3	7.53
1	14	61.49	56.5	.24957E-01	1.4	TR	3.09	3	6.59
1	15	65.88	59.2	.24957E-01	1.5	TR	3.09	3	6.28
1	16	70.27	62.0	.24957E-01	1.5	TR	3.09	3	5.99
1	17	74.67	64.9	.24957E-01	1.6	TR	3.09	3	5.73

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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= 2038
*** 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	32.4	.24957E-01	0.8	TR 0.76	3	2.80
1	1	4.39	31.0	.24957E-01	0.8	TR 0.76	3	2.93
1	2	8.78	29.6	.24957E-01	0.7	TR 0.76	3	3.07
1	3	13.18	28.2	.24957E-01	0.7	TR 0.76	3	3.22
1	4	17.57	27.0	.22794E-01	0.6	TR 0.76	3	3.68
1	5	21.96	25.9	.22794E-01	0.6	TR 0.76	3	3.84
1	6	26.35	24.7	.22794E-01	0.6	TR 0.76	3	4.03
1	7	30.75	23.7	.22794E-01	0.5	TR 0.76	3	4.21
1	8	35.14	22.9	.22794E-01	0.5	TR 0.76	3	4.35
1	9	39.53	22.9	.22794E-01	0.5	TR 0.76	3	4.35
1	10	43.92	23.7	.22794E-01	0.5	TR 0.76	3	4.21
1	11	48.31	24.7	.22794E-01	0.6	TR 0.76	3	4.03
1	12	52.71	25.9	.22794E-01	0.6	TR 0.76	3	3.84
1	13	57.10	27.0	.22794E-01	0.6	TR 0.76	3	3.68
1	14	61.49	28.2	.24957E-01	0.7	TR 0.76	3	3.22
1	15	65.88	29.6	.24957E-01	0.7	TR 0.76	3	3.07
1	16	70.27	31.0	.24957E-01	0.8	TR 0.76	3	2.93
1	17	74.67	32.4	.24957E-01	0.8	TR 0.76	3	2.80

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.6.10.10.2-2 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= 2038 & No. of Cycles= 55790250
*** If ADTT Single Lane greater than or equal to 960, refer
to Fatigue I Table.

1

TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN D FROM L SUPT (ft)	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 35.14	1	2245.8	1338.7		255	>	80		
1	17 74.67	1	2245.8	1338.7		255	>	80		

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	35.137	5.000
1	35.137	74.667	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	74.7	C

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

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 L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi	OPERAT. RATING FACTOR POS. MAX. STRESS	OPERAT. RATING FACTOR NEG. [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	2985.7	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1		4.39	2985.7	144.2	56.0	240.5	0.0	8.40	HL-93	6.48	0
1	2		8.78	2985.7	269.9	105.0	447.3	0.0	4.14	HL-93	3.19	0
1	3		13.18	2985.7	378.0	147.1	620.5	0.0	2.75	HL-93	2.12	0
1	4		17.57	4215.3	468.6	182.1	758.2	0.0	3.29	HL-93	2.54	0
1	5		21.96	4215.3	540.3	210.1	865.6	0.0	2.78	HL-93	2.14	0
1	6		26.35	4215.3	594.2	231.1	945.9	0.0	2.46	HL-93	1.90	0
1	7		30.75	4215.3	630.4	245.1	993.5	0.0	2.30	HL-93	1.77	0
1	8		35.14	4215.3	648.5	252.1	1010.4	0.0	2.23	HL-93	1.72	0
1	9		39.53	4215.3	648.1	252.1	1010.4	0.0	2.24	HL-93	1.72	0
1	10		43.92	4215.3	630.0	245.1	993.5	0.0	2.30	HL-93	1.77	0
1	11		48.31	4215.3	594.4	231.1	945.9	0.0	2.46	HL-93	1.90	0
1	12		52.71	4215.3	539.8	210.1	865.6	0.0	2.78	HL-93	2.14	0
1	13		57.10	4215.3	467.4	182.1	758.2	0.0	3.30	HL-93	2.54	0
1	14		61.49	2985.7	377.5	147.1	620.5	0.0	2.75	HL-93	2.12	0
1	15		65.88	2985.7	269.6	105.0	447.3	0.0	4.14	HL-93	3.19	0
1	16		70.27	2985.7	143.8	56.0	240.5	0.0	8.40	HL-93	6.48	0
1	17		74.67	2985.7	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

1

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]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	30.4	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	4.39	30.4	-3.03	3.44	-0.70	3.19	0.00	0.00	8.44	HL-93	6.50
1	2	8.78	30.4	-5.67	6.43	-1.30	5.94	0.00	0.00	4.03	HL-93	3.10
1	3	13.18	30.4	-7.93	9.01	-1.81	8.24	0.00	0.00	2.60	HL-93	2.00
1	4	17.57	30.4	-8.96	7.30	-2.28	6.62	0.00	0.00	3.49	HL-93	2.69
1	5	21.96	30.4	-10.33	8.42	-2.60	7.55	0.00	0.00	2.91	HL-93	2.24
1	6	26.35	30.4	-11.36	9.26	-2.84	8.25	0.00	0.00	2.56	HL-93	1.97
1	7	30.75	30.4	-12.05	9.82	-2.99	8.67	0.00	0.00	2.37	HL-93	1.83
1	8	35.14	30.4	-12.40	10.11	-3.04	8.82	0.00	0.00	2.30	HL-93	1.77
1	9	39.53	30.4	-12.39	10.10	-3.04	8.82	0.00	0.00	2.30	HL-93	1.77
1	10	43.92	30.4	-12.04	9.82	-2.99	8.67	0.00	0.00	2.37	HL-93	1.83
1	11	48.31	30.4	-11.36	9.26	-2.84	8.25	0.00	0.00	2.56	HL-93	1.97
1	12	52.71	30.4	-10.32	8.41	-2.60	7.55	0.00	0.00	2.91	HL-93	2.24
1	13	57.10	30.4	-8.94	7.29	-2.28	6.62	0.00	0.00	3.49	HL-93	2.69
1	14	61.49	30.4	-7.92	9.00	-1.81	8.24	0.00	0.00	2.60	HL-93	2.00
1	15	65.88	30.4	-5.66	6.43	-1.30	5.94	0.00	0.00	4.04	HL-93	3.10
1	16	70.27	30.4	-3.02	3.43	-0.70	3.19	0.00	0.00	8.45	HL-93	6.50
1	17	74.67	30.4	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

[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	74.67	35.14	STRENGTH L.S.	2.23	1.72	HL-93
1	74.67	35.14	SERVICEBILITY	2.30	1.77	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.235
Inventory= 1.724

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	74.67	30.75	STRENGTH L.S.	2.98	1035.32
1	74.67	30.75	SERVICEBILITY	3.01	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 2.976

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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 FACTOR POS.	MAX. SHEAR FACTOR NEG.	OPERAT. RATING [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	481.9	35.6	13.6	105.4	0.0	2.94	HL-93	2.27

1	1	4.39	481.9	30.6	12.0	97.2	-2.7	3.25	HL-93	2.51
1	2	8.78	481.9	26.6	10.4	89.2	-6.2	3.60	HL-93	2.78
1	3	13.18	481.9	22.6	8.8	81.3	-10.5	4.02	HL-93	3.10
1	4	17.57	481.9	18.6	7.2	73.6	-14.9	4.51	HL-93	3.48
1	5	21.96	481.9	14.3	5.6	66.1	-19.4	5.11	HL-93	3.94
1	6	26.35	481.9	10.3	4.0	58.7	-24.5	5.85	HL-93	4.51
1	7	30.75	481.9	6.2	2.4	51.5	-30.8	6.77	HL-93	5.22
1	8	35.14	481.9	1.9	0.8	44.5	-37.6	7.97	HL-93	6.15
1	9	39.53	481.9	-2.1	-0.8	37.6	-44.5	7.97	HL-93	6.15
1	10	43.92	481.9	-6.1	-2.4	30.8	-51.5	6.77	HL-93	5.22
1	11	48.31	481.9	-10.1	-4.0	24.5	-58.7	5.85	HL-93	4.51
1	12	52.71	481.9	-14.5	-5.6	19.4	-66.1	5.11	HL-93	3.94
1	13	57.10	481.9	-18.5	-7.2	14.9	-73.6	4.51	HL-93	3.48
1	14	61.49	481.9	-22.5	-8.8	10.5	-81.3	4.02	HL-93	3.10
1	15	65.88	481.9	-26.7	-10.4	6.2	-89.2	3.60	HL-93	2.78
1	16	70.27	481.9	-30.7	-12.0	2.7	-97.2	3.25	HL-93	2.51
1	17	74.67	481.9	-35.4	-13.6	0.0	-105.4	2.94	HL-93	2.27

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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	74.67	0.00	STRENGTH L.S.	2.94	2.27	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.939
Inventory= 2.267

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	74.67	0.00	STRENGTH L.S.	3.43	121.85

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 3.432

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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 GOVERN. (inch)	LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)

1	37.33	2	0.400	-0.48	HL-93	2.33
		3	0.510	-0.61		1.83

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. 5800 Span 1 Interior Beam Midspan	7/15/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 1(4)	IAS/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 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

TABLE 0.0.3.1 STRUCTURAL DETAILS
****

BEAM NUMBER	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
5	1	31.33	2.50	1.83	1.62	18.12	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
74.67	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
7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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	36	194			0.00	0.0000	0.00	0.0000		
2	W	36	194			0.00	0.0000	11.00	1.2500		

NOTE: [1] maximum allowable section number is 70

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

TABLE 0.0.5.1 DEFINITION OF MEMBERS

MEMBER NUMBER (IN ORDER)	MEMB SECT LEFT	END ID RIGHT	MEMBER DESCRIPTN LNGTH -->TYPE<-- (ft)	PARAMETERS FOR NONPRISMATIC MEMB 0=PRISMAT	S(0)	S(1)	YIELD STRESS (KSI) WEB	TOP	BOT
1	1	1	13.83				32.	32.	32.
2	2	2	47.00				32.	32.	32.
3	1	1	13.83				32.	32.	32.

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	2	2398	2038	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 2398

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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	0.33	0.00	15.00	1 1 1 1 1	0.54	2 2 2 2 2
1	0.00	0	0.00	0.69	0.00	0.00	0 0 0 0 0	0.98	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	7.0	0	30.0	10.0	0.69	0.00	74.67

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	0	wearing surf	0.238	0.00	74.67	0.00	0.00
2	1	rail	0.010	0.00	74.67	0.00	0.00
3	1	curb	0.115	0.00	74.67	0.00	0.00
4	2	end diaphrag	0.000	0.00	0.00	0.76	0.45
5	2	int diaphrag	0.000	0.00	0.00	0.17	3.65
6	2	int diaphrag	0.000	0.00	0.00	0.35	18.65
7	2	int diaphrag	0.000	0.00	0.00	0.35	33.65
8	2	int diaphrag	0.000	0.00	0.00	0.35	48.65
9	2	int diaphrag	0.000	0.00	0.00	0.17	63.65
10	2	end diaphrag	0.000	0.00	0.00	0.76	74.21

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 YIELD Fy (ksi)	BAR AREA PER FOOT OF SLAB (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	2.50	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		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	3.65	33.0	33.0	3.20
3.65	63.65	33.0	33.0	15.00
63.65	74.21	33.0	33.0	10.56

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

TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.688		0.2234		0.9114

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.24	0.00	74.67
1	DC2	0.010	0.000	74.667
2	DC2	0.115	0.000	74.667

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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	1524.5	56.5	228.7	83.4	312.07

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	7.50	90.00

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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	0.544	0.544	0.664	0.664	FOR STRENGTH POSITIVE MOMENT
	0.979				FOR STRENGTH POSITIVE SHEAR
1	0.328				FOR FATIGUE POSITIVE MOMENT
	0.691				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)	STEEL TOP. (in**3)
1	0	0.00	12100.0	36.5	18.25	663.0	663.0

1	1	3.73	12100.0	36.5	18.25	663.0	663.0
1	2	7.47	12100.0	36.5	18.25	663.0	663.0
1	3	11.20	12100.0	36.5	18.25	663.0	663.0
1	4	14.93	16070.6	36.5	15.83	1015.1	733.2
1	5	18.67	16070.6	36.5	15.83	1015.1	733.2
1	6	22.40	16070.6	36.5	15.83	1015.1	733.2
1	7	26.13	16070.6	36.5	15.83	1015.1	733.2
1	8	29.87	16070.6	36.5	15.83	1015.1	733.2
1	9	33.60	16070.6	36.5	15.83	1015.1	733.2
1	10	37.33	16070.6	36.5	15.83	1015.1	733.2
1	11	41.07	16070.6	36.5	15.83	1015.1	733.2
1	12	44.80	16070.6	36.5	15.83	1015.1	733.2
1	13	48.53	16070.6	36.5	15.83	1015.1	733.2
1	14	52.27	16070.6	36.5	15.83	1015.1	733.2
1	15	56.00	16070.6	36.5	15.83	1015.1	733.2
1	16	59.73	16070.6	36.5	15.83	1015.1	733.2
1	17	63.47	12100.0	36.5	18.25	663.0	663.0
1	18	67.20	12100.0	36.5	18.25	663.0	663.0
1	19	70.93	12100.0	36.5	18.25	663.0	663.0
1	20	74.67	12100.0	36.5	18.25	663.0	663.0

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 = 30.00

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	19760.3	0.017683	814.0	1616.7	1008.7
1	1	3.73	19760.3	0.017683	814.0	1616.7	1008.7
1	2	7.47	19760.3	0.017683	814.0	1616.7	1008.7
1	3	11.20	19760.3	0.017683	814.0	1616.7	1008.7
1	4	14.93	27039.6	0.015941	1239.5	1696.8	1160.4
1	5	18.67	27039.6	0.015941	1239.5	1696.8	1160.4
1	6	22.40	27039.6	0.015941	1239.5	1696.8	1160.4
1	7	26.13	27039.6	0.015941	1239.5	1696.8	1160.4
1	8	29.87	27039.6	0.015941	1239.5	1696.8	1160.4
1	9	33.60	27039.6	0.015941	1239.5	1696.8	1160.4
1	10	37.33	27039.6	0.015941	1239.5	1696.8	1160.4
1	11	41.07	27039.6	0.015941	1239.5	1696.8	1160.4
1	12	44.80	27039.6	0.015941	1239.5	1696.8	1160.4
1	13	48.53	27039.6	0.015941	1239.5	1696.8	1160.4
1	14	52.27	27039.6	0.015941	1239.5	1696.8	1160.4
1	15	56.00	27039.6	0.015941	1239.5	1696.8	1160.4
1	16	59.73	27039.6	0.015941	1239.5	1696.8	1160.4
1	17	63.47	19760.3	0.017683	814.0	1616.7	1008.7
1	18	67.20	19760.3	0.017683	814.0	1616.7	1008.7
1	19	70.93	19760.3	0.017683	814.0	1616.7	1008.7
1	20	74.67	19760.3	0.017683	814.0	1616.7	1008.7

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 10.00

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA Ix (in**4)	Q/Ix 1ST. MOMENT OF INERTIA (1/in)	ELASTIC SECTION MODULUS,(in**3) STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	27052.0	0.024957	903.7	4121.1	1941.9
1	1	3.73	27052.0	0.024957	903.7	4121.1	1941.9
1	2	7.47	27052.0	0.024957	903.7	4121.1	1941.9
1	3	11.20	27052.0	0.024957	903.7	4121.1	1941.9
1	4	14.93	38611.6	0.022794	1375.2	3991.7	2266.1
1	5	18.67	38611.6	0.022794	1375.2	3991.7	2266.1
1	6	22.40	38611.6	0.022794	1375.2	3991.7	2266.1
1	7	26.13	38611.6	0.022794	1375.2	3991.7	2266.1
1	8	29.87	38611.6	0.022794	1375.2	3991.7	2266.1
1	9	33.60	38611.6	0.022794	1375.2	3991.7	2266.1
1	10	37.33	38611.6	0.022794	1375.2	3991.7	2266.1
1	11	41.07	38611.6	0.022794	1375.2	3991.7	2266.1
1	12	44.80	38611.6	0.022794	1375.2	3991.7	2266.1
1	13	48.53	38611.6	0.022794	1375.2	3991.7	2266.1
1	14	52.27	38611.6	0.022794	1375.2	3991.7	2266.1
1	15	56.00	38611.6	0.022794	1375.2	3991.7	2266.1
1	16	59.73	38611.6	0.022794	1375.2	3991.7	2266.1
1	17	63.47	27052.0	0.024957	903.7	4121.1	1941.9
1	18	67.20	27052.0	0.024957	903.7	4121.1	1941.9
1	19	70.93	27052.0	0.024957	903.7	4121.1	1941.9
1	20	74.67	27052.0	0.024957	903.7	4121.1	1941.9

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 NO	IN NO	D FROM L SUPT (ft)	BEAM (k-ft)	SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	NONCOMPOSITE DEAD	NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	3.73	29.6	91.1	3.2	0.0		123.9
1	2	7.47	56.0	172.6	5.4	0.0		234.1
1	3	11.20	79.4	244.5	7.6	0.0		331.6
1	4	14.93	99.6	306.9	9.9	0.0		416.4
1	5	18.67	116.8	359.6	12.1	0.0		488.4
1	6	22.40	130.8	402.7	13.0	0.0		546.5
1	7	26.13	141.7	436.3	13.9	0.0		591.9
1	8	29.87	149.5	460.3	14.8	0.0		624.6
1	9	33.60	154.1	474.7	15.7	0.0		644.5
1	10	37.33	155.7	479.5	15.3	0.0		650.5
1	11	41.07	154.1	474.7	14.9	0.0		643.7
1	12	44.80	149.5	460.3	14.5	0.0		624.3
1	13	48.53	141.7	436.3	14.1	0.0		592.1
1	14	52.27	130.8	402.7	12.5	0.0		546.0
1	15	56.00	116.8	359.6	10.8	0.0		487.2
1	16	59.73	99.6	306.9	9.1	0.0		415.6
1	17	63.47	79.4	244.5	7.4	0.0		331.3
1	18	67.20	56.0	172.6	5.1	0.0		233.7
1	19	70.93	29.6	91.1	2.7	0.0		123.4
1	20	74.67	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 = 30.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD (k-ft) DW	UNIFORM SUPERIMPOSED DEAD (k-ft) DC2	CONCENTRATED LOADS (k-ft) DW	CONCENTRATED LOADS (k-ft) DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD	TOTAL (k-ft) -- COMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0		0.0
1	1	3.73	31.5	16.6	0.0	0.0	0.0		48.1
1	2	7.47	59.7	31.4	0.0	0.0	0.0		91.1
1	3	11.20	84.6	44.4	0.0	0.0	0.0		129.0
1	4	14.93	106.2	55.8	0.0	0.0	0.0		161.9
1	5	18.67	124.4	65.3	0.0	0.0	0.0		189.7
1	6	22.40	139.3	73.2	0.0	0.0	0.0		212.5
1	7	26.13	150.9	79.3	0.0	0.0	0.0		230.2
1	8	29.87	159.2	83.6	0.0	0.0	0.0		242.9
1	9	33.60	164.2	86.2	0.0	0.0	0.0		250.4
1	10	37.33	165.9	87.1	0.0	0.0	0.0		253.0
1	11	41.07	164.2	86.2	0.0	0.0	0.0		250.4
1	12	44.80	159.2	83.6	0.0	0.0	0.0		242.9
1	13	48.53	150.9	79.3	0.0	0.0	0.0		230.2
1	14	52.27	139.3	73.2	0.0	0.0	0.0		212.5
1	15	56.00	124.4	65.3	0.0	0.0	0.0		189.7
1	16	59.73	106.2	55.8	0.0	0.0	0.0		161.9
1	17	63.47	84.6	44.4	0.0	0.0	0.0		129.0
1	18	67.20	59.7	31.4	0.0	0.0	0.0		91.1
1	19	70.93	31.5	16.6	0.0	0.0	0.0		48.1
1	20	74.67	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 = 10.00 (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	3.73	0.0	0.0	206.5	HL-93	0.0	HL-93
1	2	7.47	0.0	0.0	388.8	HL-93	0.0	HL-93
1	3	11.20	0.0	0.0	546.7	HL-93	0.0	HL-93
1	4	14.93	0.0	0.0	680.4	HL-93	0.0	HL-93
1	5	18.67	0.0	0.0	787.7	HL-93	0.0	HL-93
1	6	22.40	0.0	0.0	874.8	HL-93	0.0	HL-93
1	7	26.13	0.0	0.0	942.6	HL-93	0.0	HL-93
1	8	29.87	0.0	0.0	988.2	HL-93	0.0	HL-93
1	9	33.60	0.0	0.0	1006.4	HL-93	0.0	HL-93
1	10	37.33	0.0	0.0	1012.5	HL-93	0.0	HL-93
1	11	41.07	0.0	0.0	1006.4	HL-93	0.0	HL-93
1	12	44.80	0.0	0.0	988.2	HL-93	0.0	HL-93
1	13	48.53	0.0	0.0	942.6	HL-93	0.0	HL-93
1	14	52.27	0.0	0.0	874.8	HL-93	0.0	HL-93
1	15	56.00	0.0	0.0	787.7	HL-93	0.0	HL-93
1	16	59.73	0.0	0.0	680.4	HL-93	0.0	HL-93
1	17	63.47	0.0	0.0	546.7	HL-93	0.0	HL-93
1	18	67.20	0.0	0.0	388.8	HL-93	0.0	HL-93
1	19	70.93	0.0	0.0	206.5	HL-93	0.0	HL-93
1	20	74.67	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 = 10.0 (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	3.73	72.	0.	72.
1	2	7.47	133.	0.	133.
1	3	11.20	183.	0.	183.
1	4	14.93	225.	0.	225.
1	5	18.67	261.	0.	261.
1	6	22.40	288.	0.	288.
1	7	26.13	307.	0.	307.
1	8	29.87	316.	0.	316.
1	9	33.60	308.	0.	308.
1	10	37.33	305.	0.	305.
1	11	41.07	308.	0.	308.
1	12	44.80	316.	0.	316.
1	13	48.53	307.	0.	307.
1	14	52.27	288.	0.	288.
1	15	56.00	261.	0.	261.
1	16	59.73	225.	0.	225.
1	17	63.47	183.	0.	183.
1	18	67.20	133.	0.	133.
1	19	70.93	72.	0.	72.
1	20	74.67	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=30.0 (k-ft)	LL+I:N=10.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	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	3.73	123.9	48.1	206.5	HL-93	0.0	HL-93	378.5	171.9
1	2	7.47	234.1	91.1	388.8	HL-93	0.0	HL-93	713.9	325.1
1	3	11.20	331.6	129.0	546.7	HL-93	0.0	HL-93	1007.3	460.6
1	4	14.93	416.4	161.9	680.4	HL-93	0.0	HL-93	1258.6	578.3
1	5	18.67	488.4	189.7	787.7	HL-93	0.0	HL-93	1465.9	678.2
1	6	22.40	546.5	212.5	874.8	HL-93	0.0	HL-93	1633.8	759.0
1	7	26.13	591.9	230.2	942.6	HL-93	0.0	HL-93	1764.7	822.1
1	8	29.87	624.6	242.9	988.2	HL-93	0.0	HL-93	1855.6	867.4
1	9	33.60	644.5	250.4	1006.4	HL-93	0.0	HL-93	1901.3	895.0
1	10	37.33	650.5	253.0	1012.5	HL-93	0.0	HL-93	1915.9	903.5
1	11	41.07	643.7	250.4	1006.4	HL-93	0.0	HL-93	1900.6	894.2
1	12	44.80	624.3	242.9	988.2	HL-93	0.0	HL-93	1855.3	867.1
1	13	48.53	592.1	230.2	942.6	HL-93	0.0	HL-93	1765.0	822.3
1	14	52.27	546.0	212.5	874.8	HL-93	0.0	HL-93	1633.3	758.5
1	15	56.00	487.2	189.7	787.7	HL-93	0.0	HL-93	1464.6	676.9
1	16	59.73	415.6	161.9	680.4	HL-93	0.0	HL-93	1257.9	577.5
1	17	63.47	331.3	129.0	546.7	HL-93	0.0	HL-93	1007.1	460.3
1	18	67.20	233.7	91.1	388.8	HL-93	0.0	HL-93	713.6	324.8
1	19	70.93	123.4	48.1	206.5	HL-93	0.0	HL-93	378.0	171.4
1	20	74.67	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	3.73	378.5	440.5	584.3	501.7	257.9	53.7
1	2	7.47	713.9	830.6	1101.8	946.2	487.7	99.8
1	3	11.20	1007.3	1171.3	1553.7	1335.0	690.9	137.3
1	4	14.93	1258.6	1462.8	1940.0	1667.9	867.4	169.1
1	5	18.67	1465.9	1702.2	2257.3	1942.2	1017.2	195.5
1	6	22.40	1633.8	1896.2	2514.4	2164.5	1138.5	215.7
1	7	26.13	1764.7	2047.5	2714.9	2337.9	1233.1	230.3
1	8	29.87	1855.6	2152.0	2853.4	2458.1	1301.1	237.3
1	9	33.60	1901.3	2203.3	2920.9	2518.4	1342.4	230.7
1	10	37.33	1915.9	2219.7	2942.6	2537.6	1355.2	228.5
1	11	41.07	1900.6	2202.5	2920.0	2517.4	1341.3	230.7
1	12	44.80	1855.3	2151.8	2853.0	2457.8	1300.7	237.3
1	13	48.53	1765.0	2047.7	2715.2	2338.2	1233.5	230.3
1	14	52.27	1633.3	1895.7	2513.8	2163.9	1137.8	215.7
1	15	56.00	1464.6	1700.9	2255.7	1940.6	1015.3	195.5
1	16	59.73	1257.9	1462.0	1939.1	1666.9	866.2	169.1
1	17	63.47	1007.1	1171.1	1553.4	1334.7	690.5	137.3
1	18	67.20	713.6	830.2	1101.3	945.8	487.2	99.8
1	19	70.93	378.0	440.0	583.6	501.0	257.2	53.7
1	20	74.67	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	8.3	25.7	1.5	0.0	35.6
1	1	3.73	7.5	23.1	0.6	0.0	31.2
1	2	7.47	6.7	20.5	0.6	0.0	27.8
1	3	11.20	5.8	18.0	0.6	0.0	24.4
1	4	14.93	5.0	15.4	0.6	0.0	21.0
1	5	18.67	4.2	12.8	0.2	0.0	17.3
1	6	22.40	3.3	10.3	0.2	0.0	13.9
1	7	26.13	2.5	7.7	0.2	0.0	10.5
1	8	29.87	1.7	5.1	0.2	0.0	7.0
1	9	33.60	0.8	2.6	0.2	0.0	3.6
1	10	37.33	0.0	0.0	-0.1	0.0	-0.1
1	11	41.07	-0.8	-2.6	-0.1	0.0	-3.5
1	12	44.80	-1.7	-5.1	-0.1	0.0	-6.9
1	13	48.53	-2.5	-7.7	-0.1	0.0	-10.3
1	14	52.27	-3.3	-10.3	-0.5	0.0	-14.1
1	15	56.00	-4.2	-12.8	-0.5	0.0	-17.5
1	16	59.73	-5.0	-15.4	-0.5	0.0	-20.9
1	17	63.47	-5.8	-18.0	-0.5	0.0	-24.3
1	18	67.20	-6.7	-20.5	-0.6	0.0	-27.9
1	19	70.93	-7.5	-23.1	-0.6	0.0	-31.3
1	20	74.67	-8.3	-25.7	-1.4	0.0	-35.4

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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	4.7	0.0	0.0	13.6	35.6	49.1
1	1	3.73	8.0	4.2	0.0	0.0	12.2	31.2	43.4
1	2	7.47	7.1	3.7	0.0	0.0	10.8	27.8	38.7
1	3	11.20	6.2	3.3	0.0	0.0	9.5	24.4	33.9
1	4	14.93	5.3	2.8	0.0	0.0	8.1	21.0	29.1
1	5	18.67	4.4	2.3	0.0	0.0	6.8	17.3	24.0
1	6	22.40	3.6	1.9	0.0	0.0	5.4	13.9	19.3
1	7	26.13	2.7	1.4	0.0	0.0	4.1	10.5	14.5
1	8	29.87	1.8	0.9	0.0	0.0	2.7	7.0	9.8
1	9	33.60	0.9	0.5	0.0	0.0	1.4	3.6	5.0
1	10	37.33	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1
1	11	41.07	-0.9	-0.5	0.0	0.0	-1.4	-3.5	-4.9
1	12	44.80	-1.8	-0.9	0.0	0.0	-2.7	-6.9	-9.6
1	13	48.53	-2.7	-1.4	0.0	0.0	-4.1	-10.3	-14.4
1	14	52.27	-3.6	-1.9	0.0	0.0	-5.4	-14.1	-19.5
1	15	56.00	-4.4	-2.3	0.0	0.0	-6.8	-17.5	-24.2
1	16	59.73	-5.3	-2.8	0.0	0.0	-8.1	-20.9	-29.0
1	17	63.47	-6.2	-3.3	0.0	0.0	-9.5	-24.3	-33.8
1	18	67.20	-7.1	-3.7	0.0	0.0	-10.8	-27.9	-38.7
1	19	70.93	-8.0	-4.2	0.0	0.0	-12.2	-31.3	-43.5
1	20	74.67	-8.9	-4.7	0.0	0.0	-13.6	-35.4	-49.0

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	105.4	HL-93	0.0	HL-93
1	1	3.73	0.0	0.0	98.4	HL-93	-2.2	HL-93
1	2	7.47	0.0	0.0	91.6	HL-93	-5.0	HL-93
1	3	11.20	0.0	0.0	84.9	HL-93	-8.5	HL-93
1	4	14.93	0.0	0.0	78.2	HL-93	-12.2	HL-93
1	5	18.67	0.0	0.0	71.7	HL-93	-16.0	HL-93
1	6	22.40	0.0	0.0	65.4	HL-93	-19.9	HL-93
1	7	26.13	0.0	0.0	59.1	HL-93	-24.2	HL-93
1	8	29.87	0.0	0.0	52.9	HL-93	-29.5	HL-93
1	9	33.60	0.0	0.0	46.9	HL-93	-35.2	HL-93
1	10	37.33	0.0	0.0	41.0	HL-93	-41.0	HL-93
1	11	41.07	0.0	0.0	35.2	HL-93	-46.9	HL-93
1	12	44.80	0.0	0.0	29.5	HL-93	-52.9	HL-93
1	13	48.53	0.0	0.0	24.2	HL-93	-59.1	HL-93
1	14	52.27	0.0	0.0	19.9	HL-93	-65.4	HL-93
1	15	56.00	0.0	0.0	16.0	HL-93	-71.7	HL-93
1	16	59.73	0.0	0.0	12.2	HL-93	-78.2	HL-93
1	17	63.47	0.0	0.0	8.5	HL-93	-84.9	HL-93
1	18	67.20	0.0	0.0	5.0	HL-93	-91.6	HL-93
1	19	70.93	0.0	0.0	2.2	HL-93	-98.4	HL-93
1	20	74.67	0.0	0.0	0.0	HL-93	-105.4	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 10.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0	0.00	43.	0.	43.
1	1	3.73	40.	-1.	42.
1	2	7.47	38.	-3.	40.
1	3	11.20	35.	-4.	38.
1	4	14.93	32.	-5.	37.
1	5	18.67	29.	-7.	36.
1	6	22.40	26.	-8.	34.
1	7	26.13	23.	-10.	33.
1	8	29.87	20.	-12.	32.
1	9	33.60	18.	-13.	31.
1	10	37.33	15.	-15.	30.
1	11	41.07	13.	-18.	31.
1	12	44.80	12.	-20.	32.
1	13	48.53	10.	-23.	33.
1	14	52.27	8.	-26.	34.
1	15	56.00	7.	-29.	36.
1	16	59.73	5.	-32.	37.
1	17	63.47	4.	-35.	38.
1	18	67.20	3.	-38.	40.
1	19	70.93	1.	-40.	42.
1	20	74.67	0.	-43.	43.

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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	43.2	.27984E-01	0.60	0.3125*.15400E-01		0.33	0.3125*
1	1		3.73	41.6	.27984E-01	0.58	0.3125*.15400E-01		0.32	0.3125*
1	2		7.47	40.1	.27984E-01	0.56	0.3125*.15400E-01		0.31	0.3125*
1	3		11.20	38.4	.27984E-01	0.54	0.3125*.15400E-01		0.30	0.3125*
1	4		14.93	37.0	.26097E-01	0.48	0.3125*.96076E-02		0.18	0.3125*
1	5		18.67	35.7	.26097E-01	0.47	0.3125*.96076E-02		0.17	0.3125*
1	6		22.40	34.4	.26097E-01	0.45	0.3125*.96076E-02		0.17	0.3125*
1	7		26.13	33.0	.26097E-01	0.43	0.3125*.96076E-02		0.16	0.3125*
1	8		29.87	31.9	.26097E-01	0.42	0.3125*.96076E-02		0.15	0.3125*
1	9		33.60	30.8	.26097E-01	0.40	0.3125*.96076E-02		0.15	0.3125*
1	10		37.33	30.4	.26097E-01	0.40	0.3125*.96076E-02		0.15	0.3125*
1	11		41.07	30.8	.26097E-01	0.40	0.3125*.96076E-02		0.15	0.3125*
1	12		44.80	31.9	.26097E-01	0.42	0.3125*.96076E-02		0.15	0.3125*
1	13		48.53	33.0	.26097E-01	0.43	0.3125*.96076E-02		0.16	0.3125*
1	14		52.27	34.4	.26097E-01	0.45	0.3125*.96076E-02		0.17	0.3125*
1	15		56.00	35.7	.26097E-01	0.47	0.3125*.96076E-02		0.17	0.3125*
1	16		59.73	37.0	.26097E-01	0.48	0.3125*.96076E-02		0.18	0.3125*
1	17		63.47	38.4	.27984E-01	0.54	0.3125*.15400E-01		0.30	0.3125*
1	18		67.20	40.1	.27984E-01	0.56	0.3125*.15400E-01		0.31	0.3125*
1	19		70.93	41.6	.27984E-01	0.58	0.3125*.15400E-01		0.32	0.3125*
1	20		74.67	43.2	.27984E-01	0.60	0.3125*.15400E-01		0.33	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	49.1	105.4	HL-93	0.0	HL-93	154.5	49.1
1	1	3.73	43.4	98.4	HL-93	-2.2	HL-93	141.9	41.2
1	2	7.47	38.7	91.6	HL-93	-5.0	HL-93	130.2	33.7
1	3	11.20	33.9	84.9	HL-93	-8.5	HL-93	118.8	25.4
1	4	14.93	29.1	78.2	HL-93	-12.2	HL-93	107.4	16.9
1	5	18.67	24.0	71.7	HL-93	-16.0	HL-93	95.8	8.0
1	6	22.40	19.3	65.4	HL-93	-19.9	HL-93	84.6	-0.6
1	7	26.13	14.5	59.1	HL-93	-24.2	HL-93	73.6	-9.7
1	8	29.87	9.8	52.9	HL-93	-29.5	HL-93	62.7	-19.8
1	9	33.60	5.0	46.9	HL-93	-35.2	HL-93	51.9	-30.2
1	10	37.33	-0.1	41.0	HL-93	-41.0	HL-93	40.9	-41.1
1	11	41.07	-4.9	35.2	HL-93	-46.9	HL-93	30.3	-51.8
1	12	44.80	-9.6	29.5	HL-93	-52.9	HL-93	19.9	-62.6
1	13	48.53	-14.4	24.2	HL-93	-59.1	HL-93	9.9	-73.5
1	14	52.27	-19.5	19.9	HL-93	-65.4	HL-93	0.4	-84.8
1	15	56.00	-24.2	16.0	HL-93	-71.7	HL-93	-8.3	-96.0
1	16	59.73	-29.0	12.2	HL-93	-78.2	HL-93	-16.8	-107.2
1	17	63.47	-33.8	8.5	HL-93	-84.9	HL-93	-25.2	-118.6
1	18	67.20	-38.7	5.0	HL-93	-91.6	HL-93	-33.7	-130.3
1	19	70.93	-43.5	2.2	HL-93	-98.4	HL-93	-41.2	-141.9
1	20	74.67	-49.0	0.0	HL-93	-105.4	HL-93	-49.0	-154.4

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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	154.5	186.1	248.1	205.9	73.7	32.4
1	1	3.73	141.9	171.4	228.5	189.2	65.1	31.2
1	2	7.47	130.2	157.7	210.4	173.7	58.0	30.0
1	3	11.20	118.8	144.2	192.4	158.5	50.8	28.8
1	4	14.93	107.4	130.9	174.7	143.4	43.7	27.7
1	5	18.67	95.8	117.3	156.7	128.0	36.0	26.8
1	6	22.40	84.6	104.2	139.4	113.2	28.9	25.8
1	7	26.13	73.6	91.3	122.2	98.6	21.8	24.8
1	8	29.87	62.7	78.6	105.3	84.1	14.6	23.9
1	9	33.60	51.9	66.0	88.6	69.8	7.5	23.1
1	10	37.33	-41.1	-53.4	-71.9	-55.5	-0.2	22.8
1	11	41.07	-51.8	-65.8	-88.4	-69.6	-7.3	23.1
1	12	44.80	-62.6	-78.4	-105.1	-83.9	-14.4	23.9
1	13	48.53	-73.5	-91.2	-122.0	-98.4	-21.6	24.8
1	14	52.27	-84.8	-104.4	-139.6	-113.5	-29.2	25.8
1	15	56.00	-96.0	-117.5	-157.0	-128.3	-36.4	26.8
1	16	59.73	-107.2	-130.7	-174.5	-143.2	-43.5	27.7
1	17	63.47	-118.6	-144.1	-192.3	-158.3	-50.6	28.8
1	18	67.20	-130.3	-157.8	-210.4	-173.8	-58.0	30.0
1	19	70.93	-141.9	-171.4	-228.6	-189.2	-65.2	31.2
1	20	74.67	-154.4	-186.0	-247.9	-205.7	-73.5	32.4

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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	49.11	0.00	HL-93	105.41	HL-93	ST1	41.98	248.07
						ST2	41.98	205.91
						ST4	60.33	73.66
						SE1	49.11	154.52
						SE2	49.11	186.14
2	48.97	0.00	HL-93	105.41	HL-93	ST1	41.85	247.90
						ST2	41.85	205.73
						ST4	60.13	73.45
						SE1	48.97	154.38
						SE2	48.97	186.00

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 = 30.0 (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	3.73	-0.0565	-0.1739	0.0000	-0.0549	-0.2359	-0.0549	-0.2908
1	2	7.47	-0.1109	-0.3416	0.0000	-0.1078	-0.4634	-0.1078	-0.5712
1	3	11.20	-0.1616	-0.4975	0.0000	-0.1569	-0.6749	-0.1569	-0.8318
1	4	14.93	-0.2068	-0.6369	0.0000	-0.2006	-0.8641	-0.2006	-1.0647
1	5	18.67	-0.2466	-0.7593	0.0000	-0.2390	-1.0301	-0.2390	-1.2692
1	6	22.40	-0.2803	-0.8632	0.0000	-0.2716	-1.1711	-0.2716	-1.4427
1	7	26.13	-0.3073	-0.9463	0.0000	-0.2977	-1.2838	-0.2977	-1.5814
1	8	29.87	-0.3270	-1.0069	0.0000	-0.3167	-1.3660	-0.3167	-1.6826
1	9	33.60	-0.3389	-1.0438	0.0000	-0.3282	-1.4159	-0.3282	-1.7442
1	10	37.33	-0.3430	-1.0562	0.0000	-0.3321	-1.4327	-0.3321	-1.7648
1	11	41.07	-0.3389	-1.0438	0.0000	-0.3282	-1.4158	-0.3282	-1.7441
1	12	44.80	-0.3270	-1.0069	0.0000	-0.3167	-1.3658	-0.3167	-1.6825
1	13	48.53	-0.3073	-0.9463	0.0000	-0.2977	-1.2835	-0.2977	-1.5812
1	14	52.27	-0.2803	-0.8632	0.0000	-0.2716	-1.1707	-0.2716	-1.4424
1	15	56.00	-0.2466	-0.7593	0.0000	-0.2390	-1.0298	-0.2390	-1.2688
1	16	59.73	-0.2068	-0.6369	0.0000	-0.2006	-0.8637	-0.2006	-1.0644
1	17	63.47	-0.1616	-0.4975	0.0000	-0.1569	-0.6746	-0.1569	-0.8315
1	18	67.20	-0.1109	-0.3416	0.0000	-0.1078	-0.4632	-0.1078	-0.5710
1	19	70.93	-0.0565	-0.1739	0.0000	-0.0549	-0.2358	-0.0549	-0.2907
1	20	74.67	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 STEEL DEFL. (in)	CAMBER SIZE	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	3.7	-0.056	0 1/16	-0.179	0 3/16	-0.055	0 1/16	-0.291	0 5/16
1	2	7.5	-0.111	0 1/ 8	-0.352	0 3/ 8	-0.108	0 1/ 8	-0.571	0 5/ 8
1	3	11.2	-0.162	0 3/16	-0.513	0 9/16	-0.157	0 3/16	-0.832	0 7/ 8
1	4	14.9	-0.207	0 1/ 4	-0.657	0 11/16	-0.201	0 1/ 4	-1.065	1 1/ 8
1	5	18.7	-0.247	0 1/ 4	-0.784	0 13/16	-0.239	0 1/ 4	-1.269	1 5/16
1	6	22.4	-0.280	0 5/16	-0.891	0 15/16	-0.272	0 5/16	-1.443	1 1/ 2
1	7	26.1	-0.307	0 5/16	-0.976	1 0/16	-0.298	0 5/16	-1.581	1 5/ 8
1	8	29.9	-0.327	0 3/ 8	-1.039	1 1/16	-0.317	0 3/ 8	-1.683	1 11/16
1	9	33.6	-0.339	0 3/ 8	-1.077	1 1/ 8	-0.328	0 3/ 8	-1.744	1 3/ 4
1	10	37.3	-0.343	0 3/ 8	-1.090	1 1/ 8	-0.332	0 3/ 8	-1.765	1 13/16
1	11	41.1	-0.339	0 3/ 8	-1.077	1 1/ 8	-0.328	0 3/ 8	-1.744	1 3/ 4
1	12	44.8	-0.327	0 3/ 8	-1.039	1 1/16	-0.317	0 3/ 8	-1.682	1 11/16
1	13	48.5	-0.307	0 5/16	-0.976	1 0/16	-0.298	0 5/16	-1.581	1 5/ 8
1	14	52.3	-0.280	0 5/16	-0.890	0 15/16	-0.272	0 5/16	-1.442	1 1/ 2
1	15	56.0	-0.247	0 1/ 4	-0.783	0 13/16	-0.239	0 1/ 4	-1.269	1 5/16
1	16	59.7	-0.207	0 1/ 4	-0.657	0 11/16	-0.201	0 1/ 4	-1.064	1 1/ 8
1	17	63.5	-0.162	0 3/16	-0.513	0 9/16	-0.157	0 3/16	-0.832	0 7/ 8
1	18	67.2	-0.111	0 1/ 8	-0.352	0 3/ 8	-0.108	0 1/ 8	-0.571	0 5/ 8
1	19	70.9	-0.056	0 1/16	-0.179	0 3/16	-0.055	0 1/16	-0.291	0 5/16
1	20	74.7	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	37.33	2 0.400	-0.480 MAX	HL-93	1.12	0.00330
			0.030 MIN			
			-0.165 MAX	LANE		
			0.030 MIN	LANE		
		3 0.510	-0.612 MAX	HL-93	1.12	0.00330
			0.039 MIN			
			-0.210 MAX	LANE		
			0.039 MIN	LANE		
		2 0.400	-0.390 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	3.73	-0.54	0.54	-1.71	1.71	-2.24	2.24
1	2	7.47	-1.01	1.01	-3.22	3.22	-4.24	4.24
1	3	11.20	-1.44	1.44	-4.56	4.56	-6.00	6.00
1	4	14.93	-1.63	1.18	-5.18	3.74	-6.81	4.92
1	5	18.67	-1.91	1.38	-6.08	4.39	-7.99	5.77
1	6	22.40	-2.14	1.55	-6.80	4.91	-8.94	6.46
1	7	26.13	-2.32	1.67	-7.37	5.32	-9.69	7.00
1	8	29.87	-2.45	1.77	-7.78	5.62	-10.22	7.38
1	9	33.60	-2.52	1.82	-8.03	5.80	-10.55	7.62
1	10	37.33	-2.55	1.84	-8.10	5.85	-10.65	7.69
1	11	41.07	-2.52	1.82	-8.01	5.79	-10.54	7.61
1	12	44.80	-2.45	1.77	-7.77	5.61	-10.22	7.38
1	13	48.53	-2.32	1.67	-7.37	5.33	-9.69	7.00
1	14	52.27	-2.14	1.55	-6.80	4.91	-8.94	6.45
1	15	56.00	-1.91	1.38	-6.06	4.38	-7.97	5.76
1	16	59.73	-1.63	1.18	-5.17	3.73	-6.80	4.91
1	17	63.47	-1.44	1.44	-4.56	4.56	-6.00	6.00
1	18	67.20	-1.01	1.01	-3.22	3.22	-4.23	4.23
1	19	70.93	-0.54	0.54	-1.70	1.70	-2.23	2.23
1	20	74.67	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 = 30.0 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	3.73	-0.02	-0.36	0.71	-0.02	-2.60	2.95
1	2	7.47	-0.04	-0.68	1.34	-0.04	-4.91	5.58
1	3	11.20	-0.05	-0.96	1.90	-0.05	-6.96	7.90

1	4	14.93	-0.06	-1.15	1.57	-0.06	-7.96	6.49
1	5	18.67	-0.07	-1.34	1.84	-0.07	-9.34	7.61
1	6	22.40	-0.07	-1.50	2.06	-0.07	-10.45	8.52
1	7	26.13	-0.08	-1.63	2.23	-0.08	-11.32	9.23
1	8	29.87	-0.08	-1.72	2.35	-0.08	-11.94	9.73
1	9	33.60	-0.09	-1.77	2.42	-0.09	-12.32	10.04
1	10	37.33	-0.09	-1.79	2.45	-0.09	-12.44	10.14
1	11	41.07	-0.09	-1.77	2.42	-0.09	-12.31	10.03
1	12	44.80	-0.08	-1.72	2.35	-0.08	-11.93	9.73
1	13	48.53	-0.08	-1.63	2.23	-0.08	-11.32	9.23
1	14	52.27	-0.07	-1.50	2.06	-0.07	-10.44	8.51
1	15	56.00	-0.07	-1.34	1.84	-0.07	-9.31	7.60
1	16	59.73	-0.06	-1.15	1.57	-0.06	-7.95	6.48
1	17	63.47	-0.05	-0.96	1.90	-0.05	-6.95	7.90
1	18	67.20	-0.04	-0.68	1.34	-0.04	-4.91	5.57
1	19	70.93	-0.02	-0.36	0.71	-0.02	-2.59	2.94
1	20	74.67	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 = 10.0 (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	3.73	-0.13	-0.60	2.74	0.00	0.00	0.00
1	2	7.47	-0.24	-1.13	5.16	0.00	0.00	0.00
1	3	11.20	-0.34	-1.59	7.26	0.00	0.00	0.00
1	4	14.93	-0.36	-2.05	5.94	0.00	0.00	0.00
1	5	18.67	-0.42	-2.37	6.87	0.00	0.00	0.00
1	6	22.40	-0.46	-2.63	7.63	0.00	0.00	0.00
1	7	26.13	-0.50	-2.83	8.23	0.00	0.00	0.00
1	8	29.87	-0.52	-2.97	8.62	0.00	0.00	0.00
1	9	33.60	-0.53	-3.03	8.78	0.00	0.00	0.00
1	10	37.33	-0.54	-3.04	8.83	0.00	0.00	0.00
1	11	41.07	-0.53	-3.03	8.78	0.00	0.00	0.00
1	12	44.80	-0.52	-2.97	8.62	0.00	0.00	0.00
1	13	48.53	-0.50	-2.83	8.23	0.00	0.00	0.00
1	14	52.27	-0.46	-2.63	7.63	0.00	0.00	0.00
1	15	56.00	-0.42	-2.37	6.87	0.00	0.00	0.00
1	16	59.73	-0.36	-2.05	5.94	0.00	0.00	0.00
1	17	63.47	-0.34	-1.59	7.26	0.00	0.00	0.00
1	18	67.20	-0.24	-1.13	5.16	0.00	0.00	0.00
1	19	70.93	-0.13	-0.60	2.74	0.00	0.00	0.00
1	20	74.67	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 = 10.0 (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	3.73	0.2	0.0	0.2	C	HL	1.0	0.0	1.0	T HL
1	2	7.47	0.4	0.0	0.4	C	HL	1.8	0.0	1.8	T HL
1	3	11.20	0.5	0.0	0.5	C	HL	2.4	0.0	2.4	T HL
1	4	14.93	0.7	0.0	0.7	C	HL	2.0	0.0	2.0	T HL
1	5	18.67	0.8	0.0	0.8	C	HL	2.3	0.0	2.3	T HL
1	6	22.40	0.9	0.0	0.9	C	HL	2.5	0.0	2.5	T HL
1	7	26.13	0.9	0.0	0.9	C	HL	2.7	0.0	2.7	T HL
1	8	29.87	1.0	0.0	1.0	C	HL	2.8	0.0	2.8	T HL
1	9	33.60	0.9	0.0	0.9	C	HL	2.7	0.0	2.7	T HL
1	10	37.33	0.9	0.0	0.9	C	HL	2.7	0.0	2.7	T HL
1	11	41.07	0.9	0.0	0.9	C	HL	2.7	0.0	2.7	T HL
1	12	44.80	1.0	0.0	1.0	C	HL	2.8	0.0	2.8	T HL
1	13	48.53	0.9	0.0	0.9	C	HL	2.7	0.0	2.7	T HL
1	14	52.27	0.9	0.0	0.9	C	HL	2.5	0.0	2.5	T HL
1	15	56.00	0.8	0.0	0.8	C	HL	2.3	0.0	2.3	T HL
1	16	59.73	0.7	0.0	0.7	C	HL	2.0	0.0	2.0	T HL
1	17	63.47	0.5	0.0	0.5	C	HL	2.4	0.0	2.4	T HL
1	18	67.20	0.4	0.0	0.4	C	HL	1.8	0.0	1.8	T HL
1	19	70.93	0.2	0.0	0.2	C	HL	1.0	0.0	1.0	T HL
1	20	74.67	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.15	-3.20	5.69	-0.02	-2.60	2.95
1	2	7.47	32.	32.	-0.28	-6.04	10.74	-0.04	-4.91	5.58
1	3	11.20	32.	32.	-0.39	-8.55	15.16	-0.05	-6.96	7.90
1	4	14.93	32.	32.	-0.42	-10.00	12.43	-0.06	-7.96	6.49
1	5	18.67	32.	32.	-0.48	-11.70	14.48	-0.07	-9.34	7.61
1	6	22.40	32.	32.	-0.54	-13.08	16.15	-0.07	-10.45	8.52
1	7	26.13	32.	32.	-0.58	-14.15	17.45	-0.08	-11.32	9.23
1	8	29.87	32.	32.	-0.61	-14.91	18.36	-0.08	-11.94	9.73
1	9	33.60	32.	32.	-0.62	-15.35	18.83	-0.09	-12.32	10.04
1	10	37.33	32.	32.	-0.62	-15.48	18.97	-0.09	-12.44	10.14
1	11	41.07	32.	32.	-0.62	-15.33	18.82	-0.09	-12.31	10.03
1	12	44.80	32.	32.	-0.61	-14.91	18.35	-0.08	-11.93	9.73
1	13	48.53	32.	32.	-0.58	-14.15	17.45	-0.08	-11.32	9.23
1	14	52.27	32.	32.	-0.54	-13.07	16.15	-0.07	-10.44	8.51
1	15	56.00	32.	32.	-0.48	-11.68	14.47	-0.07	-9.31	7.60
1	16	59.73	32.	32.	-0.42	-9.99	12.42	-0.06	-7.95	6.48
1	17	63.47	32.	32.	-0.39	-8.55	15.16	-0.05	-6.95	7.90
1	18	67.20	32.	32.	-0.28	-6.04	10.74	-0.04	-4.91	5.57
1	19	70.93	32.	32.	-0.15	-3.19	5.68	-0.02	-2.59	2.94
1	20	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.18	-3.38	6.52	-0.02	-2.60	2.95
1	2	7.47	32.	32.	-0.35	-6.38	12.29	-0.04	-4.91	5.58
1	3	11.20	32.	32.	-0.49	-9.03	17.34	-0.05	-6.96	7.90
1	4	14.93	32.	32.	-0.52	-10.62	14.21	-0.06	-7.96	6.49
1	5	18.67	32.	32.	-0.61	-12.41	16.55	-0.07	-9.34	7.61
1	6	22.40	32.	32.	-0.68	-13.87	18.44	-0.07	-10.45	8.52
1	7	26.13	32.	32.	-0.73	-15.00	19.92	-0.08	-11.32	9.23
1	8	29.87	32.	32.	-0.76	-15.80	20.94	-0.08	-11.94	9.73
1	9	33.60	32.	32.	-0.78	-16.25	21.46	-0.09	-12.32	10.04
1	10	37.33	32.	32.	-0.78	-16.39	21.62	-0.09	-12.44	10.14
1	11	41.07	32.	32.	-0.78	-16.24	21.45	-0.09	-12.31	10.03
1	12	44.80	32.	32.	-0.76	-15.80	20.94	-0.08	-11.93	9.73
1	13	48.53	32.	32.	-0.73	-15.00	19.92	-0.08	-11.32	9.23
1	14	52.27	32.	32.	-0.68	-13.86	18.44	-0.07	-10.44	8.51
1	15	56.00	32.	32.	-0.61	-12.39	16.53	-0.07	-9.31	7.60
1	16	59.73	32.	32.	-0.52	-10.61	14.20	-0.06	-7.95	6.48
1	17	63.47	32.	32.	-0.49	-9.02	17.34	-0.05	-6.95	7.90
1	18	67.20	32.	32.	-0.35	-6.38	12.28	-0.04	-4.91	5.57
1	19	70.93	32.	32.	-0.18	-3.37	6.51	-0.02	-2.59	2.94
1	20	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.25	-4.36	8.60	-0.03	-3.31	3.80
1	2	7.47	32.	32.	-0.47	-8.23	16.23	-0.05	-6.25	7.19
1	3	11.20	32.	32.	-0.66	-11.64	22.90	-0.07	-8.86	10.19
1	4	14.93	32.	32.	-0.71	-13.72	18.76	-0.08	-10.14	8.37
1	5	18.67	32.	32.	-0.82	-16.03	21.84	-0.09	-11.89	9.81
1	6	22.40	32.	32.	-0.91	-17.91	24.34	-0.10	-13.31	10.98
1	7	26.13	32.	32.	-0.99	-19.37	26.29	-0.11	-14.41	11.90
1	8	29.87	32.	32.	-1.03	-20.40	27.64	-0.12	-15.21	12.55
1	9	33.60	32.	32.	-1.05	-20.98	28.32	-0.12	-15.69	12.95
1	10	37.33	32.	32.	-1.06	-21.16	28.54	-0.12	-15.84	13.08
1	11	41.07	32.	32.	-1.05	-20.97	28.31	-0.12	-15.67	12.94
1	12	44.80	32.	32.	-1.03	-20.40	27.64	-0.12	-15.20	12.55
1	13	48.53	32.	32.	-0.99	-19.37	26.30	-0.11	-14.42	11.90
1	14	52.27	32.	32.	-0.91	-17.90	24.34	-0.10	-13.30	10.98
1	15	56.00	32.	32.	-0.82	-16.01	21.82	-0.09	-11.86	9.80
1	16	59.73	32.	32.	-0.71	-13.70	18.75	-0.08	-10.12	8.36
1	17	63.47	32.	32.	-0.66	-11.64	22.89	-0.07	-8.85	10.18
1	18	67.20	32.	32.	-0.47	-8.22	16.22	-0.05	-6.24	7.19
1	19	70.93	32.	32.	-0.25	-4.35	8.59	-0.03	-3.30	3.79
1	20	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.24	-3.33	7.34	-0.01	-2.28	2.54
1	2	7.47	32.	32.	-0.45	-6.29	13.84	-0.03	-4.31	4.80
1	3	11.20	32.	32.	-0.63	-8.89	19.51	-0.04	-6.11	6.80
1	4	14.93	32.	32.	-0.67	-10.56	15.97	-0.04	-6.98	5.58
1	5	18.67	32.	32.	-0.78	-12.33	18.58	-0.05	-8.18	6.55
1	6	22.40	32.	32.	-0.86	-13.76	20.69	-0.05	-9.16	7.33
1	7	26.13	32.	32.	-0.93	-14.88	22.33	-0.06	-9.92	7.94
1	8	29.87	32.	32.	-0.98	-15.66	23.47	-0.06	-10.46	8.38
1	9	33.60	32.	32.	-1.00	-16.09	24.01	-0.06	-10.80	8.64
1	10	37.33	32.	32.	-1.00	-16.22	24.18	-0.06	-10.90	8.72
1	11	41.07	32.	32.	-1.00	-16.08	24.00	-0.06	-10.79	8.63
1	12	44.80	32.	32.	-0.98	-15.66	23.46	-0.06	-10.46	8.37
1	13	48.53	32.	32.	-0.93	-14.88	22.33	-0.06	-9.92	7.94
1	14	52.27	32.	32.	-0.86	-13.75	20.68	-0.05	-9.15	7.32
1	15	56.00	32.	32.	-0.78	-12.31	18.56	-0.05	-8.16	6.54
1	16	59.73	32.	32.	-0.67	-10.54	15.97	-0.04	-6.96	5.58
1	17	63.47	32.	32.	-0.63	-8.89	19.50	-0.04	-6.10	6.80
1	18	67.20	32.	32.	-0.45	-6.29	13.83	-0.03	-4.30	4.80
1	19	70.93	32.	32.	-0.24	-3.32	7.33	-0.01	-2.27	2.53
1	20	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.20	-4.12	7.51	-0.03	-3.31	3.80
1	2	7.47	32.	32.	-0.38	-7.78	14.16	-0.05	-6.25	7.19
1	3	11.20	32.	32.	-0.53	-11.00	19.99	-0.07	-8.86	10.19
1	4	14.93	32.	32.	-0.57	-12.90	16.38	-0.08	-10.14	8.37
1	5	18.67	32.	32.	-0.66	-15.09	19.09	-0.09	-11.89	9.81
1	6	22.40	32.	32.	-0.73	-16.86	21.29	-0.10	-13.31	10.98
1	7	26.13	32.	32.	-0.79	-18.24	23.00	-0.11	-14.41	11.90
1	8	29.87	32.	32.	-0.82	-19.22	24.19	-0.12	-15.21	12.55
1	9	33.60	32.	32.	-0.84	-19.77	24.81	-0.12	-15.69	12.95
1	10	37.33	32.	32.	-0.85	-19.95	25.00	-0.12	-15.84	13.08
1	11	41.07	32.	32.	-0.84	-19.76	24.80	-0.12	-15.67	12.94
1	12	44.80	32.	32.	-0.82	-19.21	24.19	-0.12	-15.20	12.55
1	13	48.53	32.	32.	-0.79	-18.24	23.01	-0.11	-14.42	11.90
1	14	52.27	32.	32.	-0.73	-16.85	21.28	-0.10	-13.30	10.98
1	15	56.00	32.	32.	-0.66	-15.06	19.07	-0.09	-11.86	9.80
1	16	59.73	32.	32.	-0.57	-12.88	16.37	-0.08	-10.12	8.36
1	17	63.47	32.	32.	-0.53	-11.00	19.99	-0.07	-8.85	10.18
1	18	67.20	32.	32.	-0.38	-7.77	14.16	-0.05	-6.24	7.19
1	19	70.93	32.	32.	-0.20	-4.11	7.50	-0.03	-3.30	3.79
1	20	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	3.73	32.	32.	-0.19	-3.09	6.24	-0.01	-2.28	2.54
1	2	7.47	32.	32.	-0.35	-5.84	11.77	-0.03	-4.31	4.80
1	3	11.20	32.	32.	-0.49	-8.26	16.60	-0.04	-6.11	6.80
1	4	14.93	32.	32.	-0.53	-9.74	13.60	-0.04	-6.98	5.58
1	5	18.67	32.	32.	-0.61	-11.38	15.83	-0.05	-8.18	6.55
1	6	22.40	32.	32.	-0.68	-12.71	17.63	-0.05	-9.16	7.33
1	7	26.13	32.	32.	-0.73	-13.74	19.04	-0.06	-9.92	7.94
1	8	29.87	32.	32.	-0.77	-14.47	20.02	-0.06	-10.46	8.38
1	9	33.60	32.	32.	-0.78	-14.88	20.50	-0.06	-10.80	8.64
1	10	37.33	32.	32.	-0.79	-15.01	20.65	-0.06	-10.90	8.72
1	11	41.07	32.	32.	-0.78	-14.87	20.49	-0.06	-10.79	8.63
1	12	44.80	32.	32.	-0.77	-14.47	20.01	-0.06	-10.46	8.37
1	13	48.53	32.	32.	-0.73	-13.75	19.04	-0.06	-9.92	7.94
1	14	52.27	32.	32.	-0.68	-12.70	17.63	-0.05	-9.15	7.32
1	15	56.00	32.	32.	-0.61	-11.36	15.81	-0.05	-8.16	6.54
1	16	59.73	32.	32.	-0.53	-9.73	13.59	-0.04	-6.96	5.58
1	17	63.47	32.	32.	-0.49	-8.25	16.60	-0.04	-6.10	6.80
1	18	67.20	32.	32.	-0.35	-5.83	11.77	-0.03	-4.30	4.80
1	19	70.93	32.	32.	-0.19	-3.08	6.23	-0.01	-2.27	2.53
1	20	74.67	32.	32.	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=30.0 MOM. (k-ft)	DL2, N=30.0 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	13.83	391.	-7.1	152.	-1.1	641.	-1.9	0.	0.0	-10.08
											1.87
	LB			7.1		2.2		8.5		0.0	17.84
											8.51
	RT			-6.4		-1.1		-1.9		0.0	-9.41
											1.93
	RB			4.6		1.5		5.6		0.0	11.69
											5.59
1	LT	60.83	391.	-6.4	152.	-1.1	641.	-1.9	0.	0.0	-9.40
											1.93
	LB			4.6		1.5		5.6		0.0	11.69
											5.59
	RT			-7.1		-1.1		-1.9		0.0	-10.07
											1.87
	RB			7.1		2.2		8.5		0.0	17.83
											8.51

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	13.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	0.0000
2	47.00	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	11.00	1.2500
3	13.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	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) = 74.7

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	13.83	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	0.03300	OK
2	47.00	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	0.03300	OK
3	13.83	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	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 [1]	Lo ng	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	33.98	0.764	44.5	150.0	0.00	113.19	0
1	1	3.73	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	2	7.47	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	3	11.20	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	4	14.93	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	5	18.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	6	22.40	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	7	26.13	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	8	29.87	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	9	33.60	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	10	37.33	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	11	41.07	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	12	44.80	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	13	48.53	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	14	52.27	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	15	56.00	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	16	59.73	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	17	63.47	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	18	67.20	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	19	70.93	0	1	33.98	0.764	44.5	150.0	0.00	113.19	0
1	20	74.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19	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	33.98	0.764	44.48	171.59	2
1	1	3.73	0	1	33.98	0.764	44.48	171.59	2
1	2	7.47	0	1	33.98	0.764	44.48	171.59	2
1	3	11.20	0	1	33.98	0.764	44.48	171.59	2
1	4	14.93	0	1	33.98	0.764	54.16	171.59	2
1	5	18.67	0	1	33.98	0.764	54.16	171.59	2
1	6	22.40	0	1	33.98	0.764	54.16	171.59	2
1	7	26.13	0	1	33.98	0.764	54.16	171.59	2
1	8	29.87	0	1	33.98	0.764	54.16	171.59	2
1	9	33.60	0	1	33.98	0.764	54.16	171.59	2
1	10	37.33	0	1	33.98	0.764	54.16	171.59	2
1	11	41.07	0	1	33.98	0.764	54.16	171.59	2
1	12	44.80	0	1	33.98	0.764	54.16	171.59	2
1	13	48.53	0	1	33.98	0.764	54.16	171.59	2
1	14	52.27	0	1	33.98	0.764	54.16	171.59	2
1	15	56.00	0	1	33.98	0.764	54.16	171.59	2
1	16	59.73	0	1	33.98	0.764	54.16	171.59	2
1	17	63.47	0	1	33.98	0.764	44.48	171.59	2
1	18	67.20	0	1	33.98	0.764	44.48	171.59	2
1	19	70.93	0	1	33.98	0.764	44.48	171.59	2
1	20	74.67	0	1	33.98	0.764	44.48	171.59	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	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 1	3.73	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 2	7.47	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 3	11.20	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 4	14.93	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 5	18.67	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 6	22.40	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 7	26.13	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 8	29.87	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 9	33.60	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 10	37.33	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 11	41.07	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 12	44.80	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 13	48.53	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 14	52.27	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 15	56.00	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 16	59.73	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 17	63.47	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 18	67.20	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 19	70.93	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	1.00 0
1 20	74.67	4.81	12.	12.1	5.7	1.259	0.840	
		4.81	12.	12.1	5.7	1.259	0.840	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

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	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	1		3.73	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	2		7.47	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	3		11.20	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	4		14.93	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	5		18.67	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	6		22.40	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	7		26.13	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	8		29.87	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	9		33.60	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	10		37.33	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	11		41.07	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	12		44.80	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	13		48.53	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	14		52.27	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	15		56.00	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	16		59.73	1	4.81	11.44	20.15	0	0.00	7.55	28.36	0	0
1	17		63.47	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	18		67.20	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	19		70.93	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	20		74.67	1	4.81	11.44	20.15	0	0.00	7.73	29.04	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	32.00	32.00	32.00
1	1		3.73	1	1.000	1.000	1.00	32.00	32.00	32.00
1	2		7.47	1	1.000	1.000	1.00	32.00	32.00	32.00
1	3		11.20	1	1.000	1.000	1.00	32.00	32.00	32.00
1	4		14.93	1	1.000	1.000	1.00	32.00	32.00	32.00
1	5		18.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	6		22.40	1	1.000	1.000	1.00	32.00	32.00	32.00
1	7		26.13	1	1.000	1.000	1.00	32.00	32.00	32.00
1	8		29.87	1	1.000	1.000	1.00	32.00	32.00	32.00
1	9		33.60	1	1.000	1.000	1.00	32.00	32.00	32.00
1	10		37.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	11		41.07	1	1.000	1.000	1.00	32.00	32.00	32.00
1	12		44.80	1	1.000	1.000	1.00	32.00	32.00	32.00
1	13		48.53	1	1.000	1.000	1.00	32.00	32.00	32.00
1	14		52.27	1	1.000	1.000	1.00	32.00	32.00	32.00
1	15		56.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	16		59.73	1	1.000	1.000	1.00	32.00	32.00	32.00
1	17		63.47	1	1.000	1.000	1.00	32.00	32.00	32.00
1	18		67.20	1	1.000	1.000	1.00	32.00	32.00	32.00
1	19		70.93	1	1.000	1.000	1.00	32.00	32.00	32.00
1	20		74.67	1	1.000	1.000	1.00	32.00	32.00	32.00

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	7.97	18.42
1	1		3.73	1	7.97	18.42
1	2		7.47	1	7.97	18.42
1	3		11.20	1	7.97	18.42
1	4		14.93	1	8.54	18.95
1	5		18.67	1	8.54	18.95
1	6		22.40	1	8.54	18.95
1	7		26.13	1	8.54	18.95
1	8		29.87	1	8.54	18.95
1	9		33.60	1	8.54	18.95

1	10	37.33	1	8.54	18.95
1	11	41.07	1	8.54	18.95
1	12	44.80	1	8.54	18.95
1	13	48.53	1	8.54	18.95
1	14	52.27	1	8.54	18.95
1	15	56.00	1	8.54	18.95
1	16	59.73	1	8.54	18.95
1	17	63.47	1	7.97	18.42
1	18	67.20	1	7.97	18.42
1	19	70.93	1	7.97	18.42
1	20	74.67	1	7.97	18.42

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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.	Comp.
1	0	0.00	1	0	0	0	0		0	0
1	1	3.73	1	0	0	0	0		0	0
1	2	7.47	1	0	0	0	0		0	0
1	3	11.20	1	0	0	0	0		0	0
1	4	14.93	1	0	0	0	0		0	0
1	5	18.67	1	0	0	0	0		0	0
1	6	22.40	1	0	0	0	0		0	0
1	7	26.13	1	0	0	0	0		0	0
1	8	29.87	1	0	0	0	0		0	0
1	9	33.60	1	0	0	0	0		0	0
1	10	37.33	1	0	0	0	0		0	0
1	11	41.07	1	0	0	0	0		0	0
1	12	44.80	1	0	0	0	0		0	0
1	13	48.53	1	0	0	0	0		0	0
1	14	52.27	1	0	0	0	0		0	0
1	15	56.00	1	0	0	0	0		0	0
1	16	59.73	1	0	0	0	0		0	0
1	17	63.47	1	0	0	0	0		0	0
1	18	67.20	1	0	0	0	0		0	0
1	19	70.93	1	0	0	0	0		0	0
1	20	74.67	1	0	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	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	0
1	1		3.73	0.0	19.2	3.4	32.0	3.4	32.0	3.4	32.0	0
				0.0	-	3.4	-	3.4	32.0	-	-	0
1	2		7.47	0.0	19.2	6.4	32.0	6.4	32.0	6.4	32.0	0
				0.0	-	6.4	-	6.4	32.0	-	-	0
1	3		11.20	0.0	19.2	9.0	32.0	9.0	32.0	9.0	32.0	0
				0.0	-	9.0	-	9.0	32.0	-	-	0
1	4		14.93	0.0	19.2	10.2	32.0	10.2	32.0	10.2	32.0	0
				0.0	-	7.4	-	7.4	32.0	-	-	0
1	5		18.67	0.0	19.2	12.0	32.0	12.0	32.0	12.0	32.0	0
				0.0	-	8.7	-	8.7	32.0	-	-	0
1	6		22.40	0.0	19.2	13.4	32.0	13.4	32.0	13.4	32.0	0
				0.0	-	9.7	-	9.7	32.0	-	-	0
1	7		26.13	0.0	19.2	14.5	32.0	14.5	32.0	14.5	32.0	0
				0.0	-	10.5	-	10.5	32.0	-	-	0
1	8		29.87	0.0	19.2	15.3	32.0	15.3	32.0	15.3	32.0	0
				0.0	-	11.1	-	11.1	32.0	-	-	0
1	9		33.60	0.0	19.2	15.8	32.0	15.8	32.0	15.8	32.0	0
				0.0	-	11.4	-	11.4	32.0	-	-	0
1	10		37.33	0.0	19.2	16.0	32.0	16.0	32.0	16.0	32.0	0
				0.0	-	11.5	-	11.5	32.0	-	-	0
1	11		41.07	0.0	19.2	15.8	32.0	15.8	32.0	15.8	32.0	0
				0.0	-	11.4	-	11.4	32.0	-	-	0
1	12		44.80	0.0	19.2	15.3	32.0	15.3	32.0	15.3	32.0	0
				0.0	-	11.1	-	11.1	32.0	-	-	0
1	13		48.53	0.0	19.2	14.5	32.0	14.5	32.0	14.5	32.0	0
				0.0	-	10.5	-	10.5	32.0	-	-	0
1	14		52.27	0.0	19.2	13.4	32.0	13.4	32.0	13.4	32.0	0
				0.0	-	9.7	-	9.7	32.0	-	-	0
1	15		56.00	0.0	19.2	12.0	32.0	12.0	32.0	12.0	32.0	0
				0.0	-	8.6	-	8.6	32.0	-	-	0
1	16		59.73	0.0	19.2	10.2	32.0	10.2	32.0	10.2	32.0	0
				0.0	-	7.4	-	7.4	32.0	-	-	0
1	17		63.47	0.0	19.2	9.0	32.0	9.0	32.0	9.0	32.0	0
				0.0	-	9.0	-	9.0	32.0	-	-	0
1	18		67.20	0.0	19.2	6.3	32.0	6.3	32.0	6.3	32.0	0
				0.0	-	6.3	-	6.3	32.0	-	-	0
1	19		70.93	0.0	19.2	3.3	32.0	3.3	32.0	3.3	32.0	0
				0.0	-	3.3	-	3.3	32.0	-	-	0
1	20		74.67	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	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	3.73	0.000	0.105	0.105	0.105	0.105	2
					0.105		0.105	3
1	2	7.47	0.000	0.199	0.199	0.199	0.199	2
					0.199		0.199	3
1	3	11.20	0.000	0.281	0.281	0.281	0.281	2
					0.281		0.281	3
1	4	14.93	0.000	0.319	0.319	0.319	0.319	2
					0.231		0.231	3
1	5	18.67	0.000	0.375	0.375	0.375	0.375	2
					0.271		0.271	3
1	6	22.40	0.000	0.419	0.419	0.419	0.419	2
					0.303		0.303	3
1	7	26.13	0.000	0.454	0.454	0.454	0.454	2
					0.328		0.328	3
1	8	29.87	0.000	0.479	0.479	0.479	0.479	2
					0.346		0.346	3
1	9	33.60	0.000	0.494	0.494	0.494	0.494	2
					0.357		0.357	3
1	10	37.33	0.000	0.499	0.499	0.499	0.499	2
					0.360		0.360	3
1	11	41.07	0.000	0.494	0.494	0.494	0.494	2
					0.357		0.357	3
1	12	44.80	0.000	0.479	0.479	0.479	0.479	2
					0.346		0.346	3
1	13	48.53	0.000	0.454	0.454	0.454	0.454	2
					0.328		0.328	3
1	14	52.27	0.000	0.419	0.419	0.419	0.419	2
					0.303		0.303	3
1	15	56.00	0.000	0.374	0.374	0.374	0.374	2
					0.270		0.270	3
1	16	59.73	0.000	0.319	0.319	0.319	0.319	2
					0.230		0.230	3
1	17	63.47	0.000	0.281	0.281	0.281	0.281	2
					0.281		0.281	3
1	18	67.20	0.000	0.198	0.198	0.198	0.198	2
					0.198		0.198	3
1	19	70.93	0.000	0.105	0.105	0.105	0.105	2
					0.105		0.105	3
1	20	74.67	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	2985.7	-	-	0.0	19.2	-	-	0--0
1	1		3.73	0	584.3	2985.7	-	-	0.0	19.2	-	-	0--0
1	2		7.47	0	1101.8	2985.7	-	-	0.0	19.2	-	-	0--0
1	3		11.20	0	1553.7	2985.7	-	-	0.0	19.2	-	-	0--0
1	4		14.93	0	1940.0	4215.3	-	-	0.0	19.2	-	-	0--0
1	5		18.67	0	2257.3	4215.3	-	-	0.0	19.2	-	-	0--0
1	6		22.40	0	2514.4	4215.3	-	-	0.0	19.2	-	-	0--0
1	7		26.13	0	2714.9	4215.3	-	-	0.0	19.2	-	-	0--0
1	8		29.87	0	2853.4	4215.3	-	-	0.0	19.2	-	-	0--0
1	9		33.60	0	2920.9	4215.3	-	-	0.0	19.2	-	-	0--0
1	10		37.33	0	2942.6	4215.3	-	-	0.0	19.2	-	-	0--0
1	11		41.07	0	2920.0	4215.3	-	-	0.0	19.2	-	-	0--0
1	12		44.80	0	2853.0	4215.3	-	-	0.0	19.2	-	-	0--0
1	13		48.53	0	2715.2	4215.3	-	-	0.0	19.2	-	-	0--0
1	14		52.27	0	2513.8	4215.3	-	-	0.0	19.2	-	-	0--0
1	15		56.00	0	2255.7	4215.3	-	-	0.0	19.2	-	-	0--0
1	16		59.73	0	1939.1	4215.3	-	-	0.0	19.2	-	-	0--0
1	17		63.47	0	1553.4	2985.7	-	-	0.0	19.2	-	-	0--0
1	18		67.20	0	1101.3	2985.7	-	-	0.0	19.2	-	-	0--0
1	19		70.93	0	583.6	2985.7	-	-	0.0	19.2	-	-	0--0
1	20		74.67	0	0.0	2985.7	-	-	0.0	19.2	-	-	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	3.73	0	0.196			0.196		1
1	2	7.47	0	0.369			0.369		1
1	3	11.20	0	0.520			0.520		1
1	4	14.93	0	0.460			0.460		1
1	5	18.67	0	0.535			0.535		1
1	6	22.40	0	0.597			0.597		1
1	7	26.13	0	0.644			0.644		1
1	8	29.87	0	0.677			0.677		1
1	9	33.60	0	0.693			0.693		1
1	10	37.33	0	0.698			0.698		1
1	11	41.07	0	0.693			0.693		1
1	12	44.80	0	0.677			0.677		1
1	13	48.53	0	0.644			0.644		1
1	14	52.27	0	0.596			0.596		1
1	15	56.00	0	0.535			0.535		1
1	16	59.73	0	0.460			0.460		1
1	17	63.47	0	0.520			0.520		1
1	18	67.20	0	0.369			0.369		1
1	19	70.93	0	0.195			0.195		1
1	20	74.67	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	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	1	3.73	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	2	7.47	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	3	11.20	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	4	14.93	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	5	18.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	6	22.40	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	7	26.13	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	8	29.87	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	9	33.60	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	10	37.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9

1	11	41.07	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	12	44.80	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	13	48.53	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	14	52.27	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	15	56.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	16	59.73	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	17	63.47	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	18	67.20	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	19	70.93	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	20	74.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9

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 L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----		LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA-CITY, [2]	CATEGORY	OVERALL WEB
					STRENGTH CATEGORY; SEE NOTE				
1	0	0.00	0		248.1	<	481.9	0	0
1	1	3.73	0		228.5	<	481.9	0	0
1	2	7.47	0		210.4	<	481.9	0	0
1	3	11.20	0		192.4	<	481.9	0	0
1	4	14.93	0		174.7	<	481.9	0	0

1	5	18.67	0	156.7	<	481.9	0	0
1	6	22.40	0	139.4	<	481.9	0	0
1	7	26.13	0	122.2	<	481.9	0	0
1	8	29.87	0	105.3	<	481.9	0	0
1	9	33.60	0	88.6	<	481.9	0	0
1	10	37.33	0	71.9	<	481.9	0	0
1	11	41.07	0	88.4	<	481.9	0	0
1	12	44.80	0	105.1	<	481.9	0	0
1	13	48.53	0	122.0	<	481.9	0	0
1	14	52.27	0	139.6	<	481.9	0	0
1	15	56.00	0	157.0	<	481.9	0	0
1	16	59.73	0	174.5	<	481.9	0	0
1	17	63.47	0	192.3	<	481.9	0	0
1	18	67.20	0	210.4	<	481.9	0	0
1	19	70.93	0	228.6	<	481.9	0	0
1	20	74.67	0	247.9	<	481.9	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

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TABLE 1.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D L (ft)	FROM SUPT	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	32.	248.07	481.86	0	
1	1		3.73	32.	228.54	481.86	0	
1	2		7.47	32.	210.38	481.86	0	
1	3		11.20	32.	192.42	481.86	0	
1	4		14.93	32.	174.68	481.86	0	
1	5		18.67	32.	156.69	481.86	0	
1	6		22.40	32.	139.35	481.86	0	
1	7		26.13	32.	122.22	481.86	0	
1	8		29.87	32.	105.29	481.86	0	
1	9		33.60	32.	88.56	481.86	0	
1	10		37.33	32.	71.87	481.86	0	
1	11		41.07	32.	88.39	481.86	0	
1	12		44.80	32.	105.11	481.86	0	
1	13		48.53	32.	122.04	481.86	0	
1	14		52.27	32.	139.62	481.86	0	
1	15		56.00	32.	156.96	481.86	0	
1	16		59.73	32.	174.50	481.86	0	
1	17		63.47	32.	192.25	481.86	0	
1	18		67.20	32.	210.43	481.86	0	
1	19		70.93	32.	228.58	481.86	0	
1	20		74.67	32.	247.90	481.86	0	

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L (ft)	FROM SUPT	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	-	30.4	0.0	30.4	-	-	00-
1	1		3.73	3.4	-	30.4	6.5	30.4	-	-	00-
1	2		7.47	6.4	-	30.4	12.3	30.4	-	-	00-
1	3		11.20	9.0	-	30.4	17.3	30.4	-	-	00-
1	4		14.93	10.6	-	30.4	14.2	30.4	-	-	00-
1	5		18.67	12.4	-	30.4	16.5	30.4	-	-	00-
1	6		22.40	13.9	-	30.4	18.4	30.4	-	-	00-
1	7		26.13	15.0	-	30.4	19.9	30.4	-	-	00-
1	8		29.87	15.8	-	30.4	20.9	30.4	-	-	00-
1	9		33.60	16.3	-	30.4	21.5	30.4	-	-	00-
1	10		37.33	16.4	-	30.4	21.6	30.4	-	-	00-
1	11		41.07	16.2	-	30.4	21.5	30.4	-	-	00-
1	12		44.80	15.8	-	30.4	20.9	30.4	-	-	00-
1	13		48.53	15.0	-	30.4	19.9	30.4	-	-	00-
1	14		52.27	13.9	-	30.4	18.4	30.4	-	-	00-
1	15		56.00	12.4	-	30.4	16.5	30.4	-	-	00-
1	16		59.73	10.6	-	30.4	14.2	30.4	-	-	00-
1	17		63.47	9.0	-	30.4	17.3	30.4	-	-	00-
1	18		67.20	6.4	-	30.4	12.3	30.4	-	-	00-
1	19		70.93	3.4	-	30.4	6.5	30.4	-	-	00-
1	20		74.67	0.0	-	30.4	0.0	30.4	-	-	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 = II -- Urban Interstate

SP NO	IN NO	D L (ft)	FROM SUPT	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	3.73		TR	-0.3	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	2	7.47		TR	-0.6	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	3	11.20		TR	-0.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1		13.83	SCG						
1	4	14.93		TR	-1.0	A B B^C^C D E E^	TR	3.0	A B B^C^C D E
1	5	18.67		TR	-1.2	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	6	22.40		TR	-1.3	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	7	26.13		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	8	29.87		TR	-1.4	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	9	33.60		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	10	37.33		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	11	41.07		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	12	44.80		TR	-1.4	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	13	48.53		TR	-1.4	A B B^C^C D E E^	TR	4.0	A B B^C^C D E
1	14	52.27		TR	-1.3	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	15	56.00		TR	-1.2	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	16	59.73		TR	-1.0	A B B^C^C D E E^	TR	3.0	A B B^C^C D E
1		60.83	SCG						
1	17	63.47		TR	-0.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	18	67.20		TR	-0.6	A B B^C^C D E E^	TR	2.6	A B B^C^C D E
1	19	70.93		TR	-0.3	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	20	74.67		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 = II -- Urban 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		3.73	TR	-0.2	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	2		7.47	TR	-0.3	A B B^C^C D E E^	TR	1.3	A B B^C^C D E E^
1	3		11.20	TR	-0.4	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1			13.83	SCG					
1	4		14.93	TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	5		18.67	TR	-0.6	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	6		22.40	TR	-0.6	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	7		26.13	TR	-0.7	A B B^C^C D E E^	TR	2.0	A B B^C^C D E
1	8		29.87	TR	-0.7	A B B^C^C D E E^	TR	2.1	A B B^C^C D E
1	9		33.60	TR	-0.7	A B B^C^C D E E^	TR	2.0	A B B^C^C D E
1	10		37.33	TR	-0.7	A B B^C^C D E E^	TR	2.0	A B B^C^C D E
1	11		41.07	TR	-0.7	A B B^C^C D E E^	TR	2.0	A B B^C^C D E
1	12		44.80	TR	-0.7	A B B^C^C D E E^	TR	2.1	A B B^C^C D E
1	13		48.53	TR	-0.7	A B B^C^C D E E^	TR	2.0	A B B^C^C D E
1	14		52.27	TR	-0.6	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	15		56.00	TR	-0.6	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1	16		59.73	TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1			60.83	SCG					
1	17		63.47	TR	-0.4	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	18		67.20	TR	-0.3	A B B^C^C D E E^	TR	1.3	A B B^C^C D E E^
1	19		70.93	TR	-0.2	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	20		74.67	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= 2038 & No. of Cycles= 55790250
* 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	64.9	.24957E-01	1.6	TR	3.09	3	5.73
1	1	3.73	62.5	.24957E-01	1.6	TR	3.09	3	5.95
1	2	7.47	60.1	.24957E-01	1.5	TR	3.09	3	6.19
1	3	11.20	57.7	.24957E-01	1.4	TR	3.09	3	6.45
1	4	14.93	55.4	.22794E-01	1.3	TR	3.09	3	7.34
1	5	18.67	53.5	.22794E-01	1.2	TR	3.09	3	7.61
1	6	22.40	51.6	.22794E-01	1.2	TR	3.09	3	7.89
1	7	26.13	49.5	.22794E-01	1.1	TR	3.09	3	8.22
1	8	29.87	47.8	.22794E-01	1.1	TR	3.09	3	8.52
1	9	33.60	46.2	.22794E-01	1.1	TR	3.09	3	8.82
1	10	37.33	45.6	.22794E-01	1.0	TR	3.09	3	8.93
1	11	41.07	46.2	.22794E-01	1.1	TR	3.09	3	8.82
1	12	44.80	47.8	.22794E-01	1.1	TR	3.09	3	8.52
1	13	48.53	49.5	.22794E-01	1.1	TR	3.09	3	8.22
1	14	52.27	51.6	.22794E-01	1.2	TR	3.09	3	7.89
1	15	56.00	53.5	.22794E-01	1.2	TR	3.09	3	7.61
1	16	59.73	55.4	.22794E-01	1.3	TR	3.09	3	7.34
1	17	63.47	57.7	.24957E-01	1.4	TR	3.09	3	6.45
1	18	67.20	60.1	.24957E-01	1.5	TR	3.09	3	6.19
1	19	70.93	62.5	.24957E-01	1.6	TR	3.09	3	5.95
1	20	74.67	64.9	.24957E-01	1.6	TR	3.09	3	5.73

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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= 2038
*** 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]	AASHTO LRFD Eqn. 6.10.10.2-2 Vsr [3]	[4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	32.4	.24957E-01	0.8	TR	0.76	3	2.80
1	1	3.73	31.2	.24957E-01	0.8	TR	0.76	3	2.91
1	2	7.47	30.0	.24957E-01	0.7	TR	0.76	3	3.03
1	3	11.20	28.8	.24957E-01	0.7	TR	0.76	3	3.15
1	4	14.93	27.7	.22794E-01	0.6	TR	0.76	3	3.59
1	5	18.67	26.8	.22794E-01	0.6	TR	0.76	3	3.72
1	6	22.40	25.8	.22794E-01	0.6	TR	0.76	3	3.86
1	7	26.13	24.8	.22794E-01	0.6	TR	0.76	3	4.02
1	8	29.87	23.9	.22794E-01	0.5	TR	0.76	3	4.16
1	9	33.60	23.1	.22794E-01	0.5	TR	0.76	3	4.31
1	10	37.33	22.8	.22794E-01	0.5	TR	0.76	3	4.36
1	11	41.07	23.1	.22794E-01	0.5	TR	0.76	3	4.31
1	12	44.80	23.9	.22794E-01	0.5	TR	0.76	3	4.16
1	13	48.53	24.8	.22794E-01	0.6	TR	0.76	3	4.02
1	14	52.27	25.8	.22794E-01	0.6	TR	0.76	3	3.86
1	15	56.00	26.8	.22794E-01	0.6	TR	0.76	3	3.72
1	16	59.73	27.7	.22794E-01	0.6	TR	0.76	3	3.59
1	17	63.47	28.8	.24957E-01	0.7	TR	0.76	3	3.15
1	18	67.20	30.0	.24957E-01	0.7	TR	0.76	3	3.03
1	19	70.93	31.2	.24957E-01	0.8	TR	0.76	3	2.91
1	20	74.67	32.4	.24957E-01	0.8	TR	0.76	3	2.80

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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.6.10.10.2-2 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= 2038 & No. of Cycles= 55790250

*** 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 FROM L SUPT (ft)	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k) FATIG. CRI.	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	SEE NOTE N2	STATUS BLANK=OK CHECK=**
1 10	37.33	1	2245.8	1338.7		270	>	80		
1 20	74.67	1	2245.8	1338.7		270	>	80		

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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	37.333	5.000
1	37.333	74.667	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	74.7	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) or STRESS	MOMENT CAP. (k-ft)	UNFACTORED DEAD LOAD MOMENT D1; k-ft	UNFACTORED L+I MAX. MOMENT D2 ksi POS.	UNFACTORED L+I MAX. MOMENT NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	2985.7	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1		3.73	2985.7	123.9	48.1	206.5	9.91	HL-93	7.64	0
1	2		7.47	2985.7	234.1	91.1	388.8	4.89	HL-93	3.77	0
1	3		11.20	2985.7	331.6	129.0	546.7	3.24	HL-93	2.50	0
1	4		14.93	4215.3	416.4	161.9	680.4	3.77	HL-93	2.91	0
1	5		18.67	4215.3	488.4	189.7	787.7	3.14	HL-93	2.42	0
1	6		22.40	4215.3	546.5	212.5	874.8	2.74	HL-93	2.11	0
1	7		26.13	4215.3	591.9	230.2	942.6	2.48	HL-93	1.91	0
1	8		29.87	4215.3	624.6	242.9	988.2	2.32	HL-93	1.79	0
1	9		33.60	4215.3	644.5	250.4	1006.4	2.25	HL-93	1.73	0
1	10		37.33	4215.3	650.5	253.0	1012.5	2.23	HL-93	1.72	0
1	11		41.07	4215.3	643.7	250.4	1006.4	2.25	HL-93	1.74	0
1	12		44.80	4215.3	624.3	242.9	988.2	2.32	HL-93	1.79	0
1	13		48.53	4215.3	592.1	230.2	942.6	2.48	HL-93	1.91	0
1	14		52.27	4215.3	546.0	212.5	874.8	2.74	HL-93	2.11	0
1	15		56.00	4215.3	487.2	189.7	787.7	3.14	HL-93	2.42	0
1	16		59.73	4215.3	415.6	161.9	680.4	3.77	HL-93	2.91	0
1	17		63.47	2985.7	331.3	129.0	546.7	3.24	HL-93	2.50	0
1	18		67.20	2985.7	233.7	91.1	388.8	4.89	HL-93	3.77	0
1	19		70.93	2985.7	123.4	48.1	206.5	9.91	HL-93	7.65	0
1	20		74.67	2985.7	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	UNFACTORED L+I STRESS MAX. NEG. STEEL TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	30.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	3.73	30.4	-2.60 2.95	-0.60 2.74	0.00 0.00	9.99	HL-93	7.68
1	2	7.47	30.4	-4.91 5.58	-1.13 5.16	0.00 0.00	4.81	HL-93	3.70
1	3	11.20	30.4	-6.96 7.90	-1.59 7.26	0.00 0.00	3.10	HL-93	2.38
1	4	14.93	30.4	-7.96 6.49	-2.05 5.94	0.00 0.00	4.03	HL-93	3.10
1	5	18.67	30.4	-9.34 7.61	-2.37 6.87	0.00 0.00	3.32	HL-93	2.55
1	6	22.40	30.4	-10.45 8.52	-2.63 7.63	0.00 0.00	2.87	HL-93	2.21
1	7	26.13	30.4	-11.32 9.23	-2.83 8.23	0.00 0.00	2.57	HL-93	1.98
1	8	29.87	30.4	-11.94 9.73	-2.97 8.62	0.00 0.00	2.40	HL-93	1.84
1	9	33.60	30.4	-12.32 10.04	-3.03 8.78	0.00 0.00	2.32	HL-93	1.78
1	10	37.33	30.4	-12.44 10.14	-3.04 8.83	0.00 0.00	2.29	HL-93	1.76
1	11	41.07	30.4	-12.31 10.03	-3.03 8.78	0.00 0.00	2.32	HL-93	1.78
1	12	44.80	30.4	-11.93 9.73	-2.97 8.62	0.00 0.00	2.40	HL-93	1.84
1	13	48.53	30.4	-11.32 9.23	-2.83 8.23	0.00 0.00	2.57	HL-93	1.98
1	14	52.27	30.4	-10.44 8.51	-2.63 7.63	0.00 0.00	2.87	HL-93	2.21
1	15	56.00	30.4	-9.31 7.60	-2.37 6.87	0.00 0.00	3.32	HL-93	2.55
1	16	59.73	30.4	-7.95 6.48	-2.05 5.94	0.00 0.00	4.03	HL-93	3.10
1	17	63.47	30.4	-6.95 7.90	-1.59 7.26	0.00 0.00	3.10	HL-93	2.38
1	18	67.20	30.4	-4.91 5.57	-1.13 5.16	0.00 0.00	4.81	HL-93	3.70
1	19	70.93	30.4	-2.59 2.94	-0.60 2.74	0.00 0.00	9.99	HL-93	7.68
1	20	74.67	30.4	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	74.67	37.33	STRENGTH L.S.	2.23	1.72	HL-93
1	74.67	37.33	SERVICEBILITY	2.29	1.76	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.227
Inventory= 1.718

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	74.67	29.87	STRENGTH L.S.	2.96	1043.88
1	74.67	29.87	SERVICEBILITY	3.00	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 2.961

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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	OPERAT. RATING MAX. SHEAR POS.	OPERAT. RATING MAX. SHEAR NEG.	FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	481.9	35.6	13.6	105.4	0.0	2.94	HL-93	2.27
1	1	3.73	481.9	31.2	12.2	98.4	-2.2	3.20	HL-93	2.47
1	2	7.47	481.9	27.8	10.8	91.6	-5.0	3.49	HL-93	2.69

1	3	11.20	481.9	24.4	9.5	84.9	-8.5	3.82	HL-93	2.95
1	4	14.93	481.9	21.0	8.1	78.2	-12.2	4.20	HL-93	3.24
1	5	18.67	481.9	17.3	6.8	71.7	-16.0	4.65	HL-93	3.59
1	6	22.40	481.9	13.9	5.4	65.4	-19.9	5.18	HL-93	3.99
1	7	26.13	481.9	10.5	4.1	59.1	-24.2	5.80	HL-93	4.48
1	8	29.87	481.9	7.0	2.7	52.9	-29.5	6.57	HL-93	5.06
1	9	33.60	481.9	3.6	1.4	46.9	-35.2	7.51	HL-93	5.79
1	10	37.33	481.9	-0.1	0.0	41.0	-41.0	8.71	HL-93	6.72
1	11	41.07	481.9	-3.5	-1.4	35.2	-46.9	7.51	HL-93	5.79
1	12	44.80	481.9	-6.9	-2.7	29.5	-52.9	6.57	HL-93	5.07
1	13	48.53	481.9	-10.3	-4.1	24.2	-59.1	5.81	HL-93	4.48
1	14	52.27	481.9	-14.1	-5.4	19.9	-65.4	5.18	HL-93	3.99
1	15	56.00	481.9	-17.5	-6.8	16.0	-71.7	4.65	HL-93	3.59
1	16	59.73	481.9	-20.9	-8.1	12.2	-78.2	4.21	HL-93	3.24
1	17	63.47	481.9	-24.3	-9.5	8.5	-84.9	3.82	HL-93	2.95
1	18	67.20	481.9	-27.9	-10.8	5.0	-91.6	3.49	HL-93	2.69
1	19	70.93	481.9	-31.3	-12.2	2.2	-98.4	3.20	HL-93	2.47
1	20	74.67	481.9	-35.4	-13.6	0.0	-105.4	2.94	HL-93	2.27

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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	74.67	0.00	STRENGTH L.S.	2.94	2.27	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.939
Inventory= 2.267

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	74.67	0.00	STRENGTH L.S.	3.43	121.85

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 3.432

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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	37.33	2	0.400	-0.48	HL-93	2.33
		3	0.510	-0.61		1.83

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. 5800 Span 2 Interior Beam	7/17/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 2(5)	IAS/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 DESIGN TYPE 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

TABLE 0.0.3.1 STRUCTURAL DETAILS
****

BEAM NUMBER	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
5	1	31.33	2.50	1.83	1.57	22.60	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
91.42	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
7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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	36	280			0.00	0.0000	0.00	0.0000		
2	W	36	280			0.00	0.0000	15.00	1.5000		

NOTE: [1] maximum allowable section number is 70

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

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	17.71				32.	32.	32.
2	2	2	56.00				32.	32.	32.
3	1	1	17.71				32.	32.	32.

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	2	2398	2038	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 2398

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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 (%)	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	0.32	0.00	15.00	1 1 1 1 1	0.54	2 2 2 2 2
1	0.00	0	0.00	0.68	0.00	0.00	0 0 0 0 0	0.97	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	7.0	0	30.0	10.0	0.69	0.00	91.42

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	0	wearing surf	0.238	0.00	91.42	0.00	0.00
2	1	rail	0.010	0.00	91.42	0.00	0.00
3	1	curb	0.115	0.00	91.42	0.00	0.00
4	2	end diaphrag	0.000	0.00	0.00	0.76	0.45
5	2	int diaphrag	0.000	0.00	0.00	0.35	13.71
6	2	int diaphrag	0.000	0.00	0.00	0.35	29.71
7	2	int diaphrag	0.000	0.00	0.00	0.35	45.71
8	2	int diaphrag	0.000	0.00	0.00	0.35	61.71
9	2	int diaphrag	0.000	0.00	0.00	0.35	77.71
10	2	end diaphrag	0.000	0.00	0.00	0.76	90.96

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 YIELD Fy (ksi)	BAR AREA PER FOOT OF SLAB (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	2.50	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	0.45	33.0	33.0	0.45
0.45	9.03	33.0	33.0	8.57
9.03	89.03	33.0	33.0	16.00
89.03	90.96	33.0	33.0	1.94

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

TABLE 1.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.688		0.3273		1.0153

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.24	0.00	91.42
1	DC2	0.010	0.000	91.417
2	DC2	0.115	0.000	91.417

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

CONCRETE DECK				STEEL	SUPERSTRUCTURE TOTAL WEIGHT
	VOLUME		TOTAL	TOTAL	
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	WEIGHT (kip)	WEIGHT (kip)	(kip)
150.00	1866.4	69.1	280.0	149.6	429.56

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	7.50	90.00

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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	0.539	0.539	0.653	0.653	FOR STRENGTH POSITIVE MOMENT
	0.969				FOR STRENGTH POSITIVE SHEAR
1	0.319				FOR FATIGUE POSITIVE MOMENT
	0.684				FOR FATIGUE POSITIVE SHEAR

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

SP NO	IN D 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)	STEEL TOP. (in**3)
1	0	0.00	18900.0	36.5	18.26	1035.0	1035.0

1	1	5.71	18900.0	36.5	18.26	1035.0	1035.0
1	2	11.43	18900.0	36.5	18.26	1035.0	1035.0
1	3	17.14	18900.0	36.5	18.26	1035.0	1035.0
1	4	22.85	25335.0	36.5	15.68	1615.5	1134.2
1	5	28.57	25335.0	36.5	15.68	1615.5	1134.2
1	6	34.28	25335.0	36.5	15.68	1615.5	1134.2
1	7	39.99	25335.0	36.5	15.68	1615.5	1134.2
1	8	45.71	25335.0	36.5	15.68	1615.5	1134.2
1	9	51.42	25335.0	36.5	15.68	1615.5	1134.2
1	10	57.14	25335.0	36.5	15.68	1615.5	1134.2
1	11	62.85	25335.0	36.5	15.68	1615.5	1134.2
1	12	68.56	25335.0	36.5	15.68	1615.5	1134.2
1	13	74.28	18900.0	36.5	18.26	1035.0	1035.0
1	14	79.99	18900.0	36.5	18.26	1035.0	1035.0
1	15	85.70	18900.0	36.5	18.26	1035.0	1035.0
1	16	91.42	18900.0	36.5	18.26	1035.0	1035.0

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 = 30.00

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	26963.4	0.014046	1186.8	1953.7	1296.2
1	1	5.71	26963.4	0.014046	1186.8	1953.7	1296.2
1	2	11.43	26963.4	0.014046	1186.8	1953.7	1296.2
1	3	17.14	26963.4	0.014046	1186.8	1953.7	1296.2
1	4	22.85	37194.9	0.012677	1856.5	2068.1	1488.7
1	5	28.57	37194.9	0.012677	1856.5	2068.1	1488.7
1	6	34.28	37194.9	0.012677	1856.5	2068.1	1488.7
1	7	39.99	37194.9	0.012677	1856.5	2068.1	1488.7
1	8	45.71	37194.9	0.012677	1856.5	2068.1	1488.7
1	9	51.42	37194.9	0.012677	1856.5	2068.1	1488.7
1	10	57.14	37194.9	0.012677	1856.5	2068.1	1488.7
1	11	62.85	37194.9	0.012677	1856.5	2068.1	1488.7
1	12	68.56	37194.9	0.012677	1856.5	2068.1	1488.7
1	13	74.28	26963.4	0.014046	1186.8	1953.7	1296.2
1	14	79.99	26963.4	0.014046	1186.8	1953.7	1296.2
1	15	85.70	26963.4	0.014046	1186.8	1953.7	1296.2
1	16	91.42	26963.4	0.014046	1186.8	1953.7	1296.2

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 10.00

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	36123.9	0.022201	1302.2	4114.4	2289.2
1	1	5.71	36123.9	0.022201	1302.2	4114.4	2289.2
1	2	11.43	36123.9	0.022201	1302.2	4114.4	2289.2
1	3	17.14	36123.9	0.022201	1302.2	4114.4	2289.2
1	4	22.85	52001.2	0.020281	2043.8	4134.8	2656.3
1	5	28.57	52001.2	0.020281	2043.8	4134.8	2656.3
1	6	34.28	52001.2	0.020281	2043.8	4134.8	2656.3
1	7	39.99	52001.2	0.020281	2043.8	4134.8	2656.3
1	8	45.71	52001.2	0.020281	2043.8	4134.8	2656.3
1	9	51.42	52001.2	0.020281	2043.8	4134.8	2656.3
1	10	57.14	52001.2	0.020281	2043.8	4134.8	2656.3
1	11	62.85	52001.2	0.020281	2043.8	4134.8	2656.3
1	12	68.56	52001.2	0.020281	2043.8	4134.8	2656.3
1	13	74.28	36123.9	0.022201	1302.2	4114.4	2289.2
1	14	79.99	36123.9	0.022201	1302.2	4114.4	2289.2
1	15	85.70	36123.9	0.022201	1302.2	4114.4	2289.2
1	16	91.42	36123.9	0.022201	1302.2	4114.4	2289.2

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	5.71	80.1	168.4	5.3	0.0	253.9	
1	2	11.43	149.6	314.4	10.3	0.0	474.4	
1	3	17.14	208.3	438.0	14.1	0.0	660.4	
1	4	22.85	256.4	539.0	17.1	0.0	812.6	
1	5	28.57	293.8	617.6	20.1	0.0	931.6	
1	6	34.28	320.5	673.8	21.5	0.0	1015.9	

1	7	39.99	336.6	707.5	22.5	0.0	1066.6
1	8	45.71	341.9	718.7	23.5	0.0	1084.1
1	9	51.42	336.6	707.5	22.5	0.0	1066.6
1	10	57.14	320.5	673.8	21.5	0.0	1015.9
1	11	62.85	293.8	617.6	20.1	0.0	931.6
1	12	68.56	256.4	539.0	17.1	0.0	812.6
1	13	74.28	208.3	438.0	14.1	0.0	660.4
1	14	79.99	149.6	314.4	10.3	0.0	474.4
1	15	85.70	80.1	168.4	5.3	0.0	253.9
1	16	91.42	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 = 30.00 (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	(k-ft) 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	5.71	58.3	30.6	0.0	0.0	0.0	88.9
1	2	11.43	108.8	57.1	0.0	0.0	0.0	165.9
1	3	17.14	151.5	79.6	0.0	0.0	0.0	231.1
1	4	22.85	186.5	97.9	0.0	0.0	0.0	284.4
1	5	28.57	213.7	112.2	0.0	0.0	0.0	325.9
1	6	34.28	233.1	122.4	0.0	0.0	0.0	355.5
1	7	39.99	244.7	128.5	0.0	0.0	0.0	373.3
1	8	45.71	248.6	130.6	0.0	0.0	0.0	379.2
1	9	51.42	244.7	128.5	0.0	0.0	0.0	373.3
1	10	57.14	233.1	122.4	0.0	0.0	0.0	355.5
1	11	62.85	213.7	112.2	0.0	0.0	0.0	325.9
1	12	68.56	186.5	97.9	0.0	0.0	0.0	284.4
1	13	74.28	151.5	79.6	0.0	0.0	0.0	231.1
1	14	79.99	108.8	57.1	0.0	0.0	0.0	165.9
1	15	85.70	58.3	30.6	0.0	0.0	0.0	88.9
1	16	91.42	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 = 10.00 (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	5.71	0.0	0.0	330.8	HL-93	0.0	HL-93
1	2	11.43	0.0	0.0	613.5	HL-93	0.0	HL-93
1	3	17.14	0.0	0.0	848.1	HL-93	0.0	HL-93
1	4	22.85	0.0	0.0	1032.5	HL-93	0.0	HL-93
1	5	28.57	0.0	0.0	1174.4	HL-93	0.0	HL-93
1	6	34.28	0.0	0.0	1274.4	HL-93	0.0	HL-93
1	7	39.99	0.0	0.0	1324.6	HL-93	0.0	HL-93
1	8	45.71	0.0	0.0	1339.2	HL-93	0.0	HL-93
1	9	51.42	0.0	0.0	1324.6	HL-93	0.0	HL-93
1	10	57.14	0.0	0.0	1274.4	HL-93	0.0	HL-93

1	11	62.85	0.0	0.0	1174.4	HL-93	0.0	HL-93
1	12	68.56	0.0	0.0	1032.5	HL-93	0.0	HL-93
1	13	74.28	0.0	0.0	848.1	HL-93	0.0	HL-93
1	14	79.99	0.0	0.0	613.5	HL-93	0.0	HL-93
1	15	85.70	0.0	0.0	330.8	HL-93	0.0	HL-93
1	16	91.42	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 = 10.0 (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	5.71	111.	0.	111.
1	2	11.43	203.	0.	203.
1	3	17.14	277.	0.	277.
1	4	22.85	336.	0.	336.
1	5	28.57	380.	0.	380.
1	6	34.28	408.	0.	408.
1	7	39.99	410.	0.	410.
1	8	45.71	407.	0.	407.
1	9	51.42	410.	0.	410.
1	10	57.14	408.	0.	408.
1	11	62.85	380.	0.	380.
1	12	68.56	336.	0.	336.
1	13	74.28	277.	0.	277.
1	14	79.99	203.	0.	203.
1	15	85.70	111.	0.	111.
1	16	91.42	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=30.0 (k-ft)	LL+I:N=10.0 LOAD TYPE= HL - 93 MAXIMUM POSITIVE (k-ft)	GOVERN LOAD TYPE	MAXIMUM NEGATIVE LOAD (k-ft)	GOVERN LOAD TYPE	TOTAL MAXIMUM POSITIVE (k-ft)	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	5.71	253.9	88.9	330.8	HL-93	0.0	HL-93	673.6	342.8
1	2	11.43	474.4	165.9	613.5	HL-93	0.0	HL-93	1253.8	640.3
1	3	17.14	660.4	231.1	848.1	HL-93	0.0	HL-93	1739.6	891.5
1	4	22.85	812.6	284.4	1032.5	HL-93	0.0	HL-93	2129.5	1097.0
1	5	28.57	931.6	325.9	1174.4	HL-93	0.0	HL-93	2431.9	1257.5
1	6	34.28	1015.9	355.5	1274.4	HL-93	0.0	HL-93	2645.7	1371.4
1	7	39.99	1066.6	373.3	1324.6	HL-93	0.0	HL-93	2764.4	1439.9
1	8	45.71	1084.1	379.2	1339.2	HL-93	0.0	HL-93	2802.6	1463.3
1	9	51.42	1066.6	373.3	1324.6	HL-93	0.0	HL-93	2764.4	1439.9
1	10	57.14	1015.9	355.5	1274.4	HL-93	0.0	HL-93	2645.7	1371.4
1	11	62.85	931.6	325.9	1174.4	HL-93	0.0	HL-93	2431.9	1257.5
1	12	68.56	812.6	284.4	1032.5	HL-93	0.0	HL-93	2129.5	1097.0
1	13	74.28	660.4	231.1	848.1	HL-93	0.0	HL-93	1739.6	891.5
1	14	79.99	474.4	165.9	613.5	HL-93	0.0	HL-93	1253.8	640.3
1	15	85.70	253.9	88.9	330.8	HL-93	0.0	HL-93	673.6	342.8
1	16	91.42	0.0	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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COMPOSITE
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ANALYSIS

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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	5.71	673.6	772.9	1022.0	889.7	514.2	83.5
1	2	11.43	1253.8	1437.8	1901.2	1655.8	960.4	152.2
1	3	17.14	1739.6	1994.0	2636.4	2297.2	1337.3	207.8
1	4	22.85	2129.5	2439.3	3224.8	2811.8	1645.5	252.4
1	5	28.57	2431.9	2784.2	3680.5	3210.7	1886.2	285.3
1	6	34.28	2645.7	3028.0	4002.6	3492.9	2057.0	306.1
1	7	39.99	2764.4	3161.8	4179.0	3649.2	2159.8	307.2
1	8	45.71	2802.6	3204.4	4235.0	3699.3	2195.0	305.2
1	9	51.42	2764.4	3161.8	4179.0	3649.2	2159.8	307.2
1	10	57.14	2645.7	3028.0	4002.6	3492.9	2057.0	306.1
1	11	62.85	2431.9	2784.2	3680.5	3210.8	1886.2	285.3
1	12	68.56	2129.5	2439.3	3224.8	2811.8	1645.5	252.4
1	13	74.28	1739.6	1994.1	2636.5	2297.2	1337.3	207.8
1	14	79.99	1253.8	1437.8	1901.2	1655.8	960.4	152.2
1	15	85.70	673.6	772.9	1022.0	889.7	514.2	83.5
1	16	91.42	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	15.0	31.4	1.6	0.0	48.0
1	1	5.71	13.1	27.5	0.9	0.0	41.5
1	2	11.43	11.2	23.6	0.9	0.0	35.7
1	3	17.14	9.3	19.7	0.5	0.0	29.5
1	4	22.85	7.5	15.7	0.5	0.0	23.7
1	5	28.57	5.6	11.8	0.5	0.0	17.9
1	6	34.28	3.7	7.9	0.2	0.0	11.8
1	7	39.99	1.9	3.9	0.2	0.0	6.0
1	8	45.71	0.0	0.0	0.2	0.0	0.2
1	9	51.42	-1.9	-3.9	-0.2	0.0	-6.0
1	10	57.14	-3.7	-7.9	-0.2	0.0	-11.8
1	11	62.85	-5.6	-11.8	-0.5	0.0	-17.9
1	12	68.56	-7.5	-15.7	-0.5	0.0	-23.7
1	13	74.28	-9.3	-19.7	-0.5	0.0	-29.5
1	14	79.99	-11.2	-23.6	-0.9	0.0	-35.7
1	15	85.70	-13.1	-27.5	-0.9	0.0	-41.5
1	16	91.42	-15.0	-31.4	-1.6	0.0	-48.0

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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	COMPOSITE (k) UNIFORM LOADS DC2	COMPOSITE (k) CONCENTRATED LOADS DW	COMPOSITE (k) CONCENTRATED LOADS DC2	TOTAL COMPOSITE (k)	TOTAL NONCOMPOSITE (k)	TOTAL DEAD LOADS (k)
1	0	0.00	10.9	5.7	0.0	0.0	16.6	48.0	64.6
1	1	5.71	9.5	5.0	0.0	0.0	14.5	41.5	56.0
1	2	11.43	8.2	4.3	0.0	0.0	12.4	35.7	48.1
1	3	17.14	6.8	3.6	0.0	0.0	10.4	29.5	39.9
1	4	22.85	5.4	2.9	0.0	0.0	8.3	23.7	32.0
1	5	28.57	4.1	2.1	0.0	0.0	6.2	17.9	24.1
1	6	34.28	2.7	1.4	0.0	0.0	4.1	11.8	15.9
1	7	39.99	1.4	0.7	0.0	0.0	2.1	6.0	8.0
1	8	45.71	0.0	0.0	0.0	0.0	0.0	0.2	0.2
1	9	51.42	-1.4	-0.7	0.0	0.0	-2.1	-6.0	-8.0
1	10	57.14	-2.7	-1.4	0.0	0.0	-4.1	-11.8	-15.9
1	11	62.85	-4.1	-2.1	0.0	0.0	-6.2	-17.9	-24.1
1	12	68.56	-5.4	-2.9	0.0	0.0	-8.3	-23.7	-32.0
1	13	74.28	-6.8	-3.6	0.0	0.0	-10.4	-29.5	-39.9
1	14	79.99	-8.2	-4.3	0.0	0.0	-12.4	-35.7	-48.1
1	15	85.70	-9.5	-5.0	0.0	0.0	-14.5	-41.5	-56.0
1	16	91.42	-10.9	-5.7	0.0	0.0	-16.6	-48.0	-64.6

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	111.7	HL-93	0.0	HL-93
1	1	5.71	0.0	0.0	102.4	HL-93	-3.1	HL-93
1	2	11.43	0.0	0.0	93.4	HL-93	-7.1	HL-93
1	3	17.14	0.0	0.0	84.6	HL-93	-11.7	HL-93
1	4	22.85	0.0	0.0	76.1	HL-93	-16.5	HL-93
1	5	28.57	0.0	0.0	67.7	HL-93	-22.3	HL-93
1	6	34.28	0.0	0.0	59.6	HL-93	-29.3	HL-93
1	7	39.99	0.0	0.0	51.7	HL-93	-36.5	HL-93
1	8	45.71	0.0	0.0	44.0	HL-93	-44.0	HL-93
1	9	51.42	0.0	0.0	36.5	HL-93	-51.7	HL-93
1	10	57.14	0.0	0.0	29.3	HL-93	-59.6	HL-93
1	11	62.85	0.0	0.0	22.3	HL-93	-67.7	HL-93
1	12	68.56	0.0	0.0	16.5	HL-93	-76.1	HL-93
1	13	74.28	0.0	0.0	11.7	HL-93	-84.6	HL-93
1	14	79.99	0.0	0.0	7.1	HL-93	-93.4	HL-93
1	15	85.70	0.0	0.0	3.1	HL-93	-102.4	HL-93
1	16	91.42	0.0	0.0	0.0	HL-93	-111.7	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 10.0 (kips) (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (ft)	TRUCK ONLY POS	NEG	RANGE
1	0		0.00	45.	0.	45.
1	1		5.71	42.	-2.	43.
1	2		11.43	38.	-3.	41.
1	3		17.14	35.	-5.	40.
1	4		22.85	31.	-7.	38.
1	5		28.57	28.	-9.	36.
1	6		34.28	24.	-11.	35.
1	7		39.99	21.	-14.	34.
1	8		45.71	17.	-17.	34.
1	9		51.42	14.	-21.	34.
1	10		57.14	11.	-24.	35.
1	11		62.85	9.	-28.	36.
1	12		68.56	7.	-31.	38.
1	13		74.28	5.	-35.	40.
1	14		79.99	3.	-38.	41.
1	15		85.70	2.	-42.	43.
1	16		91.42	0.	-45.	45.

TABLE 1.2.6.3B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE Vr [1] (Kip)	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	45.3	.27204E-01	0.62	0.3125*	.17938E-01	0.41	0.3125*
1	1		5.71	43.3	.27204E-01	0.59	0.3125*	.17938E-01	0.39	0.3125*
1	2		11.43	41.4	.27204E-01	0.56	0.3125*	.17938E-01	0.37	0.3125*
1	3		17.14	39.7	.27204E-01	0.54	0.3125*	.17938E-01	0.36	0.3125*
1	4		22.85	38.0	.25568E-01	0.49	0.3125*	.10650E-01	0.20	0.3125*
1	5		28.57	36.4	.25568E-01	0.47	0.3125*	.10650E-01	0.19	0.3125*
1	6		34.28	35.0	.25568E-01	0.45	0.3125*	.10650E-01	0.19	0.3125*
1	7		39.99	34.2	.25568E-01	0.44	0.3125*	.10650E-01	0.18	0.3125*
1	8		45.71	34.0	.25568E-01	0.44	0.3125*	.10650E-01	0.18	0.3125*
1	9		51.42	34.2	.25568E-01	0.44	0.3125*	.10650E-01	0.18	0.3125*
1	10		57.14	35.0	.25568E-01	0.45	0.3125*	.10650E-01	0.19	0.3125*
1	11		62.85	36.4	.25568E-01	0.47	0.3125*	.10650E-01	0.19	0.3125*
1	12		68.56	38.0	.25568E-01	0.49	0.3125*	.10650E-01	0.20	0.3125*
1	13		74.28	39.7	.27204E-01	0.54	0.3125*	.17938E-01	0.36	0.3125*
1	14		79.99	41.4	.27204E-01	0.56	0.3125*	.17938E-01	0.37	0.3125*
1	15		85.70	43.3	.27204E-01	0.59	0.3125*	.17938E-01	0.39	0.3125*
1	16		91.42	45.3	.27204E-01	0.62	0.3125*	.17938E-01	0.41	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

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	64.6	111.7	HL-93	0.0	HL-93	176.3	64.6
1	1	5.71	56.0	102.4	HL-93	-3.1	HL-93	158.4	52.9
1	2	11.43	48.1	93.4	HL-93	-7.1	HL-93	141.5	41.0
1	3	17.14	39.9	84.6	HL-93	-11.7	HL-93	124.5	28.2
1	4	22.85	32.0	76.1	HL-93	-16.5	HL-93	108.1	15.6
1	5	28.57	24.1	67.7	HL-93	-22.3	HL-93	91.9	1.8
1	6	34.28	15.9	59.6	HL-93	-29.3	HL-93	75.5	-13.4
1	7	39.99	8.0	51.7	HL-93	-36.5	HL-93	59.7	-28.5
1	8	45.71	0.2	44.0	HL-93	-44.0	HL-93	44.2	-43.8
1	9	51.42	-8.0	36.5	HL-93	-51.7	HL-93	28.5	-59.7
1	10	57.14	-15.9	29.3	HL-93	-59.6	HL-93	13.4	-75.5
1	11	62.85	-24.1	22.3	HL-93	-67.7	HL-93	-1.8	-91.9
1	12	68.56	-32.0	16.5	HL-93	-76.1	HL-93	-15.6	-108.1
1	13	74.28	-39.9	11.7	HL-93	-84.6	HL-93	-28.2	-124.5
1	14	79.99	-48.1	7.1	HL-93	-93.4	HL-93	-41.0	-141.5
1	15	85.70	-56.0	3.1	HL-93	-102.4	HL-93	-52.9	-158.4
1	16	91.42	-64.6	0.0	HL-93	-111.7	HL-93	-64.6	-176.3

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	176.3	209.8	278.9	234.2	97.0	34.0
1	1	5.71	158.4	189.1	251.6	210.6	84.0	32.5
1	2	11.43	141.5	169.6	225.7	188.3	72.2	31.1
1	3	17.14	124.5	149.9	199.7	165.8	59.8	29.8
1	4	22.85	108.1	130.9	174.5	144.1	48.0	28.5
1	5	28.57	91.9	112.2	149.7	122.6	36.2	27.3
1	6	34.28	75.5	93.4	124.9	101.0	23.9	26.2
1	7	39.99	59.7	75.2	100.8	80.2	12.1	25.7
1	8	45.71	44.2	57.4	77.2	59.6	0.3	25.5
1	9	51.42	-59.7	-75.2	-100.8	-80.2	-12.1	25.7
1	10	57.14	-75.5	-93.4	-124.9	-101.0	-23.9	26.2
1	11	62.85	-91.9	-112.2	-149.7	-122.6	-36.2	27.3
1	12	68.56	-108.1	-130.9	-174.5	-144.1	-48.0	28.5

1	13	74.28	-124.5	-149.9	-199.7	-165.8	-59.8	29.8
1	14	79.99	-141.5	-169.6	-225.7	-188.3	-72.2	31.1
1	15	85.70	-158.4	-189.1	-251.6	-210.6	-84.0	32.5
1	16	91.42	-176.3	-209.8	-278.9	-234.2	-97.0	34.0

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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	64.63	0.00	HL-93	111.65	HL-93	ST1	55.45	278.91
						ST2	55.45	234.25
						ST4	80.63	96.95
						SE1	64.63	176.29
						SE2	64.63	209.78
2	64.63	0.00	HL-93	111.65	HL-93	ST1	55.45	278.91
						ST2	55.45	234.25
						ST4	80.63	96.95
						SE1	64.63	176.29
						SE2	64.63	209.78

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 = 30.0 (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	5.71	-0.1479	-0.3110	0.0000	-0.1125	-0.4689	-0.1125	-0.5814
1	2	11.43	-0.2877	-0.6048	0.0000	-0.2187	-0.9120	-0.2187	-1.1307
1	3	17.14	-0.4122	-0.8664	0.0000	-0.3130	-1.3065	-0.3130	-1.6195
1	4	22.85	-0.5177	-1.0883	0.0000	-0.3927	-1.6410	-0.3927	-2.0338
1	5	28.57	-0.6036	-1.2688	0.0000	-0.4576	-1.9133	-0.4576	-2.3709
1	6	34.28	-0.6670	-1.4021	0.0000	-0.5055	-2.1143	-0.5055	-2.6198
1	7	39.99	-0.7059	-1.4838	0.0000	-0.5349	-2.2374	-0.5349	-2.7723
1	8	45.71	-0.7190	-1.5113	0.0000	-0.5448	-2.2789	-0.5448	-2.8237
1	9	51.42	-0.7059	-1.4838	0.0000	-0.5349	-2.2374	-0.5349	-2.7723
1	10	57.14	-0.6670	-1.4021	0.0000	-0.5055	-2.1143	-0.5055	-2.6198
1	11	62.85	-0.6036	-1.2688	0.0000	-0.4576	-1.9133	-0.4576	-2.3709
1	12	68.56	-0.5177	-1.0883	0.0000	-0.3927	-1.6410	-0.3927	-2.0338
1	13	74.28	-0.4122	-0.8664	0.0000	-0.3130	-1.3065	-0.3130	-1.6195
1	14	79.99	-0.2877	-0.6048	0.0000	-0.2187	-0.9120	-0.2187	-1.1307
1	15	85.70	-0.1479	-0.3110	0.0000	-0.1125	-0.4689	-0.1125	-0.5814
1	16	91.42	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	T O T A L (in) DEFL.	CAMBER SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	5.7	-0.148	0 3/16	-0.321	0 3/ 8	-0.113	0 1/ 8	-0.581	0 5/ 8
1	2	11.4	-0.288	0 5/16	-0.624	0 5/ 8	-0.219	0 1/ 4	-1.131	1 3/16
1	3	17.1	-0.412	0 7/16	-0.894	0 15/16	-0.313	0 3/ 8	-1.620	1 5/ 8
1	4	22.9	-0.518	0 9/16	-1.123	1 1/ 8	-0.393	0 7/16	-2.034	2 1/16
1	5	28.6	-0.604	0 5/ 8	-1.310	1 5/16	-0.458	0 1/ 2	-2.371	2 3/ 8
1	6	34.3	-0.667	0 11/16	-1.447	1 1/ 2	-0.506	0 9/16	-2.620	2 5/ 8
1	7	40.0	-0.706	0 3/ 4	-1.532	1 9/16	-0.535	0 9/16	-2.772	2 13/16
1	8	45.7	-0.719	0 3/ 4	-1.560	1 9/16	-0.545	0 9/16	-2.824	2 7/ 8
1	9	51.4	-0.706	0 3/ 4	-1.532	1 9/16	-0.535	0 9/16	-2.772	2 13/16
1	10	57.1	-0.667	0 11/16	-1.447	1 1/ 2	-0.506	0 9/16	-2.620	2 5/ 8
1	11	62.8	-0.604	0 5/ 8	-1.310	1 5/16	-0.458	0 1/ 2	-2.371	2 3/ 8
1	12	68.6	-0.518	0 9/16	-1.123	1 1/ 8	-0.393	0 7/16	-2.034	2 1/16
1	13	74.3	-0.412	0 7/16	-0.894	0 15/16	-0.313	0 3/ 8	-1.620	1 5/ 8
1	14	80.0	-0.288	0 5/16	-0.624	0 5/ 8	-0.219	0 1/ 4	-1.131	1 3/16
1	15	85.7	-0.148	0 3/16	-0.321	0 3/ 8	-0.113	0 1/ 8	-0.581	0 5/ 8
1	16	91.4	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	45.71	2 0.400	-0.677 MAX	HL-93	1.37	0.00378
			0.034 MIN			
			-0.276 MAX	LANE		
			0.034 MIN	LANE		
		3 0.510	-0.863 MAX	HL-93	1.37	0.00378
			0.043 MIN			
			-0.352 MAX	LANE		
			0.043 MIN	LANE		
		2 0.400	-0.594 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	5.71	-0.93	0.93	-2.01	2.01	-2.94	2.94
1	2	11.43	-1.73	1.73	-3.77	3.77	-5.50	5.50
1	3	17.14	-2.42	2.42	-5.24	5.24	-7.66	7.66
1	4	22.85	-2.71	1.90	-5.88	4.13	-8.60	6.04
1	5	28.57	-3.11	2.18	-6.75	4.74	-9.86	6.92
1	6	34.28	-3.39	2.38	-7.36	5.16	-10.75	7.55
1	7	39.99	-3.56	2.50	-7.72	5.42	-11.28	7.92
1	8	45.71	-3.62	2.54	-7.85	5.51	-11.47	8.05
1	9	51.42	-3.56	2.50	-7.72	5.42	-11.28	7.92
1	10	57.14	-3.39	2.38	-7.36	5.16	-10.75	7.55
1	11	62.85	-3.11	2.18	-6.75	4.74	-9.86	6.92
1	12	68.56	-2.71	1.90	-5.88	4.13	-8.60	6.04
1	13	74.28	-2.42	2.42	-5.24	5.24	-7.66	7.66
1	14	79.99	-1.73	1.73	-3.77	3.77	-5.50	5.50
1	15	85.70	-0.93	0.93	-2.01	2.01	-2.94	2.94
1	16	91.42	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 = 30.0 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	5.71	-0.03	-0.55	0.90	-0.03	-3.49	3.84
1	2	11.43	-0.05	-1.02	1.68	-0.05	-6.52	7.18
1	3	17.14	-0.07	-1.42	2.34	-0.07	-9.08	9.99
1	4	22.85	-0.08	-1.65	1.84	-0.08	-10.25	7.87
1	5	28.57	-0.09	-1.89	2.11	-0.09	-11.75	9.03
1	6	34.28	-0.10	-2.06	2.30	-0.10	-12.81	9.84
1	7	39.99	-0.10	-2.17	2.41	-0.10	-13.45	10.34
1	8	45.71	-0.10	-2.20	2.45	-0.10	-13.67	10.50
1	9	51.42	-0.10	-2.17	2.41	-0.10	-13.45	10.34
1	10	57.14	-0.10	-2.06	2.30	-0.10	-12.81	9.84
1	11	62.85	-0.09	-1.89	2.11	-0.09	-11.75	9.03
1	12	68.56	-0.08	-1.65	1.84	-0.08	-10.25	7.87
1	13	74.28	-0.07	-1.42	2.34	-0.07	-9.08	9.99
1	14	79.99	-0.05	-1.02	1.68	-0.05	-6.52	7.18
1	15	85.70	-0.03	-0.55	0.90	-0.03	-3.49	3.84
1	16	91.42	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 = 10.0 (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	5.71	-0.17	-0.96	3.05	0.00	0.00	0.00
1	2	11.43	-0.32	-1.79	5.65	0.00	0.00	0.00
1	3	17.14	-0.44	-2.47	7.82	0.00	0.00	0.00
1	4	22.85	-0.47	-3.00	6.06	0.00	0.00	0.00
1	5	28.57	-0.53	-3.41	6.90	0.00	0.00	0.00
1	6	34.28	-0.58	-3.70	7.48	0.00	0.00	0.00
1	7	39.99	-0.60	-3.84	7.78	0.00	0.00	0.00
1	8	45.71	-0.61	-3.89	7.86	0.00	0.00	0.00
1	9	51.42	-0.60	-3.84	7.78	0.00	0.00	0.00
1	10	57.14	-0.58	-3.70	7.48	0.00	0.00	0.00
1	11	62.85	-0.53	-3.41	6.90	0.00	0.00	0.00
1	12	68.56	-0.47	-3.00	6.06	0.00	0.00	0.00
1	13	74.28	-0.44	-2.47	7.82	0.00	0.00	0.00
1	14	79.99	-0.32	-1.79	5.65	0.00	0.00	0.00
1	15	85.70	-0.17	-0.96	3.05	0.00	0.00	0.00
1	16	91.42	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 = 10.0 (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	5.71	0.3	0.0	0.3	C	HL	1.0	0.0	1.0	T	HL
1	2	11.43	0.6	0.0	0.6	C	HL	1.9	0.0	1.9	T	HL
1	3	17.14	0.8	0.0	0.8	C	HL	2.6	0.0	2.6	T	HL
1	4	22.85	1.0	0.0	1.0	C	HL	2.0	0.0	2.0	T	HL
1	5	28.57	1.1	0.0	1.1	C	HL	2.2	0.0	2.2	T	HL

1	6	34.28	1.2	0.0	1.2	C	HL	2.4	0.0	2.4	T HL
1	7	39.99	1.2	0.0	1.2	C	HL	2.4	0.0	2.4	T HL
1	8	45.71	1.2	0.0	1.2	C	HL	2.4	0.0	2.4	T HL
1	9	51.42	1.2	0.0	1.2	C	HL	2.4	0.0	2.4	T HL
1	10	57.14	1.2	0.0	1.2	C	HL	2.4	0.0	2.4	T HL
1	11	62.85	1.1	0.0	1.1	C	HL	2.2	0.0	2.2	T HL
1	12	68.56	1.0	0.0	1.0	C	HL	2.0	0.0	2.0	T HL
1	13	74.28	0.8	0.0	0.8	C	HL	2.6	0.0	2.6	T HL
1	14	79.99	0.6	0.0	0.6	C	HL	1.9	0.0	1.9	T HL
1	15	85.70	0.3	0.0	0.3	C	HL	1.0	0.0	1.0	T HL
1	16	91.42	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.20	-4.45	6.89	-0.03	-3.49	3.84
1	2	11.43	32.	32.	-0.37	-8.31	12.83	-0.05	-6.52	7.18
1	3	17.14	32.	32.	-0.52	-11.55	17.81	-0.07	-9.08	9.99
1	4	22.85	32.	32.	-0.54	-13.24	13.94	-0.08	-10.25	7.87
1	5	28.57	32.	32.	-0.62	-15.16	15.92	-0.09	-11.75	9.03
1	6	34.28	32.	32.	-0.67	-16.51	17.33	-0.10	-12.81	9.84
1	7	39.99	32.	32.	-0.70	-17.29	18.11	-0.10	-13.45	10.34
1	8	45.71	32.	32.	-0.71	-17.56	18.37	-0.10	-13.67	10.50
1	9	51.42	32.	32.	-0.70	-17.29	18.11	-0.10	-13.45	10.34
1	10	57.14	32.	32.	-0.67	-16.51	17.33	-0.10	-12.81	9.84
1	11	62.85	32.	32.	-0.62	-15.16	15.92	-0.09	-11.75	9.03
1	12	68.56	32.	32.	-0.54	-13.24	13.94	-0.08	-10.25	7.87
1	13	74.28	32.	32.	-0.52	-11.55	17.81	-0.07	-9.08	9.99
1	14	79.99	32.	32.	-0.37	-8.31	12.83	-0.05	-6.52	7.18
1	15	85.70	32.	32.	-0.20	-4.45	6.89	-0.03	-3.49	3.84
1	16	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.25	-4.74	7.81	-0.03	-3.49	3.84
1	2	11.43	32.	32.	-0.47	-8.84	14.53	-0.05	-6.52	7.18
1	3	17.14	32.	32.	-0.65	-12.29	20.15	-0.07	-9.08	9.99
1	4	22.85	32.	32.	-0.68	-14.14	15.76	-0.08	-10.25	7.87
1	5	28.57	32.	32.	-0.78	-16.18	17.99	-0.09	-11.75	9.03

1	6	34.28	32.	32.	-0.84	-17.62	19.57	-0.10	-12.81	9.84
1	7	39.99	32.	32.	-0.88	-18.45	20.45	-0.10	-13.45	10.34
1	8	45.71	32.	32.	-0.89	-18.72	20.73	-0.10	-13.67	10.50
1	9	51.42	32.	32.	-0.88	-18.45	20.45	-0.10	-13.45	10.34
1	10	57.14	32.	32.	-0.84	-17.62	19.57	-0.10	-12.81	9.84
1	11	62.85	32.	32.	-0.78	-16.18	17.99	-0.09	-11.75	9.03
1	12	68.56	32.	32.	-0.68	-14.14	15.76	-0.08	-10.25	7.87
1	13	74.28	32.	32.	-0.65	-12.29	20.15	-0.07	-9.08	9.99
1	14	79.99	32.	32.	-0.47	-8.84	14.53	-0.05	-6.52	7.18
1	15	85.70	32.	32.	-0.25	-4.74	7.81	-0.03	-3.49	3.84
1	16	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.34	-6.14	10.29	-0.04	-4.45	4.95
1	2	11.43	32.	32.	-0.64	-11.45	19.14	-0.07	-8.32	9.25
1	3	17.14	32.	32.	-0.88	-15.91	26.55	-0.10	-11.58	12.87
1	4	22.85	32.	32.	-0.92	-18.32	20.75	-0.11	-13.08	10.14
1	5	28.57	32.	32.	-1.05	-20.96	23.70	-0.12	-14.99	11.63
1	6	34.28	32.	32.	-1.14	-22.82	25.78	-0.14	-16.35	12.68
1	7	39.99	32.	32.	-1.19	-23.90	26.93	-0.14	-17.17	13.31
1	8	45.71	32.	32.	-1.20	-24.25	27.29	-0.14	-17.45	13.53
1	9	51.42	32.	32.	-1.19	-23.90	26.93	-0.14	-17.17	13.31
1	10	57.14	32.	32.	-1.14	-22.82	25.78	-0.14	-16.35	12.68
1	11	62.85	32.	32.	-1.05	-20.96	23.70	-0.12	-14.99	11.63
1	12	68.56	32.	32.	-0.92	-18.32	20.75	-0.11	-13.08	10.14
1	13	74.28	32.	32.	-0.88	-15.91	26.55	-0.10	-11.58	12.87
1	14	79.99	32.	32.	-0.64	-11.45	19.14	-0.07	-8.32	9.25
1	15	85.70	32.	32.	-0.34	-6.14	10.29	-0.04	-4.45	4.95
1	16	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.32	-4.74	8.65	-0.02	-3.05	3.31
1	2	11.43	32.	32.	-0.60	-8.83	16.08	-0.04	-5.70	6.18
1	3	17.14	32.	32.	-0.83	-12.26	22.29	-0.05	-7.94	8.61
1	4	22.85	32.	32.	-0.87	-14.20	17.39	-0.06	-8.95	6.79
1	5	28.57	32.	32.	-0.99	-16.23	19.85	-0.06	-10.26	7.78
1	6	34.28	32.	32.	-1.08	-17.66	21.58	-0.07	-11.19	8.48
1	7	39.99	32.	32.	-1.12	-18.48	22.52	-0.07	-11.75	8.91
1	8	45.71	32.	32.	-1.13	-18.74	22.81	-0.07	-11.94	9.05
1	9	51.42	32.	32.	-1.12	-18.48	22.52	-0.07	-11.75	8.91
1	10	57.14	32.	32.	-1.08	-17.66	21.58	-0.07	-11.19	8.48
1	11	62.85	32.	32.	-0.99	-16.23	19.85	-0.06	-10.26	7.78
1	12	68.56	32.	32.	-0.87	-14.20	17.39	-0.06	-8.95	6.79

1	13	74.28	32.	32.	-0.83	-12.26	22.29	-0.05	-7.94	8.61
1	14	79.99	32.	32.	-0.60	-8.83	16.08	-0.04	-5.70	6.18
1	15	85.70	32.	32.	-0.32	-4.74	8.65	-0.02	-3.05	3.31
1	16	91.42	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00

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MERLIN V 10.4
COMPOSITE
LRF -- 2010
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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.27	-5.75	9.07	-0.04	-4.45	4.95
1	2	11.43	32.	32.	-0.51	-10.73	16.88	-0.07	-8.32	9.25
1	3	17.14	32.	32.	-0.70	-14.92	23.43	-0.10	-11.58	12.87
1	4	22.85	32.	32.	-0.74	-17.13	18.33	-0.11	-13.08	10.14
1	5	28.57	32.	32.	-0.84	-19.60	20.94	-0.12	-14.99	11.63
1	6	34.28	32.	32.	-0.91	-21.34	22.78	-0.14	-16.35	12.68
1	7	39.99	32.	32.	-0.95	-22.36	23.81	-0.14	-17.17	13.31
1	8	45.71	32.	32.	-0.96	-22.70	24.15	-0.14	-17.45	13.53
1	9	51.42	32.	32.	-0.95	-22.36	23.81	-0.14	-17.17	13.31
1	10	57.14	32.	32.	-0.91	-21.34	22.78	-0.14	-16.35	12.68
1	11	62.85	32.	32.	-0.84	-19.60	20.94	-0.12	-14.99	11.63
1	12	68.56	32.	32.	-0.74	-17.13	18.33	-0.11	-13.08	10.14
1	13	74.28	32.	32.	-0.70	-14.92	23.43	-0.10	-11.58	12.87
1	14	79.99	32.	32.	-0.51	-10.73	16.88	-0.07	-8.32	9.25
1	15	85.70	32.	32.	-0.27	-5.75	9.07	-0.04	-4.45	4.95
1	16	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.71	32.	32.	-0.25	-4.35	7.43	-0.02	-3.05	3.31
1	2	11.43	32.	32.	-0.47	-8.12	13.82	-0.04	-5.70	6.18
1	3	17.14	32.	32.	-0.65	-11.28	19.16	-0.05	-7.94	8.61
1	4	22.85	32.	32.	-0.69	-13.00	14.97	-0.06	-8.95	6.79
1	5	28.57	32.	32.	-0.78	-14.86	17.09	-0.06	-10.26	7.78
1	6	34.28	32.	32.	-0.85	-16.18	18.58	-0.07	-11.19	8.48
1	7	39.99	32.	32.	-0.88	-16.94	19.41	-0.07	-11.75	8.91
1	8	45.71	32.	32.	-0.89	-17.19	19.67	-0.07	-11.94	9.05
1	9	51.42	32.	32.	-0.88	-16.94	19.41	-0.07	-11.75	8.91
1	10	57.14	32.	32.	-0.85	-16.18	18.58	-0.07	-11.19	8.48
1	11	62.85	32.	32.	-0.78	-14.86	17.09	-0.06	-10.26	7.78
1	12	68.56	32.	32.	-0.69	-13.00	14.97	-0.06	-8.95	6.79
1	13	74.28	32.	32.	-0.65	-11.28	19.16	-0.05	-7.94	8.61
1	14	79.99	32.	32.	-0.47	-8.12	13.82	-0.04	-5.70	6.18
1	15	85.70	32.	32.	-0.25	-4.35	7.43	-0.02	-3.05	3.31
1	16	91.42	32.	32.	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=30.0 MOM. (k-ft)	DL2, N=30.0 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	17.71	676.	-7.8	236.	-1.5	866.	-2.5	0.	0.0	-11.81
											2.53
	LB		7.8		2.4		8.0			0.0	18.21
											7.98
	RT		-7.1		-1.4		-2.5			0.0	-11.03
											2.51
	RB		5.0		1.5		5.1			0.0	11.63
											5.09
1	LT	73.71	676.	-7.1	236.	-1.4	866.	-2.5	0.	0.0	-11.03
											2.51
	LB		5.0		1.5		5.1			0.0	11.63
											5.09
	RT		-7.8		-1.5		-2.5			0.0	-11.81
											2.53
	RB		7.8		2.4		8.0			0.0	18.21
											7.98

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	17.71	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	16.59	0.0000
2	56.00	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	15.00	1.5000
3	17.71	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	16.59	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) = 91.4

MEMBER NO	LENGTH (ft)	SLAB THICK. (in)	GIRDER DEPTH [2]=OVERALL D. (in)	GIRDER DEPTH [3]= GIRDER D. (in)	[2]/(12*[1])	<=>	LIMITATION 1/25	[3]/(12*[1])	LIMITATION 1/30	STATUS

1	17.71	7.00	36.52	[2]=	43.52	0.03967	<	0.04000<--=NG
				[3]=	36.52	0.03329	<	0.03300<--=NG
2	56.00	7.00	36.52	[2]=	43.52	0.03967	<	0.04000<--=NG
				[3]=	36.52	0.03329	<	0.03300<--=NG
3	17.71	7.00	36.52	[2]=	43.52	0.03967	<	0.04000<--=NG
				[3]=	36.52	0.03329	<	0.03300<--=NG

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]	2Dcp/tw [3]	2Dc/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0		0.00	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	1		5.71	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	2		11.43	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	3		17.14	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	4		22.85	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	5		28.57	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	6		34.28	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	7		39.99	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	8		45.71	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	9		51.42	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	10		57.14	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	11		62.85	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	12		68.56	0	1	33.38	0.884	37.8	150.0	13.02	113.19			0
1	13		74.28	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	14		79.99	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	15		85.70	0	1	33.38	0.884	37.8	150.0	0.00	113.19			0
1	16		91.42	0	1	33.38	0.884	37.8	150.0	0.00	113.19			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

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	33.38	0.884	37.76	171.59	2
1	1	5.71	0	1	33.38	0.884	37.76	171.59	2
1	2	11.43	0	1	33.38	0.884	37.76	171.59	2
1	3	17.14	0	1	33.38	0.884	37.76	171.59	2
1	4	22.85	0	1	33.38	0.884	47.05	171.59	2
1	5	28.57	0	1	33.38	0.884	47.05	171.59	2
1	6	34.28	0	1	33.38	0.884	47.05	171.59	2
1	7	39.99	0	1	33.38	0.884	47.05	171.59	2
1	8	45.71	0	1	33.38	0.884	47.05	171.59	2
1	9	51.42	0	1	33.38	0.884	47.05	171.59	2
1	10	57.14	0	1	33.38	0.884	47.05	171.59	2
1	11	62.85	0	1	33.38	0.884	47.05	171.59	2
1	12	68.56	0	1	33.38	0.884	47.05	171.59	2
1	13	74.28	0	1	33.38	0.884	37.76	171.59	2
1	14	79.99	0	1	33.38	0.884	37.76	171.59	2
1	15	85.70	0	1	33.38	0.884	37.76	171.59	2
1	16	91.42	0	1	33.38	0.884	37.76	171.59	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	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	1 5.71	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	2 11.43	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	3 17.14	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	4 22.85	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	5 28.57	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1	6 34.28	5.28	12.	16.6	5.6	1.570	0.972	

			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	7	39.99	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	8	45.71	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	9	51.42	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	10	57.14	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	11	62.85	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	12	68.56	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	13	74.28	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	14	79.99	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	15	85.70	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	1.00	0
1	16	91.42	5.28	12.	16.6	5.6	1.570	0.972		
			5.28	12.	16.6	5.6	1.570	0.972	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

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

LATERAL BRACING NO	DIST. LEFT FROM SUPT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb

---

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

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

SP NO	IN NO	D L (ft)	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	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0	
1	1	5.71	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0	
1	2	11.43	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0	
1	3	17.14	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0	
1	4	22.85	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0	
1	5	28.57	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0	
1	6	34.28	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0	
1	7	39.99	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0	

1	8	45.71	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0
1	9	51.42	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0
1	10	57.14	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0
1	11	62.85	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0
1	12	68.56	1	5.28	11.44	20.15	0	0.00	10.81	40.60	0	0
1	13	74.28	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	14	79.99	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	15	85.70	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	16	91.42	1	5.28	11.44	20.15	0	0.00	11.02	41.39	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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — COMPOSITE
UNIVERSITY OF MARYLAND — LRF -- 2010
CODE CHECK

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 54\5800\Rating\MD\5

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	32.00	32.00	32.00
1	1		5.71	1	1.000	1.000	1.00	32.00	32.00	32.00
1	2		11.43	1	1.000	1.000	1.00	32.00	32.00	32.00
1	3		17.14	1	1.000	1.000	1.00	32.00	32.00	32.00
1	4		22.85	1	1.000	1.000	1.00	32.00	32.00	32.00
1	5		28.57	1	1.000	1.000	1.00	32.00	32.00	32.00
1	6		34.28	1	1.000	1.000	1.00	32.00	32.00	32.00
1	7		39.99	1	1.000	1.000	1.00	32.00	32.00	32.00
1	8		45.71	1	1.000	1.000	1.00	32.00	32.00	32.00
1	9		51.42	1	1.000	1.000	1.00	32.00	32.00	32.00
1	10		57.14	1	1.000	1.000	1.00	32.00	32.00	32.00
1	11		62.85	1	1.000	1.000	1.00	32.00	32.00	32.00
1	12		68.56	1	1.000	1.000	1.00	32.00	32.00	32.00
1	13		74.28	1	1.000	1.000	1.00	32.00	32.00	32.00
1	14		79.99	1	1.000	1.000	1.00	32.00	32.00	32.00
1	15		85.70	1	1.000	1.000	1.00	32.00	32.00	32.00
1	16		91.42	1	1.000	1.000	1.00	32.00	32.00	32.00

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 L	FROM SUPT (ft)	Comp	Dp (in)	0.42Dt (in)
1	0		0.00	1	8.20	18.28
1	1		5.71	1	8.20	18.28
1	2		11.43	1	8.20	18.28
1	3		17.14	1	8.20	18.28
1	4		22.85	1	14.32	18.91
1	5		28.57	1	14.32	18.91
1	6		34.28	1	14.32	18.91
1	7		39.99	1	14.32	18.91
1	8		45.71	1	14.32	18.91
1	9		51.42	1	14.32	18.91
1	10		57.14	1	14.32	18.91
1	11		62.85	1	14.32	18.91
1	12		68.56	1	14.32	18.91
1	13		74.28	1	8.20	18.28
1	14		79.99	1	8.20	18.28
1	15		85.70	1	8.20	18.28
1	16		91.42	1	8.20	18.28

1

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	0	0	0	0	0
1	1		5.71	1	0	0	0	0	0	0
1	2		11.43	1	0	0	0	0	0	0
1	3		17.14	1	0	0	0	0	0	0
1	4		22.85	1	0	0	0	0	0	0
1	5		28.57	1	0	0	0	0	0	0
1	6		34.28	1	0	0	0	0	0	0
1	7		39.99	1	0	0	0	0	0	0
1	8		45.71	1	0	0	0	0	0	0
1	9		51.42	1	0	0	0	0	0	0
1	10		57.14	1	0	0	0	0	0	0
1	11		62.85	1	0	0	0	0	0	0
1	12		68.56	1	0	0	0	0	0	0
1	13		74.28	1	0	0	0	0	0	0
1	14		79.99	1	0	0	0	0	0	0
1	15		85.70	1	0	0	0	0	0	0
1	16		91.42	1	0	0	0	0	0	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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MERLIN V 10.4
COMPOSITE
LRF -- 2010
CODE CHECK

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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	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	0
1	1		5.71	0.0	19.2	4.4	32.0	4.4	32.0	4.4	32.0	0
				0.0	-	4.4	-	4.4	32.0	-	-	0
1	2		11.43	0.0	19.2	8.2	32.0	8.2	32.0	8.2	32.0	0
				0.0	-	8.2	-	8.2	32.0	-	-	0
1	3		17.14	0.0	19.2	11.5	32.0	11.5	32.0	11.5	32.0	0
				0.0	-	11.5	-	11.5	32.0	-	-	0
1	4		22.85	0.0	19.2	12.9	32.0	12.9	32.0	12.9	32.0	0
				0.0	-	9.1	-	9.1	32.0	-	-	0
1	5		28.57	0.0	19.2	14.8	32.0	14.8	32.0	14.8	32.0	0
				0.0	-	10.4	-	10.4	32.0	-	-	0
1	6		34.28	0.0	19.2	16.1	32.0	16.1	32.0	16.1	32.0	0
				0.0	-	11.3	-	11.3	32.0	-	-	0
1	7		39.99	0.0	19.2	16.9	32.0	16.9	32.0	16.9	32.0	0
				0.0	-	11.9	-	11.9	32.0	-	-	0
1	8		45.71	0.0	19.2	17.2	32.0	17.2	32.0	17.2	32.0	0
				0.0	-	12.1	-	12.1	32.0	-	-	0
1	9		51.42	0.0	19.2	16.9	32.0	16.9	32.0	16.9	32.0	0
				0.0	-	11.9	-	11.9	32.0	-	-	0
1	10		57.14	0.0	19.2	16.1	32.0	16.1	32.0	16.1	32.0	0
				0.0	-	11.3	-	11.3	32.0	-	-	0
1	11		62.85	0.0	19.2	14.8	32.0	14.8	32.0	14.8	32.0	0
				0.0	-	10.4	-	10.4	32.0	-	-	0
1	12		68.56	0.0	19.2	12.9	32.0	12.9	32.0	12.9	32.0	0
				0.0	-	9.1	-	9.1	32.0	-	-	0
1	13		74.28	0.0	19.2	11.5	32.0	11.5	32.0	11.5	32.0	0
				0.0	-	11.5	-	11.5	32.0	-	-	0
1	14		79.99	0.0	19.2	8.2	32.0	8.2	32.0	8.2	32.0	0
				0.0	-	8.2	-	8.2	32.0	-	-	0
1	15		85.70	0.0	19.2	4.4	32.0	4.4	32.0	4.4	32.0	0
				0.0	-	4.4	-	4.4	32.0	-	-	0
1	16		91.42	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	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

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	5.71	0.000	0.138	0.138	0.138	0.138	2
					0.138		0.138	3
1	2	11.43	0.000	0.258	0.258	0.258	0.258	2
					0.258		0.258	3
1	3	17.14	0.000	0.359	0.359	0.359	0.359	2
					0.359		0.359	3
1	4	22.85	0.000	0.403	0.403	0.403	0.403	2
					0.283		0.283	3
1	5	28.57	0.000	0.462	0.462	0.462	0.462	2
					0.324		0.324	3
1	6	34.28	0.000	0.504	0.504	0.504	0.504	2
					0.354		0.354	3
1	7	39.99	0.000	0.529	0.529	0.529	0.529	2
					0.371		0.371	3
1	8	45.71	0.000	0.538	0.538	0.538	0.538	2
					0.377		0.377	3
1	9	51.42	0.000	0.529	0.529	0.529	0.529	2
					0.371		0.371	3
1	10	57.14	0.000	0.504	0.504	0.504	0.504	2
					0.354		0.354	3
1	11	62.85	0.000	0.462	0.462	0.462	0.462	2
					0.324		0.324	3
1	12	68.56	0.000	0.403	0.403	0.403	0.403	2
					0.283		0.283	3
1	13	74.28	0.000	0.359	0.359	0.359	0.359	2
					0.359		0.359	3
1	14	79.99	0.000	0.258	0.258	0.258	0.258	2
					0.258		0.258	3
1	15	85.70	0.000	0.138	0.138	0.138	0.138	2
					0.138		0.138	3
1	16	91.42	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] fbu (ksi)	[2] fl (ksi)	[3] fbu+1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	0.0	4034.8	- -	0.0 19.2	- -	0--0
1	1		5.71	0	1022.0	4034.8	- -	0.0 19.2	- -	0--0
1	2		11.43	0	1901.2	4034.8	- -	0.0 19.2	- -	0--0
1	3		17.14	0	2636.4	4034.8	- -	0.0 19.2	- -	0--0
1	4		22.85	0	3224.8	5398.1	- -	0.0 19.2	- -	0--0
1	5		28.57	0	3680.5	5398.1	- -	0.0 19.2	- -	0--0
1	6		34.28	0	4002.6	5398.1	- -	0.0 19.2	- -	0--0
1	7		39.99	0	4179.0	5398.1	- -	0.0 19.2	- -	0--0
1	8		45.71	0	4235.0	5398.1	- -	0.0 19.2	- -	0--0
1	9		51.42	0	4179.0	5398.1	- -	0.0 19.2	- -	0--0
1	10		57.14	0	4002.6	5398.1	- -	0.0 19.2	- -	0--0
1	11		62.85	0	3680.5	5398.1	- -	0.0 19.2	- -	0--0
1	12		68.56	0	3224.8	5398.1	- -	0.0 19.2	- -	0--0
1	13		74.28	0	2636.5	4034.8	- -	0.0 19.2	- -	0--0
1	14		79.99	0	1901.2	4034.8	- -	0.0 19.2	- -	0--0
1	15		85.70	0	1022.0	4034.8	- -	0.0 19.2	- -	0--0
1	16		91.42	0	0.0	4034.8	- -	0.0 19.2	- -	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 NO	IN NO	D L	FROM SUPT	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	5.71	0	0.253		0.253	1
1	2	11.43	0	0.471		0.471	1
1	3	17.14	0	0.653		0.653	1
1	4	22.85	0	0.597		0.597	1
1	5	28.57	0	0.682		0.682	1
1	6	34.28	0	0.741		0.741	1
1	7	39.99	0	0.774		0.774	1
1	8	45.71	0	0.785		0.785	1
1	9	51.42	0	0.774		0.774	1
1	10	57.14	0	0.741		0.741	1
1	11	62.85	0	0.682		0.682	1
1	12	68.56	0	0.597		0.597	1
1	13	74.28	0	0.653		0.653	1
1	14	79.99	0	0.471		0.471	1
1	15	85.70	0	0.253		0.253	1
1	16	91.42	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	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	1	5.71	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	2	11.43	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	3	17.14	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	4	22.85	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	5	28.57	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	6	34.28	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	7	39.99	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	8	45.71	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	9	51.42	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	10	57.14	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	11	62.85	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	12	68.56	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	13	74.28	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	14	79.99	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	15	85.70	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	16	91.42	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7

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 L SUPT (ft)	STRENGTH CATEGORY; SEE NOTE: WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPACITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	278.9	<	547.7	0	0
1	1	5.71	0	251.6	<	547.7	0	0
1	2	11.43	0	225.7	<	547.7	0	0
1	3	17.14	0	199.7	<	547.7	0	0
1	4	22.85	0	174.5	<	547.7	0	0
1	5	28.57	0	149.7	<	547.7	0	0
1	6	34.28	0	124.9	<	547.7	0	0
1	7	39.99	0	100.8	<	547.7	0	0
1	8	45.71	0	77.2	<	547.7	0	0
1	9	51.42	0	100.8	<	547.7	0	0
1	10	57.14	0	124.9	<	547.7	0	0
1	11	62.85	0	149.7	<	547.7	0	0
1	12	68.56	0	174.5	<	547.7	0	0
1	13	74.28	0	199.7	<	547.7	0	0
1	14	79.99	0	225.7	<	547.7	0	0
1	15	85.70	0	251.6	<	547.7	0	0
1	16	91.42	0	278.9	<	547.7	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 UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF SHEAR CAPACITY (k)	TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	32.	278.91	547.67	0	
1	1	5.71	32.	251.62	547.67	0	
1	2	11.43	32.	225.66	547.67	0	
1	3	17.14	32.	199.66	547.67	0	
1	4	22.85	32.	174.49	547.67	0	
1	5	28.57	32.	149.70	547.67	0	
1	6	34.28	32.	124.86	547.67	0	
1	7	39.99	32.	100.84	547.67	0	
1	8	45.71	32.	77.21	547.67	0	
1	9	51.42	32.	100.84	547.67	0	
1	10	57.14	32.	124.86	547.67	0	

1 11	62.85	32.	149.70	547.67	0
1 12	68.56	32.	174.49	547.67	0
1 13	74.28	32.	199.66	547.67	0
1 14	79.99	32.	225.66	547.67	0
1 15	85.70	32.	251.62	547.67	0
1 16	91.42	32.	278.91	547.67	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] (ksi)	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-		30.4	0.0	30.4	-	- 00-
1	1	5.71	4.7	-		30.4	7.8	30.4	-	- 00-
1	2	11.43	8.8	-		30.4	14.5	30.4	-	- 00-
1	3	17.14	12.3	-		30.4	20.2	30.4	-	- 00-
1	4	22.85	14.1	-		30.4	15.8	30.4	-	- 00-
1	5	28.57	16.2	-		30.4	18.0	30.4	-	- 00-
1	6	34.28	17.6	-		30.4	19.6	30.4	-	- 00-
1	7	39.99	18.4	-		30.4	20.4	30.4	-	- 00-
1	8	45.71	18.7	-		30.4	20.7	30.4	-	- 00-
1	9	51.42	18.4	-		30.4	20.4	30.4	-	- 00-
1	10	57.14	17.6	-		30.4	19.6	30.4	-	- 00-
1	11	62.85	16.2	-		30.4	18.0	30.4	-	- 00-
1	12	68.56	14.1	-		30.4	15.8	30.4	-	- 00-
1	13	74.28	12.3	-		30.4	20.2	30.4	-	- 00-
1	14	79.99	8.8	-		30.4	14.5	30.4	-	- 00-
1	15	85.70	4.7	-		30.4	7.8	30.4	-	- 00-
1	16	91.42	0.0	-		30.4	0.0	30.4	-	- 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 = II -- Urban 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		5.71	TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	2		11.43	TR	-0.9	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	3		17.14	TR	-1.2	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1			17.71	SCG					
1	4		22.85	TR	-1.5	A B B^C^C D E E^	TR	3.0	A B B^C^C D E
1	5		28.57	TR	-1.7	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	6		34.28	TR	-1.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	7		39.99	TR	-1.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	8		45.71	TR	-1.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	9		51.42	TR	-1.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	10		57.14	TR	-1.8	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	11		62.85	TR	-1.7	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	12		68.56	TR	-1.5	A B B^C^C D E E^	TR	3.0	A B B^C^C D E
1			73.71	SCG					
1	13		74.28	TR	-1.2	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	14		79.99	TR	-0.9	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	15		85.70	TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	16		91.42	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 = II -- Urban 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		5.71	TR	-0.2	A B B^C^C D E E^	TR	0.8	A B B^C^C D E E^
1	2		11.43	TR	-0.4	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^

| | | |

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= 2038
*** 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	SHEAR CONN. PER TRANS. SECTION <--Input	[5], input	MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	34.0	.22201E-01	0.8	TR	0.76	3	3.01
1	1	5.71	32.5	.22201E-01	0.7	TR	0.76	3	3.15
1	2	11.43	31.1	.22201E-01	0.7	TR	0.76	3	3.29
1	3	17.14	29.8	.22201E-01	0.7	TR	0.76	3	3.44
1	4	22.85	28.5	.20281E-01	0.6	TR	0.76	3	3.92
1	5	28.57	27.3	.20281E-01	0.6	TR	0.76	3	4.09
1	6	34.28	26.2	.20281E-01	0.5	TR	0.76	3	4.27
1	7	39.99	25.7	.20281E-01	0.5	TR	0.76	3	4.36
1	8	45.71	25.5	.20281E-01	0.5	TR	0.76	3	4.38
1	9	51.42	25.7	.20281E-01	0.5	TR	0.76	3	4.36
1	10	57.14	26.2	.20281E-01	0.5	TR	0.76	3	4.27
1	11	62.85	27.3	.20281E-01	0.6	TR	0.76	3	4.09
1	12	68.56	28.5	.20281E-01	0.6	TR	0.76	3	3.92
1	13	74.28	29.8	.22201E-01	0.7	TR	0.76	3	3.44
1	14	79.99	31.1	.22201E-01	0.7	TR	0.76	3	3.29
1	15	85.70	32.5	.22201E-01	0.7	TR	0.76	3	3.15
1	16	91.42	34.0	.22201E-01	0.8	TR	0.76	3	3.01

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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= 2038 & No. of Cycles= 55790250

*** 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 FROM SUPT (ft)	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k) FATIG. CRI.	NO OF SHEAR CONNECTOR N(PER ZONE)	SEE NOTE <=>	N1	N2	STATUS BLANK=OK CHECK=**
1	8 45.71	1	3331.6	1338.7		276	>	80		
1	16 91.42	1	3331.6	1338.7		276	>	80		

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	45.708	6.000
1	45.708	91.417	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	91.4	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
NG	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 IN D	FROM	UNFACTORED DEAD LOAD MOMENT	UNFACTORED L+I MAX. MOMENT	OPERAT. RATING	LIVE	INVENT. RATING	C A

NO	NO	L (ft)	SUPT CAP. (k-ft) or STRESS	D1; k-ft ksi	D2	FACTOR POS. MAX. STRESS	FACTOR NEG.	LOAD [3]	FACTOR TYPE	T
1	0	0.00	4034.8	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	5.71	4034.8	253.9	88.9	330.8	0.0	8.04	HL-93	6.20	0
1	2	11.43	4034.8	474.4	165.9	613.5	0.0	3.87	HL-93	2.99	0
1	3	17.14	4034.8	660.4	231.1	848.1	0.0	2.52	HL-93	1.94	0
1	4	22.85	5398.1	812.6	284.4	1032.5	0.0	2.86	HL-93	2.20	0
1	5	28.57	5398.1	931.6	325.9	1174.4	0.0	2.38	HL-93	1.84	0
1	6	34.28	5398.1	1015.9	355.5	1274.4	0.0	2.11	HL-93	1.63	0
1	7	39.99	5398.1	1066.6	373.3	1324.6	0.0	1.98	HL-93	1.53	0
1	8	45.71	5398.1	1084.1	379.2	1339.2	0.0	1.94	HL-93	1.50	0
1	9	51.42	5398.1	1066.6	373.3	1324.6	0.0	1.98	HL-93	1.53	0
1	10	57.14	5398.1	1015.9	355.5	1274.4	0.0	2.11	HL-93	1.63	0
1	11	62.85	5398.1	931.6	325.9	1174.4	0.0	2.38	HL-93	1.84	0
1	12	68.56	5398.1	812.6	284.4	1032.5	0.0	2.86	HL-93	2.20	0
1	13	74.28	4034.8	660.4	231.1	848.1	0.0	2.52	HL-93	1.94	0
1	14	79.99	4034.8	474.4	165.9	613.5	0.0	3.87	HL-93	2.99	0
1	15	85.70	4034.8	253.9	88.9	330.8	0.0	8.04	HL-93	6.20	0
1	16	91.42	4034.8	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	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. RATING LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	30.4	0.00 0.00 0.00	0.00 0.00 0.00	0.00 0.00 0.00	9.99	HL-93	7.68
1	1	5.71	30.4	-3.49 3.84 -0.96	3.05 0.00 0.00	0.00 0.00 0.00	8.71	HL-93	6.70
1	2	11.43	30.4	-6.52 7.18 -1.79	5.65 0.00 0.00	0.00 0.00 0.00	4.11	HL-93	3.16
1	3	17.14	30.4	-9.08 9.99 -2.47	7.82 0.00 0.00	0.00 0.00 0.00	2.61	HL-93	2.01

1	4	22.85	30.4	-10.25	7.87	-3.00	6.06	0.00	0.00	3.72	HL-93	2.86
1	5	28.57	30.4	-11.75	9.03	-3.41	6.90	0.00	0.00	3.10	HL-93	2.38
1	6	34.28	30.4	-12.81	9.84	-3.70	7.48	0.00	0.00	2.75	HL-93	2.11
1	7	39.99	30.4	-13.45	10.34	-3.84	7.78	0.00	0.00	2.58	HL-93	1.98
1	8	45.71	30.4	-13.67	10.50	-3.89	7.86	0.00	0.00	2.53	HL-93	1.95
1	9	51.42	30.4	-13.45	10.34	-3.84	7.78	0.00	0.00	2.58	HL-93	1.98
1	10	57.14	30.4	-12.81	9.84	-3.70	7.48	0.00	0.00	2.75	HL-93	2.11
1	11	62.85	30.4	-11.75	9.03	-3.41	6.90	0.00	0.00	3.10	HL-93	2.38
1	12	68.56	30.4	-10.25	7.87	-3.00	6.06	0.00	0.00	3.72	HL-93	2.86
1	13	74.28	30.4	-9.08	9.99	-2.47	7.82	0.00	0.00	2.61	HL-93	2.01
1	14	79.99	30.4	-6.52	7.18	-1.79	5.65	0.00	0.00	4.11	HL-93	3.16
1	15	85.70	30.4	-3.49	3.84	-0.96	3.05	0.00	0.00	8.71	HL-93	6.70
1	16	91.42	30.4	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	91.42	45.71	STRENGTH L.S.	1.94	1.50	HL-93
1	91.42	45.71	SERVICEBILITY	2.53	1.95	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 1.940
Inventory= 1.496

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	91.42	45.71	STRENGTH L.S.	2.56	1367.62
1	91.42	74.28	SERVICEBILITY	3.15	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 2.564

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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	OPERAT. RATING FACTOR MAX. SHEAR POS.	OPERAT. RATING FACTOR SHEAR NEG.	[3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	547.7	48.0	16.6	111.7	0.0	3.08	HL-93	2.38
1	1	5.71	547.7	41.5	14.5	102.4	-3.1	3.44	HL-93	2.65
1	2	11.43	547.7	35.7	12.4	93.4	-7.1	3.85	HL-93	2.97
1	3	17.14	547.7	29.5	10.4	84.6	-11.7	4.34	HL-93	3.35
1	4	22.85	547.7	23.7	8.3	76.1	-16.5	4.93	HL-93	3.80
1	5	28.57	547.7	17.9	6.2	67.7	-22.3	5.65	HL-93	4.36
1	6	34.28	547.7	11.8	4.1	59.6	-29.3	6.55	HL-93	5.05
1	7	39.99	547.7	6.0	2.1	51.7	-36.5	7.70	HL-93	5.94
1	8	45.71	547.7	0.2	0.0	44.0	-44.0	9.22	HL-93	7.11
1	9	51.42	547.7	-6.0	-2.1	36.5	-51.7	7.70	HL-93	5.94
1	10	57.14	547.7	-11.8	-4.1	29.3	-59.6	6.55	HL-93	5.05
1	11	62.85	547.7	-17.9	-6.2	22.3	-67.7	5.65	HL-93	4.36
1	12	68.56	547.7	-23.7	-8.3	16.5	-76.1	4.93	HL-93	3.80
1	13	74.28	547.7	-29.5	-10.4	11.7	-84.6	4.34	HL-93	3.35
1	14	79.99	547.7	-35.7	-12.4	7.1	-93.4	3.85	HL-93	2.97
1	15	85.70	547.7	-41.5	-14.5	3.1	-102.4	3.44	HL-93	2.65
1	16	91.42	547.7	-48.0	-16.6	0.0	-111.7	3.08	HL-93	2.38

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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	91.42	91.42	STRENGTH L.S.	3.08	2.38	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.079
Inventory= 2.375

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

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

NO.	LENGTH (ft)	D FROM L SUPT (ft)	CRITERION	RATING	LL SHEAR (kips)
1	91.42	91.42	STRENGTH L.S.	3.63	127.74

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 3.633

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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	45.71	2 0.400	-0.68	HL-93	2.03
		3 0.510	-0.86		1.59

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. 5800 Span 3 Interior Beam	7/17/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 3(6)	IAS/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	2	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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INPUT
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TABLE 0.0.3.1 STRUCTURAL DETAILS
****

BEAM NUMBER	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
5	1	31.33	2.50	1.83	0.00	0.00	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
41.62	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
7.50	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 DEPTH ID.	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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	36	150		0.00	0.0000	0.00	0.0000		

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	MEMB END	MEMBER DESCRIPTN	PARAMETERS FOR	YIELD STRESS

NUMBER (IN ORDER)	SECT LEFT	SECT RIGHT	ID	LNGTH (ft)	NONPRISMATIC MEMB →TYPE← 0=PRISMAT	S(0)	S(1)	(KSI) WEB	TOP	BOT
1	1	1		41.62				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	2	2398	2038	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 2398

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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 IMP FCTR	DF M-	DF M+	DF M-	IMP F	A	D	M	G	C	DF M+ LOADING TYPES

NO STR/SER (%)	ST/SE	FA	FA	FA (%)	NO=0 ;YES= 1	ST/SE	A D M G C
1 33.00	0	0.00	0.00	0.00	0.00	1 1 1 1 1	0.75	2 2 2 2 2
1 0.00	0	0.00	0.00	0.00	0.00	0 0 0 0 0	0.83	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.

TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n	MODULAR RATIO N2=n	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	DECK+haunch	0.0	7.0	0	30.0	10.0	0.69	0.00	41.62

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.

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

LOAD NO.	IDENTIFICATION TYPE	UNIFROM LOAD DATA INTENSITY (k/ft)	UNIFROM LOAD DATA POSITION FROM (ft)	UNIFROM LOAD DATA POSITION TO (ft)	CONCENTRATED LOAD DATA INTENSITY (Kips)	CONCENTRATED LOAD DATA DISTANCE FROM L SUPT (ft)

1	0	wearing surf	0.341	0.00	41.62	0.00	0.00
2	0	end diaphrag	0.000	0.00	0.00	0.76	0.45
3	0	int diaphrag	0.000	0.00	0.00	0.18	2.13
4	0	int diaphrag	0.000	0.00	0.00	0.35	17.13
5	0	int diaphrag	0.000	0.00	0.00	0.18	32.13
6	0	end diaphrag	0.000	0.00	0.00	0.76	41.17

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 YIELD Fy (ksi)	BAR AREA PER FOOT OF SLAB (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
0	0.000	0	0.00	0.0	0.0	0.0	0.00	0.00	2.50	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)

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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	DISTANCE		

FROM (ft)	TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	2.13	33.0	33.0	1.68
2.13	32.13	33.0	33.0	15.00
32.13	41.17	33.0	33.0	9.04

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 2.2.1.1=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

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TABLE 2.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)

1	0.691	0.1504	0.8414

NONCOMPOSITE SUPERIMPOSED DEAD LOADS

LOAD	INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	0.341	0.000	41.625

TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES

CONCRETE DECK				STEEL	SUPERSTRUCTURE TOTAL WEIGHT
	VOLUME		TOTAL	TOTAL	
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	WEIGHT (kip)	WEIGHT (kip)	(kip)
150.00	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.760	0.452
2	0.176	2.131
3	0.352	17.131
4	0.176	32.131
5	0.760	41.171

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

TABLE 2.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
****

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.750	0.750	0.534	0.534	FOR STRENGTH POSITIVE MOMENT
	0.833				FOR STRENGTH POSITIVE SHEAR
1	0.339				FOR FATIGUE POSITIVE MOMENT
	0.550				FOR FATIGUE POSITIVE SHEAR

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TABLE 2.2.4.1=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)	STEEL TOP. (in**3)
1	0	0.00	9040.0	35.9	17.95	503.6	503.6
1	1	2.08	9040.0	35.9	17.95	503.6	503.6
1	2	4.16	9040.0	35.9	17.95	503.6	503.6
1	3	6.24	9040.0	35.9	17.95	503.6	503.6
1	4	8.32	9040.0	35.9	17.95	503.6	503.6
1	5	10.41	9040.0	35.9	17.95	503.6	503.6
1	6	12.49	9040.0	35.9	17.95	503.6	503.6
1	7	14.57	9040.0	35.9	17.95	503.6	503.6
1	8	16.65	9040.0	35.9	17.95	503.6	503.6
1	9	18.73	9040.0	35.9	17.95	503.6	503.6
1	10	20.81	9040.0	35.9	17.95	503.6	503.6
1	11	22.89	9040.0	35.9	17.95	503.6	503.6
1	12	24.97	9040.0	35.9	17.95	503.6	503.6
1	13	27.06	9040.0	35.9	17.95	503.6	503.6
1	14	29.14	9040.0	35.9	17.95	503.6	503.6
1	15	31.22	9040.0	35.9	17.95	503.6	503.6
1	16	33.30	9040.0	35.9	17.95	503.6	503.6
1	17	35.38	9040.0	35.9	17.95	503.6	503.6
1	18	37.46	9040.0	35.9	17.95	503.6	503.6
1	19	39.54	9040.0	35.9	17.95	503.6	503.6
1	20	41.62	9040.0	35.9	17.95	503.6	503.6

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TABLE 2.2.5.1=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	2.08	6.2	28.4	1.2	14.0	49.9	

1	2	4.16	11.7	53.9	1.7	26.6	93.9
1	3	6.24	16.6	76.3	2.2	37.7	132.8
1	4	8.32	20.8	95.8	2.7	47.3	166.6
1	5	10.41	24.4	112.2	3.2	55.4	195.3
1	6	12.49	27.4	125.7	3.7	62.0	218.8
1	7	14.57	29.6	136.2	4.2	67.2	237.2
1	8	16.65	31.3	143.7	4.7	70.9	250.5
1	9	18.73	32.2	148.2	4.6	73.1	258.1
1	10	20.81	32.6	149.7	4.4	73.9	260.5
1	11	22.89	32.2	148.2	4.1	73.1	257.7
1	12	24.97	31.3	143.7	3.9	70.9	249.8
1	13	27.06	29.6	136.2	3.7	67.2	236.7
1	14	29.14	27.4	125.7	3.4	62.0	218.5
1	15	31.22	24.4	112.2	3.2	55.4	195.3
1	16	33.30	20.8	95.8	2.8	47.3	166.7
1	17	35.38	16.6	76.3	2.2	37.7	132.8
1	18	37.46	11.7	53.9	1.6	26.6	93.7
1	19	39.54	6.2	28.4	0.9	14.0	49.6
1	20	41.62	0.0	0.0	0.0	0.0	0.0

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TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (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	2.08	0.0	0.0	128.2	HL-93	0.0	HL-93
1	2	4.16	0.0	0.0	239.4	HL-93	0.0	HL-93
1	3	6.24	0.0	0.0	333.6	HL-93	0.0	HL-93
1	4	8.32	0.0	0.0	410.8	HL-93	0.0	HL-93
1	5	10.41	0.0	0.0	468.1	HL-93	0.0	HL-93
1	6	12.49	0.0	0.0	514.0	HL-93	0.0	HL-93
1	7	14.57	0.0	0.0	549.9	HL-93	0.0	HL-93
1	8	16.65	0.0	0.0	571.5	HL-93	0.0	HL-93
1	9	18.73	0.0	0.0	571.9	HL-93	0.0	HL-93
1	10	20.81	0.0	0.0	573.1	HL-93	0.0	HL-93
1	11	22.89	0.0	0.0	571.9	HL-93	0.0	HL-93
1	12	24.97	0.0	0.0	571.5	HL-93	0.0	HL-93
1	13	27.06	0.0	0.0	549.9	HL-93	0.0	HL-93
1	14	29.14	0.0	0.0	514.0	HL-93	0.0	HL-93
1	15	31.22	0.0	0.0	468.1	HL-93	0.0	HL-93
1	16	33.30	0.0	0.0	410.8	HL-93	0.0	HL-93
1	17	35.38	0.0	0.0	333.6	HL-93	0.0	HL-93
1	18	37.46	0.0	0.0	239.4	HL-93	0.0	HL-93
1	19	39.54	0.0	0.0	128.2	HL-93	0.0	HL-93
1	20	41.62	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 2.2.5.2A=FATIGUE LIVE LOAD MOMENT RANGE (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE

1	0	0.00	0.	0.	0.
1	1	2.08	31.	0.	31.
1	2	4.16	56.	0.	56.
1	3	6.24	74.	0.	74.
1	4	8.32	91.	0.	91.
1	5	10.41	109.	0.	109.
1	6	12.49	123.	0.	123.
1	7	14.57	132.	0.	132.
1	8	16.65	138.	0.	138.
1	9	18.73	140.	0.	140.
1	10	20.81	141.	0.	141.
1	11	22.89	140.	0.	140.
1	12	24.97	138.	0.	138.
1	13	27.06	132.	0.	132.
1	14	29.14	123.	0.	123.
1	15	31.22	109.	0.	109.
1	16	33.30	91.	0.	91.
1	17	35.38	74.	0.	74.
1	18	37.46	56.	0.	56.
1	19	39.54	31.	0.	31.
1	20	41.62	0.	0.	0.

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TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0	0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.08	49.9	128.2	HL-93	0.0	HL-93	178.1	49.9
1	2	4.16	93.9	239.4	HL-93	0.0	HL-93	333.3	93.9
1	3	6.24	132.8	333.6	HL-93	0.0	HL-93	466.4	132.8
1	4	8.32	166.6	410.8	HL-93	0.0	HL-93	577.4	166.6
1	5	10.41	195.3	468.1	HL-93	0.0	HL-93	663.4	195.3
1	6	12.49	218.8	514.0	HL-93	0.0	HL-93	732.8	218.8
1	7	14.57	237.2	549.9	HL-93	0.0	HL-93	787.1	237.2
1	8	16.65	250.5	571.5	HL-93	0.0	HL-93	822.0	250.5
1	9	18.73	258.1	571.9	HL-93	0.0	HL-93	830.0	258.1
1	10	20.81	260.5	573.1	HL-93	0.0	HL-93	833.6	260.5
1	11	22.89	257.7	571.9	HL-93	0.0	HL-93	829.6	257.7
1	12	24.97	249.8	571.5	HL-93	0.0	HL-93	821.2	249.8
1	13	27.06	236.7	549.9	HL-93	0.0	HL-93	786.6	236.7
1	14	29.14	218.5	514.0	HL-93	0.0	HL-93	732.6	218.5
1	15	31.22	195.3	468.1	HL-93	0.0	HL-93	663.4	195.3
1	16	33.30	166.7	410.8	HL-93	0.0	HL-93	577.4	166.7
1	17	35.38	132.8	333.6	HL-93	0.0	HL-93	466.4	132.8
1	18	37.46	93.7	239.4	HL-93	0.0	HL-93	333.2	93.7
1	19	39.54	49.6	128.2	HL-93	0.0	HL-93	177.8	49.6
1	20	41.62	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE 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	2.08	178.1	216.6	286.7	235.4	74.8	23.0
1	2	4.16	333.3	405.2	536.4	440.6	140.9	42.1
1	3	6.24	466.4	566.5	749.9	616.4	199.2	55.6
1	4	8.32	577.4	700.6	927.1	762.8	249.9	68.5
1	5	10.41	663.4	803.8	1063.3	876.0	292.9	82.1
1	6	12.49	732.8	887.0	1173.0	967.4	328.2	92.5
1	7	14.57	787.1	952.1	1258.8	1038.9	355.8	99.4
1	8	16.65	822.0	993.5	1313.3	1084.7	375.8	103.8
1	9	18.73	830.0	1001.6	1323.5	1094.7	387.2	105.0
1	10	20.81	833.6	1005.5	1328.5	1099.3	390.7	105.4
1	11	22.89	829.6	1001.1	1322.9	1094.1	386.5	105.0
1	12	24.97	821.2	992.7	1312.3	1083.7	374.6	103.8
1	13	27.06	786.6	951.5	1258.2	1038.2	355.1	99.4
1	14	29.14	732.6	886.8	1172.7	967.1	327.8	92.5
1	15	31.22	663.4	803.8	1063.3	876.0	292.9	82.1
1	16	33.30	577.4	700.7	927.2	762.9	250.0	68.5
1	17	35.38	466.4	566.5	749.8	616.3	199.1	55.6
1	18	37.46	333.2	405.0	536.2	440.4	140.6	42.1
1	19	39.54	177.8	216.3	286.4	235.1	74.4	23.0
1	20	41.62	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 2.2.6.1=NONCOMPOSITE DEAD LOAD SHEARS FOR N=INFINITY (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k)	DEAD LOAD SLAB (k)	CONCENTRATED LOADS (k)	UNIFORM LOADS (k)	TOTAL NONCOMPOSITE DEAD LOAD (k)
1	0	0.00	3.13	14.38	1.2	7.10	25.78
1	1	2.08	2.82	12.94	0.4	6.39	22.56
1	2	4.16	2.50	11.51	0.2	5.68	19.93
1	3	6.24	2.19	10.07	0.2	4.97	17.46
1	4	8.32	1.88	8.63	0.2	4.26	15.00
1	5	10.41	1.57	7.19	0.2	3.55	12.54
1	6	12.49	1.25	5.75	0.2	2.84	10.08
1	7	14.57	0.94	4.31	0.2	2.13	7.62
1	8	16.65	0.63	2.88	0.2	1.42	5.16
1	9	18.73	0.31	1.44	-0.1	0.71	2.35
1	10	20.81	0.00	0.00	-0.1	0.00	-0.11
1	11	22.89	-0.31	-1.44	-0.1	-0.71	-2.57
1	12	24.97	-0.63	-2.88	-0.1	-1.42	-5.04
1	13	27.06	-0.94	-4.31	-0.1	-2.13	-7.50
1	14	29.14	-1.25	-5.75	-0.1	-2.84	-9.96
1	15	31.22	-1.57	-7.19	-0.1	-3.55	-12.42
1	16	33.30	-1.88	-8.63	-0.3	-4.26	-15.05
1	17	35.38	-2.19	-10.07	-0.3	-4.97	-17.52
1	18	37.46	-2.50	-11.51	-0.3	-5.68	-19.98
1	19	39.54	-2.82	-12.94	-0.3	-6.39	-22.44
1	20	41.62	-3.13	-14.38	-1.0	-7.10	-25.66

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TABLE 2.2.6.2=LIVE LOAD SHEARS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

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****

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	72.96	HL-93	0.00	HL-93
1	1	2.08	0.0	0.0	67.89	HL-93	-1.80	HL-93
1	2	4.16	0.0	0.0	62.87	HL-93	-3.65	HL-93
1	3	6.24	0.0	0.0	57.91	HL-93	-5.88	HL-93
1	4	8.32	0.0	0.0	53.01	HL-93	-8.85	HL-93
1	5	10.41	0.0	0.0	48.08	HL-93	-11.87	HL-93
1	6	12.49	0.0	0.0	43.37	HL-93	-14.94	HL-93
1	7	14.57	0.0	0.0	38.92	HL-93	-18.07	HL-93
1	8	16.65	0.0	0.0	34.60	HL-93	-21.26	HL-93
1	9	18.73	0.0	0.0	31.15	HL-93	-24.50	HL-93
1	10	20.81	0.0	0.0	27.80	HL-93	-27.80	HL-93
1	11	22.89	0.0	0.0	24.50	HL-93	-31.15	HL-93
1	12	24.97	0.0	0.0	21.26	HL-93	-34.60	HL-93
1	13	27.06	0.0	0.0	18.07	HL-93	-38.92	HL-93
1	14	29.14	0.0	0.0	14.94	HL-93	-43.37	HL-93
1	15	31.22	0.0	0.0	11.87	HL-93	-48.08	HL-93
1	16	33.30	0.0	0.0	8.85	HL-93	-53.01	HL-93
1	17	35.38	0.0	0.0	5.88	HL-93	-57.91	HL-93
1	18	37.46	0.0	0.0	3.65	HL-93	-62.87	HL-93
1	19	39.54	0.0	0.0	1.80	HL-93	-67.89	HL-93
1	20	41.62	0.0	0.0	0.00	HL-93	-72.96	HL-93

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TABLE 2.2.6.2A=LIVE LOAD SHEAR RANGE (in kips) (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	26.	0.	26.
1	1	2.08	24.	-1.	25.
1	2	4.16	22.	-2.	24.
1	3	6.24	20.	-3.	23.
1	4	8.32	18.	-4.	22.
1	5	10.41	17.	-5.	22.
1	6	12.49	16.	-6.	22.
1	7	14.57	15.	-7.	22.
1	8	16.65	13.	-8.	22.
1	9	18.73	12.	-10.	22.
1	10	20.81	11.	-11.	22.
1	11	22.89	10.	-12.	22.
1	12	24.97	8.	-13.	22.
1	13	27.06	7.	-15.	22.
1	14	29.14	6.	-16.	22.
1	15	31.22	5.	-17.	22.
1	16	33.30	4.	-18.	22.
1	17	35.38	3.	-20.	23.
1	18	37.46	2.	-22.	24.
1	19	39.54	1.	-24.	25.
1	20	41.62	0.	-26.	26.

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TABLE 2.2.6.2B=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	25.9	.20618E-01	0.27	0.3125*	.20618E-01	0.27	0.3125*
1	1		2.08	24.9	.20618E-01	0.26	0.3125*	.20618E-01	0.26	0.3125*
1	2		4.16	23.9	.20618E-01	0.25	0.3125*	.20618E-01	0.25	0.3125*
1	3		6.24	22.6	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	4		8.32	21.8	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	5		10.41	22.0	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	6		12.49	22.1	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	7		14.57	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	8		16.65	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	9		18.73	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	10		20.81	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	11		22.89	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	12		24.97	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	13		27.06	21.9	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	14		29.14	22.1	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	15		31.22	22.0	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	16		33.30	21.8	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	17		35.38	22.6	.20618E-01	0.23	0.3125*	.20618E-01	0.23	0.3125*
1	18		37.46	23.9	.20618E-01	0.25	0.3125*	.20618E-01	0.25	0.3125*
1	19		39.54	24.9	.20618E-01	0.26	0.3125*	.20618E-01	0.26	0.3125*
1	20		41.62	25.9	.20618E-01	0.27	0.3125*	.20618E-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 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	GOVERN TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0		0.00	25.8	73.0	HL-93	0.0	HL-93	98.7	25.8
1	1		2.08	22.6	67.9	HL-93	-1.8	HL-93	90.4	20.8

1	2	4.16	19.9	62.9	HL-93	-3.6	HL-93	82.8	16.3
1	3	6.24	17.5	57.9	HL-93	-5.9	HL-93	75.4	11.6
1	4	8.32	15.0	53.0	HL-93	-8.8	HL-93	68.0	6.2
1	5	10.41	12.5	48.1	HL-93	-11.9	HL-93	60.6	0.7
1	6	12.49	10.1	43.4	HL-93	-14.9	HL-93	53.4	-4.9
1	7	14.57	7.6	38.9	HL-93	-18.1	HL-93	46.5	-10.5
1	8	16.65	5.2	34.6	HL-93	-21.3	HL-93	39.8	-16.1
1	9	18.73	2.3	31.1	HL-93	-24.5	HL-93	33.5	-22.2
1	10	20.81	-0.1	27.8	HL-93	-27.8	HL-93	-27.9	27.7
1	11	22.89	-2.6	24.5	HL-93	-31.1	HL-93	-33.7	21.9
1	12	24.97	-5.0	21.3	HL-93	-34.6	HL-93	-39.6	16.2
1	13	27.06	-7.5	18.1	HL-93	-38.9	HL-93	-46.4	10.6
1	14	29.14	-10.0	14.9	HL-93	-43.4	HL-93	-53.3	5.0
1	15	31.22	-12.4	11.9	HL-93	-48.1	HL-93	-60.5	-0.6
1	16	33.30	-15.1	8.8	HL-93	-53.0	HL-93	-68.1	-6.2
1	17	35.38	-17.5	5.9	HL-93	-57.9	HL-93	-75.4	-11.6
1	18	37.46	-20.0	3.6	HL-93	-62.9	HL-93	-82.8	-16.3
1	19	39.54	-22.4	1.8	HL-93	-67.9	HL-93	-90.3	-20.6
1	20	41.62	-25.7	0.0	HL-93	-73.0	HL-93	-98.6	-25.7

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TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE 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	98.7	120.6	159.9	130.7	38.7	19.4
1	1	2.08	90.4	110.8	147.0	119.9	33.8	18.7
1	2	4.16	82.8	101.7	134.9	109.8	29.9	17.9
1	3	6.24	75.4	92.8	123.2	100.0	26.2	16.9
1	4	8.32	68.0	83.9	111.5	90.3	22.5	16.4
1	5	10.41	60.6	75.0	99.8	80.6	18.8	16.5
1	6	12.49	53.4	66.5	88.5	71.1	15.1	16.5
1	7	14.57	46.5	58.2	77.6	62.1	11.4	16.5
1	8	16.65	39.8	50.1	67.0	53.2	7.7	16.4
1	9	18.73	33.5	42.8	57.4	45.0	3.5	16.4
1	10	20.81	-27.9	-36.2	-48.8	-37.7	-0.2	16.4
1	11	22.89	-33.7	-43.1	-57.7	-45.3	-3.9	16.4
1	12	24.97	-39.6	-50.0	-66.8	-53.0	-7.6	16.4
1	13	27.06	-46.4	-58.1	-77.5	-61.9	-11.2	16.5
1	14	29.14	-53.3	-66.3	-88.3	-71.0	-14.9	16.5
1	15	31.22	-60.5	-74.9	-99.7	-80.4	-18.6	16.5
1	16	33.30	-68.1	-84.0	-111.6	-90.4	-22.6	16.4
1	17	35.38	-75.4	-92.8	-123.2	-100.1	-26.3	16.9
1	18	37.46	-82.8	-101.7	-135.0	-109.8	-30.0	17.9
1	19	39.54	-90.3	-110.7	-146.8	-119.7	-33.7	18.7
1	20	41.62	-98.6	-120.5	-159.7	-130.6	-38.5	19.4

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TABLE 2.2.7.2=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	25.78	0.00	HL-93	72.96	HL-93	ST1	23.20	159.90
						ST2	23.20	130.72
						ST4	38.67	38.67
						SE1	25.78	98.74
						SE2	25.78	120.63
2	25.66	0.00	HL-93	72.96	HL-93	ST1	23.09	159.75
						ST2	23.09	130.57
						ST4	38.49	38.49
						SE1	25.66	98.62
						SE2	25.66	120.50

NOTE: [1] " - " Indicates Uplift

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

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TABLE 2.2.8.1=DEAD LOAD DEFLECTIONS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (in)	DEAD LOAD SLAB (in)	CONCENTRATED LOADS (in)	UNIFORM LOADS (in)	T O T A L +=UP , -=DOWN
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.08	-0.0062	-0.0283	-0.0008	-0.0140	-0.0493
1	2	4.16	-0.0122	-0.0559	-0.0017	-0.0276	-0.0973
1	3	6.24	-0.0178	-0.0819	-0.0024	-0.0404	-0.1426
1	4	8.32	-0.0230	-0.1057	-0.0031	-0.0522	-0.1840
1	5	10.41	-0.0276	-0.1269	-0.0037	-0.0626	-0.2208
1	6	12.49	-0.0315	-0.1448	-0.0043	-0.0714	-0.2520
1	7	14.57	-0.0346	-0.1591	-0.0047	-0.0785	-0.2769
1	8	16.65	-0.0369	-0.1695	-0.0050	-0.0837	-0.2951
1	9	18.73	-0.0383	-0.1759	-0.0052	-0.0868	-0.3062
1	10	20.81	-0.0388	-0.1780	-0.0052	-0.0879	-0.3098
1	11	22.89	-0.0383	-0.1759	-0.0051	-0.0868	-0.3061
1	12	24.97	-0.0369	-0.1695	-0.0049	-0.0837	-0.2950
1	13	27.06	-0.0346	-0.1591	-0.0046	-0.0785	-0.2768
1	14	29.14	-0.0315	-0.1448	-0.0042	-0.0714	-0.2519
1	15	31.22	-0.0276	-0.1269	-0.0037	-0.0626	-0.2207
1	16	33.30	-0.0230	-0.1057	-0.0031	-0.0522	-0.1840
1	17	35.38	-0.0178	-0.0819	-0.0024	-0.0404	-0.1425
1	18	37.46	-0.0122	-0.0559	-0.0016	-0.0276	-0.0972
1	19	39.54	-0.0062	-0.0283	-0.0008	-0.0140	-0.0493
1	20	41.62	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " indicates downward deflection

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TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (FT)	NONCOMPOSITE DEAD LOADS STEEL DEFL.	CAMBER (IN) SIZE	SLAB DEFL.	CAMBER (IN) SIZE	OTHER DL DEFL.	CAMBER (IN) SIZE	T O T A L DEFL.	CAMBER (IN) SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	2.1	-0.006	0 1/16	-0.028	0 1/16	-0.015	0 1/16	-0.049	0 1/16
1	2	4.2	-0.012	0 1/16	-0.056	0 1/16	-0.029	0 1/16	-0.097	0 1/ 8
1	3	6.2	-0.018	0 1/16	-0.082	0 1/ 8	-0.043	0 1/16	-0.143	0 3/16
1	4	8.3	-0.023	0 1/16	-0.106	0 1/ 8	-0.055	0 1/16	-0.184	0 3/16

1	5	10.4	-0.028	0	1/16	-0.127	0	3/16	-0.066	0	1/ 8	-0.221	0	1/ 4
1	6	12.5	-0.032	0	1/16	-0.145	0	3/16	-0.076	0	1/ 8	-0.252	0	5/16
1	7	14.6	-0.035	0	1/16	-0.159	0	3/16	-0.083	0	1/ 8	-0.277	0	5/16
1	8	16.6	-0.037	0	1/16	-0.170	0	3/16	-0.089	0	1/ 8	-0.295	0	5/16
1	9	18.7	-0.038	0	1/16	-0.176	0	3/16	-0.092	0	1/ 8	-0.306	0	5/16
1	10	20.8	-0.039	0	1/16	-0.178	0	3/16	-0.093	0	1/ 8	-0.310	0	5/16
1	11	22.9	-0.038	0	1/16	-0.176	0	3/16	-0.092	0	1/ 8	-0.306	0	5/16
1	12	25.0	-0.037	0	1/16	-0.170	0	3/16	-0.089	0	1/ 8	-0.295	0	5/16
1	13	27.1	-0.035	0	1/16	-0.159	0	3/16	-0.083	0	1/ 8	-0.277	0	5/16
1	14	29.1	-0.032	0	1/16	-0.145	0	3/16	-0.076	0	1/ 8	-0.252	0	5/16
1	15	31.2	-0.028	0	1/16	-0.127	0	3/16	-0.066	0	1/ 8	-0.221	0	1/ 4
1	16	33.3	-0.023	0	1/16	-0.106	0	1/ 8	-0.055	0	1/16	-0.184	0	3/16
1	17	35.4	-0.018	0	1/16	-0.082	0	1/ 8	-0.043	0	1/16	-0.143	0	3/16
1	18	37.5	-0.012	0	1/16	-0.056	0	1/16	-0.029	0	1/16	-0.097	0	1/ 8
1	19	39.5	-0.006	0	1/16	-0.028	0	1/16	-0.015	0	1/16	-0.049	0	1/16
1	20	41.6	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 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****
(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	20.81	2 0.400	-0.288 MAX	HL-93	0.62	0.00350
			0.038 MIN			
			-0.066 MAX	LANE		
			0.038 MIN	LANE		
		3 0.510	-0.368 MAX	HL-93	0.62	0.00350
			0.048 MIN			
			-0.084 MAX	LANE		
			0.048 MIN	LANE		
		2 0.400	-0.189 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 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	N.COMP DEAD LOAD STEEL BEAM TOP (ksi)	N.COMP 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	2.08	-0.83	0.83	-0.36	0.36	-1.19	1.19
1	2	4.16	-1.56	1.56	-0.67	0.67	-2.24	2.24
1	3	6.24	-2.21	2.21	-0.95	0.95	-3.16	3.16
1	4	8.32	-2.78	2.78	-1.19	1.19	-3.97	3.97
1	5	10.41	-3.26	3.26	-1.40	1.40	-4.65	4.65
1	6	12.49	-3.65	3.65	-1.57	1.57	-5.21	5.21
1	7	14.57	-3.95	3.95	-1.70	1.70	-5.65	5.65
1	8	16.65	-4.17	4.17	-1.80	1.80	-5.97	5.97
1	9	18.73	-4.30	4.30	-1.85	1.85	-6.15	6.15
1	10	20.81	-4.34	4.34	-1.86	1.86	-6.21	6.21
1	11	22.89	-4.30	4.30	-1.84	1.84	-6.14	6.14
1	12	24.97	-4.17	4.17	-1.78	1.78	-5.95	5.95
1	13	27.06	-3.95	3.95	-1.69	1.69	-5.64	5.64
1	14	29.14	-3.65	3.65	-1.56	1.56	-5.21	5.21
1	15	31.22	-3.26	3.26	-1.40	1.40	-4.65	4.65
1	16	33.30	-2.78	2.78	-1.19	1.19	-3.97	3.97
1	17	35.38	-2.21	2.21	-0.95	0.95	-3.16	3.16
1	18	37.46	-1.56	1.56	-0.67	0.67	-2.23	2.23
1	19	39.54	-0.83	0.83	-0.36	0.36	-1.18	1.18
1	20	41.62	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 2.2.9.2=LIVE + I STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	0.00	0.00	0.00	0.00
1	1	2.08	-3.06	3.06	0.00	0.00
1	2	4.16	-5.71	5.71	0.00	0.00
1	3	6.24	-7.95	7.95	0.00	0.00
1	4	8.32	-9.79	9.79	0.00	0.00
1	5	10.41	-11.15	11.15	0.00	0.00
1	6	12.49	-12.25	12.25	0.00	0.00
1	7	14.57	-13.10	13.10	0.00	0.00
1	8	16.65	-13.62	13.62	0.00	0.00
1	9	18.73	-13.63	13.63	0.00	0.00
1	10	20.81	-13.66	13.66	0.00	0.00
1	11	22.89	-13.63	13.63	0.00	0.00
1	12	24.97	-13.62	13.62	0.00	0.00
1	13	27.06	-13.10	13.10	0.00	0.00
1	14	29.14	-12.25	12.25	0.00	0.00
1	15	31.22	-11.15	11.15	0.00	0.00
1	16	33.30	-9.79	9.79	0.00	0.00
1	17	35.38	-7.95	7.95	0.00	0.00
1	18	37.46	-5.71	5.71	0.00	0.00
1	19	39.54	-3.06	3.06	0.00	0.00
1	20	41.62	0.00	0.00	0.00	0.00

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TABLE 2.2.9.2A=FATIGUE LIVE LOAD STRESS RANGE (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	2.08	0.7	0.0	0.7	C	HL	0.7	0.0	0.7	T	HL
1	2	4.16	1.3	0.0	1.3	C	HL	1.3	0.0	1.3	T	HL
1	3	6.24	1.8	0.0	1.8	C	HL	1.8	0.0	1.8	T	HL
1	4	8.32	2.2	0.0	2.2	C	HL	2.2	0.0	2.2	T	HL
1	5	10.41	2.6	0.0	2.6	C	HL	2.6	0.0	2.6	T	HL
1	6	12.49	2.9	0.0	2.9	C	HL	2.9	0.0	2.9	T	HL
1	7	14.57	3.2	0.0	3.2	C	HL	3.2	0.0	3.2	T	HL
1	8	16.65	3.3	0.0	3.3	C	HL	3.3	0.0	3.3	T	HL
1	9	18.73	3.3	0.0	3.3	C	HL	3.3	0.0	3.3	T	HL
1	10	20.81	3.3	0.0	3.3	C	HL	3.3	0.0	3.3	T	HL
1	11	22.89	3.3	0.0	3.3	C	HL	3.3	0.0	3.3	T	HL
1	12	24.97	3.3	0.0	3.3	C	HL	3.3	0.0	3.3	T	HL
1	13	27.06	3.2	0.0	3.2	C	HL	3.2	0.0	3.2	T	HL
1	14	29.14	2.9	0.0	2.9	C	HL	2.9	0.0	2.9	T	HL
1	15	31.22	2.6	0.0	2.6	C	HL	2.6	0.0	2.6	T	HL
1	16	33.30	2.2	0.0	2.2	C	HL	2.2	0.0	2.2	T	HL
1	17	35.38	1.8	0.0	1.8	C	HL	1.8	0.0	1.8	T	HL
1	18	37.46	1.3	0.0	1.3	C	HL	1.3	0.0	1.3	T	HL
1	19	39.54	0.7	0.0	0.7	C	HL	0.7	0.0	0.7	T	HL
1	20	41.62	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 2.2.9.3A=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	MAXIMUM POSITIVE,(ksi) TOP ------STEEL-----	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP -----STEEL------	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-4.24	4.24	-1.19	1.19
1	2	4.16	33.	33.	-7.94	7.94	-2.24	2.24
1	3	6.24	33.	33.	-11.11	11.11	-3.16	3.16
1	4	8.32	33.	33.	-13.76	13.76	-3.97	3.97
1	5	10.41	33.	33.	-15.81	15.81	-4.65	4.65
1	6	12.49	33.	33.	-17.46	17.46	-5.21	5.21
1	7	14.57	33.	33.	-18.75	18.75	-5.65	5.65
1	8	16.65	33.	33.	-19.59	19.59	-5.97	5.97
1	9	18.73	33.	33.	-19.78	19.78	-6.15	6.15
1	10	20.81	33.	33.	-19.86	19.86	-6.21	6.21
1	11	22.89	33.	33.	-19.77	19.77	-6.14	6.14
1	12	24.97	33.	33.	-19.57	19.57	-5.95	5.95

1	13	27.06	33.	33.	-18.74	18.74	-5.64	5.64
1	14	29.14	33.	33.	-17.46	17.46	-5.21	5.21
1	15	31.22	33.	33.	-15.81	15.81	-4.65	4.65
1	16	33.30	33.	33.	-13.76	13.76	-3.97	3.97
1	17	35.38	33.	33.	-11.11	11.11	-3.16	3.16
1	18	37.46	33.	33.	-7.94	7.94	-2.23	2.23
1	19	39.54	33.	33.	-4.24	4.24	-1.18	1.18
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-5.16	5.16	-1.19	1.19
1	2	4.16	33.	33.	-9.65	9.65	-2.24	2.24
1	3	6.24	33.	33.	-13.50	13.50	-3.16	3.16
1	4	8.32	33.	33.	-16.69	16.69	-3.97	3.97
1	5	10.41	33.	33.	-19.15	19.15	-4.65	4.65
1	6	12.49	33.	33.	-21.14	21.14	-5.21	5.21
1	7	14.57	33.	33.	-22.69	22.69	-5.65	5.65
1	8	16.65	33.	33.	-23.67	23.67	-5.97	5.97
1	9	18.73	33.	33.	-23.87	23.87	-6.15	6.15
1	10	20.81	33.	33.	-23.96	23.96	-6.21	6.21
1	11	22.89	33.	33.	-23.85	23.85	-6.14	6.14
1	12	24.97	33.	33.	-23.65	23.65	-5.95	5.95
1	13	27.06	33.	33.	-22.67	22.67	-5.64	5.64
1	14	29.14	33.	33.	-21.13	21.13	-5.21	5.21
1	15	31.22	33.	33.	-19.15	19.15	-4.65	4.65
1	16	33.30	33.	33.	-16.70	16.70	-3.97	3.97
1	17	35.38	33.	33.	-13.50	13.50	-3.16	3.16
1	18	37.46	33.	33.	-9.65	9.65	-2.23	2.23
1	19	39.54	33.	33.	-5.15	5.15	-1.18	1.18
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-6.83	6.83	-1.49	1.49
1	2	4.16	33.	33.	-12.78	12.78	-2.80	2.80
1	3	6.24	33.	33.	-17.87	17.87	-3.96	3.96
1	4	8.32	33.	33.	-22.09	22.09	-4.96	4.96
1	5	10.41	33.	33.	-25.34	25.34	-5.82	5.82
1	6	12.49	33.	33.	-27.95	27.95	-6.52	6.52
1	7	14.57	33.	33.	-29.99	29.99	-7.07	7.07
1	8	16.65	33.	33.	-31.29	31.29	-7.46	7.46
1	9	18.73	33.	33.	-31.54	31.54	-7.69	7.69
1	10	20.81	33.	33.	-31.66	31.66	-7.76	7.76
1	11	22.89	33.	33.	-31.52	31.52	-7.67	7.67
1	12	24.97	33.	33.	-31.27	31.27	-7.44	7.44
1	13	27.06	33.	33.	-29.98	29.98	-7.05	7.05
1	14	29.14	33.	33.	-27.94	27.94	-6.51	6.51
1	15	31.22	33.	33.	-25.34	25.34	-5.82	5.82
1	16	33.30	33.	33.	-22.09	22.09	-4.96	4.96
1	17	35.38	33.	33.	-17.87	17.87	-3.95	3.95
1	18	37.46	33.	33.	-12.78	12.78	-2.79	2.79
1	19	39.54	33.	33.	-6.82	6.82	-1.48	1.48
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-6.42	6.42	-1.07	1.07
1	2	4.16	33.	33.	-12.00	12.00	-2.01	2.01
1	3	6.24	33.	33.	-16.76	16.76	-2.85	2.85
1	4	8.32	33.	33.	-20.70	20.70	-3.57	3.57
1	5	10.41	33.	33.	-23.71	23.71	-4.19	4.19
1	6	12.49	33.	33.	-26.13	26.13	-4.69	4.69
1	7	14.57	33.	33.	-28.02	28.02	-5.09	5.09
1	8	16.65	33.	33.	-29.20	29.20	-5.37	5.37
1	9	18.73	33.	33.	-29.38	29.38	-5.54	5.54
1	10	20.81	33.	33.	-29.48	29.48	-5.59	5.59
1	11	22.89	33.	33.	-29.37	29.37	-5.53	5.53
1	12	24.97	33.	33.	-29.19	29.19	-5.36	5.36
1	13	27.06	33.	33.	-28.00	28.00	-5.08	5.08
1	14	29.14	33.	33.	-26.12	26.12	-4.69	4.69
1	15	31.22	33.	33.	-23.71	23.71	-4.19	4.19
1	16	33.30	33.	33.	-20.70	20.70	-3.57	3.57
1	17	35.38	33.	33.	-16.76	16.76	-2.85	2.85
1	18	37.46	33.	33.	-11.99	11.99	-2.01	2.01
1	19	39.54	33.	33.	-6.41	6.41	-1.06	1.06
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-5.61	5.61	-1.49	1.49
1	2	4.16	33.	33.	-10.50	10.50	-2.80	2.80
1	3	6.24	33.	33.	-14.69	14.69	-3.96	3.96
1	4	8.32	33.	33.	-18.18	18.18	-4.96	4.96
1	5	10.41	33.	33.	-20.87	20.87	-5.82	5.82
1	6	12.49	33.	33.	-23.05	23.05	-6.52	6.52
1	7	14.57	33.	33.	-24.75	24.75	-7.07	7.07
1	8	16.65	33.	33.	-25.84	25.84	-7.46	7.46
1	9	18.73	33.	33.	-26.08	26.08	-7.69	7.69
1	10	20.81	33.	33.	-26.19	26.19	-7.76	7.76
1	11	22.89	33.	33.	-26.07	26.07	-7.67	7.67
1	12	24.97	33.	33.	-25.82	25.82	-7.44	7.44
1	13	27.06	33.	33.	-24.74	24.74	-7.05	7.05
1	14	29.14	33.	33.	-23.04	23.04	-6.51	6.51
1	15	31.22	33.	33.	-20.87	20.87	-5.82	5.82
1	16	33.30	33.	33.	-18.18	18.18	-4.96	4.96
1	17	35.38	33.	33.	-14.69	14.69	-3.95	3.95
1	18	37.46	33.	33.	-10.49	10.49	-2.79	2.79
1	19	39.54	33.	33.	-5.60	5.60	-1.48	1.48
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00

1	1	2.08	33.	33.	-5.19	5.19	-1.07	1.07
1	2	4.16	33.	33.	-9.72	9.72	-2.01	2.01
1	3	6.24	33.	33.	-13.58	13.58	-2.85	2.85
1	4	8.32	33.	33.	-16.79	16.79	-3.57	3.57
1	5	10.41	33.	33.	-19.25	19.25	-4.19	4.19
1	6	12.49	33.	33.	-21.23	21.23	-4.69	4.69
1	7	14.57	33.	33.	-22.77	22.77	-5.09	5.09
1	8	16.65	33.	33.	-23.76	23.76	-5.37	5.37
1	9	18.73	33.	33.	-23.93	23.93	-5.54	5.54
1	10	20.81	33.	33.	-24.02	24.02	-5.59	5.59
1	11	22.89	33.	33.	-23.92	23.92	-5.53	5.53
1	12	24.97	33.	33.	-23.74	23.74	-5.36	5.36
1	13	27.06	33.	33.	-22.76	22.76	-5.08	5.08
1	14	29.14	33.	33.	-21.22	21.22	-4.69	4.69
1	15	31.22	33.	33.	-19.25	19.25	-4.19	4.19
1	16	33.30	33.	33.	-16.79	16.79	-3.57	3.57
1	17	35.38	33.	33.	-13.58	13.58	-2.85	2.85
1	18	37.46	33.	33.	-9.71	9.71	-2.01	2.01
1	19	39.54	33.	33.	-5.19	5.19	-1.06	1.06
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

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TABLE 2.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY
****

MEMBER		TOP FLANGE		WEB		BOT FLANGE		COVER PLATE -- (in) TOP		COVER PLATE -- (in) BOT	
NO	LENGTH (ft)	STEEL TYPE	WIDTH (in)	THICK	DEPTH (in)	THICK	WIDTH (in)	THICK	WIDTH	THICK	WIDTH	THICK
1	41.62	W	12.0	0.9390	34.0	0.6250	12.0	0.9390	0.00	0.0000	0.00	0.0000

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

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TABLE 2.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 41.6

MEMBER NO	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	41.62	35.90	[2] does not apply			
			[3]= 35.90	0.07187	> 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	34.02	0.625	54.4	150.0	0.00	111.46			0
1	1		2.08	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	2		4.16	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	3		6.24	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	4		8.32	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	5		10.41	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	6		12.49	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	7		14.57	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	8		16.65	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	9		18.73	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	10		20.81	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	11		22.89	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	12		24.97	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	13		27.06	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	14		29.14	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	15		31.22	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	16		33.30	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	17		35.38	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	18		37.46	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	19		39.54	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	20		41.62	0	1	34.02	0.625	54.4	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	34.02	0.625	54.44	168.97	2
1	1		2.08	0	1	34.02	0.625	54.44	168.97	2
1	2		4.16	0	1	34.02	0.625	54.44	168.97	2

1	3	6.24	0	1	34.02	0.625	54.44	168.97	2
1	4	8.32	0	1	34.02	0.625	54.44	168.97	2
1	5	10.41	0	1	34.02	0.625	54.44	168.97	2
1	6	12.49	0	1	34.02	0.625	54.44	168.97	2
1	7	14.57	0	1	34.02	0.625	54.44	168.97	2
1	8	16.65	0	1	34.02	0.625	54.44	168.97	2
1	9	18.73	0	1	34.02	0.625	54.44	168.97	2
1	10	20.81	0	1	34.02	0.625	54.44	168.97	2
1	11	22.89	0	1	34.02	0.625	54.44	168.97	2
1	12	24.97	0	1	34.02	0.625	54.44	168.97	2
1	13	27.06	0	1	34.02	0.625	54.44	168.97	2
1	14	29.14	0	1	34.02	0.625	54.44	168.97	2
1	15	31.22	0	1	34.02	0.625	54.44	168.97	2
1	16	33.30	0	1	34.02	0.625	54.44	168.97	2
1	17	35.38	0	1	34.02	0.625	54.44	168.97	2
1	18	37.46	0	1	34.02	0.625	54.44	168.97	2
1	19	39.54	0	1	34.02	0.625	54.44	168.97	2
1	20	41.62	0	1	34.02	0.625	54.44	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 2.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2]	tf (in)	[3]	Iyc/Iyt	FLAG
1 0	0.00	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 1	2.08	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 2	4.16	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 3	6.24	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 4	8.32	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 5	10.41	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 6	12.49	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 7	14.57	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 8	16.65	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 9	18.73	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 10	20.81	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 11	22.89	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 12	24.97	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 13	27.06	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 14	29.14	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 15	31.22	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0

1	16	33.30	6.39	12.	12.0	5.7	0.939	0.688	
			6.39	12.	12.0	5.7	0.939	0.688	1.00	0
1	17	35.38	6.39	12.	12.0	5.7	0.939	0.688	
			6.39	12.	12.0	5.7	0.939	0.688	1.00	0
1	18	37.46	6.39	12.	12.0	5.7	0.939	0.688	
			6.39	12.	12.0	5.7	0.939	0.688	1.00	0
1	19	39.54	6.39	12.	12.0	5.7	0.939	0.688	
			6.39	12.	12.0	5.7	0.939	0.688	1.00	0
1	20	41.62	6.39	12.	12.0	5.7	0.939	0.688	
			6.39	12.	12.0	5.7	0.939	0.688	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 SUPT FROM (FT)	TO (FT)	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb

---

Note: f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

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

SP NO	IN NO	D L	FROM SUPT (ft)	Comp	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	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	1		2.08	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	2		4.16	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	3		6.24	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	4		8.32	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	5		10.41	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	6		12.49	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	7		14.57	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	8		16.65	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	9		18.73	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	10		20.81	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	11		22.89	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	12		24.97	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	13		27.06	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	14		29.14	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	15		31.22	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	16		33.30	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	17		35.38	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	18		37.46	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	19		39.54	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	20		41.62	1	6.39	11.26	19.84	0	0.00	7.46	28.03	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	33.00	33.00
1	1		2.08	1	1.000	1.000	1.00	33.00	33.00	33.00
1	2		4.16	1	1.000	1.000	1.00	33.00	33.00	33.00
1	3		6.24	1	1.000	1.000	1.00	33.00	33.00	33.00
1	4		8.32	1	1.000	1.000	1.00	33.00	33.00	33.00
1	5		10.41	1	1.000	1.000	1.00	33.00	33.00	33.00
1	6		12.49	1	1.000	1.000	1.00	33.00	33.00	33.00
1	7		14.57	1	1.000	1.000	1.00	33.00	33.00	33.00
1	8		16.65	1	1.000	1.000	1.00	33.00	33.00	33.00
1	9		18.73	1	1.000	1.000	1.00	33.00	33.00	33.00
1	10		20.81	1	1.000	1.000	1.00	33.00	33.00	33.00
1	11		22.89	1	1.000	1.000	1.00	33.00	33.00	33.00
1	12		24.97	1	1.000	1.000	1.00	33.00	33.00	33.00
1	13		27.06	1	1.000	1.000	1.00	33.00	33.00	33.00
1	14		29.14	1	1.000	1.000	1.00	33.00	33.00	33.00
1	15		31.22	1	1.000	1.000	1.00	33.00	33.00	33.00
1	16		33.30	1	1.000	1.000	1.00	33.00	33.00	33.00
1	17		35.38	1	1.000	1.000	1.00	33.00	33.00	33.00
1	18		37.46	1	1.000	1.000	1.00	33.00	33.00	33.00
1	19		39.54	1	1.000	1.000	1.00	33.00	33.00	33.00
1	20		41.62	1	1.000	1.000	1.00	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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BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING
UNIVERSITY OF MARYLAND

MERLIN V 10.4
NONCOMPOSITE
LRF -- 2010
INPUT

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 43\5800\Rating\MD\5

TABLE 2.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION
****

	STRENGTH CATEGORY,	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.
1	0	0.00	1	2	0			2
1	1	2.08	1	2	0			2
1	2	4.16	1	2	0			2
1	3	6.24	1	2	0			2
1	4	8.32	1	2	0			2
1	5	10.41	1	2	0			2
1	6	12.49	1	2	0			2
1	7	14.57	1	2	0			2
1	8	16.65	1	2	0			2
1	9	18.73	1	2	0			2
1	10	20.81	1	2	0			2
1	11	22.89	1	2	0			2
1	12	24.97	1	2	0			2
1	13	27.06	1	2	0			2
1	14	29.14	1	2	0			2
1	15	31.22	1	2	0			2
1	16	33.30	1	2	0			2
1	17	35.38	1	2	0			2
1	18	37.46	1	2	0			2
1	19	39.54	1	2	0			2
1	20	41.62	1	2	0			2

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

TABLE 2.2.22.10=CONSTRUCBILITY CHECK (STRENGTH IV)
****

SP NO	IN NO	D FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1]	fbu+fl (ksi)	[2]	fbu+1/3fl (ksi)	[3]	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	2.08	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	2	4.16	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	3	6.24	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	4	8.32	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	5	10.41	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	6	12.49	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	7	14.57	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	8	16.65	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	9	18.73	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	10	20.81	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	11	22.89	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	12	24.97	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	13	27.06	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	14	29.14	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
1	15	31.22	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	16	33.30	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	17	35.38	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	18	37.46	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	19	39.54	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	20	41.62	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

TABLE 2.2.22.10A=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		2.08	0.000	0.000	0.000	0.000 0.000		1

					0.000			0.000	3
1	2	4.16	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	3	6.24	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	4	8.32	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	5	10.41	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	6	12.49	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	7	14.57	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	8	16.65	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	9	18.73	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	10	20.81	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	11	22.89	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	12	24.97	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	13	27.06	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	14	29.14	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	15	31.22	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	16	33.30	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	17	35.38	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	18	37.46	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	19	39.54	0.000	0.000	0.000	0.000	0.000		1
					0.000			0.000	3
1	20	41.62	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 2.2.22.14=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	-	0.0	19.8	0.0	33.0	-0--
					-	-	0.0	-	0.0	19.8	0.0	33.0	--0-
1	1		2.08	0	-	-	6.8	-	0.0	19.8	6.8	33.0	-0--
					-	-	6.8	-	0.0	19.8	6.8	33.0	--0-
1	2		4.16	0	-	-	12.8	-	0.0	19.8	12.8	33.0	-0--
					-	-	12.8	-	0.0	19.8	12.8	33.0	--0-
1	3		6.24	0	-	-	17.9	-	0.0	19.8	17.9	33.0	-0--
					-	-	17.9	-	0.0	19.8	17.9	33.0	--0-
1	4		8.32	0	-	-	22.1	-	0.0	19.8	22.1	33.0	-0--
					-	-	22.1	-	0.0	19.8	22.1	33.0	--0-

1	5	10.41	0	-	-	25.3	-	0.0	19.8	25.3	33.0	-0--
				-	-	25.3	-	0.0	19.8	25.3	33.0	--0-
1	6	12.49	0	-	-	28.0	-	0.0	19.8	28.0	33.0	-0--
				-	-	28.0	-	0.0	19.8	28.0	33.0	--0-
1	7	14.57	0	-	-	30.0	-	0.0	19.8	30.0	33.0	-0--
				-	-	30.0	-	0.0	19.8	30.0	33.0	--0-
1	8	16.65	0	-	-	31.3	-	0.0	19.8	31.3	33.0	-0--
				-	-	31.3	-	0.0	19.8	31.3	33.0	--0-
1	9	18.73	0	-	-	31.5	-	0.0	19.8	31.5	33.0	-0--
				-	-	31.5	-	0.0	19.8	31.5	33.0	--0-
1	10	20.81	0	-	-	31.7	-	0.0	19.8	31.7	33.0	-0--
				-	-	31.7	-	0.0	19.8	31.7	33.0	--0-
1	11	22.89	0	-	-	31.5	-	0.0	19.8	31.5	33.0	-0--
				-	-	31.5	-	0.0	19.8	31.5	33.0	--0-
1	12	24.97	0	-	-	31.3	-	0.0	19.8	31.3	33.0	-0--
				-	-	31.3	-	0.0	19.8	31.3	33.0	--0-
1	13	27.06	0	-	-	30.0	-	0.0	19.8	30.0	33.0	-0--
				-	-	30.0	-	0.0	19.8	30.0	33.0	--0-
1	14	29.14	0	-	-	27.9	-	0.0	19.8	27.9	33.0	-0--
				-	-	27.9	-	0.0	19.8	27.9	33.0	--0-
1	15	31.22	0	-	-	25.3	-	0.0	19.8	25.3	33.0	-0--
				-	-	25.3	-	0.0	19.8	25.3	33.0	--0-
1	16	33.30	0	-	-	22.1	-	0.0	19.8	22.1	33.0	-0--
				-	-	22.1	-	0.0	19.8	22.1	33.0	--0-
1	17	35.38	0	-	-	17.9	-	0.0	19.8	17.9	33.0	-0--
				-	-	17.9	-	0.0	19.8	17.9	33.0	--0-
1	18	37.46	0	-	-	12.8	-	0.0	19.8	12.8	33.0	-0--
				-	-	12.8	-	0.0	19.8	12.8	33.0	--0-
1	19	39.54	0	-	-	6.8	-	0.0	19.8	6.8	33.0	-0--
				-	-	6.8	-	0.0	19.8	6.8	33.0	--0-
1	20	41.62	0	-	-	0.0	-	0.0	19.8	0.0	33.0	-0--
				-	-	0.0	-	0.0	19.8	0.0	33.0	--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 2.2.22.14A=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	0.000	4
						0.000	0.000	0.000	4
1	1	2.08	0			0.000	0.207	0.207	4
						0.000	0.207	0.207	4
1	2	4.16	0			0.000	0.387	0.387	4
						0.000	0.387	0.387	4
1	3	6.24	0			0.000	0.541	0.541	4
						0.000	0.541	0.541	4
1	4	8.32	0			0.000	0.669	0.669	4
						0.000	0.669	0.669	4
1	5	10.41	0			0.000	0.768	0.768	4
						0.000	0.768	0.768	4
1	6	12.49	0			0.000	0.847	0.847	4
						0.000	0.847	0.847	4
1	7	14.57	0			0.000	0.909	0.909	4
						0.000	0.909	0.909	4
1	8	16.65	0			0.000	0.948	0.948	4
						0.000	0.948	0.948	4
1	9	18.73	0			0.000	0.956	0.956	4
						0.000	0.956	0.956	4
1	10	20.81	0			0.000	0.959	0.959	4
						0.000	0.959	0.959	4
1	11	22.89	0			0.000	0.955	0.955	4
						0.000	0.955	0.955	4
1	12	24.97	0			0.000	0.948	0.948	4
						0.000	0.948	0.948	4
1	13	27.06	0			0.000	0.908	0.908	4
						0.000	0.908	0.908	4
1	14	29.14	0			0.000	0.847	0.847	4
						0.000	0.847	0.847	4
1	15	31.22	0			0.000	0.768	0.768	4
						0.000	0.768	0.768	4
1	16	33.30	0			0.000	0.669	0.669	4
						0.000	0.669	0.669	4
1	17	35.38	0			0.000	0.541	0.541	4
						0.000	0.541	0.541	4
1	18	37.46	0			0.000	0.387	0.387	4
						0.000	0.387	0.387	4
1	19	39.54	0			0.000	0.207	0.207	4
						0.000	0.207	0.207	4
1	20	41.62	0			0.000	0.000	0.000	4
						0.000	0.000	0.000	4

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

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	54.4	1.000	407.0	407.0
1	1	2.08	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	2	4.16	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	3	6.24	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	4	8.32	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	5	10.41	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	6	12.49	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	7	14.57	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	8	16.65	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	9	18.73	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	10	20.81	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	11	22.89	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	12	24.97	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	13	27.06	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	14	29.14	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	15	31.22	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	16	33.30	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	17	35.38	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	18	37.46	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	19	39.54	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	20	41.62	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0

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 2.2.22.16=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	OVERALL WEB
1	0	0.00	2	159.9	<	407.0	2	2
1	1	2.08	2	147.0	<	407.0	2	2
1	2	4.16	2	134.9	<	407.0	2	2
1	3	6.24	2	123.2	<	407.0	2	2
1	4	8.32	2	111.5	<	407.0	2	2
1	5	10.41	2	99.8	<	407.0	2	2
1	6	12.49	2	88.5	<	407.0	2	2
1	7	14.57	2	77.6	<	407.0	2	2
1	8	16.65	2	67.0	<	407.0	2	2
1	9	18.73	2	57.4	<	407.0	2	2
1	10	20.81	2	48.8	<	407.0	2	2
1	11	22.89	2	57.7	<	407.0	2	2
1	12	24.97	2	66.8	<	407.0	2	2
1	13	27.06	2	77.5	<	407.0	2	2
1	14	29.14	2	88.3	<	407.0	2	2
1	15	31.22	2	99.7	<	407.0	2	2
1	16	33.30	2	111.6	<	407.0	2	2
1	17	35.38	2	123.2	<	407.0	2	2
1	18	37.46	2	135.0	<	407.0	2	2
1	19	39.54	2	146.8	<	407.0	2	2
1	20	41.62	2	159.7	<	407.0	2	2

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 2.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF SHEAR CAPACITY (k)	TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	33.	159.90	406.99	0	
1	1	2.08	33.	147.01	406.99	0	
1	2	4.16	33.	134.93	406.99	0	
1	3	6.24	33.	123.18	406.99	0	
1	4	8.32	33.	111.52	406.99	0	
1	5	10.41	33.	99.82	406.99	0	
1	6	12.49	33.	88.50	406.99	0	
1	7	14.57	33.	77.64	406.99	0	
1	8	16.65	33.	66.99	406.99	0	
1	9	18.73	33.	57.44	406.99	0	
1	10	20.81	33.	48.79	406.99	0	
1	11	22.89	33.	57.73	406.99	0	
1	12	24.97	33.	66.84	406.99	0	
1	13	27.06	33.	77.49	406.99	0	
1	14	29.14	33.	88.34	406.99	0	
1	15	31.22	33.	99.66	406.99	0	
1	16	33.30	33.	111.58	406.99	0	
1	17	35.38	33.	123.24	406.99	0	
1	18	37.46	33.	135.00	406.99	0	
1	19	39.54	33.	146.85	406.99	0	
1	20	41.62	33.	159.75	406.99	0	

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TABLE 2.2.22.21=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)	FLG fcrw [3] (ksi)	FLAG TBC
1	0	0.00	-	0.0	26.4	0.0	26.4	0.0	33.0	000
1	1	2.08	-	5.2	26.4	5.2	26.4	5.2	33.0	000
1	2	4.16	-	9.7	26.4	9.7	26.4	9.7	33.0	000
1	3	6.24	-	13.5	26.4	13.5	26.4	13.5	33.0	000
1	4	8.32	-	16.7	26.4	16.7	26.4	16.7	33.0	000
1	5	10.41	-	19.2	26.4	19.2	26.4	19.2	33.0	000
1	6	12.49	-	21.1	26.4	21.1	26.4	21.1	33.0	000
1	7	14.57	-	22.7	26.4	22.7	26.4	22.7	33.0	000
1	8	16.65	-	23.7	26.4	23.7	26.4	23.7	33.0	000
1	9	18.73	-	23.9	26.4	23.9	26.4	23.9	33.0	000
1	10	20.81	-	24.0	26.4	24.0	26.4	24.0	33.0	000
1	11	22.89	-	23.9	26.4	23.9	26.4	23.9	33.0	000
1	12	24.97	-	23.7	26.4	23.7	26.4	23.7	33.0	000
1	13	27.06	-	22.7	26.4	22.7	26.4	22.7	33.0	000
1	14	29.14	-	21.1	26.4	21.1	26.4	21.1	33.0	000

1	15	31.22	-	19.2	26.4	19.2	26.4	19.2	33.0 000
1	16	33.30	-	16.7	26.4	16.7	26.4	16.7	33.0 000
1	17	35.38	-	13.5	26.4	13.5	26.4	13.5	33.0 000
1	18	37.46	-	9.7	26.4	9.7	26.4	9.7	33.0 000
1	19	39.54	-	5.2	26.4	5.2	26.4	5.2	33.0 000
1	20	41.62	-	0.0	26.4	0.0	26.4	0.0	33.0 000

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = II -- Urban Interstate

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. LOADING	TOP OF TOP FLANGE STRESS RANGE (ksi)	TOP OF TOP FLANGE ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. LOADING	BOTTOM OF BOTTOM FLANGE STRESS RANGE (ksi)	BOTTOM OF BOTTOM FLANGE 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		2.08	TR	-1.1	A B B^C^C D E E^	TR	1.1	A B B^C^C D E E^
1	2		4.16	TR	-2.0	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	3		6.24	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	4		8.32	TR	-3.3	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1	5		10.41	TR	-3.9	A B B^C^C D E E^	TR	3.9	A B B^C^C D E
1	6		12.49	TR	-4.4	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	7		14.57	TR	-4.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	8		16.65	TR	-4.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	9		18.73	TR	-5.0	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	10		20.81	TR	-5.0	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	11		22.89	TR	-5.0	A B B^C^C D E E^	TR	5.0	A B B^C^C D
1	12		24.97	TR	-4.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	13		27.06	TR	-4.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	14		29.14	TR	-4.4	A B B^C^C D E E^	TR	4.4	A B B^C^C D E
1	15		31.22	TR	-3.9	A B B^C^C D E E^	TR	3.9	A B B^C^C D E
1	16		33.30	TR	-3.3	A B B^C^C D E E^	TR	3.3	A B B^C^C D E
1	17		35.38	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	18		37.46	TR	-2.0	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^

| | | |

S:P = Point where SCG coincides with POC

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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 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT

SP NO	IN NO	D L (ft)	FROM SUPT or STRESS	MOMENT CAP. (k-ft)		UNFACTORED DEAD LOAD MOMENT D1; k-ft D2 ksi	UNFACTORED L+I MAX. MOMENT POS.	UNFACTORED L+I MAX. MOMENT NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00		33.00	TOP	0.00	0.00	0.00	9.99	HL-93	9.99	0
				33.00	BOT	0.00	0.00	0.00	9.99	HL-93	9.99	0
1	1	2.08		33.00	TOP	-1.19	-3.06	0.00	7.64	HL-93	5.89	0
				33.00	BOT	1.19	3.06	0.00	7.64	HL-93	5.89	0
1	2	4.16		33.00	TOP	-2.24	-5.71	0.00	3.92	HL-93	3.03	0
				33.00	BOT	2.24	5.71	0.00	3.92	HL-93	3.03	0
1	3	6.24		33.00	TOP	-3.16	-7.95	0.00	2.71	HL-93	2.09	0
				33.00	BOT	3.16	7.95	0.00	2.71	HL-93	2.09	0
1	4	8.32		33.00	TOP	-3.97	-9.79	0.00	2.12	HL-93	1.64	0
				33.00	BOT	3.97	9.79	0.00	2.12	HL-93	1.64	0
1	5	10.41		33.00	TOP	-4.65	-11.15	0.00	1.81	HL-93	1.39	0
				33.00	BOT	4.65	11.15	0.00	1.81	HL-93	1.39	0
1	6	12.49		33.00	TOP	-5.21	-12.25	0.00	1.60	HL-93	1.24	0
				33.00	BOT	5.21	12.25	0.00	1.60	HL-93	1.24	0
1	7	14.57		33.00	TOP	-5.65	-13.10	0.00	1.47	HL-93	1.13	0
				33.00	BOT	5.65	13.10	0.00	1.47	HL-93	1.13	0
1	8	16.65		33.00	TOP	-5.97	-13.62	0.00	1.39	HL-93	1.07	0
				33.00	BOT	5.97	13.62	0.00	1.39	HL-93	1.07	0
1	9	18.73		33.00	TOP	-6.15	-13.63	0.00	1.38	HL-93	1.06	0
				33.00	BOT	6.15	13.63	0.00	1.38	HL-93	1.06	0
1	10	20.81		33.00	TOP	-6.21	-13.66	0.00	1.37	HL-93	1.06	0
				33.00	BOT	6.21	13.66	0.00	1.37	HL-93	1.06	0
1	11	22.89		33.00	TOP	-6.14	-13.63	0.00	1.38	HL-93	1.06	0

		33.00	BOT	6.14	13.63	0.00	1.38	HL-93	1.06	0
1 12	24.97	33.00	TOP	-5.95	-13.62	0.00	1.39	HL-93	1.07	0
		33.00	BOT	5.95	13.62	0.00	1.39	HL-93	1.07	0
1 13	27.06	33.00	TOP	-5.64	-13.10	0.00	1.47	HL-93	1.13	0
		33.00	BOT	5.64	13.10	0.00	1.47	HL-93	1.13	0
1 14	29.14	33.00	TOP	-5.21	-12.25	0.00	1.60	HL-93	1.24	0
		33.00	BOT	5.21	12.25	0.00	1.60	HL-93	1.24	0
1 15	31.22	33.00	TOP	-4.65	-11.15	0.00	1.81	HL-93	1.39	0
		33.00	BOT	4.65	11.15	0.00	1.81	HL-93	1.39	0
1 16	33.30	33.00	TOP	-3.97	-9.79	0.00	2.12	HL-93	1.64	0
		33.00	BOT	3.97	9.79	0.00	2.12	HL-93	1.64	0
1 17	35.38	33.00	TOP	-3.16	-7.95	0.00	2.71	HL-93	2.09	0
		33.00	BOT	3.16	7.95	0.00	2.71	HL-93	2.09	0
1 18	37.46	33.00	TOP	-2.23	-5.71	0.00	3.92	HL-93	3.03	0
		33.00	BOT	2.23	5.71	0.00	3.92	HL-93	3.03	0
1 19	39.54	33.00	TOP	-1.18	-3.06	0.00	7.64	HL-93	5.90	0
		33.00	BOT	1.18	3.06	0.00	7.64	HL-93	5.90	0
1 20	41.62	33.00	TOP	0.00	0.00	0.00	9.99	HL-93	9.99	0
		33.00	BOT	0.00	0.00	0.00	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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MERLIN V 10.4
NONCOMPOSITE
LRF -- 2010
RATING

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 61\5800\Rating\MD\5

TABLE 2.2.32.2=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	26.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	2.08	26.4	-1.19 1.19	-3.06 3.06	0.00 0.00	8.25	HL-93	6.35
1	2	4.16	26.4	-2.24 2.24	-5.71 5.71	0.00 0.00	4.24	HL-93	3.26
1	3	6.24	26.4	-3.16 3.16	-7.95 7.95	0.00 0.00	2.92	HL-93	2.25
1	4	8.32	26.4	-3.97 3.97	-9.79 9.79	0.00 0.00	2.29	HL-93	1.76
1	5	10.41	26.4	-4.65 4.65	-11.15 11.15	0.00 0.00	1.95	HL-93	1.50
1	6	12.49	26.4	-5.21 5.21	-12.25 12.25	0.00 0.00	1.73	HL-93	1.33

1	7	14.57	26.4	-5.65	5.65	-13.10	13.10	0.00	0.00	1.58	HL-93	1.22
1	8	16.65	26.4	-5.97	5.97	-13.62	13.62	0.00	0.00	1.50	HL-93	1.15
1	9	18.73	26.4	-6.15	6.15	-13.63	13.63	0.00	0.00	1.49	HL-93	1.14
1	10	20.81	26.4	-6.21	6.21	-13.66	13.66	0.00	0.00	1.48	HL-93	1.14
1	11	22.89	26.4	-6.14	6.14	-13.63	13.63	0.00	0.00	1.49	HL-93	1.14
1	12	24.97	26.4	-5.95	5.95	-13.62	13.62	0.00	0.00	1.50	HL-93	1.16
1	13	27.06	26.4	-5.64	5.64	-13.10	13.10	0.00	0.00	1.58	HL-93	1.22
1	14	29.14	26.4	-5.21	5.21	-12.25	12.25	0.00	0.00	1.73	HL-93	1.33
1	15	31.22	26.4	-4.65	4.65	-11.15	11.15	0.00	0.00	1.95	HL-93	1.50
1	16	33.30	26.4	-3.97	3.97	-9.79	9.79	0.00	0.00	2.29	HL-93	1.76
1	17	35.38	26.4	-3.16	3.16	-7.95	7.95	0.00	0.00	2.92	HL-93	2.25
1	18	37.46	26.4	-2.23	2.23	-5.71	5.71	0.00	0.00	4.24	HL-93	3.26
1	19	39.54	26.4	-1.18	1.18	-3.06	3.06	0.00	0.00	8.25	HL-93	6.35
1	20	41.62	26.4	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 2.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING CRITERION	RATING OPERATING RATING	RATING INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	41.62	20.81	STRENGTH L.S.	1.37	1.06	HL-93
1	41.62	20.81	SERVICEBILITY	1.48	1.14	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 1.369
Inventory= 1.056

TABLE 2.2.32.3B=BRIDGE MOMENT RATING INFORMATION FOR DUMP TRUCK FA

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

	(ft)	(ft)			(k-ft)
1	41.62	20.81	STRENGTH L.S.	1.68	617.94
1	41.62	20.81	SERVICEBILITY	1.81	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 1.685

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TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR

SP NO	IN NO	D L	FROM 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	407.0	0.0	25.8	73.0	0.0	3.74	HL-93	2.88
1	1		2.08	407.0	0.0	22.6	67.9	-1.8	4.07	HL-93	3.14
1	2		4.16	407.0	0.0	19.9	62.9	-3.6	4.44	HL-93	3.43
1	3		6.24	407.0	0.0	17.5	57.9	-5.9	4.87	HL-93	3.76
1	4		8.32	407.0	0.0	15.0	53.0	-8.8	5.37	HL-93	4.14
1	5		10.41	407.0	0.0	12.5	48.1	-11.9	5.98	HL-93	4.61
1	6		12.49	407.0	0.0	10.1	43.4	-14.9	6.69	HL-93	5.16
1	7		14.57	407.0	0.0	7.6	38.9	-18.1	7.53	HL-93	5.81
1	8		16.65	407.0	0.0	5.2	34.6	-21.3	8.55	HL-93	6.59
1	9		18.73	407.0	0.0	2.3	31.1	-24.5	9.59	HL-93	7.40
1	10		20.81	407.0	0.0	-0.1	27.8	-27.8	9.99	HL-93	8.36
1	11		22.89	407.0	0.0	-2.6	24.5	-31.1	9.59	HL-93	7.40
1	12		24.97	407.0	0.0	-5.0	21.3	-34.6	8.55	HL-93	6.60
1	13		27.06	407.0	0.0	-7.5	18.1	-38.9	7.53	HL-93	5.81
1	14		29.14	407.0	0.0	-10.0	14.9	-43.4	6.70	HL-93	5.17
1	15		31.22	407.0	0.0	-12.4	11.9	-48.1	5.98	HL-93	4.62
1	16		33.30	407.0	0.0	-15.1	8.8	-53.0	5.37	HL-93	4.14
1	17		35.38	407.0	0.0	-17.5	5.9	-57.9	4.87	HL-93	3.76
1	18		37.46	407.0	0.0	-20.0	3.6	-62.9	4.44	HL-93	3.43
1	19		39.54	407.0	0.0	-22.4	1.8	-67.9	4.07	HL-93	3.14
1	20		41.62	407.0	0.0	-25.7	0.0	-73.0	3.74	HL-93	2.89

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TABLE 2.2.33.3A=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	41.62	0.00	STRENGTH L.S.	3.74	2.88	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 3.740
Inventory= 2.885

TABLE 2.2.33.3B=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	41.62	0.00	STRENGTH L.S.	4.72	77.97

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 4.724

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TABLE 2.2.34.1=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	20.81	2 0.400	-0.29	HL-93	2.16
		3 0.510	-0.37		1.70

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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                HAVE PROVEN INEFFECTIVE.
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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5800 Span 1 Exterior Beam	7/17/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 1(4)	IAS/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				PROGRAM FLOW CONTROL
			CODE ID	YEAR	UNIT TYPE	DESIGN 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 NUMBER	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
5	2	31.33	2.50	1.83	1.62	18.12	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
74.67	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
7.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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	36		194		0.00	0.0000	0.00	0.0000		
2	W	36		194		0.00	0.0000	11.00	1.0625		

NOTE: [1] maximum allowable section number is 70

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

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	14.83				32.	32.	32.
2	2	2	45.00				32.	32.	32.
3	1	1	14.83				32.	32.	32.

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	2	2398	2038	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 2398

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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	0.47	2 2 2 2 2
1	0.00	0	0.00	0.55	0.00	0.00	0 0 0 0 0	0.70	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.

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	7.0	0	30.0	10.0	0.60	0.00	74.67

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.

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

LOAD NO.	TYPE	DESCRIPTION	UNIFROM LOAD DATA INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)	CONCENTRATED LOAD DATA INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	0	wearing surf	0.238	0.00	74.67	0.00	0.00
2	1	rail	0.010	0.00	74.67	0.00	0.00
3	1	curb	0.115	0.00	74.67	0.00	0.00
4	2	end diaphrag	0.000	0.00	0.00	0.38	0.45
5	2	int diaphrag	0.000	0.00	0.00	0.17	14.97
6	2	int diaphrag	0.000	0.00	0.00	0.17	29.97
7	2	int diaphrag	0.000	0.00	0.00	0.17	44.97
8	2	int diaphrag	0.000	0.00	0.00	0.17	59.97
9	2	end diaphrag	0.000	0.00	0.00	0.38	74.21

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 NO. PER TRAN. SEC	DIA. (in)	CONNECTOR IN NEGAT. M. REGION 0=NO 1=YES	Qn VALUE AASHTO ART. 6.10.10.4 (kip / per connect.)	Zr VALUE AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	SLAB REINFORCEMENT REBAR YIELD STRESS Fy (ksi)	BAR AREA PER FOOT OF SLAB (in**2)	DIST. FROM TOP (in)	CONCRETE 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	2.50	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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UNIVERSITY OF MARYLAND — LRF -- 2010
INPUT

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

LOCATION		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	14.97	33.0	33.0	14.52
14.97	59.97	33.0	33.0	15.00
59.97	74.21	33.0	33.0	14.24

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.	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

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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
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AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.599		0.2179		0.8169

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.24	0.00	74.67
1	DC2	0.010	0.000	74.667
2	DC2	0.115	0.000	74.667

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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	1524.5	56.5	228.7	81.4	310.03

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	7.50	75.00

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	0.466	0.466	0.649	0.649	FOR STRENGTH POSITIVE MOMENT
	0.704				FOR STRENGTH POSITIVE SHEAR
1	0.366				FOR FATIGUE POSITIVE MOMENT
	0.553				FOR FATIGUE POSITIVE SHEAR

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)	STEEL TOP. (in**3)
1	0	0.00	12100.0	36.5	18.25	663.0	663.0
1	1	4.67	12100.0	36.5	18.25	663.0	663.0
1	2	9.33	12100.0	36.5	18.25	663.0	663.0

1	3	14.00	12100.0	36.5	18.25	663.0	663.0
1	4	18.67	15540.1	36.5	16.12	964.2	724.6
1	5	23.33	15540.1	36.5	16.12	964.2	724.6
1	6	28.00	15540.1	36.5	16.12	964.2	724.6
1	7	32.67	15540.1	36.5	16.12	964.2	724.6
1	8	37.33	15540.1	36.5	16.12	964.2	724.6
1	9	42.00	15540.1	36.5	16.12	964.2	724.6
1	10	46.67	15540.1	36.5	16.12	964.2	724.6
1	11	51.33	15540.1	36.5	16.12	964.2	724.6
1	12	56.00	15540.1	36.5	16.12	964.2	724.6
1	13	60.67	12100.0	36.5	18.25	663.0	663.0
1	14	65.33	12100.0	36.5	18.25	663.0	663.0
1	15	70.00	12100.0	36.5	18.25	663.0	663.0
1	16	74.67	12100.0	36.5	18.25	663.0	663.0

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 = 30.00

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	18797.7	0.016332	799.0	1448.9	924.2
1	1	4.67	18797.7	0.016332	799.0	1448.9	924.2
1	2	9.33	18797.7	0.016332	799.0	1448.9	924.2
1	3	14.00	18797.7	0.016332	799.0	1448.9	924.2
1	4	18.67	24668.6	0.014876	1155.8	1521.0	1046.0
1	5	23.33	24668.6	0.014876	1155.8	1521.0	1046.0
1	6	28.00	24668.6	0.014876	1155.8	1521.0	1046.0
1	7	32.67	24668.6	0.014876	1155.8	1521.0	1046.0
1	8	37.33	24668.6	0.014876	1155.8	1521.0	1046.0
1	9	42.00	24668.6	0.014876	1155.8	1521.0	1046.0
1	10	46.67	24668.6	0.014876	1155.8	1521.0	1046.0
1	11	51.33	24668.6	0.014876	1155.8	1521.0	1046.0
1	12	56.00	24668.6	0.014876	1155.8	1521.0	1046.0
1	13	60.67	18797.7	0.016332	799.0	1448.9	924.2
1	14	65.33	18797.7	0.016332	799.0	1448.9	924.2
1	15	70.00	18797.7	0.016332	799.0	1448.9	924.2
1	16	74.67	18797.7	0.016332	799.0	1448.9	924.2

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 10.00

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	25758.6	0.024083	889.8	3411.0	1726.7
1	1	4.67	25758.6	0.024083	889.8	3411.0	1726.7

1	2	9.33	25758.6	0.024083	889.8	3411.0	1726.7
1	3	14.00	25758.6	0.024083	889.8	3411.0	1726.7
1	4	18.67	34966.4	0.022197	1285.5	3374.6	1972.4
1	5	23.33	34966.4	0.022197	1285.5	3374.6	1972.4
1	6	28.00	34966.4	0.022197	1285.5	3374.6	1972.4
1	7	32.67	34966.4	0.022197	1285.5	3374.6	1972.4
1	8	37.33	34966.4	0.022197	1285.5	3374.6	1972.4
1	9	42.00	34966.4	0.022197	1285.5	3374.6	1972.4
1	10	46.67	34966.4	0.022197	1285.5	3374.6	1972.4
1	11	51.33	34966.4	0.022197	1285.5	3374.6	1972.4
1	12	56.00	34966.4	0.022197	1285.5	3374.6	1972.4
1	13	60.67	25758.6	0.024083	889.8	3411.0	1726.7
1	14	65.33	25758.6	0.024083	889.8	3411.0	1726.7
1	15	70.00	25758.6	0.024083	889.8	3411.0	1726.7
1	16	74.67	25758.6	0.024083	889.8	3411.0	1726.7

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	4.67	35.6	97.8	1.8	0.0		135.2
1	2	9.33	66.4	182.6	3.4	0.0		252.5
1	3	14.00	92.5	254.4	5.1	0.0		352.0
1	4	18.67	113.9	313.1	6.0	0.0		433.0
1	5	23.33	130.5	358.7	6.8	0.0		496.1
1	6	28.00	142.4	391.4	7.7	0.0		541.4
1	7	32.67	149.5	410.9	8.0	0.0		568.4
1	8	37.33	151.9	417.4	8.0	0.0		577.3

1	9	42.00	149.5	410.9	8.0	0.0	568.4
1	10	46.67	142.4	391.4	7.7	0.0	541.4
1	11	51.33	130.5	358.7	6.9	0.0	496.1
1	12	56.00	113.9	313.1	6.0	0.0	433.0
1	13	60.67	92.5	254.4	5.1	0.0	352.0
1	14	65.33	66.4	182.6	3.5	0.0	252.5
1	15	70.00	35.6	97.8	1.8	0.0	135.2
1	16	74.67	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 = 30.00 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD LOAD (k-ft) DW	(k-ft) 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	4.67	38.9	20.4	0.0	0.0	0.0	59.3
1	2	9.33	72.6	38.1	0.0	0.0	0.0	110.7
1	3	14.00	101.1	53.1	0.0	0.0	0.0	154.2
1	4	18.67	124.4	65.3	0.0	0.0	0.0	189.7
1	5	23.33	142.5	74.9	0.0	0.0	0.0	217.4
1	6	28.00	155.5	81.7	0.0	0.0	0.0	237.2
1	7	32.67	163.3	85.8	0.0	0.0	0.0	249.0
1	8	37.33	165.9	87.1	0.0	0.0	0.0	253.0
1	9	42.00	163.3	85.8	0.0	0.0	0.0	249.0
1	10	46.67	155.5	81.7	0.0	0.0	0.0	237.2
1	11	51.33	142.5	74.9	0.0	0.0	0.0	217.4
1	12	56.00	124.4	65.3	0.0	0.0	0.0	189.7
1	13	60.67	101.1	53.1	0.0	0.0	0.0	154.2
1	14	65.33	72.6	38.1	0.0	0.0	0.0	110.7
1	15	70.00	38.9	20.4	0.0	0.0	0.0	59.3
1	16	74.67	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 = 10.00 (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	4.67	0.0	0.0	217.9	HL-93	0.0	HL-93
1	2	9.33	0.0	0.0	403.3	HL-93	0.0	HL-93
1	3	14.00	0.0	0.0	556.2	HL-93	0.0	HL-93
1	4	18.67	0.0	0.0	674.8	HL-93	0.0	HL-93
1	5	23.33	0.0	0.0	765.7	HL-93	0.0	HL-93
1	6	28.00	0.0	0.0	829.4	HL-93	0.0	HL-93
1	7	32.67	0.0	0.0	859.2	HL-93	0.0	HL-93
1	8	37.33	0.0	0.0	867.3	HL-93	0.0	HL-93
1	9	42.00	0.0	0.0	859.2	HL-93	0.0	HL-93
1	10	46.67	0.0	0.0	829.4	HL-93	0.0	HL-93
1	11	51.33	0.0	0.0	765.7	HL-93	0.0	HL-93
1	12	56.00	0.0	0.0	674.8	HL-93	0.0	HL-93

1	13	60.67	0.0	0.0	556.2	HL-93	0.0	HL-93
1	14	65.33	0.0	0.0	403.3	HL-93	0.0	HL-93
1	15	70.00	0.0	0.0	217.9	HL-93	0.0	HL-93
1	16	74.67	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 = 10.0 (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	4.67	98.	0.	98.
1	2	9.33	177.	0.	177.
1	3	14.00	241.	0.	241.
1	4	18.67	291.	0.	291.
1	5	23.33	327.	0.	327.
1	6	28.00	349.	0.	349.
1	7	32.67	345.	0.	345.
1	8	37.33	340.	0.	340.
1	9	42.00	345.	0.	345.
1	10	46.67	349.	0.	349.
1	11	51.33	327.	0.	327.
1	12	56.00	291.	0.	291.
1	13	60.67	241.	0.	241.
1	14	65.33	177.	0.	177.
1	15	70.00	98.	0.	98.
1	16	74.67	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=30.0 (k-ft)	LL+I:N=10.0 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	4.67	135.2	59.3	217.9	HL-93	0.0	HL-93	412.4	194.5
1	2	9.33	252.5	110.7	403.3	HL-93	0.0	HL-93	766.5	363.2
1	3	14.00	352.0	154.2	556.2	HL-93	0.0	HL-93	1062.3	506.1
1	4	18.67	433.0	189.7	674.8	HL-93	0.0	HL-93	1297.5	622.7
1	5	23.33	496.1	217.4	765.7	HL-93	0.0	HL-93	1479.2	713.5
1	6	28.00	541.4	237.2	829.4	HL-93	0.0	HL-93	1607.9	778.5
1	7	32.67	568.4	249.0	859.2	HL-93	0.0	HL-93	1676.6	817.4
1	8	37.33	577.3	253.0	867.3	HL-93	0.0	HL-93	1697.6	830.3
1	9	42.00	568.4	249.0	859.2	HL-93	0.0	HL-93	1676.6	817.4
1	10	46.67	541.4	237.2	829.4	HL-93	0.0	HL-93	1607.9	778.6
1	11	51.33	496.1	217.4	765.7	HL-93	0.0	HL-93	1479.2	713.5
1	12	56.00	433.0	189.7	674.8	HL-93	0.0	HL-93	1297.5	622.8
1	13	60.67	352.0	154.2	556.2	HL-93	0.0	HL-93	1062.3	506.2
1	14	65.33	252.5	110.7	403.3	HL-93	0.0	HL-93	766.5	363.2
1	15	70.00	135.2	59.3	217.9	HL-93	0.0	HL-93	412.5	194.5
1	16	74.67	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	4.67	412.4	477.8	634.2	547.1	291.8	73.5
1	2	9.33	766.5	887.5	1177.9	1016.6	544.8	133.0
1	3	14.00	1062.3	1229.1	1631.2	1408.8	759.2	180.4
1	4	18.67	1297.5	1499.9	1990.4	1720.5	934.1	218.2
1	5	23.33	1479.2	1708.9	2267.4	1961.2	1070.2	245.6
1	6	28.00	1607.9	1856.7	2463.4	2131.7	1167.8	262.0
1	7	32.67	1676.6	1934.3	2566.1	2222.5	1226.1	258.8
1	8	37.33	1697.6	1957.7	2597.1	2250.1	1245.4	255.0
1	9	42.00	1676.6	1934.3	2566.1	2222.5	1226.1	258.8
1	10	46.67	1607.9	1856.7	2463.5	2131.7	1167.9	262.0
1	11	51.33	1479.2	1708.9	2267.4	1961.2	1070.3	245.6
1	12	56.00	1297.5	1499.9	1990.4	1720.5	934.1	218.2
1	13	60.67	1062.3	1229.2	1631.3	1408.8	759.3	180.4
1	14	65.33	766.5	887.5	1177.9	1016.6	544.8	133.0
1	15	70.00	412.5	477.8	634.2	547.1	291.8	73.5
1	16	74.67	0.0	0.0	0.0	0.0	0.0	0.0

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — COMPOSITE
UNIVERSITY OF MARYLAND — LRF -- 2010
ANALYSIS
FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 25\5800\Rating\MD\5

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	8.1	22.4	0.7	0.0	31.2
1	1	4.67	7.1	19.6	0.3	0.0	27.0
1	2	9.33	6.1	16.8	0.3	0.0	23.2
1	3	14.00	5.1	14.0	0.3	0.0	19.4
1	4	18.67	4.1	11.2	0.2	0.0	15.4
1	5	23.33	3.1	8.4	0.2	0.0	11.6
1	6	28.00	2.0	5.6	0.2	0.0	7.8
1	7	32.67	1.0	2.8	0.0	0.0	3.8
1	8	37.33	0.0	0.0	0.0	0.0	0.0
1	9	42.00	-1.0	-2.8	0.0	0.0	-3.8
1	10	46.67	-2.0	-5.6	-0.2	0.0	-7.8
1	11	51.33	-3.1	-8.4	-0.2	0.0	-11.6
1	12	56.00	-4.1	-11.2	-0.2	0.0	-15.4
1	13	60.67	-5.1	-14.0	-0.4	0.0	-19.4
1	14	65.33	-6.1	-16.8	-0.4	0.0	-23.2
1	15	70.00	-7.1	-19.6	-0.4	0.0	-27.0
1	16	74.67	-8.1	-22.4	-0.7	0.0	-31.2

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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	8.9	4.7	0.0	0.0	13.6	31.2	44.8
1	1	4.67	7.8	4.1	0.0	0.0	11.9	27.0	38.9
1	2	9.33	6.7	3.5	0.0	0.0	10.2	23.2	33.4
1	3	14.00	5.6	2.9	0.0	0.0	8.5	19.4	27.9
1	4	18.67	4.4	2.3	0.0	0.0	6.8	15.4	22.2
1	5	23.33	3.3	1.8	0.0	0.0	5.1	11.6	16.7
1	6	28.00	2.2	1.2	0.0	0.0	3.4	7.8	11.2
1	7	32.67	1.1	0.6	0.0	0.0	1.7	3.8	5.5
1	8	37.33	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	9	42.00	-1.1	-0.6	0.0	0.0	-1.7	-3.8	-5.5
1	10	46.67	-2.2	-1.2	0.0	0.0	-3.4	-7.8	-11.2
1	11	51.33	-3.3	-1.8	0.0	0.0	-5.1	-11.6	-16.7
1	12	56.00	-4.4	-2.3	0.0	0.0	-6.8	-15.4	-22.2
1	13	60.67	-5.6	-2.9	0.0	0.0	-8.5	-19.4	-27.9
1	14	65.33	-6.7	-3.5	0.0	0.0	-10.2	-23.2	-33.4
1	15	70.00	-7.8	-4.1	0.0	0.0	-11.9	-27.0	-38.9
1	16	74.67	-8.9	-4.7	0.0	0.0	-13.6	-31.2	-44.8

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	75.8	HL-93	0.0	HL-93
1	1	4.67	0.0	0.0	69.5	HL-93	-2.1	HL-93
1	2	9.33	0.0	0.0	63.4	HL-93	-4.9	HL-93
1	3	14.00	0.0	0.0	57.4	HL-93	-8.1	HL-93
1	4	18.67	0.0	0.0	51.6	HL-93	-11.5	HL-93
1	5	23.33	0.0	0.0	45.9	HL-93	-15.0	HL-93
1	6	28.00	0.0	0.0	40.3	HL-93	-19.3	HL-93
1	7	32.67	0.0	0.0	34.8	HL-93	-24.3	HL-93
1	8	37.33	0.0	0.0	29.5	HL-93	-29.5	HL-93
1	9	42.00	0.0	0.0	24.3	HL-93	-34.8	HL-93
1	10	46.67	0.0	0.0	19.3	HL-93	-40.3	HL-93
1	11	51.33	0.0	0.0	15.0	HL-93	-45.9	HL-93
1	12	56.00	0.0	0.0	11.5	HL-93	-51.6	HL-93
1	13	60.67	0.0	0.0	8.1	HL-93	-57.4	HL-93
1	14	65.33	0.0	0.0	4.9	HL-93	-63.4	HL-93
1	15	70.00	0.0	0.0	2.1	HL-93	-69.5	HL-93
1	16	74.67	0.0	0.0	0.0	HL-93	-75.8	HL-93

TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 10.0 (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	4.67	32.	-1.	33.
1	2	9.33	29.	-3.	31.
1	3	14.00	26.	-4.	30.
1	4	18.67	23.	-5.	29.
1	5	23.33	20.	-7.	27.
1	6	28.00	17.	-9.	26.
1	7	32.67	15.	-10.	25.
1	8	37.33	12.	-12.	24.
1	9	42.00	10.	-15.	25.
1	10	46.67	9.	-17.	26.
1	11	51.33	7.	-20.	27.
1	12	56.00	5.	-23.	29.
1	13	60.67	4.	-26.	30.
1	14	65.33	3.	-29.	31.
1	15	70.00	1.	-32.	33.
1	16	74.67	0.	-35.	35.

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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	34.6	.27814E-01	0.48	0.3125*	.15622E-01	0.27	0.3125*
1	1	4.67	33.0	.27814E-01	0.46	0.3125*	.15622E-01	0.26	0.3125*
1	2	9.33	31.4	.27814E-01	0.44	0.3125*	.15622E-01	0.25	0.3125*
1	3	14.00	29.9	.27814E-01	0.42	0.3125*	.15622E-01	0.23	0.3125*
1	4	18.67	28.6	.26128E-01	0.37	0.3125*	.10325E-01	0.15	0.3125*
1	5	23.33	27.2	.26128E-01	0.36	0.3125*	.10325E-01	0.14	0.3125*
1	6	28.00	25.9	.26128E-01	0.34	0.3125*	.10325E-01	0.13	0.3125*
1	7	32.67	24.8	.26128E-01	0.32	0.3125*	.10325E-01	0.13	0.3125*
1	8	37.33	24.3	.26128E-01	0.32	0.3125*	.10325E-01	0.13	0.3125*
1	9	42.00	24.8	.26128E-01	0.32	0.3125*	.10325E-01	0.13	0.3125*
1	10	46.67	25.9	.26128E-01	0.34	0.3125*	.10325E-01	0.13	0.3125*
1	11	51.33	27.2	.26128E-01	0.36	0.3125*	.10325E-01	0.14	0.3125*
1	12	56.00	28.6	.26128E-01	0.37	0.3125*	.10325E-01	0.15	0.3125*
1	13	60.67	29.9	.27814E-01	0.42	0.3125*	.15622E-01	0.23	0.3125*
1	14	65.33	31.4	.27814E-01	0.44	0.3125*	.15622E-01	0.25	0.3125*
1	15	70.00	33.0	.27814E-01	0.46	0.3125*	.15622E-01	0.26	0.3125*
1	16	74.67	34.6	.27814E-01	0.48	0.3125*	.15622E-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	44.8	75.8	HL-93	0.0	HL-93	120.6	44.8
1	1	4.67	38.9	69.5	HL-93	-2.1	HL-93	108.4	36.8
1	2	9.33	33.4	63.4	HL-93	-4.9	HL-93	96.8	28.5
1	3	14.00	27.9	57.4	HL-93	-8.1	HL-93	85.3	19.8
1	4	18.67	22.2	51.6	HL-93	-11.5	HL-93	73.8	10.7
1	5	23.33	16.7	45.9	HL-93	-15.0	HL-93	62.6	1.7
1	6	28.00	11.2	40.3	HL-93	-19.3	HL-93	51.5	-8.1
1	7	32.67	5.5	34.8	HL-93	-24.3	HL-93	40.3	-18.8
1	8	37.33	0.0	29.5	HL-93	-29.5	HL-93	29.5	-29.5
1	9	42.00	-5.5	24.3	HL-93	-34.8	HL-93	18.8	-40.3
1	10	46.67	-11.2	19.3	HL-93	-40.3	HL-93	8.1	-51.5
1	11	51.33	-16.7	15.0	HL-93	-45.9	HL-93	-1.7	-62.6
1	12	56.00	-22.2	11.5	HL-93	-51.6	HL-93	-10.7	-73.8
1	13	60.67	-27.9	8.1	HL-93	-57.4	HL-93	-19.8	-85.3
1	14	65.33	-33.4	4.9	HL-93	-63.4	HL-93	-28.5	-96.8
1	15	70.00	-38.9	2.1	HL-93	-69.5	HL-93	-36.8	-108.4
1	16	74.67	-44.8	0.0	HL-93	-75.8	HL-93	-44.8	-120.6

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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	120.6	143.3	190.8	160.5	67.2	26.0
1	1	4.67	108.4	129.3	172.3	144.5	58.3	24.8
1	2	9.33	96.8	115.8	154.4	129.0	50.1	23.5
1	3	14.00	85.3	102.6	136.8	113.8	41.8	22.4
1	4	18.67	73.8	89.3	119.1	98.5	33.3	21.4
1	5	23.33	62.6	76.3	102.0	83.6	25.0	20.4
1	6	28.00	51.5	63.5	85.0	68.9	16.8	19.5
1	7	32.67	40.3	50.8	68.1	54.1	8.3	18.6
1	8	37.33	-29.5	-38.3	-51.6	-39.8	0.0	18.3
1	9	42.00	-40.3	-50.8	-68.1	-54.2	-8.3	18.6
1	10	46.67	-51.5	-63.5	-85.0	-68.9	-16.8	19.5
1	11	51.33	-62.6	-76.3	-102.0	-83.6	-25.0	20.4
1	12	56.00	-73.8	-89.3	-119.1	-98.5	-33.3	21.4
1	13	60.67	-85.3	-102.6	-136.8	-113.8	-41.8	22.4
1	14	65.33	-96.8	-115.8	-154.4	-129.0	-50.1	23.5

1	15	70.00	-108.4	-129.3	-172.3	-144.5	-58.3	24.8
1	16	74.67	-120.6	-143.3	-190.8	-160.5	-67.2	26.0

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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	44.78	0.00	HL-93	75.80	HL-93	ST1	38.08	190.85
						ST2	38.08	160.53
						ST4	53.84	67.17
						SE1	44.78	120.58
						SE2	44.78	143.32
2	44.78	0.00	HL-93	75.80	HL-93	ST1	38.08	190.85
						ST2	38.08	160.53
						ST4	53.84	67.17
						SE1	44.78	120.58
						SE2	44.78	143.32

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 = 30.0 (UNFACTORED)
****

SP NO	IN D FROM NO 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
1	1	4.67	-0.0709	-0.1950	0.0000	-0.0748	-0.2697	-0.0748	-0.3445
1	2	9.33	-0.1381	-0.3795	0.0000	-0.1456	-0.5250	-0.1456	-0.6706
1	3	14.00	-0.1982	-0.5447	0.0000	-0.2088	-0.7534	-0.2088	-0.9621
1	4	18.67	-0.2492	-0.6849	0.0000	-0.2623	-0.9473	-0.2623	-1.2096
1	5	23.33	-0.2907	-0.7989	0.0000	-0.3058	-1.1051	-0.3058	-1.4109
1	6	28.00	-0.3213	-0.8831	0.0000	-0.3380	-1.2215	-0.3380	-1.5595
1	7	32.67	-0.3401	-0.9347	0.0000	-0.3577	-1.2929	-0.3577	-1.6506
1	8	37.33	-0.3464	-0.9521	0.0000	-0.3643	-1.3170	-0.3643	-1.6813
1	9	42.00	-0.3401	-0.9347	0.0000	-0.3577	-1.2929	-0.3577	-1.6506
1	10	46.67	-0.3213	-0.8831	0.0000	-0.3380	-1.2216	-0.3380	-1.5595
1	11	51.33	-0.2907	-0.7989	0.0000	-0.3059	-1.1051	-0.3059	-1.4109
1	12	56.00	-0.2492	-0.6849	0.0000	-0.2623	-0.9473	-0.2623	-1.2096
1	13	60.67	-0.1982	-0.5447	0.0000	-0.2088	-0.7534	-0.2088	-0.9622
1	14	65.33	-0.1381	-0.3796	0.0000	-0.1456	-0.5250	-0.1456	-0.6706
1	15	70.00	-0.0709	-0.1950	0.0000	-0.0748	-0.2697	-0.0748	-0.3445
1	16	74.67	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 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	4.7	-0.071	0 1/ 8	-0.199	0 1/ 4	-0.075	0 1/ 8	-0.344	0 3/ 8
1	2	9.3	-0.138	0 3/16	-0.387	0 7/16	-0.146	0 3/16	-0.671	0 11/16
1	3	14.0	-0.198	0 1/ 4	-0.555	0 9/16	-0.209	0 1/ 4	-0.962	1 0/16
1	4	18.7	-0.249	0 1/ 4	-0.698	0 3/ 4	-0.262	0 5/16	-1.210	1 1/ 4
1	5	23.3	-0.291	0 5/16	-0.814	0 7/ 8	-0.306	0 5/16	-1.411	1 7/16
1	6	28.0	-0.321	0 3/ 8	-0.900	0 15/16	-0.338	0 3/ 8	-1.560	1 9/16
1	7	32.7	-0.340	0 3/ 8	-0.953	1 0/16	-0.358	0 3/ 8	-1.651	1 11/16
1	8	37.3	-0.346	0 3/ 8	-0.971	1 0/16	-0.364	0 3/ 8	-1.681	1 11/16
1	9	42.0	-0.340	0 3/ 8	-0.953	1 0/16	-0.358	0 3/ 8	-1.651	1 11/16
1	10	46.7	-0.321	0 3/ 8	-0.900	0 15/16	-0.338	0 3/ 8	-1.560	1 9/16
1	11	51.3	-0.291	0 5/16	-0.814	0 7/ 8	-0.306	0 5/16	-1.411	1 7/16
1	12	56.0	-0.249	0 1/ 4	-0.698	0 3/ 4	-0.262	0 5/16	-1.210	1 1/ 4
1	13	60.7	-0.198	0 1/ 4	-0.555	0 9/16	-0.209	0 1/ 4	-0.962	1 0/16
1	14	65.3	-0.138	0 3/16	-0.387	0 7/16	-0.146	0 3/16	-0.671	0 11/16
1	15	70.0	-0.071	0 1/ 8	-0.199	0 1/ 4	-0.075	0 1/ 8	-0.344	0 3/ 8
1	16	74.7	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 DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION GOVERN. (inch)	LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	37.33	2 0.400	-0.530 MAX	HL-93	1.12	0.00363
			0.033 MIN			
			-0.182 MAX	LANE		
			0.033 MIN	LANE		
		3 0.510	-0.676 MAX	HL-93	1.12	0.00363
			0.043 MIN			
			-0.232 MAX	LANE		
			0.043 MIN	LANE		
		2 0.400	-0.431 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 L (ft)	FROM SUPT	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		4.67	-0.64	0.64	-1.80	1.80	-2.45	2.45
1	2		9.33	-1.20	1.20	-3.37	3.37	-4.57	4.57
1	3		14.00	-1.68	1.68	-4.70	4.70	-6.37	6.37
1	4		18.67	-1.89	1.42	-5.28	3.97	-7.17	5.39
1	5		23.33	-2.16	1.62	-6.05	4.55	-8.22	6.17
1	6		28.00	-2.36	1.77	-6.61	4.97	-8.97	6.74
1	7		32.67	-2.48	1.86	-6.94	5.21	-9.41	7.07
1	8		37.33	-2.52	1.89	-7.05	5.29	-9.56	7.18
1	9		42.00	-2.48	1.86	-6.94	5.21	-9.41	7.07
1	10		46.67	-2.36	1.77	-6.61	4.97	-8.97	6.74
1	11		51.33	-2.16	1.62	-6.05	4.55	-8.22	6.17
1	12		56.00	-1.89	1.42	-5.28	3.97	-7.17	5.39
1	13		60.67	-1.68	1.68	-4.70	4.70	-6.37	6.37
1	14		65.33	-1.20	1.20	-3.37	3.37	-4.57	4.57
1	15		70.00	-0.64	0.64	-1.80	1.80	-2.45	2.45
1	16		74.67	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 = 30.0 AND TOTAL DL STRESSES
****
(UNFACTORED)

SP NO	IN NO	D L (ft)	FROM SUPT	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		4.67	-0.03	-0.49	0.89	-0.03	-2.94	3.34
1	2		9.33	-0.05	-0.92	1.66	-0.05	-5.49	6.23

1	3	14.00	-0.07	-1.28	2.32	-0.07	-7.65	8.69
1	4	18.67	-0.07	-1.50	1.97	-0.07	-8.67	7.36
1	5	23.33	-0.08	-1.72	2.26	-0.08	-9.93	8.43
1	6	28.00	-0.09	-1.87	2.46	-0.09	-10.84	9.20
1	7	32.67	-0.10	-1.96	2.59	-0.10	-11.38	9.66
1	8	37.33	-0.10	-2.00	2.63	-0.10	-11.56	9.81
1	9	42.00	-0.10	-1.96	2.59	-0.10	-11.38	9.66
1	10	46.67	-0.09	-1.87	2.46	-0.09	-10.84	9.20
1	11	51.33	-0.08	-1.72	2.26	-0.08	-9.93	8.43
1	12	56.00	-0.07	-1.50	1.97	-0.07	-8.67	7.36
1	13	60.67	-0.07	-1.28	2.32	-0.07	-7.65	8.69
1	14	65.33	-0.05	-0.92	1.66	-0.05	-5.49	6.23
1	15	70.00	-0.03	-0.49	0.89	-0.03	-2.94	3.34
1	16	74.67	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 = 10.0 (UNFACTORED)
****

SP NO	IN NO	D L (ft)	FROM SUPT	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	0.00
1	1	4.67	-0.15	-0.77	2.94	0.00	0.00	0.00	0.00
1	2	9.33	-0.28	-1.42	5.44	0.00	0.00	0.00	0.00
1	3	14.00	-0.39	-1.96	7.50	0.00	0.00	0.00	0.00
1	4	18.67	-0.41	-2.40	6.30	0.00	0.00	0.00	0.00
1	5	23.33	-0.47	-2.72	7.15	0.00	0.00	0.00	0.00
1	6	28.00	-0.50	-2.95	7.74	0.00	0.00	0.00	0.00
1	7	32.67	-0.52	-3.06	8.02	0.00	0.00	0.00	0.00
1	8	37.33	-0.53	-3.08	8.10	0.00	0.00	0.00	0.00
1	9	42.00	-0.52	-3.06	8.02	0.00	0.00	0.00	0.00
1	10	46.67	-0.50	-2.95	7.74	0.00	0.00	0.00	0.00
1	11	51.33	-0.47	-2.72	7.15	0.00	0.00	0.00	0.00
1	12	56.00	-0.41	-2.40	6.30	0.00	0.00	0.00	0.00
1	13	60.67	-0.39	-1.96	7.50	0.00	0.00	0.00	0.00
1	14	65.33	-0.28	-1.42	5.44	0.00	0.00	0.00	0.00
1	15	70.00	-0.15	-0.77	2.94	0.00	0.00	0.00	0.00
1	16	74.67	0.00	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 = 10.0 (ksi) (UNFACTORED)
****

SP NO	IN NO	D L (ft)	FROM SUPT	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	4.67		0.3	0.0	0.3 C	HL	1.3	0.0	1.3 T	HL
1	2	9.33		0.6	0.0	0.6 C	HL	2.4	0.0	2.4 T	HL
1	3	14.00		0.8	0.0	0.8 C	HL	3.2	0.0	3.2 T	HL
1	4	18.67		1.0	0.0	1.0 C	HL	2.7	0.0	2.7 T	HL
1	5	23.33		1.2	0.0	1.2 C	HL	3.1	0.0	3.1 T	HL
1	6	28.00		1.2	0.0	1.2 C	HL	3.3	0.0	3.3 T	HL
1	7	32.67		1.2	0.0	1.2 C	HL	3.2	0.0	3.2 T	HL

1	8	37.33	1.2	0.0	1.2	C	HL	3.2	0.0	3.2	T HL
1	9	42.00	1.2	0.0	1.2	C	HL	3.2	0.0	3.2	T HL
1	10	46.67	1.2	0.0	1.2	C	HL	3.3	0.0	3.3	T HL
1	11	51.33	1.2	0.0	1.2	C	HL	3.1	0.0	3.1	T HL
1	12	56.00	1.0	0.0	1.0	C	HL	2.7	0.0	2.7	T HL
1	13	60.67	0.8	0.0	0.8	C	HL	3.2	0.0	3.2	T HL
1	14	65.33	0.6	0.0	0.6	C	HL	2.4	0.0	2.4	T HL
1	15	70.00	0.3	0.0	0.3	C	HL	1.3	0.0	1.3	T HL
1	16	74.67	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.18	-3.71	6.28	-0.03	-2.94	3.34
1	2	9.33	32.	32.	-0.33	-6.91	11.67	-0.05	-5.49	6.23
1	3	14.00	32.	32.	-0.45	-9.60	16.19	-0.07	-7.65	8.69
1	4	18.67	32.	32.	-0.48	-11.07	13.66	-0.07	-8.67	7.36
1	5	23.33	32.	32.	-0.55	-12.65	15.58	-0.08	-9.93	8.43
1	6	28.00	32.	32.	-0.60	-13.79	16.94	-0.09	-10.84	9.20
1	7	32.67	32.	32.	-0.62	-14.43	17.68	-0.10	-11.38	9.66
1	8	37.33	32.	32.	-0.62	-14.64	17.91	-0.10	-11.56	9.81
1	9	42.00	32.	32.	-0.62	-14.43	17.68	-0.10	-11.38	9.66
1	10	46.67	32.	32.	-0.60	-13.79	16.94	-0.09	-10.84	9.20
1	11	51.33	32.	32.	-0.55	-12.65	15.58	-0.08	-9.93	8.43
1	12	56.00	32.	32.	-0.48	-11.07	13.66	-0.07	-8.67	7.36
1	13	60.67	32.	32.	-0.45	-9.60	16.19	-0.07	-7.65	8.69
1	14	65.33	32.	32.	-0.33	-6.91	11.67	-0.05	-5.49	6.23
1	15	70.00	32.	32.	-0.18	-3.71	6.28	-0.03	-2.94	3.34
1	16	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.22	-3.94	7.16	-0.03	-2.94	3.34
1	2	9.33	32.	32.	-0.41	-7.33	13.30	-0.05	-5.49	6.23
1	3	14.00	32.	32.	-0.57	-10.19	18.44	-0.07	-7.65	8.69
1	4	18.67	32.	32.	-0.61	-11.79	15.55	-0.07	-8.67	7.36
1	5	23.33	32.	32.	-0.69	-13.47	17.72	-0.08	-9.93	8.43
1	6	28.00	32.	32.	-0.75	-14.67	19.26	-0.09	-10.84	9.20
1	7	32.67	32.	32.	-0.77	-15.35	20.09	-0.10	-11.38	9.66

1	8	37.33	32.	32.	-0.78	-15.57	20.34	-0.10	-11.56	9.81
1	9	42.00	32.	32.	-0.77	-15.35	20.09	-0.10	-11.38	9.66
1	10	46.67	32.	32.	-0.75	-14.67	19.26	-0.09	-10.84	9.20
1	11	51.33	32.	32.	-0.69	-13.47	17.72	-0.08	-9.93	8.43
1	12	56.00	32.	32.	-0.61	-11.79	15.55	-0.07	-8.67	7.36
1	13	60.67	32.	32.	-0.57	-10.19	18.44	-0.07	-7.65	8.69
1	14	65.33	32.	32.	-0.41	-7.33	13.30	-0.05	-5.49	6.23
1	15	70.00	32.	32.	-0.22	-3.94	7.16	-0.03	-2.94	3.34
1	16	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.30	-5.10	9.46	-0.04	-3.75	4.32
1	2	9.33	32.	32.	-0.56	-9.49	17.58	-0.07	-7.01	8.06
1	3	14.00	32.	32.	-0.77	-13.19	24.36	-0.09	-9.77	11.24
1	4	18.67	32.	32.	-0.82	-15.28	20.54	-0.10	-11.08	9.52
1	5	23.33	32.	32.	-0.93	-17.46	23.42	-0.12	-12.69	10.91
1	6	28.00	32.	32.	-1.01	-19.01	25.45	-0.13	-13.85	11.90
1	7	32.67	32.	32.	-1.05	-19.89	26.53	-0.13	-14.54	12.50
1	8	37.33	32.	32.	-1.06	-20.17	26.86	-0.14	-14.77	12.69
1	9	42.00	32.	32.	-1.05	-19.89	26.53	-0.13	-14.54	12.50
1	10	46.67	32.	32.	-1.01	-19.01	25.45	-0.13	-13.85	11.90
1	11	51.33	32.	32.	-0.93	-17.46	23.42	-0.12	-12.69	10.91
1	12	56.00	32.	32.	-0.82	-15.28	20.54	-0.10	-11.08	9.52
1	13	60.67	32.	32.	-0.77	-13.19	24.36	-0.09	-9.77	11.24
1	14	65.33	32.	32.	-0.56	-9.49	17.58	-0.07	-7.01	8.06
1	15	70.00	32.	32.	-0.30	-5.10	9.46	-0.04	-3.75	4.32
1	16	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.28	-3.91	8.00	-0.02	-2.56	2.86
1	2	9.33	32.	32.	-0.53	-7.27	14.85	-0.04	-4.79	5.34
1	3	14.00	32.	32.	-0.73	-10.10	20.56	-0.05	-6.67	7.44
1	4	18.67	32.	32.	-0.77	-11.75	17.32	-0.05	-7.56	6.30
1	5	23.33	32.	32.	-0.88	-13.42	19.73	-0.06	-8.66	7.22
1	6	28.00	32.	32.	-0.95	-14.61	21.42	-0.07	-9.45	7.88
1	7	32.67	32.	32.	-0.98	-15.26	22.31	-0.07	-9.92	8.27
1	8	37.33	32.	32.	-0.99	-15.47	22.57	-0.07	-10.07	8.40
1	9	42.00	32.	32.	-0.98	-15.26	22.31	-0.07	-9.92	8.27
1	10	46.67	32.	32.	-0.95	-14.61	21.43	-0.07	-9.45	7.88
1	11	51.33	32.	32.	-0.88	-13.42	19.73	-0.06	-8.66	7.22
1	12	56.00	32.	32.	-0.77	-11.75	17.32	-0.05	-7.56	6.30
1	13	60.67	32.	32.	-0.73	-10.10	20.56	-0.05	-6.67	7.44
1	14	65.33	32.	32.	-0.53	-7.27	14.86	-0.04	-4.79	5.34

1	15	70.00	32.	32.	-0.28	-3.91	8.00	-0.02	-2.56	2.86
1	16	74.67	32.	32.	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 1.2.9.5E=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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.24	-4.79	8.29	-0.04	-3.75	4.32
1	2	9.33	32.	32.	-0.45	-8.92	15.41	-0.07	-7.01	8.06
1	3	14.00	32.	32.	-0.62	-12.41	21.36	-0.09	-9.77	11.24
1	4	18.67	32.	32.	-0.66	-14.32	18.03	-0.10	-11.08	9.52
1	5	23.33	32.	32.	-0.75	-16.37	20.56	-0.12	-12.69	10.91
1	6	28.00	32.	32.	-0.81	-17.83	22.36	-0.13	-13.85	11.90
1	7	32.67	32.	32.	-0.84	-18.67	23.33	-0.13	-14.54	12.50
1	8	37.33	32.	32.	-0.85	-18.94	23.62	-0.14	-14.77	12.69
1	9	42.00	32.	32.	-0.84	-18.67	23.33	-0.13	-14.54	12.50
1	10	46.67	32.	32.	-0.81	-17.83	22.36	-0.13	-13.85	11.90
1	11	51.33	32.	32.	-0.75	-16.37	20.56	-0.12	-12.69	10.91
1	12	56.00	32.	32.	-0.66	-14.32	18.03	-0.10	-11.08	9.52
1	13	60.67	32.	32.	-0.62	-12.41	21.36	-0.09	-9.77	11.24
1	14	65.33	32.	32.	-0.45	-8.92	15.41	-0.07	-7.01	8.06
1	15	70.00	32.	32.	-0.24	-4.79	8.29	-0.04	-3.75	4.32
1	16	74.67	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.67	32.	32.	-0.22	-3.60	6.83	-0.02	-2.56	2.86
1	2	9.33	32.	32.	-0.41	-6.70	12.68	-0.04	-4.79	5.34
1	3	14.00	32.	32.	-0.57	-9.31	17.56	-0.05	-6.67	7.44
1	4	18.67	32.	32.	-0.61	-10.79	14.80	-0.05	-7.56	6.30
1	5	23.33	32.	32.	-0.69	-12.33	16.87	-0.06	-8.66	7.22
1	6	28.00	32.	32.	-0.75	-13.43	18.33	-0.07	-9.45	7.88
1	7	32.67	32.	32.	-0.78	-14.04	19.10	-0.07	-9.92	8.27
1	8	37.33	32.	32.	-0.78	-14.24	19.33	-0.07	-10.07	8.40
1	9	42.00	32.	32.	-0.78	-14.04	19.10	-0.07	-9.92	8.27
1	10	46.67	32.	32.	-0.75	-13.43	18.33	-0.07	-9.45	7.88
1	11	51.33	32.	32.	-0.69	-12.33	16.87	-0.06	-8.66	7.22
1	12	56.00	32.	32.	-0.61	-10.79	14.80	-0.05	-7.56	6.30
1	13	60.67	32.	32.	-0.57	-9.32	17.56	-0.05	-6.67	7.44
1	14	65.33	32.	32.	-0.41	-6.70	12.68	-0.04	-4.79	5.34
1	15	70.00	32.	32.	-0.22	-3.60	6.83	-0.02	-2.56	2.86
1	16	74.67	32.	32.	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=30.0 MOM. (k-ft)	DL2, N=30.0 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	14.83	366.	-6.6	160.	-1.3	577.	-2.0	0.	0.0	-9.99
	LB			6.6		2.4		7.8		0.0	2.03 16.83
	RT			-6.1		-1.3		-2.1		0.0	7.78 -9.39
	RB			4.6		1.7		5.4		0.0	2.05 11.61
1	LT	59.83	367.	-6.1	161.	-1.3	577.	-2.1	0.	0.0	5.39 -9.39
	LB			4.6		1.7		5.4		0.0	2.05 11.62
	RT			-6.6		-1.3		-2.0		0.0	5.39 -10.00
	RB			6.6		2.4		7.8		0.0	2.03 16.83
											7.79

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
****

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	14.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	0.0000
2	45.00	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	11.00	1.0625
3	14.83	W	12.1	1.2590	34.0	0.7640	12.1	1.2590	12.10	0.0000	12.10	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) = 74.7

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	14.83	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	0.03300	OK
2	45.00	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	0.03300	OK
3	14.83	7.00	36.50	[2]=	43.87	0.04896	>	0.04000	OK
				[3]=	36.50	0.04074	>	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]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	1	4.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	2	9.33	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	3	14.00	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	4	18.67	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	5	23.33	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	6	28.00	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	7	32.67	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	8	37.33	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	9	42.00	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	10	46.67	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	11	51.33	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	12	56.00	0	1	33.98	0.764	44.5	150.0	4.77	113.19			0
1	13	60.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	14	65.33	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	15	70.00	0	1	33.98	0.764	44.5	150.0	0.00	113.19			0
1	16	74.67	0	1	33.98	0.764	44.5	150.0	0.00	113.19			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	33.98	0.764	44.48	171.59	2
1	1	4.67	0	1	33.98	0.764	44.48	171.59	2
1	2	9.33	0	1	33.98	0.764	44.48	171.59	2
1	3	14.00	0	1	33.98	0.764	44.48	171.59	2
1	4	18.67	0	1	33.98	0.764	52.91	171.59	2
1	5	23.33	0	1	33.98	0.764	52.91	171.59	2
1	6	28.00	0	1	33.98	0.764	52.91	171.59	2
1	7	32.67	0	1	33.98	0.764	52.91	171.59	2
1	8	37.33	0	1	33.98	0.764	52.91	171.59	2
1	9	42.00	0	1	33.98	0.764	52.91	171.59	2
1	10	46.67	0	1	33.98	0.764	52.91	171.59	2
1	11	51.33	0	1	33.98	0.764	52.91	171.59	2
1	12	56.00	0	1	33.98	0.764	52.91	171.59	2
1	13	60.67	0	1	33.98	0.764	44.48	171.59	2
1	14	65.33	0	1	33.98	0.764	44.48	171.59	2
1	15	70.00	0	1	33.98	0.764	44.48	171.59	2
1	16	74.67	0	1	33.98	0.764	44.48	171.59	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	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	1	4.67	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	2	9.33	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	3	14.00	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	4	18.67	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	5	23.33	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	6	28.00	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	7	32.67	4.81	12.	12.1	5.7	1.259	0.840	

		4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	8	37.33	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	9	42.00	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	10	46.67	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	11	51.33	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	12	56.00	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	13	60.67	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	14	65.33	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	15	70.00	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	1.00	0
1	16	74.67	4.81	12.	12.1	5.7	1.259	0.840	
			4.81	12.	12.1	5.7	1.259	0.840	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 SUPT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb

---

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	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	1		4.67	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	2		9.33	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	3		14.00	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	4		18.67	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	5		23.33	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	6		28.00	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	7		32.67	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	8		37.33	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	9		42.00	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0

1	10	46.67	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	11	51.33	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	12	56.00	1	4.81	11.44	20.15	0	0.00	7.58	28.45	0	0
1	13	60.67	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	14	65.33	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	15	70.00	1	4.81	11.44	20.15	0	0.00	7.73	29.04	0	0
1	16	74.67	1	4.81	11.44	20.15	0	0.00	7.73	29.04	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

1

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	32.00	32.00	32.00
1	1		4.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	2		9.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	3		14.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	4		18.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	5		23.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	6		28.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	7		32.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	8		37.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	9		42.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	10		46.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	11		51.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	12		56.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	13		60.67	1	1.000	1.000	1.00	32.00	32.00	32.00
1	14		65.33	1	1.000	1.000	1.00	32.00	32.00	32.00
1	15		70.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	16		74.67	1	1.000	1.000	1.00	32.00	32.00	32.00

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	8.26	18.42
1	1		4.67	1	8.26	18.42
1	2		9.33	1	8.26	18.42
1	3		14.00	1	8.26	18.42
1	4		18.67	1	10.45	18.87
1	5		23.33	1	10.45	18.87
1	6		28.00	1	10.45	18.87
1	7		32.67	1	10.45	18.87
1	8		37.33	1	10.45	18.87
1	9		42.00	1	10.45	18.87
1	10		46.67	1	10.45	18.87
1	11		51.33	1	10.45	18.87
1	12		56.00	1	10.45	18.87
1	13		60.67	1	8.26	18.42
1	14		65.33	1	8.26	18.42
1	15		70.00	1	8.26	18.42
1	16		74.67	1	8.26	18.42

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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	0	0	0	0	0
1	1		4.67	1	0	0	0	0	0	0
1	2		9.33	1	0	0	0	0	0	0
1	3		14.00	1	0	0	0	0	0	0
1	4		18.67	1	0	0	0	0	0	0
1	5		23.33	1	0	0	0	0	0	0
1	6		28.00	1	0	0	0	0	0	0
1	7		32.67	1	0	0	0	0	0	0
1	8		37.33	1	0	0	0	0	0	0
1	9		42.00	1	0	0	0	0	0	0
1	10		46.67	1	0	0	0	0	0	0
1	11		51.33	1	0	0	0	0	0	0
1	12		56.00	1	0	0	0	0	0	0
1	13		60.67	1	0	0	0	0	0	0
1	14		65.33	1	0	0	0	0	0	0
1	15		70.00	1	0	0	0	0	0	0
1	16		74.67	1	0	0	0	0	0	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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FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 58\5800\Rating\MD\5

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	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	0
1	1		4.67	0.0	19.2	3.7	32.0	3.7	32.0	3.7	32.0	0
				0.0	-	3.7	-	3.7	32.0	-	-	0
1	2		9.33	0.0	19.2	6.9	32.0	6.9	32.0	6.9	32.0	0
				0.0	-	6.9	-	6.9	32.0	-	-	0
1	3		14.00	0.0	19.2	9.6	32.0	9.6	32.0	9.6	32.0	0
				0.0	-	9.6	-	9.6	32.0	-	-	0
1	4		18.67	0.0	19.2	10.8	32.0	10.8	32.0	10.8	32.0	0
				0.0	-	8.1	-	8.1	32.0	-	-	0
1	5		23.33	0.0	19.2	12.3	32.0	12.3	32.0	12.3	32.0	0
				0.0	-	9.3	-	9.3	32.0	-	-	0
1	6		28.00	0.0	19.2	13.4	32.0	13.4	32.0	13.4	32.0	0
				0.0	-	10.1	-	10.1	32.0	-	-	0
1	7		32.67	0.0	19.2	14.1	32.0	14.1	32.0	14.1	32.0	0
				0.0	-	10.6	-	10.6	32.0	-	-	0
1	8		37.33	0.0	19.2	14.3	32.0	14.3	32.0	14.3	32.0	0
				0.0	-	10.8	-	10.8	32.0	-	-	0
1	9		42.00	0.0	19.2	14.1	32.0	14.1	32.0	14.1	32.0	0
				0.0	-	10.6	-	10.6	32.0	-	-	0
1	10		46.67	0.0	19.2	13.4	32.0	13.4	32.0	13.4	32.0	0
				0.0	-	10.1	-	10.1	32.0	-	-	0
1	11		51.33	0.0	19.2	12.3	32.0	12.3	32.0	12.3	32.0	0
				0.0	-	9.3	-	9.3	32.0	-	-	0
1	12		56.00	0.0	19.2	10.8	32.0	10.8	32.0	10.8	32.0	0
				0.0	-	8.1	-	8.1	32.0	-	-	0
1	13		60.67	0.0	19.2	9.6	32.0	9.6	32.0	9.6	32.0	0
				0.0	-	9.6	-	9.6	32.0	-	-	0
1	14		65.33	0.0	19.2	6.9	32.0	6.9	32.0	6.9	32.0	0
				0.0	-	6.9	-	6.9	32.0	-	-	0
1	15		70.00	0.0	19.2	3.7	32.0	3.7	32.0	3.7	32.0	0
				0.0	-	3.7	-	3.7	32.0	-	-	0
1	16		74.67	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	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

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		4.67	0.000	0.115	0.115	0.115	0.115	2
						0.115		0.115	3
1	2		9.33	0.000	0.214	0.214	0.214	0.214	2
						0.214		0.214	3
1	3		14.00	0.000	0.299	0.299	0.299	0.299	2
						0.299		0.299	3
1	4		18.67	0.000	0.336	0.336	0.336	0.336	2
						0.253		0.253	3
1	5		23.33	0.000	0.385	0.385	0.385	0.385	2
						0.289		0.289	3
1	6		28.00	0.000	0.420	0.420	0.420	0.420	2
						0.316		0.316	3
1	7		32.67	0.000	0.441	0.441	0.441	0.441	2
						0.332		0.332	3
1	8		37.33	0.000	0.448	0.448	0.448	0.448	2
						0.337		0.337	3
1	9		42.00	0.000	0.441	0.441	0.441	0.441	2
						0.332		0.332	3
1	10		46.67	0.000	0.420	0.420	0.420	0.420	2
						0.316		0.316	3
1	11		51.33	0.000	0.385	0.385	0.385	0.385	2
						0.289		0.289	3
1	12		56.00	0.000	0.336	0.336	0.336	0.336	2
						0.253		0.253	3
1	13		60.67	0.000	0.299	0.299	0.299	0.299	2
						0.299		0.299	3
1	14		65.33	0.000	0.214	0.214	0.214	0.214	2
						0.214		0.214	3
1	15		70.00	0.000	0.115	0.115	0.115	0.115	2
						0.115		0.115	3
1	16		74.67	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	2891.0	-	-	0.0	19.2	-	-	0--0
1	1		4.67	0	634.2	2891.0	-	-	0.0	19.2	-	-	0--0
1	2		9.33	0	1177.9	2891.0	-	-	0.0	19.2	-	-	0--0
1	3		14.00	0	1631.2	2891.0	-	-	0.0	19.2	-	-	0--0
1	4		18.67	0	1990.4	3806.8	-	-	0.0	19.2	-	-	0--0
1	5		23.33	0	2267.4	3806.8	-	-	0.0	19.2	-	-	0--0
1	6		28.00	0	2463.4	3806.8	-	-	0.0	19.2	-	-	0--0
1	7		32.67	0	2566.1	3806.8	-	-	0.0	19.2	-	-	0--0
1	8		37.33	0	2597.1	3806.8	-	-	0.0	19.2	-	-	0--0
1	9		42.00	0	2566.1	3806.8	-	-	0.0	19.2	-	-	0--0
1	10		46.67	0	2463.5	3806.8	-	-	0.0	19.2	-	-	0--0
1	11		51.33	0	2267.4	3806.8	-	-	0.0	19.2	-	-	0--0
1	12		56.00	0	1990.4	3806.8	-	-	0.0	19.2	-	-	0--0
1	13		60.67	0	1631.3	2891.0	-	-	0.0	19.2	-	-	0--0
1	14		65.33	0	1177.9	2891.0	-	-	0.0	19.2	-	-	0--0
1	15		70.00	0	634.2	2891.0	-	-	0.0	19.2	-	-	0--0
1	16		74.67	0	0.0	2891.0	-	-	0.0	19.2	-	-	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	4.67	0	0.219		0.219	1
1	2	9.33	0	0.407		0.407	1
1	3	14.00	0	0.564		0.564	1
1	4	18.67	0	0.523		0.523	1
1	5	23.33	0	0.596		0.596	1
1	6	28.00	0	0.647		0.647	1
1	7	32.67	0	0.674		0.674	1
1	8	37.33	0	0.682		0.682	1
1	9	42.00	0	0.674		0.674	1
1	10	46.67	0	0.647		0.647	1
1	11	51.33	0	0.596		0.596	1
1	12	56.00	0	0.523		0.523	1
1	13	60.67	0	0.564		0.564	1
1	14	65.33	0	0.407		0.407	1
1	15	70.00	0	0.219		0.219	1
1	16	74.67	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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CODE CHECK
FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 62\5800\Rating\MD\5

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 [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0		0.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	1		4.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	2		9.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	3		14.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	4		18.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	5		23.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	6		28.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	7		32.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	8		37.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	9		42.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	10		46.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	11		51.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	12		56.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	13		60.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	14		65.33	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	15		70.00	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9
1	16		74.67	32.0	5.0	75.4	94.2	44.5	1.000	481.9	481.9

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.

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

SP NO	IN D NO	FROM L (ft)	WEB STABILITY UNSTIFFENED,[1] SUPT ---- D/tw -----	LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPACITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00	0	190.8	<	481.9	0	0
1	1	4.67	0	172.3	<	481.9	0	0
1	2	9.33	0	154.4	<	481.9	0	0
1	3	14.00	0	136.8	<	481.9	0	0
1	4	18.67	0	119.1	<	481.9	0	0
1	5	23.33	0	102.0	<	481.9	0	0
1	6	28.00	0	85.0	<	481.9	0	0
1	7	32.67	0	68.1	<	481.9	0	0
1	8	37.33	0	51.6	<	481.9	0	0
1	9	42.00	0	68.1	<	481.9	0	0
1	10	46.67	0	85.0	<	481.9	0	0
1	11	51.33	0	102.0	<	481.9	0	0
1	12	56.00	0	119.1	<	481.9	0	0
1	13	60.67	0	136.8	<	481.9	0	0
1	14	65.33	0	154.4	<	481.9	0	0
1	15	70.00	0	172.3	<	481.9	0	0
1	16	74.67	0	190.8	<	481.9	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

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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 UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF CAPACITY (k)	TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE TRANS. STIFFENERS SPACING (ft-in)
1	0	0.00	32.	190.85	481.86	0	
1	1	4.67	32.	172.27	481.86	0	
1	2	9.33	32.	154.40	481.86	0	
1	3	14.00	32.	136.76	481.86	0	
1	4	18.67	32.	119.14	481.86	0	
1	5	23.33	32.	101.96	481.86	0	
1	6	28.00	32.	85.01	481.86	0	
1	7	32.67	32.	68.07	481.86	0	
1	8	37.33	32.	51.58	481.86	0	
1	9	42.00	32.	68.07	481.86	0	
1	10	46.67	32.	85.01	481.86	0	
1	11	51.33	32.	101.96	481.86	0	
1	12	56.00	32.	119.14	481.86	0	

1	13	60.67	32.	136.77	481.86	0
1	14	65.33	32.	154.41	481.86	0
1	15	70.00	32.	172.27	481.86	0
1	16	74.67	32.	190.85	481.86	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 [3] (ksi)	FLAG TBC
1	0	0.00	0.0	-		30.4	0.0	30.4	-	-	00-
1	1	4.67	3.9	-		30.4	7.2	30.4	-	-	00-
1	2	9.33	7.3	-		30.4	13.3	30.4	-	-	00-
1	3	14.00	10.2	-		30.4	18.4	30.4	-	-	00-
1	4	18.67	11.8	-		30.4	15.5	30.4	-	-	00-
1	5	23.33	13.5	-		30.4	17.7	30.4	-	-	00-
1	6	28.00	14.7	-		30.4	19.3	30.4	-	-	00-
1	7	32.67	15.3	-		30.4	20.1	30.4	-	-	00-
1	8	37.33	15.6	-		30.4	20.3	30.4	-	-	00-
1	9	42.00	15.3	-		30.4	20.1	30.4	-	-	00-
1	10	46.67	14.7	-		30.4	19.3	30.4	-	-	00-
1	11	51.33	13.5	-		30.4	17.7	30.4	-	-	00-
1	12	56.00	11.8	-		30.4	15.5	30.4	-	-	00-
1	13	60.67	10.2	-		30.4	18.4	30.4	-	-	00-
1	14	65.33	7.3	-		30.4	13.3	30.4	-	-	00-
1	15	70.00	3.9	-		30.4	7.2	30.4	-	-	00-
1	16	74.67	0.0	-		30.4	0.0	30.4	-	-	00-

1

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

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = II -- Urban 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		4.67	TR	-0.5	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	2		9.33	TR	-0.9	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	3		14.00	TR	-1.3	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1			14.83	SCG					
1	4		18.67	TR	-1.6	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1	5		23.33	TR	-1.7	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	6		28.00	TR	-1.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	7		32.67	TR	-1.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	8		37.33	TR	-1.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	9		42.00	TR	-1.8	A B B^C^C D E E^	TR	4.8	A B B^C^C D
1	10		46.67	TR	-1.9	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	11		51.33	TR	-1.7	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	12		56.00	TR	-1.6	A B B^C^C D E E^	TR	4.1	A B B^C^C D E
1			59.83	SCG					
1	13		60.67	TR	-1.3	A B B^C^C D E E^	TR	4.9	A B B^C^C D
1	14		65.33	TR	-0.9	A B B^C^C D E E^	TR	3.6	A B B^C^C D E
1	15		70.00	TR	-0.5	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1	16		74.67	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 = II -- Urban 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		4.67	TR	-0.3	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	2		9.33	TR	-0.5	A B B^C^C D E E^	TR	1.8	A B B^C^C D E E^
1	3		14.00	TR	-0.6	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1			14.83	SCG					

| | | |

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= 2038
*** 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	26.0	.24083E-01	0.6	TR 0.76	3	3.63
1	1	4.67	24.8	.24083E-01	0.6	TR 0.76	3	3.81
1	2	9.33	23.5	.24083E-01	0.6	TR 0.76	3	4.00
1	3	14.00	22.4	.24083E-01	0.5	TR 0.76	3	4.21
1	4	18.67	21.4	.22197E-01	0.5	TR 0.76	3	4.77
1	5	23.33	20.4	.22197E-01	0.5	TR 0.76	3	5.00
1	6	28.00	19.5	.22197E-01	0.4	TR 0.76	3	5.26
1	7	32.67	18.6	.22197E-01	0.4	TR 0.76	3	5.50
1	8	37.33	18.3	.22197E-01	0.4	TR 0.76	3	5.60
1	9	42.00	18.6	.22197E-01	0.4	TR 0.76	3	5.50
1	10	46.67	19.5	.22197E-01	0.4	TR 0.76	3	5.26
1	11	51.33	20.4	.22197E-01	0.5	TR 0.76	3	5.00
1	12	56.00	21.4	.22197E-01	0.5	TR 0.76	3	4.77
1	13	60.67	22.4	.24083E-01	0.5	TR 0.76	3	4.21
1	14	65.33	23.5	.24083E-01	0.6	TR 0.76	3	4.00
1	15	70.00	24.8	.24083E-01	0.6	TR 0.76	3	3.81
1	16	74.67	26.0	.24083E-01	0.6	TR 0.76	3	3.63

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= 2038 & No. of Cycles= 55790250
*** 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 SUPT (ft)	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k) FATIG.	P2n (k) CRI.	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	SEE NOTE N2	STATUS BLANK=OK CHECK=**
1	8	37.33	1	2179.8	1115.6		195	>	67		
1	16	74.67	1	2179.8	1115.6		195	>	67		

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	37.333	7.000
1	37.333	74.667	7.000

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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	74.7	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 D NO	FROM L (ft)	SUPT (ft)	MOMENT CAP. (k-ft)	UNFACTORED DEAD LOAD MOMENT D1; k-ft	D2	UNFACTORED L+I MOMENT MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T

				or STRESS	ksi	MAX. STRESS					
1	0	0.00	2891.0	0.0	0.0	0.0	0.0	9.99	HL-93	9.99	0
1	1	4.67	2891.0	135.2	59.3	217.9	0.0	8.97	HL-93	6.92	0
1	2	9.33	2891.0	252.5	110.7	403.3	0.0	4.44	HL-93	3.43	0
1	3	14.00	2891.0	352.0	154.2	556.2	0.0	2.97	HL-93	2.29	0
1	4	18.67	3806.8	433.0	189.7	674.8	0.0	3.29	HL-93	2.54	0
1	5	23.33	3806.8	496.1	217.4	765.7	0.0	2.79	HL-93	2.15	0
1	6	28.00	3806.8	541.4	237.2	829.4	0.0	2.50	HL-93	1.93	0
1	7	32.67	3806.8	568.4	249.0	859.2	0.0	2.37	HL-93	1.83	0
1	8	37.33	3806.8	577.3	253.0	867.3	0.0	2.33	HL-93	1.80	0
1	9	42.00	3806.8	568.4	249.0	859.2	0.0	2.37	HL-93	1.83	0
1	10	46.67	3806.8	541.4	237.2	829.4	0.0	2.50	HL-93	1.93	0
1	11	51.33	3806.8	496.1	217.4	765.7	0.0	2.79	HL-93	2.15	0
1	12	56.00	3806.8	433.0	189.7	674.8	0.0	3.29	HL-93	2.54	0
1	13	60.67	2891.0	352.0	154.2	556.2	0.0	2.97	HL-93	2.29	0
1	14	65.33	2891.0	252.5	110.7	403.3	0.0	4.44	HL-93	3.43	0
1	15	70.00	2891.0	135.2	59.3	217.9	0.0	8.97	HL-93	6.92	0
1	16	74.67	2891.0	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	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	30.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	4.67	30.4	-2.94 3.34	-0.77 2.94	0.00 0.00	9.21	HL-93	7.08
1	2	9.33	30.4	-5.49 6.23	-1.42 5.44	0.00 0.00	4.44	HL-93	3.42
1	3	14.00	30.4	-7.65 8.69	-1.96 7.50	0.00 0.00	2.90	HL-93	2.23
1	4	18.67	30.4	-8.67 7.36	-2.40 6.30	0.00 0.00	3.66	HL-93	2.81
1	5	23.33	30.4	-9.93 8.43	-2.72 7.15	0.00 0.00	3.07	HL-93	2.36

1	6	28.00	30.4	-10.84	9.20	-2.95	7.74	0.00	0.00	2.74	HL-93	2.11
1	7	32.67	30.4	-11.38	9.66	-3.06	8.02	0.00	0.00	2.59	HL-93	1.99
1	8	37.33	30.4	-11.56	9.81	-3.08	8.10	0.00	0.00	2.54	HL-93	1.96
1	9	42.00	30.4	-11.38	9.66	-3.06	8.02	0.00	0.00	2.59	HL-93	1.99
1	10	46.67	30.4	-10.84	9.20	-2.95	7.74	0.00	0.00	2.74	HL-93	2.11
1	11	51.33	30.4	-9.93	8.43	-2.72	7.15	0.00	0.00	3.07	HL-93	2.36
1	12	56.00	30.4	-8.67	7.36	-2.40	6.30	0.00	0.00	3.66	HL-93	2.81
1	13	60.67	30.4	-7.65	8.69	-1.96	7.50	0.00	0.00	2.90	HL-93	2.23
1	14	65.33	30.4	-5.49	6.23	-1.42	5.44	0.00	0.00	4.44	HL-93	3.42
1	15	70.00	30.4	-2.94	3.34	-0.77	2.94	0.00	0.00	9.21	HL-93	7.08
1	16	74.67	30.4	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	74.67	37.33	STRENGTH L.S.	2.33	1.80	HL-93
1	74.67	37.33	SERVICEBILITY	2.54	1.96	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.330
Inventory= 1.797

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	74.67	32.67	STRENGTH L.S.	3.14	874.02
1	74.67	32.67	SERVICEBILITY	3.36	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 3.140

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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	481.9	31.2	13.6	75.8	0.0	4.14	HL-93	3.19
1	1	4.67	481.9	27.0	11.9	69.5	-2.1	4.59	HL-93	3.54
1	2	9.33	481.9	23.2	10.2	63.4	-4.9	5.12	HL-93	3.95
1	3	14.00	481.9	19.4	8.5	57.4	-8.1	5.75	HL-93	4.43
1	4	18.67	481.9	15.4	6.8	51.6	-11.5	6.50	HL-93	5.02
1	5	23.33	481.9	11.6	5.1	45.9	-15.0	7.43	HL-93	5.73
1	6	28.00	481.9	7.8	3.4	40.3	-19.3	8.60	HL-93	6.63
1	7	32.67	481.9	3.8	1.7	34.8	-24.3	9.99	HL-93	7.79
1	8	37.33	481.9	0.0	0.0	29.5	-29.5	9.99	HL-93	9.34
1	9	42.00	481.9	-3.8	-1.7	24.3	-34.8	9.99	HL-93	7.79
1	10	46.67	481.9	-7.8	-3.4	19.3	-40.3	8.60	HL-93	6.63
1	11	51.33	481.9	-11.6	-5.1	15.0	-45.9	7.43	HL-93	5.73
1	12	56.00	481.9	-15.4	-6.8	11.5	-51.6	6.50	HL-93	5.02
1	13	60.67	481.9	-19.4	-8.5	8.1	-57.4	5.75	HL-93	4.43
1	14	65.33	481.9	-23.2	-10.2	4.9	-63.4	5.12	HL-93	3.95
1	15	70.00	481.9	-27.0	-11.9	2.1	-69.5	4.59	HL-93	3.54
1	16	74.67	481.9	-31.2	-13.6	0.0	-75.8	4.14	HL-93	3.19

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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	74.67	74.67	STRENGTH L.S.	4.14	3.19	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.140
Inventory= 3.194

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	74.67	74.67	STRENGTH L.S.	4.84	87.62

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 4.835

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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	37.33	2	0.400	-0.53 HL-93	2.11
		3	0.510	-0.68	1.66

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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                PROVISIONS OF THIS AGREEMENT HAVE BEEN BREACHED OR
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TABLE 0.0.1.1 PROJECT DATA
****

DESCRIPTION	DATE
MaineDOT Bridge No. 5800 Span 2 Exterior Beam	7/17/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 2(5)	IAS/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				PROGRAM FLOW CONTROL
			CODE ID	YEAR	UNIT TYPE	DESIGN OPTION	
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
****

BEAM NUMBER	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
5	2	31.33	2.50	1.83	1.57	22.60	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
91.42	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
7.50	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 DEPTH ID.	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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	36	280		0.00	0.0000	0.00	0.0000		
2	W	36	280		0.00	0.0000	15.00	1.1875		

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	19.21				32.	32.	32.
2	2	2	53.00				32.	32.	32.
3	1	1	19.21				32.	32.	32.

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	2	2398	2038	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 2398

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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 (%)	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	0.47	2 2 2 2 2
1	0.00	0	0.00	0.55	0.00	0.00	0 0 0 0 0	0.70	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	7.0	0	30.0	10.0	0.58	0.00	91.42

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	0	wearing surf	0.238	0.00	91.42	0.00	0.00
2	1	rail	0.010	0.00	91.42	0.00	0.00
3	1	curb	0.115	0.00	91.42	0.00	0.00
4	2	end diaphrag	0.000	0.00	0.00	0.38	0.45
5	2	int diaphrag	0.000	0.00	0.00	0.17	20.34
6	2	int diaphrag	0.000	0.00	0.00	0.17	36.34
7	2	int diaphrag	0.000	0.00	0.00	0.17	52.34
8	2	int diaphrag	0.000	0.00	0.00	0.17	68.34
9	2	int diaphrag	0.000	0.00	0.00	0.17	84.34
10	2	end diaphrag	0.000	0.00	0.00	0.38	90.96

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 (in)	CONNECTOR TYPE 0=NO 1=YES	AASHTO ART. 6.10.10.4 NEGAT. REGION (kip / per connect.)	AASHTO ART. 6.10.10.2 (kip / per connector) Truck	Lane	REBAR YIELD Fy (ksi)	BAR AREA PER FOOT OF SLAB (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	2.50	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		YIELD STRESS		SPACING OF LATERAL BRACING
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	20.34	33.0	33.0	19.89
20.34	84.34	33.0	33.0	16.00
84.34	90.96	33.0	33.0	6.62

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.	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 NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)
1	0.579		0.3155		0.8945

SUPERIMPOSED DEAD LOADS
****

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.24	0.00	91.42
1	DC2	0.010	0.000	91.417
2	DC2	0.115	0.000	91.417

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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	1866.4	69.1	280.0	144.2	424.19

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	7.50	75.00

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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	0.471	0.471	0.649	0.649	FOR STRENGTH POSITIVE MOMENT
	0.696				FOR STRENGTH POSITIVE SHEAR
1	0.370				FOR FATIGUE POSITIVE MOMENT
	0.547				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)	STEEL TOP. (in**3)
1	0	0.00	18900.0	36.5	18.26	1035.0	1035.0

1	1	4.57	18900.0	36.5	18.26	1035.0	1035.0
1	2	9.14	18900.0	36.5	18.26	1035.0	1035.0
1	3	13.71	18900.0	36.5	18.26	1035.0	1035.0
1	4	18.28	18900.0	36.5	18.26	1035.0	1035.0
1	5	22.85	24141.4	36.5	16.10	1499.8	1117.1
1	6	27.43	24141.4	36.5	16.10	1499.8	1117.1
1	7	32.00	24141.4	36.5	16.10	1499.8	1117.1
1	8	36.57	24141.4	36.5	16.10	1499.8	1117.1
1	9	41.14	24141.4	36.5	16.10	1499.8	1117.1
1	10	45.71	24141.4	36.5	16.10	1499.8	1117.1
1	11	50.28	24141.4	36.5	16.10	1499.8	1117.1
1	12	54.85	24141.4	36.5	16.10	1499.8	1117.1
1	13	59.42	24141.4	36.5	16.10	1499.8	1117.1
1	14	63.99	24141.4	36.5	16.10	1499.8	1117.1
1	15	68.56	24141.4	36.5	16.10	1499.8	1117.1
1	16	73.13	18900.0	36.5	18.26	1035.0	1035.0
1	17	77.70	18900.0	36.5	18.26	1035.0	1035.0
1	18	82.28	18900.0	36.5	18.26	1035.0	1035.0
1	19	86.85	18900.0	36.5	18.26	1035.0	1035.0
1	20	91.42	18900.0	36.5	18.26	1035.0	1035.0

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 = 30.00

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	25864.9	0.012737	1169.6	1795.5	1208.3
1	1	4.57	25864.9	0.012737	1169.6	1795.5	1208.3
1	2	9.14	25864.9	0.012737	1169.6	1795.5	1208.3
1	3	13.71	25864.9	0.012737	1169.6	1795.5	1208.3
1	4	18.28	25864.9	0.012737	1169.6	1795.5	1208.3
1	5	22.85	33699.2	0.011676	1695.6	1889.8	1357.1
1	6	27.43	33699.2	0.011676	1695.6	1889.8	1357.1
1	7	32.00	33699.2	0.011676	1695.6	1889.8	1357.1
1	8	36.57	33699.2	0.011676	1695.6	1889.8	1357.1
1	9	41.14	33699.2	0.011676	1695.6	1889.8	1357.1
1	10	45.71	33699.2	0.011676	1695.6	1889.8	1357.1
1	11	50.28	33699.2	0.011676	1695.6	1889.8	1357.1
1	12	54.85	33699.2	0.011676	1695.6	1889.8	1357.1
1	13	59.42	33699.2	0.011676	1695.6	1889.8	1357.1
1	14	63.99	33699.2	0.011676	1695.6	1889.8	1357.1
1	15	68.56	33699.2	0.011676	1695.6	1889.8	1357.1
1	16	73.13	25864.9	0.012737	1169.6	1795.5	1208.3
1	17	77.70	25864.9	0.012737	1169.6	1795.5	1208.3
1	18	82.28	25864.9	0.012737	1169.6	1795.5	1208.3
1	19	86.85	25864.9	0.012737	1169.6	1795.5	1208.3
1	20	91.42	25864.9	0.012737	1169.6	1795.5	1208.3

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TABLE 1.2.4.3=COMPOSITE SECTION PROPERTIES FOR N = 10.00

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	34375.3	0.021127	1283.1	3533.3	2054.8
1	1	4.57	34375.3	0.021127	1283.1	3533.3	2054.8
1	2	9.14	34375.3	0.021127	1283.1	3533.3	2054.8
1	3	13.71	34375.3	0.021127	1283.1	3533.3	2054.8
1	4	18.28	34375.3	0.021127	1283.1	3533.3	2054.8
1	5	22.85	46219.9	0.019563	1863.4	3581.9	2322.2
1	6	27.43	46219.9	0.019563	1863.4	3581.9	2322.2
1	7	32.00	46219.9	0.019563	1863.4	3581.9	2322.2
1	8	36.57	46219.9	0.019563	1863.4	3581.9	2322.2
1	9	41.14	46219.9	0.019563	1863.4	3581.9	2322.2
1	10	45.71	46219.9	0.019563	1863.4	3581.9	2322.2
1	11	50.28	46219.9	0.019563	1863.4	3581.9	2322.2
1	12	54.85	46219.9	0.019563	1863.4	3581.9	2322.2
1	13	59.42	46219.9	0.019563	1863.4	3581.9	2322.2
1	14	63.99	46219.9	0.019563	1863.4	3581.9	2322.2
1	15	68.56	46219.9	0.019563	1863.4	3581.9	2322.2
1	16	73.13	34375.3	0.021127	1283.1	3533.3	2054.8
1	17	77.70	34375.3	0.021127	1283.1	3533.3	2054.8
1	18	82.28	34375.3	0.021127	1283.1	3533.3	2054.8
1	19	86.85	34375.3	0.021127	1283.1	3533.3	2054.8
1	20	91.42	34375.3	0.021127	1283.1	3533.3	2054.8

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 NO	IN NO	D FROM L SUPT (ft)	BEAM (k-ft)	SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	NONCOMPOSITE DEAD	NONCOMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0		0.0
1	1	4.57	62.6	114.9	1.9	0.0		179.4
1	2	9.14	118.7	217.7	3.6	0.0		340.0
1	3	13.71	168.1	308.5	5.3	0.0		481.9
1	4	18.28	211.0	387.1	7.0	0.0		605.1
1	5	22.85	247.2	453.6	8.3	0.0		709.1
1	6	27.43	276.9	508.1	9.2	0.0		794.1
1	7	32.00	299.9	550.4	10.1	0.0		860.5
1	8	36.57	316.4	580.6	11.0	0.0		908.0
1	9	41.14	326.3	598.8	11.1	0.0		936.2
1	10	45.71	329.6	604.8	11.2	0.0		945.6
1	11	50.28	326.3	598.8	11.3	0.0		936.4
1	12	54.85	316.4	580.6	11.0	0.0		908.0
1	13	59.42	299.9	550.4	10.3	0.0		860.6
1	14	63.99	276.9	508.1	9.6	0.0		794.5
1	15	68.56	247.2	453.6	8.9	0.0		709.7
1	16	73.13	211.0	387.1	7.4	0.0		605.4
1	17	77.70	168.1	308.5	5.9	0.0		482.5
1	18	82.28	118.7	217.7	4.4	0.0		340.8
1	19	86.85	62.6	114.9	2.5	0.0		180.0
1	20	91.42	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 = 30.00 (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	UNIFORM SUPERIMPOSED DEAD (k-ft) DW	UNIFORM SUPERIMPOSED DEAD (k-ft) DC2	CONCENTRATED LOADS (k-ft) DW	CONCENTRATED LOADS (k-ft) DC2	OTHER UNIFORM LOADS	TOTAL (k-ft) -- COMPOSITE DEAD	TOTAL (k-ft) -- COMPOSITE LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0		0.0
1	1	4.57	47.2	24.8	0.0	0.0	0.0		72.0
1	2	9.14	89.5	47.0	0.0	0.0	0.0		136.5
1	3	13.71	126.8	66.6	0.0	0.0	0.0		193.4
1	4	18.28	159.1	83.6	0.0	0.0	0.0		242.7
1	5	22.85	186.5	97.9	0.0	0.0	0.0		284.4
1	6	27.43	208.8	109.7	0.0	0.0	0.0		318.5
1	7	32.00	226.2	118.8	0.0	0.0	0.0		345.1
1	8	36.57	238.7	125.4	0.0	0.0	0.0		364.0
1	9	41.14	246.1	129.3	0.0	0.0	0.0		375.4
1	10	45.71	248.6	130.6	0.0	0.0	0.0		379.2
1	11	50.28	246.1	129.3	0.0	0.0	0.0		375.4
1	12	54.85	238.7	125.4	0.0	0.0	0.0		364.0
1	13	59.42	226.2	118.8	0.0	0.0	0.0		345.1
1	14	63.99	208.8	109.7	0.0	0.0	0.0		318.5
1	15	68.56	186.5	97.9	0.0	0.0	0.0		284.4
1	16	73.13	159.1	83.6	0.0	0.0	0.0		242.7
1	17	77.70	126.8	66.6	0.0	0.0	0.0		193.4
1	18	82.28	89.5	47.0	0.0	0.0	0.0		136.5
1	19	86.85	47.2	24.8	0.0	0.0	0.0		72.0
1	20	91.42	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 = 10.00 (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	4.57	0.0	0.0	234.6	HL-93	0.0	HL-93
1	2	9.14	0.0	0.0	442.4	HL-93	0.0	HL-93
1	3	13.71	0.0	0.0	623.2	HL-93	0.0	HL-93
1	4	18.28	0.0	0.0	777.0	HL-93	0.0	HL-93
1	5	22.85	0.0	0.0	902.3	HL-93	0.0	HL-93
1	6	27.43	0.0	0.0	1004.1	HL-93	0.0	HL-93
1	7	32.00	0.0	0.0	1083.4	HL-93	0.0	HL-93
1	8	36.57	0.0	0.0	1137.5	HL-93	0.0	HL-93
1	9	41.14	0.0	0.0	1162.1	HL-93	0.0	HL-93
1	10	45.71	0.0	0.0	1170.3	HL-93	0.0	HL-93
1	11	50.28	0.0	0.0	1162.1	HL-93	0.0	HL-93
1	12	54.85	0.0	0.0	1137.5	HL-93	0.0	HL-93
1	13	59.42	0.0	0.0	1083.4	HL-93	0.0	HL-93
1	14	63.99	0.0	0.0	1004.1	HL-93	0.0	HL-93
1	15	68.56	0.0	0.0	902.3	HL-93	0.0	HL-93
1	16	73.13	0.0	0.0	777.0	HL-93	0.0	HL-93
1	17	77.70	0.0	0.0	623.2	HL-93	0.0	HL-93
1	18	82.28	0.0	0.0	442.4	HL-93	0.0	HL-93
1	19	86.85	0.0	0.0	234.6	HL-93	0.0	HL-93
1	20	91.42	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 = 10.0 (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	4.57	105.	0.	105.
1	2	9.14	196.	0.	196.
1	3	13.71	272.	0.	272.
1	4	18.28	336.	0.	336.
1	5	22.85	390.	0.	390.
1	6	27.43	432.	0.	432.
1	7	32.00	463.	0.	463.
1	8	36.57	480.	0.	480.
1	9	41.14	474.	0.	474.
1	10	45.71	472.	0.	472.
1	11	50.28	474.	0.	474.
1	12	54.85	480.	0.	480.
1	13	59.42	463.	0.	463.
1	14	63.99	432.	0.	432.
1	15	68.56	390.	0.	390.
1	16	73.13	336.	0.	336.
1	17	77.70	272.	0.	272.
1	18	82.28	196.	0.	196.
1	19	86.85	105.	0.	105.
1	20	91.42	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=10.0 LOAD TYPE= HL - 93 COMP. N=30.0 (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	4.57	179.4	72.0	234.6	HL-93	0.0	HL-93	486.1	251.5
1	2	9.14	340.0	136.5	442.4	HL-93	0.0	HL-93	918.9	476.5
1	3	13.71	481.9	193.4	623.2	HL-93	0.0	HL-93	1298.4	675.3
1	4	18.28	605.1	242.7	777.0	HL-93	0.0	HL-93	1624.8	847.8
1	5	22.85	709.1	284.4	902.3	HL-93	0.0	HL-93	1895.8	993.5
1	6	27.43	794.1	318.5	1004.1	HL-93	0.0	HL-93	2116.8	1112.7
1	7	32.00	860.5	345.1	1083.4	HL-93	0.0	HL-93	2288.9	1205.5
1	8	36.57	908.0	364.0	1137.5	HL-93	0.0	HL-93	2409.6	1272.1
1	9	41.14	936.2	375.4	1162.1	HL-93	0.0	HL-93	2473.7	1311.6
1	10	45.71	945.6	379.2	1170.3	HL-93	0.0	HL-93	2495.1	1324.8
1	11	50.28	936.4	375.4	1162.1	HL-93	0.0	HL-93	2473.9	1311.8
1	12	54.85	908.0	364.0	1137.5	HL-93	0.0	HL-93	2409.6	1272.1
1	13	59.42	860.6	345.1	1083.4	HL-93	0.0	HL-93	2289.1	1205.7
1	14	63.99	794.5	318.5	1004.1	HL-93	0.0	HL-93	2117.2	1113.1
1	15	68.56	709.7	284.4	902.3	HL-93	0.0	HL-93	1896.4	994.1
1	16	73.13	605.4	242.7	777.0	HL-93	0.0	HL-93	1625.2	848.1
1	17	77.70	482.5	193.4	623.2	HL-93	0.0	HL-93	1299.0	675.8
1	18	82.28	340.8	136.5	442.4	HL-93	0.0	HL-93	919.7	477.3
1	19	86.85	180.0	72.0	234.6	HL-93	0.0	HL-93	486.7	252.1
1	20	91.42	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	4.57	486.1	556.5	736.8	642.9	377.2	78.8
1	2	9.14	918.9	1051.6	1392.1	1215.2	714.8	147.2
1	3	13.71	1298.4	1485.4	1966.3	1717.0	1012.9	203.9
1	4	18.28	1624.8	1857.9	2459.3	2148.5	1271.6	252.3
1	5	22.85	1895.8	2166.5	2867.5	2506.6	1490.3	292.7
1	6	27.43	2116.8	2418.0	3200.2	2798.6	1669.0	324.1
1	7	32.00	2288.9	2613.9	3459.4	3026.0	1808.3	347.3
1	8	36.57	2409.6	2750.8	3640.4	3185.4	1908.1	360.1
1	9	41.14	2473.7	2822.3	3734.7	3269.9	1967.4	355.5
1	10	45.71	2495.1	2846.2	3766.2	3298.1	1987.3	354.0
1	11	50.28	2473.9	2822.5	3735.0	3270.1	1967.7	355.5
1	12	54.85	2409.6	2750.8	3640.4	3185.4	1908.1	360.1
1	13	59.42	2289.1	2614.1	3459.6	3026.3	1808.6	347.3
1	14	63.99	2117.2	2418.4	3200.7	2799.1	1669.6	324.1
1	15	68.56	1896.4	2167.1	2868.2	2507.3	1491.2	292.7
1	16	73.13	1625.2	1858.3	2459.8	2148.9	1272.2	252.3
1	17	77.70	1299.0	1485.9	1967.0	1717.8	1013.8	203.9
1	18	82.28	919.7	1052.4	1393.1	1216.2	716.0	147.2
1	19	86.85	486.7	557.1	737.5	643.6	378.1	78.8
1	20	91.42	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	14.4	26.5	0.8	0.0	41.6
1	1	4.57	13.0	23.8	0.4	0.0	37.2
1	2	9.14	11.5	21.2	0.4	0.0	33.1
1	3	13.71	10.1	18.5	0.4	0.0	29.0
1	4	18.28	8.7	15.9	0.4	0.0	24.9
1	5	22.85	7.2	13.2	0.2	0.0	20.6
1	6	27.43	5.8	10.6	0.2	0.0	16.6
1	7	32.00	4.3	7.9	0.2	0.0	12.5
1	8	36.57	2.9	5.3	0.0	0.0	8.2
1	9	41.14	1.4	2.6	0.0	0.0	4.1
1	10	45.71	0.0	0.0	0.0	0.0	0.0
1	11	50.28	-1.4	-2.6	0.0	0.0	-4.1
1	12	54.85	-2.9	-5.3	-0.2	0.0	-8.3
1	13	59.42	-4.3	-7.9	-0.2	0.0	-12.4
1	14	63.99	-5.8	-10.6	-0.2	0.0	-16.5
1	15	68.56	-7.2	-13.2	-0.3	0.0	-20.8
1	16	73.13	-8.7	-15.9	-0.3	0.0	-24.9
1	17	77.70	-10.1	-18.5	-0.3	0.0	-28.9
1	18	82.28	-11.5	-21.2	-0.3	0.0	-33.0
1	19	86.85	-13.0	-23.8	-0.5	0.0	-37.3
1	20	91.42	-14.4	-26.5	-0.9	0.0	-41.8

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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	10.9	5.7	0.0	0.0	16.6	41.6	58.2
1	1	4.57	9.8	5.1	0.0	0.0	14.9	37.2	52.1
1	2	9.14	8.7	4.6	0.0	0.0	13.3	33.1	46.4
1	3	13.71	7.6	4.0	0.0	0.0	11.6	29.0	40.6
1	4	18.28	6.5	3.4	0.0	0.0	10.0	24.9	34.9
1	5	22.85	5.4	2.9	0.0	0.0	8.3	20.6	28.9
1	6	27.43	4.4	2.3	0.0	0.0	6.6	16.6	23.2
1	7	32.00	3.3	1.7	0.0	0.0	5.0	12.5	17.4
1	8	36.57	2.2	1.1	0.0	0.0	3.3	8.2	11.5
1	9	41.14	1.1	0.6	0.0	0.0	1.7	4.1	5.8
1	10	45.71	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	11	50.28	-1.1	-0.6	0.0	0.0	-1.7	-4.1	-5.7
1	12	54.85	-2.2	-1.1	0.0	0.0	-3.3	-8.3	-11.6
1	13	59.42	-3.3	-1.7	0.0	0.0	-5.0	-12.4	-17.4
1	14	63.99	-4.4	-2.3	0.0	0.0	-6.6	-16.5	-23.1
1	15	68.56	-5.4	-2.9	0.0	0.0	-8.3	-20.8	-29.1
1	16	73.13	-6.5	-3.4	0.0	0.0	-10.0	-24.9	-34.8
1	17	77.70	-7.6	-4.0	0.0	0.0	-11.6	-28.9	-40.6
1	18	82.28	-8.7	-4.6	0.0	0.0	-13.3	-33.0	-46.3
1	19	86.85	-9.8	-5.1	0.0	0.0	-14.9	-37.3	-52.2
1	20	91.42	-10.9	-5.7	0.0	0.0	-16.6	-41.8	-58.4

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	80.2	HL-93	0.0	HL-93
1	1	4.57	0.0	0.0	74.9	HL-93	-1.7	HL-93
1	2	9.14	0.0	0.0	69.7	HL-93	-3.8	HL-93
1	3	13.71	0.0	0.0	64.5	HL-93	-6.4	HL-93
1	4	18.28	0.0	0.0	59.5	HL-93	-9.1	HL-93
1	5	22.85	0.0	0.0	54.6	HL-93	-11.8	HL-93
1	6	27.43	0.0	0.0	49.8	HL-93	-15.1	HL-93
1	7	32.00	0.0	0.0	45.1	HL-93	-19.0	HL-93
1	8	36.57	0.0	0.0	40.5	HL-93	-23.1	HL-93
1	9	41.14	0.0	0.0	36.0	HL-93	-27.3	HL-93
1	10	45.71	0.0	0.0	31.6	HL-93	-31.6	HL-93
1	11	50.28	0.0	0.0	27.3	HL-93	-36.0	HL-93
1	12	54.85	0.0	0.0	23.1	HL-93	-40.5	HL-93
1	13	59.42	0.0	0.0	19.0	HL-93	-45.1	HL-93
1	14	63.99	0.0	0.0	15.1	HL-93	-49.8	HL-93
1	15	68.56	0.0	0.0	11.8	HL-93	-54.6	HL-93
1	16	73.13	0.0	0.0	9.1	HL-93	-59.5	HL-93
1	17	77.70	0.0	0.0	6.4	HL-93	-64.5	HL-93
1	18	82.28	0.0	0.0	3.8	HL-93	-69.7	HL-93
1	19	86.85	0.0	0.0	1.7	HL-93	-74.9	HL-93
1	20	91.42	0.0	0.0	0.0	HL-93	-80.2	HL-93

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TABLE 1.2.6.3A=LIVE LOAD SHEAR RANGE FOR N = 10.0 (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	4.57	34.	-1.	35.
1	2	9.14	32.	-2.	34.
1	3	13.71	29.	-3.	33.
1	4	18.28	27.	-4.	31.
1	5	22.85	25.	-5.	30.
1	6	27.43	23.	-7.	29.
1	7	32.00	20.	-8.	28.
1	8	36.57	18.	-9.	28.
1	9	41.14	16.	-12.	27.
1	10	45.71	14.	-14.	27.
1	11	50.28	12.	-16.	27.
1	12	54.85	9.	-18.	28.
1	13	59.42	8.	-20.	28.
1	14	63.99	7.	-23.	29.
1	15	68.56	5.	-25.	30.
1	16	73.13	4.	-27.	31.
1	17	77.70	3.	-29.	33.
1	18	82.28	2.	-32.	34.
1	19	86.85	1.	-34.	35.
1	20	91.42	0.	-36.	36.

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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	36.3	.27065E-01	0.49	0.3125*	.18171E-01	0.33	0.3125*
1	1		4.57	35.0	.27065E-01	0.47	0.3125*	.18171E-01	0.32	0.3125*
1	2		9.14	33.7	.27065E-01	0.46	0.3125*	.18171E-01	0.31	0.3125*
1	3		13.71	32.6	.27065E-01	0.44	0.3125*	.18171E-01	0.30	0.3125*
1	4		18.28	31.5	.27065E-01	0.43	0.3125*	.18171E-01	0.29	0.3125*
1	5		22.85	30.4	.25685E-01	0.39	0.3125*	.11807E-01	0.18	0.3125*
1	6		27.43	29.4	.25685E-01	0.38	0.3125*	.11807E-01	0.17	0.3125*
1	7		32.00	28.4	.25685E-01	0.36	0.3125*	.11807E-01	0.17	0.3125*
1	8		36.57	27.6	.25685E-01	0.35	0.3125*	.11807E-01	0.16	0.3125*
1	9		41.14	27.3	.25685E-01	0.35	0.3125*	.11807E-01	0.16	0.3125*
1	10		45.71	27.2	.25685E-01	0.35	0.3125*	.11807E-01	0.16	0.3125*
1	11		50.28	27.3	.25685E-01	0.35	0.3125*	.11807E-01	0.16	0.3125*
1	12		54.85	27.6	.25685E-01	0.35	0.3125*	.11807E-01	0.16	0.3125*
1	13		59.42	28.4	.25685E-01	0.36	0.3125*	.11807E-01	0.17	0.3125*
1	14		63.99	29.4	.25685E-01	0.38	0.3125*	.11807E-01	0.17	0.3125*
1	15		68.56	30.4	.25685E-01	0.39	0.3125*	.11807E-01	0.18	0.3125*
1	16		73.13	31.5	.27065E-01	0.43	0.3125*	.18171E-01	0.29	0.3125*
1	17		77.70	32.6	.27065E-01	0.44	0.3125*	.18171E-01	0.30	0.3125*
1	18		82.28	33.7	.27065E-01	0.46	0.3125*	.18171E-01	0.31	0.3125*
1	19		86.85	35.0	.27065E-01	0.47	0.3125*	.18171E-01	0.32	0.3125*
1	20		91.42	36.3	.27065E-01	0.49	0.3125*	.18171E-01	0.33	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	58.2	80.2	HL-93	0.0	HL-93	138.4	58.2
1	1	4.57	52.1	74.9	HL-93	-1.7	HL-93	127.0	50.4
1	2	9.14	46.4	69.7	HL-93	-3.8	HL-93	116.0	42.5
1	3	13.71	40.6	64.5	HL-93	-6.4	HL-93	105.2	34.2
1	4	18.28	34.9	59.5	HL-93	-9.1	HL-93	94.4	25.8
1	5	22.85	28.9	54.6	HL-93	-11.8	HL-93	83.6	17.1
1	6	27.43	23.2	49.8	HL-93	-15.1	HL-93	73.0	8.1
1	7	32.00	17.4	45.1	HL-93	-19.0	HL-93	62.6	-1.6
1	8	36.57	11.5	40.5	HL-93	-23.1	HL-93	52.0	-11.6
1	9	41.14	5.8	36.0	HL-93	-27.3	HL-93	41.8	-21.5
1	10	45.71	0.0	31.6	HL-93	-31.6	HL-93	31.6	-31.6
1	11	50.28	-5.7	27.3	HL-93	-36.0	HL-93	21.6	-41.7
1	12	54.85	-11.6	23.1	HL-93	-40.5	HL-93	11.5	-52.2
1	13	59.42	-17.4	19.0	HL-93	-45.1	HL-93	1.6	-62.5
1	14	63.99	-23.1	15.1	HL-93	-49.8	HL-93	-8.1	-73.0
1	15	68.56	-29.1	11.8	HL-93	-54.6	HL-93	-17.2	-83.7
1	16	73.13	-34.8	9.1	HL-93	-59.5	HL-93	-25.8	-94.4
1	17	77.70	-40.6	6.4	HL-93	-64.5	HL-93	-34.2	-105.1
1	18	82.28	-46.3	3.8	HL-93	-69.7	HL-93	-42.5	-116.0
1	19	86.85	-52.2	1.7	HL-93	-74.9	HL-93	-50.5	-127.1
1	20	91.42	-58.4	0.0	HL-93	-80.2	HL-93	-58.4	-138.6

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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	138.4	162.5	215.9	183.8	87.4	27.2
1	1	4.57	127.0	149.4	198.6	168.7	78.2	26.2
1	2	9.14	116.0	136.9	182.0	154.2	69.5	25.3
1	3	13.71	105.2	124.5	165.6	139.8	60.9	24.4
1	4	18.28	94.4	112.3	149.4	125.6	52.3	23.6
1	5	22.85	83.6	100.0	133.1	111.3	43.4	22.8
1	6	27.43	73.0	88.0	117.3	97.3	34.8	22.1
1	7	32.00	62.6	76.1	101.6	83.5	26.2	21.3
1	8	36.57	52.0	64.2	85.8	69.6	17.3	20.7
1	9	41.14	41.8	52.6	70.5	56.1	8.7	20.5
1	10	45.71	31.6	41.1	55.3	42.7	0.0	20.4
1	11	50.28	-41.7	-52.5	-70.4	-56.0	-8.6	20.5
1	12	54.85	-52.2	-64.3	-86.0	-69.8	-17.5	20.7
1	13	59.42	-62.5	-76.0	-101.5	-83.5	-26.1	21.3
1	14	63.99	-73.0	-87.9	-117.2	-97.3	-34.7	22.1
1	15	68.56	-83.7	-100.1	-133.3	-111.4	-43.6	22.8
1	16	73.13	-94.4	-112.2	-149.3	-125.5	-52.2	23.6
1	17	77.70	-105.1	-124.5	-165.6	-139.7	-60.8	24.4
1	18	82.28	-116.0	-136.9	-182.0	-154.1	-69.5	25.3
1	19	86.85	-127.1	-149.6	-198.8	-168.8	-78.3	26.2
1	20	91.42	-138.6	-162.6	-216.0	-183.9	-87.5	27.2

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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	58.23	0.00	HL-93	80.20	HL-93	ST1	49.69	215.86
						ST2	49.69	183.78
						ST4	71.03	87.35
						SE1	58.23	138.43
						SE2	58.23	162.49
2	58.36	0.00	HL-93	80.20	HL-93	ST1	49.81	216.02
						ST2	49.81	183.94
						ST4	71.22	87.54
						SE1	58.36	138.56
						SE2	58.36	162.62

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 = 30.0 (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	4.57	-0.1198	-0.2198	0.0000	-0.0992	-0.3436	-0.0992	-0.4428
1	2	9.14	-0.2354	-0.4320	0.0000	-0.1950	-0.6754	-0.1950	-0.8705
1	3	13.71	-0.3433	-0.6300	0.0000	-0.2843	-0.9850	-0.2843	-1.2693
1	4	18.28	-0.4402	-0.8078	0.0000	-0.3643	-1.2629	-0.3643	-1.6272
1	5	22.85	-0.5243	-0.9620	0.0000	-0.4336	-1.5040	-0.4336	-1.9376
1	6	27.43	-0.5955	-1.0928	0.0000	-0.4923	-1.7085	-0.4923	-2.2008
1	7	32.00	-0.6525	-1.1974	0.0000	-0.5393	-1.8721	-0.5393	-2.4114
1	8	36.57	-0.6941	-1.2737	0.0000	-0.5735	-1.9914	-0.5735	-2.5650
1	9	41.14	-0.7194	-1.3201	0.0000	-0.5944	-2.0640	-0.5944	-2.6584
1	10	45.71	-0.7279	-1.3356	0.0000	-0.6014	-2.0884	-0.6014	-2.6898
1	11	50.28	-0.7194	-1.3201	0.0000	-0.5944	-2.0641	-0.5944	-2.6585
1	12	54.85	-0.6941	-1.2737	0.0000	-0.5735	-1.9916	-0.5735	-2.5651
1	13	59.42	-0.6525	-1.1974	0.0000	-0.5393	-1.8723	-0.5393	-2.4116
1	14	63.99	-0.5955	-1.0928	0.0000	-0.4923	-1.7088	-0.4923	-2.2011
1	15	68.56	-0.5243	-0.9620	0.0000	-0.4336	-1.5043	-0.4336	-1.9379
1	16	73.13	-0.4402	-0.8078	0.0000	-0.3643	-1.2632	-0.3643	-1.6275
1	17	77.70	-0.3433	-0.6300	0.0000	-0.2843	-0.9853	-0.2843	-1.2696
1	18	82.28	-0.2354	-0.4321	0.0000	-0.1950	-0.6757	-0.1950	-0.8707
1	19	86.85	-0.1198	-0.2198	0.0000	-0.0992	-0.3437	-0.0992	-0.4429
1	20	91.42	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	4.6	-0.120	0 1/ 8	-0.224	0 1/ 4	-0.099	0 1/ 8	-0.443	0 1/ 2
1	2	9.1	-0.235	0 1/ 4	-0.440	0 1/ 2	-0.195	0 1/ 4	-0.870	0 7/ 8
1	3	13.7	-0.343	0 3/ 8	-0.642	0 11/16	-0.284	0 5/16	-1.269	1 5/16
1	4	18.3	-0.440	0 1/ 2	-0.823	0 7/ 8	-0.364	0 3/ 8	-1.627	1 11/16
1	5	22.9	-0.524	0 9/16	-0.980	1 0/16	-0.434	0 7/16	-1.938	2 0/16
1	6	27.4	-0.596	0 5/ 8	-1.113	1 1/ 8	-0.492	0 1/ 2	-2.201	2 1/ 4
1	7	32.0	-0.653	0 11/16	-1.220	1 1/ 4	-0.539	0 9/16	-2.411	2 7/16
1	8	36.6	-0.694	0 3/ 4	-1.297	1 5/16	-0.574	0 5/ 8	-2.565	2 5/ 8
1	9	41.1	-0.719	0 3/ 4	-1.345	1 3/ 8	-0.594	0 5/ 8	-2.658	2 11/16
1	10	45.7	-0.728	0 3/ 4	-1.361	1 3/ 8	-0.601	0 5/ 8	-2.690	2 3/ 4
1	11	50.3	-0.719	0 3/ 4	-1.345	1 3/ 8	-0.594	0 5/ 8	-2.658	2 11/16
1	12	54.9	-0.694	0 3/ 4	-1.297	1 5/16	-0.574	0 5/ 8	-2.565	2 5/ 8
1	13	59.4	-0.653	0 11/16	-1.220	1 1/ 4	-0.539	0 9/16	-2.412	2 7/16
1	14	64.0	-0.596	0 5/ 8	-1.113	1 1/ 8	-0.492	0 1/ 2	-2.201	2 1/ 4
1	15	68.6	-0.524	0 9/16	-0.980	1 0/16	-0.434	0 7/16	-1.938	2 0/16
1	16	73.1	-0.440	0 1/ 2	-0.823	0 7/ 8	-0.364	0 3/ 8	-1.627	1 11/16
1	17	77.7	-0.343	0 3/ 8	-0.642	0 11/16	-0.284	0 5/16	-1.270	1 5/16
1	18	82.3	-0.235	0 1/ 4	-0.440	0 1/ 2	-0.195	0 1/ 4	-0.871	0 7/ 8
1	19	86.8	-0.120	0 1/ 8	-0.224	0 1/ 4	-0.099	0 1/ 8	-0.443	0 1/ 2
1	20	91.4	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	45.71	2 0.400	-0.762 MAX	HL-93	1.37	0.00422
			0.038 MIN			
			-0.310 MAX	LANE		
			0.038 MIN	LANE		
		3 0.510	-0.971 MAX	HL-93	1.37	0.00422
			0.048 MIN			
			-0.396 MAX	LANE		
			0.048 MIN	LANE		
		2 0.400	-0.668 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	4.57	-0.73	0.73	-1.35	1.35	-2.08	2.08
1	2	9.14	-1.38	1.38	-2.57	2.57	-3.94	3.94
1	3	13.71	-1.95	1.95	-3.64	3.64	-5.59	5.59
1	4	18.28	-2.45	2.45	-4.57	4.57	-7.01	7.01
1	5	22.85	-2.66	1.98	-4.96	3.70	-7.62	5.67
1	6	27.43	-2.97	2.22	-5.56	4.14	-8.53	6.35
1	7	32.00	-3.22	2.40	-6.02	4.48	-9.24	6.88
1	8	36.57	-3.40	2.53	-6.36	4.73	-9.75	7.27
1	9	41.14	-3.51	2.61	-6.55	4.88	-10.06	7.49
1	10	45.71	-3.54	2.64	-6.62	4.93	-10.16	7.57
1	11	50.28	-3.51	2.61	-6.55	4.88	-10.06	7.49
1	12	54.85	-3.40	2.53	-6.36	4.73	-9.75	7.27
1	13	59.42	-3.22	2.40	-6.02	4.49	-9.25	6.89
1	14	63.99	-2.97	2.22	-5.56	4.14	-8.54	6.36
1	15	68.56	-2.66	1.98	-4.97	3.70	-7.62	5.68
1	16	73.13	-2.45	2.45	-4.57	4.57	-7.02	7.02
1	17	77.70	-1.95	1.95	-3.64	3.64	-5.59	5.59
1	18	82.28	-1.38	1.38	-2.58	2.58	-3.95	3.95
1	19	86.85	-0.73	0.73	-1.36	1.36	-2.09	2.09
1	20	91.42	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 = 30.0 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	4.57	-0.02	-0.48	0.74	-0.02	-2.56	2.82
1	2	9.14	-0.05	-0.91	1.40	-0.05	-4.85	5.34
1	3	13.71	-0.06	-1.29	1.98	-0.06	-6.88	7.57

1	4	18.28	-0.08	-1.62	2.49	-0.08	-8.64	9.50
1	5	22.85	-0.08	-1.81	2.01	-0.08	-9.42	7.69
1	6	27.43	-0.09	-2.02	2.25	-0.09	-10.55	8.61
1	7	32.00	-0.10	-2.19	2.44	-0.10	-11.43	9.33
1	8	36.57	-0.11	-2.31	2.58	-0.11	-12.07	9.84
1	9	41.14	-0.11	-2.38	2.66	-0.11	-12.44	10.15
1	10	45.71	-0.11	-2.41	2.68	-0.11	-12.57	10.25
1	11	50.28	-0.11	-2.38	2.66	-0.11	-12.44	10.15
1	12	54.85	-0.11	-2.31	2.58	-0.11	-12.07	9.84
1	13	59.42	-0.10	-2.19	2.44	-0.10	-11.44	9.33
1	14	63.99	-0.09	-2.02	2.25	-0.09	-10.56	8.61
1	15	68.56	-0.08	-1.81	2.01	-0.08	-9.43	7.69
1	16	73.13	-0.08	-1.62	2.49	-0.08	-8.64	9.51
1	17	77.70	-0.06	-1.29	1.98	-0.06	-6.89	7.58
1	18	82.28	-0.05	-0.91	1.40	-0.05	-4.86	5.35
1	19	86.85	-0.02	-0.48	0.74	-0.02	-2.57	2.83
1	20	91.42	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 = 10.0 (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	4.57	-0.14	-0.80	2.19	0.00	0.00	0.00
1	2	9.14	-0.26	-1.50	4.14	0.00	0.00	0.00
1	3	13.71	-0.36	-2.12	5.83	0.00	0.00	0.00
1	4	18.28	-0.45	-2.64	7.27	0.00	0.00	0.00
1	5	22.85	-0.47	-3.02	5.81	0.00	0.00	0.00
1	6	27.43	-0.52	-3.36	6.47	0.00	0.00	0.00
1	7	32.00	-0.56	-3.63	6.98	0.00	0.00	0.00
1	8	36.57	-0.59	-3.81	7.33	0.00	0.00	0.00
1	9	41.14	-0.60	-3.89	7.48	0.00	0.00	0.00
1	10	45.71	-0.60	-3.92	7.54	0.00	0.00	0.00
1	11	50.28	-0.60	-3.89	7.48	0.00	0.00	0.00
1	12	54.85	-0.59	-3.81	7.33	0.00	0.00	0.00
1	13	59.42	-0.56	-3.63	6.98	0.00	0.00	0.00
1	14	63.99	-0.52	-3.36	6.47	0.00	0.00	0.00
1	15	68.56	-0.47	-3.02	5.81	0.00	0.00	0.00
1	16	73.13	-0.45	-2.64	7.27	0.00	0.00	0.00
1	17	77.70	-0.36	-2.12	5.83	0.00	0.00	0.00
1	18	82.28	-0.26	-1.50	4.14	0.00	0.00	0.00
1	19	86.85	-0.14	-0.80	2.19	0.00	0.00	0.00
1	20	91.42	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 = 10.0 (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	4.57	0.4	0.0	0.4	C	HL	1.0	0.0	1.0	T HL
1	2	9.14	0.7	0.0	0.7	C	HL	1.8	0.0	1.8	T HL
1	3	13.71	0.9	0.0	0.9	C	HL	2.5	0.0	2.5	T HL
1	4	18.28	1.1	0.0	1.1	C	HL	3.1	0.0	3.1	T HL
1	5	22.85	1.3	0.0	1.3	C	HL	2.5	0.0	2.5	T HL
1	6	27.43	1.4	0.0	1.4	C	HL	2.8	0.0	2.8	T HL
1	7	32.00	1.6	0.0	1.6	C	HL	3.0	0.0	3.0	T HL
1	8	36.57	1.6	0.0	1.6	C	HL	3.1	0.0	3.1	T HL
1	9	41.14	1.6	0.0	1.6	C	HL	3.1	0.0	3.1	T HL
1	10	45.71	1.6	0.0	1.6	C	HL	3.0	0.0	3.0	T HL
1	11	50.28	1.6	0.0	1.6	C	HL	3.1	0.0	3.1	T HL
1	12	54.85	1.6	0.0	1.6	C	HL	3.1	0.0	3.1	T HL
1	13	59.42	1.6	0.0	1.6	C	HL	3.0	0.0	3.0	T HL
1	14	63.99	1.4	0.0	1.4	C	HL	2.8	0.0	2.8	T HL
1	15	68.56	1.3	0.0	1.3	C	HL	2.5	0.0	2.5	T HL
1	16	73.13	1.1	0.0	1.1	C	HL	3.1	0.0	3.1	T HL
1	17	77.70	0.9	0.0	0.9	C	HL	2.5	0.0	2.5	T HL
1	18	82.28	0.7	0.0	0.7	C	HL	1.8	0.0	1.8	T HL
1	19	86.85	0.4	0.0	0.4	C	HL	1.0	0.0	1.0	T HL
1	20	91.42	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.16	-3.36	5.01	-0.02	-2.56	2.82
1	2	9.14	32.	32.	-0.30	-6.36	9.48	-0.05	-4.85	5.34
1	3	13.71	32.	32.	-0.43	-9.00	13.40	-0.06	-6.88	7.57
1	4	18.28	32.	32.	-0.53	-11.28	16.77	-0.08	-8.64	9.50
1	5	22.85	32.	32.	-0.55	-12.45	13.50	-0.08	-9.42	7.69
1	6	27.43	32.	32.	-0.61	-13.92	15.07	-0.09	-10.55	8.61
1	7	32.00	32.	32.	-0.66	-15.06	16.30	-0.10	-11.43	9.33
1	8	36.57	32.	32.	-0.70	-15.88	17.17	-0.11	-12.07	9.84
1	9	41.14	32.	32.	-0.71	-16.33	17.63	-0.11	-12.44	10.15
1	10	45.71	32.	32.	-0.72	-16.49	17.79	-0.11	-12.57	10.25
1	11	50.28	32.	32.	-0.71	-16.34	17.63	-0.11	-12.44	10.15
1	12	54.85	32.	32.	-0.70	-15.88	17.17	-0.11	-12.07	9.84
1	13	59.42	32.	32.	-0.66	-15.07	16.30	-0.10	-11.44	9.33
1	14	63.99	32.	32.	-0.61	-13.92	15.08	-0.09	-10.56	8.61
1	15	68.56	32.	32.	-0.55	-12.45	13.50	-0.08	-9.43	7.69
1	16	73.13	32.	32.	-0.53	-11.28	16.78	-0.08	-8.64	9.51
1	17	77.70	32.	32.	-0.43	-9.00	13.41	-0.06	-6.89	7.58
1	18	82.28	32.	32.	-0.30	-6.37	9.49	-0.05	-4.86	5.35
1	19	86.85	32.	32.	-0.16	-3.37	5.02	-0.02	-2.57	2.83
1	20	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.20	-3.60	5.67	-0.02	-2.56	2.82
1	2	9.14	32.	32.	-0.38	-6.81	10.72	-0.05	-4.85	5.34
1	3	13.71	32.	32.	-0.54	-9.63	15.15	-0.06	-6.88	7.57
1	4	18.28	32.	32.	-0.67	-12.07	18.95	-0.08	-8.64	9.50
1	5	22.85	32.	32.	-0.69	-13.35	15.24	-0.08	-9.42	7.69
1	6	27.43	32.	32.	-0.77	-14.93	17.01	-0.09	-10.55	8.61
1	7	32.00	32.	32.	-0.83	-16.15	18.40	-0.10	-11.43	9.33
1	8	36.57	32.	32.	-0.87	-17.02	19.36	-0.11	-12.07	9.84
1	9	41.14	32.	32.	-0.89	-17.50	19.88	-0.11	-12.44	10.15
1	10	45.71	32.	32.	-0.90	-17.66	20.05	-0.11	-12.57	10.25
1	11	50.28	32.	32.	-0.89	-17.50	19.88	-0.11	-12.44	10.15
1	12	54.85	32.	32.	-0.87	-17.02	19.36	-0.11	-12.07	9.84
1	13	59.42	32.	32.	-0.83	-16.15	18.40	-0.10	-11.44	9.33
1	14	63.99	32.	32.	-0.77	-14.93	17.02	-0.09	-10.56	8.61
1	15	68.56	32.	32.	-0.69	-13.36	15.24	-0.08	-9.43	7.69
1	16	73.13	32.	32.	-0.67	-12.07	18.96	-0.08	-8.64	9.51
1	17	77.70	32.	32.	-0.54	-9.64	15.15	-0.06	-6.89	7.58
1	18	82.28	32.	32.	-0.38	-6.82	10.73	-0.05	-4.86	5.35
1	19	86.85	32.	32.	-0.20	-3.60	5.68	-0.02	-2.57	2.83
1	20	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.27	-4.68	7.49	-0.03	-3.28	3.65
1	2	9.14	32.	32.	-0.52	-8.85	14.15	-0.06	-6.22	6.91
1	3	13.71	32.	32.	-0.73	-12.51	19.99	-0.09	-8.81	9.79
1	4	18.28	32.	32.	-0.91	-15.68	25.01	-0.11	-11.06	12.29
1	5	22.85	32.	32.	-0.93	-17.37	20.11	-0.12	-12.08	9.94
1	6	27.43	32.	32.	-1.04	-19.41	22.45	-0.13	-13.52	11.13
1	7	32.00	32.	32.	-1.12	-21.00	24.27	-0.14	-14.65	12.06
1	8	36.57	32.	32.	-1.18	-22.13	25.54	-0.15	-15.46	12.72
1	9	41.14	32.	32.	-1.21	-22.75	26.22	-0.16	-15.94	13.12
1	10	45.71	32.	32.	-1.22	-22.96	26.44	-0.16	-16.10	13.25
1	11	50.28	32.	32.	-1.21	-22.76	26.22	-0.16	-15.94	13.12
1	12	54.85	32.	32.	-1.18	-22.13	25.54	-0.15	-15.46	12.72
1	13	59.42	32.	32.	-1.12	-21.01	24.27	-0.14	-14.65	12.06
1	14	63.99	32.	32.	-1.04	-19.42	22.45	-0.13	-13.53	11.13
1	15	68.56	32.	32.	-0.93	-17.37	20.11	-0.12	-12.08	9.94
1	16	73.13	32.	32.	-0.91	-15.69	25.01	-0.11	-11.07	12.29
1	17	77.70	32.	32.	-0.73	-12.52	20.00	-0.09	-8.82	9.80
1	18	82.28	32.	32.	-0.52	-8.86	14.16	-0.06	-6.23	6.92
1	19	86.85	32.	32.	-0.27	-4.68	7.49	-0.03	-3.29	3.65
1	20	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.26	-3.62	6.26	-0.02	-2.23	2.42
1	2	9.14	32.	32.	-0.49	-6.85	11.82	-0.03	-4.22	4.58
1	3	13.71	32.	32.	-0.68	-9.68	16.69	-0.05	-5.98	6.49
1	4	18.28	32.	32.	-0.85	-12.13	20.86	-0.06	-7.51	8.15
1	5	22.85	32.	32.	-0.88	-13.47	16.76	-0.06	-8.19	6.59
1	6	27.43	32.	32.	-0.98	-15.05	18.69	-0.07	-9.17	7.38
1	7	32.00	32.	32.	-1.05	-16.28	20.20	-0.07	-9.93	7.99
1	8	36.57	32.	32.	-1.11	-17.15	21.25	-0.08	-10.48	8.44
1	9	41.14	32.	32.	-1.13	-17.62	21.79	-0.08	-10.81	8.70
1	10	45.71	32.	32.	-1.14	-17.78	21.97	-0.08	-10.92	8.78
1	11	50.28	32.	32.	-1.13	-17.62	21.80	-0.08	-10.81	8.70
1	12	54.85	32.	32.	-1.11	-17.15	21.25	-0.08	-10.48	8.44
1	13	59.42	32.	32.	-1.05	-16.29	20.20	-0.07	-9.93	8.00
1	14	63.99	32.	32.	-0.98	-15.06	18.70	-0.07	-9.17	7.38
1	15	68.56	32.	32.	-0.88	-13.48	16.76	-0.06	-8.19	6.59
1	16	73.13	32.	32.	-0.85	-12.13	20.87	-0.06	-7.51	8.15
1	17	77.70	32.	32.	-0.68	-9.69	16.69	-0.05	-5.99	6.49
1	18	82.28	32.	32.	-0.49	-6.86	11.83	-0.03	-4.23	4.59
1	19	86.85	32.	32.	-0.26	-3.63	6.26	-0.02	-2.23	2.42
1	20	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.22	-4.36	6.61	-0.03	-3.28	3.65
1	2	9.14	32.	32.	-0.41	-8.25	12.49	-0.06	-6.22	6.91
1	3	13.71	32.	32.	-0.58	-11.67	17.66	-0.09	-8.81	9.79
1	4	18.28	32.	32.	-0.73	-14.62	22.10	-0.11	-11.06	12.29
1	5	22.85	32.	32.	-0.75	-16.16	17.78	-0.12	-12.08	9.94
1	6	27.43	32.	32.	-0.83	-18.06	19.86	-0.13	-13.52	11.13
1	7	32.00	32.	32.	-0.90	-19.55	21.48	-0.14	-14.65	12.06
1	8	36.57	32.	32.	-0.95	-20.61	22.61	-0.15	-15.46	12.72
1	9	41.14	32.	32.	-0.97	-21.20	23.22	-0.16	-15.94	13.12
1	10	45.71	32.	32.	-0.97	-21.40	23.43	-0.16	-16.10	13.25
1	11	50.28	32.	32.	-0.97	-21.20	23.22	-0.16	-15.94	13.12
1	12	54.85	32.	32.	-0.95	-20.61	22.61	-0.15	-15.46	12.72
1	13	59.42	32.	32.	-0.90	-19.55	21.48	-0.14	-14.65	12.06
1	14	63.99	32.	32.	-0.83	-18.07	19.86	-0.13	-13.53	11.13
1	15	68.56	32.	32.	-0.75	-16.16	17.79	-0.12	-12.08	9.94
1	16	73.13	32.	32.	-0.73	-14.63	22.11	-0.11	-11.07	12.29
1	17	77.70	32.	32.	-0.58	-11.68	17.67	-0.09	-8.82	9.80
1	18	82.28	32.	32.	-0.41	-8.26	12.50	-0.06	-6.23	6.92
1	19	86.85	32.	32.	-0.22	-4.37	6.62	-0.03	-3.29	3.65
1	20	91.42	32.	32.	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	32.	32.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.57	32.	32.	-0.20	-3.30	5.38	-0.02	-2.23	2.42
1	2	9.14	32.	32.	-0.38	-6.25	10.16	-0.03	-4.22	4.58
1	3	13.71	32.	32.	-0.54	-8.84	14.36	-0.05	-5.98	6.49
1	4	18.28	32.	32.	-0.67	-11.07	17.96	-0.06	-7.51	8.15
1	5	22.85	32.	32.	-0.69	-12.27	14.43	-0.06	-8.19	6.59
1	6	27.43	32.	32.	-0.77	-13.71	16.11	-0.07	-9.17	7.38
1	7	32.00	32.	32.	-0.83	-14.83	17.41	-0.07	-9.93	7.99
1	8	36.57	32.	32.	-0.87	-15.63	18.32	-0.08	-10.48	8.44
1	9	41.14	32.	32.	-0.89	-16.06	18.80	-0.08	-10.81	8.70
1	10	45.71	32.	32.	-0.90	-16.21	18.96	-0.08	-10.92	8.78
1	11	50.28	32.	32.	-0.89	-16.06	18.80	-0.08	-10.81	8.70
1	12	54.85	32.	32.	-0.87	-15.63	18.32	-0.08	-10.48	8.44
1	13	59.42	32.	32.	-0.83	-14.83	17.41	-0.07	-9.93	8.00
1	14	63.99	32.	32.	-0.77	-13.71	16.11	-0.07	-9.17	7.38
1	15	68.56	32.	32.	-0.69	-12.27	14.44	-0.06	-8.19	6.59
1	16	73.13	32.	32.	-0.67	-11.07	17.96	-0.06	-7.51	8.15
1	17	77.70	32.	32.	-0.54	-8.84	14.36	-0.05	-5.99	6.49
1	18	82.28	32.	32.	-0.38	-6.26	10.17	-0.03	-4.23	4.59
1	19	86.85	32.	32.	-0.20	-3.31	5.38	-0.02	-2.23	2.42
1	20	91.42	32.	32.	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=30.0 MOM. (k-ft)	DL2, N=30.0 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	19.21	626.	-7.3	251.	-1.7	802.	-2.7	0.	0.0	-11.66
											2.73
	LB			7.3		2.6		7.5		0.0	17.34
											7.50
	RT			-6.7		-1.6		-2.7		0.0	-11.01
											2.69
	RB			5.0		1.8		5.2		0.0	11.95
											5.17
1	LT	72.21	627.	-6.7	251.	-1.6	802.	-2.7	0.	0.0	-11.01
											2.69
	LB			5.0		1.8		5.2		0.0	11.96
											5.17
	RT			-7.3		-1.7		-2.7		0.0	-11.67
											2.73
	RB			7.3		2.6		7.5		0.0	17.34
											7.50

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	19.21	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	16.59	0.0000
2	53.00	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	15.00	1.1875
3	19.21	W	16.6	1.5700	33.4	0.8840	16.6	1.5700	16.59	0.0000	16.59	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) = 91.4

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	19.21	7.00	36.52	[2]=	43.52	0.03967	< 0.04000	<--=NG
				[3]=	36.52	0.03329	< 0.03300	<--=NG
2	53.00	7.00	36.52	[2]=	43.52	0.03967	< 0.04000	<--=NG
				[3]=	36.52	0.03329	< 0.03300	<--=NG
3	19.21	7.00	36.52	[2]=	43.52	0.03967	< 0.04000	<--=NG
				[3]=	36.52	0.03329	< 0.03300	<--=NG

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 [1]	Lo ng	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	33.38	0.884	37.8	150.0	0.00	113.19	0
1	1	4.57	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	2	9.14	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	3	13.71	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	4	18.28	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	5	22.85	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	6	27.43	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	7	32.00	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	8	36.57	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	9	41.14	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	10	45.71	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	11	50.28	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	12	54.85	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	13	59.42	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	14	63.99	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	15	68.56	0	1	33.38	0.884	37.8	150.0	15.94	113.19	0
1	16	73.13	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	17	77.70	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	18	82.28	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	19	86.85	0	1	33.38	0.884	37.8	150.0	0.00	113.19	0
1	20	91.42	0	1	33.38	0.884	37.8	150.0	0.00	113.19	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	33.38	0.884	37.76	171.59	2
1	1	4.57	0	1	33.38	0.884	37.76	171.59	2
1	2	9.14	0	1	33.38	0.884	37.76	171.59	2
1	3	13.71	0	1	33.38	0.884	37.76	171.59	2
1	4	18.28	0	1	33.38	0.884	37.76	171.59	2
1	5	22.85	0	1	33.38	0.884	45.40	171.59	2
1	6	27.43	0	1	33.38	0.884	45.40	171.59	2
1	7	32.00	0	1	33.38	0.884	45.40	171.59	2
1	8	36.57	0	1	33.38	0.884	45.40	171.59	2
1	9	41.14	0	1	33.38	0.884	45.40	171.59	2
1	10	45.71	0	1	33.38	0.884	45.40	171.59	2
1	11	50.28	0	1	33.38	0.884	45.40	171.59	2
1	12	54.85	0	1	33.38	0.884	45.40	171.59	2
1	13	59.42	0	1	33.38	0.884	45.40	171.59	2
1	14	63.99	0	1	33.38	0.884	45.40	171.59	2
1	15	68.56	0	1	33.38	0.884	45.40	171.59	2
1	16	73.13	0	1	33.38	0.884	37.76	171.59	2
1	17	77.70	0	1	33.38	0.884	37.76	171.59	2
1	18	82.28	0	1	33.38	0.884	37.76	171.59	2
1	19	86.85	0	1	33.38	0.884	37.76	171.59	2
1	20	91.42	0	1	33.38	0.884	37.76	171.59	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	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 1	4.57	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 2	9.14	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 3	13.71	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 4	18.28	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 5	22.85	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 6	27.43	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 7	32.00	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 8	36.57	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 9	41.14	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 10	45.71	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 11	50.28	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 12	54.85	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 13	59.42	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 14	63.99	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 15	68.56	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 16	73.13	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 17	77.70	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 18	82.28	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 19	86.85	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	1.00 0
1 20	91.42	5.28	12.	16.6	5.6	1.570	0.972	
		5.28	12.	16.6	5.6	1.570	0.972	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

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	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	1		4.57	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	2		9.14	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	3		13.71	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	4		18.28	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	5		22.85	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	6		27.43	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	7		32.00	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	8		36.57	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	9		41.14	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	10		45.71	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	11		50.28	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	12		54.85	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	13		59.42	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	14		63.99	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	15		68.56	1	5.28	11.44	20.15	0	0.00	10.85	40.74	0	0
1	16		73.13	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	17		77.70	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	18		82.28	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	19		86.85	1	5.28	11.44	20.15	0	0.00	11.02	41.39	0	0
1	20		91.42	1	5.28	11.44	20.15	0	0.00	11.02	41.39	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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MERLIN V 10.4
COMPOSITE
LRF -- 2010
CODE CHECK

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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	32.00	32.00	32.00
1	1		4.57	1	1.000	1.000	1.00	32.00	32.00	32.00
1	2		9.14	1	1.000	1.000	1.00	32.00	32.00	32.00
1	3		13.71	1	1.000	1.000	1.00	32.00	32.00	32.00
1	4		18.28	1	1.000	1.000	1.00	32.00	32.00	32.00
1	5		22.85	1	1.000	1.000	1.00	32.00	32.00	32.00
1	6		27.43	1	1.000	1.000	1.00	32.00	32.00	32.00
1	7		32.00	1	1.000	1.000	1.00	32.00	32.00	32.00
1	8		36.57	1	1.000	1.000	1.00	32.00	32.00	32.00
1	9		41.14	1	1.000	1.000	1.00	32.00	32.00	32.00
1	10		45.71	1	1.000	1.000	1.00	32.00	32.00	32.00
1	11		50.28	1	1.000	1.000	1.00	32.00	32.00	32.00
1	12		54.85	1	1.000	1.000	1.00	32.00	32.00	32.00
1	13		59.42	1	1.000	1.000	1.00	32.00	32.00	32.00
1	14		63.99	1	1.000	1.000	1.00	32.00	32.00	32.00
1	15		68.56	1	1.000	1.000	1.00	32.00	32.00	32.00
1	16		73.13	1	1.000	1.000	1.00	32.00	32.00	32.00
1	17		77.70	1	1.000	1.000	1.00	32.00	32.00	32.00
1	18		82.28	1	1.000	1.000	1.00	32.00	32.00	32.00
1	19		86.85	1	1.000	1.000	1.00	32.00	32.00	32.00
1	20		91.42	1	1.000	1.000	1.00	32.00	32.00	32.00

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	8.41	18.28
1	1		4.57	1	8.41	18.28
1	2		9.14	1	8.41	18.28
1	3		13.71	1	8.41	18.28
1	4		18.28	1	8.41	18.28
1	5		22.85	1	15.62	18.78
1	6		27.43	1	15.62	18.78
1	7		32.00	1	15.62	18.78
1	8		36.57	1	15.62	18.78
1	9		41.14	1	15.62	18.78

1	10	45.71	1	15.62	18.78
1	11	50.28	1	15.62	18.78
1	12	54.85	1	15.62	18.78
1	13	59.42	1	15.62	18.78
1	14	63.99	1	15.62	18.78
1	15	68.56	1	15.62	18.78
1	16	73.13	1	8.41	18.28
1	17	77.70	1	8.41	18.28
1	18	82.28	1	8.41	18.28
1	19	86.85	1	8.41	18.28
1	20	91.42	1	8.41	18.28

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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.	Comp.
1	0	0.00	1	0	0	0	0		0	0
1	1	4.57	1	0	0	0	0		0	0
1	2	9.14	1	0	0	0	0		0	0
1	3	13.71	1	0	0	0	0		0	0
1	4	18.28	1	0	0	0	0		0	0
1	5	22.85	1	0	0	0	0		0	0
1	6	27.43	1	0	0	0	0		0	0
1	7	32.00	1	0	0	0	0		0	0
1	8	36.57	1	0	0	0	0		0	0
1	9	41.14	1	0	0	0	0		0	0
1	10	45.71	1	0	0	0	0		0	0
1	11	50.28	1	0	0	0	0		0	0
1	12	54.85	1	0	0	0	0		0	0
1	13	59.42	1	0	0	0	0		0	0
1	14	63.99	1	0	0	0	0		0	0
1	15	68.56	1	0	0	0	0		0	0
1	16	73.13	1	0	0	0	0		0	0
1	17	77.70	1	0	0	0	0		0	0
1	18	82.28	1	0	0	0	0		0	0
1	19	86.85	1	0	0	0	0		0	0
1	20	91.42	1	0	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	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	0
1	1		4.57	0.0	19.2	3.1	32.0	3.1	32.0	3.1	32.0	0
				0.0	-	3.1	-	3.1	32.0	-	-	0
1	2		9.14	0.0	19.2	5.9	32.0	5.9	32.0	5.9	32.0	0
				0.0	-	5.9	-	5.9	32.0	-	-	0
1	3		13.71	0.0	19.2	8.4	32.0	8.4	32.0	8.4	32.0	0
				0.0	-	8.4	-	8.4	32.0	-	-	0
1	4		18.28	0.0	19.2	10.5	32.0	10.5	32.0	10.5	32.0	0
				0.0	-	10.5	-	10.5	32.0	-	-	0
1	5		22.85	0.0	19.2	11.4	32.0	11.4	32.0	11.4	32.0	0
				0.0	-	8.5	-	8.5	32.0	-	-	0
1	6		27.43	0.0	19.2	12.8	32.0	12.8	32.0	12.8	32.0	0
				0.0	-	9.5	-	9.5	32.0	-	-	0
1	7		32.00	0.0	19.2	13.9	32.0	13.9	32.0	13.9	32.0	0
				0.0	-	10.3	-	10.3	32.0	-	-	0
1	8		36.57	0.0	19.2	14.6	32.0	14.6	32.0	14.6	32.0	0
				0.0	-	10.9	-	10.9	32.0	-	-	0
1	9		41.14	0.0	19.2	15.1	32.0	15.1	32.0	15.1	32.0	0
				0.0	-	11.2	-	11.2	32.0	-	-	0
1	10		45.71	0.0	19.2	15.2	32.0	15.2	32.0	15.2	32.0	0
				0.0	-	11.3	-	11.3	32.0	-	-	0
1	11		50.28	0.0	19.2	15.1	32.0	15.1	32.0	15.1	32.0	0
				0.0	-	11.2	-	11.2	32.0	-	-	0
1	12		54.85	0.0	19.2	14.6	32.0	14.6	32.0	14.6	32.0	0
				0.0	-	10.9	-	10.9	32.0	-	-	0
1	13		59.42	0.0	19.2	13.9	32.0	13.9	32.0	13.9	32.0	0
				0.0	-	10.3	-	10.3	32.0	-	-	0
1	14		63.99	0.0	19.2	12.8	32.0	12.8	32.0	12.8	32.0	0
				0.0	-	9.5	-	9.5	32.0	-	-	0
1	15		68.56	0.0	19.2	11.4	32.0	11.4	32.0	11.4	32.0	0
				0.0	-	8.5	-	8.5	32.0	-	-	0
1	16		73.13	0.0	19.2	10.5	32.0	10.5	32.0	10.5	32.0	0
				0.0	-	10.5	-	10.5	32.0	-	-	0
1	17		77.70	0.0	19.2	8.4	32.0	8.4	32.0	8.4	32.0	0
				0.0	-	8.4	-	8.4	32.0	-	-	0
1	18		82.28	0.0	19.2	5.9	32.0	5.9	32.0	5.9	32.0	0
				0.0	-	5.9	-	5.9	32.0	-	-	0
1	19		86.85	0.0	19.2	3.1	32.0	3.1	32.0	3.1	32.0	0
				0.0	-	3.1	-	3.1	32.0	-	-	0
1	20		91.42	0.0	19.2	0.0	32.0	0.0	32.0	0.0	32.0	0
				0.0	-	0.0	-	0.0	32.0	-	-	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

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	4.57	0.000	0.098	0.098	0.098	0.098	2
					0.098		0.098	3
1	2	9.14	0.000	0.185	0.185	0.185	0.185	2
					0.185		0.185	3
1	3	13.71	0.000	0.262	0.262	0.262	0.262	2
					0.262		0.262	3
1	4	18.28	0.000	0.329	0.329	0.329	0.329	2
					0.329		0.329	3
1	5	22.85	0.000	0.357	0.357	0.357	0.357	2
					0.266		0.266	3
1	6	27.43	0.000	0.400	0.400	0.400	0.400	2
					0.298		0.298	3
1	7	32.00	0.000	0.433	0.433	0.433	0.433	2
					0.323		0.323	3
1	8	36.57	0.000	0.457	0.457	0.457	0.457	2
					0.341		0.341	3
1	9	41.14	0.000	0.471	0.471	0.471	0.471	2
					0.351		0.351	3
1	10	45.71	0.000	0.476	0.476	0.476	0.476	2
					0.355		0.355	3
1	11	50.28	0.000	0.472	0.472	0.472	0.472	2
					0.351		0.351	3
1	12	54.85	0.000	0.457	0.457	0.457	0.457	2
					0.341		0.341	3
1	13	59.42	0.000	0.433	0.433	0.433	0.433	2
					0.323		0.323	3
1	14	63.99	0.000	0.400	0.400	0.400	0.400	2
					0.298		0.298	3
1	15	68.56	0.000	0.357	0.357	0.357	0.357	2
					0.266		0.266	3
1	16	73.13	0.000	0.329	0.329	0.329	0.329	2
					0.329		0.329	3
1	17	77.70	0.000	0.262	0.262	0.262	0.262	2
					0.262		0.262	3
1	18	82.28	0.000	0.185	0.185	0.185	0.185	2
					0.185		0.185	3
1	19	86.85	0.000	0.098	0.098	0.098	0.098	2
					0.098		0.098	3
1	20	91.42	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	3936.8	-	-	0.0	19.2	-	-	0--0
1	1		4.57	0	736.8	3936.8	-	-	0.0	19.2	-	-	0--0
1	2		9.14	0	1392.1	3936.8	-	-	0.0	19.2	-	-	0--0
1	3		13.71	0	1966.3	3936.8	-	-	0.0	19.2	-	-	0--0
1	4		18.28	0	2459.3	3936.8	-	-	0.0	19.2	-	-	0--0
1	5		22.85	0	2867.5	4774.7	-	-	0.0	19.2	-	-	0--0
1	6		27.43	0	3200.2	4774.7	-	-	0.0	19.2	-	-	0--0
1	7		32.00	0	3459.4	4774.7	-	-	0.0	19.2	-	-	0--0
1	8		36.57	0	3640.4	4774.7	-	-	0.0	19.2	-	-	0--0
1	9		41.14	0	3734.7	4774.7	-	-	0.0	19.2	-	-	0--0
1	10		45.71	0	3766.2	4774.7	-	-	0.0	19.2	-	-	0--0
1	11		50.28	0	3735.0	4774.7	-	-	0.0	19.2	-	-	0--0
1	12		54.85	0	3640.4	4774.7	-	-	0.0	19.2	-	-	0--0
1	13		59.42	0	3459.6	4774.7	-	-	0.0	19.2	-	-	0--0
1	14		63.99	0	3200.7	4774.7	-	-	0.0	19.2	-	-	0--0
1	15		68.56	0	2868.2	4774.7	-	-	0.0	19.2	-	-	0--0
1	16		73.13	0	2459.8	3936.8	-	-	0.0	19.2	-	-	0--0
1	17		77.70	0	1967.0	3936.8	-	-	0.0	19.2	-	-	0--0
1	18		82.28	0	1393.1	3936.8	-	-	0.0	19.2	-	-	0--0
1	19		86.85	0	737.5	3936.8	-	-	0.0	19.2	-	-	0--0
1	20		91.42	0	0.0	3936.8	-	-	0.0	19.2	-	-	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/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0	0.000			0.000		1
1	1	4.57	0	0.187			0.187		1
1	2	9.14	0	0.354			0.354		1
1	3	13.71	0	0.499			0.499		1
1	4	18.28	0	0.625			0.625		1
1	5	22.85	0	0.601			0.601		1
1	6	27.43	0	0.670			0.670		1
1	7	32.00	0	0.725			0.725		1
1	8	36.57	0	0.762			0.762		1
1	9	41.14	0	0.782			0.782		1
1	10	45.71	0	0.789			0.789		1
1	11	50.28	0	0.782			0.782		1
1	12	54.85	0	0.762			0.762		1
1	13	59.42	0	0.725			0.725		1
1	14	63.99	0	0.670			0.670		1
1	15	68.56	0	0.601			0.601		1
1	16	73.13	0	0.625			0.625		1
1	17	77.70	0	0.500			0.500		1
1	18	82.28	0	0.354			0.354		1
1	19	86.85	0	0.187			0.187		1
1	20	91.42	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	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	1	4.57	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	2	9.14	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	3	13.71	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	4	18.28	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	5	22.85	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	6	27.43	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	7	32.00	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	8	36.57	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	9	41.14	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	10	45.71	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7

1	11	50.28	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	12	54.85	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	13	59.42	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	14	63.99	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	15	68.56	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	16	73.13	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	17	77.70	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	18	82.28	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	19	86.85	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7
1	20	91.42	32.0	5.0	75.4	94.2	37.8	1.000	547.7	547.7

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 L SUPT (ft)	WEB STABILITY UNSTIFFENED,[1] ---- D/tw -----		LFD/LRFD MAX. SHEAR (kip)	<=>	STRENGTH CATEGORY; SEE NOTE UNSTIFFENED SHEAR CAPACITY, [2]		OVERALL WEB
1	0	0.00	0		215.9	<	547.7	0	0
1	1	4.57	0		198.6	<	547.7	0	0
1	2	9.14	0		182.0	<	547.7	0	0
1	3	13.71	0		165.6	<	547.7	0	0
1	4	18.28	0		149.4	<	547.7	0	0

1	5	22.85	0	133.1	<	547.7	0	0
1	6	27.43	0	117.3	<	547.7	0	0
1	7	32.00	0	101.6	<	547.7	0	0
1	8	36.57	0	85.8	<	547.7	0	0
1	9	41.14	0	70.5	<	547.7	0	0
1	10	45.71	0	55.3	<	547.7	0	0
1	11	50.28	0	70.4	<	547.7	0	0
1	12	54.85	0	86.0	<	547.7	0	0
1	13	59.42	0	101.5	<	547.7	0	0
1	14	63.99	0	117.2	<	547.7	0	0
1	15	68.56	0	133.3	<	547.7	0	0
1	16	73.13	0	149.3	<	547.7	0	0
1	17	77.70	0	165.6	<	547.7	0	0
1	18	82.28	0	182.0	<	547.7	0	0
1	19	86.85	0	198.8	<	547.7	0	0
1	20	91.42	0	216.0	<	547.7	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 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	32.	215.86	547.67	0	
1	1		4.57	32.	198.62	547.67	0	
1	2		9.14	32.	182.03	547.67	0	
1	3		13.71	32.	165.63	547.67	0	
1	4		18.28	32.	149.40	547.67	0	
1	5		22.85	32.	133.13	547.67	0	
1	6		27.43	32.	117.26	547.67	0	
1	7		32.00	32.	101.56	547.67	0	
1	8		36.57	32.	85.83	547.67	0	
1	9		41.14	32.	70.49	547.67	0	
1	10		45.71	32.	55.33	547.67	0	
1	11		50.28	32.	70.43	547.67	0	
1	12		54.85	32.	85.99	547.67	0	
1	13		59.42	32.	101.51	547.67	0	
1	14		63.99	32.	117.20	547.67	0	
1	15		68.56	32.	133.29	547.67	0	
1	16		73.13	32.	149.34	547.67	0	
1	17		77.70	32.	165.57	547.67	0	
1	18		82.28	32.	181.97	547.67	0	
1	19		86.85	32.	198.78	547.67	0	
1	20		91.42	32.	216.02	547.67	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)	FLG fcrw [3] (ksi)	FLAG TBC
1	0		0.00	0.0	-	30.4	0.0	30.4	-	-	00-
1	1		4.57	3.6	-	30.4	5.7	30.4	-	-	00-
1	2		9.14	6.8	-	30.4	10.7	30.4	-	-	00-
1	3		13.71	9.6	-	30.4	15.1	30.4	-	-	00-
1	4		18.28	12.1	-	30.4	19.0	30.4	-	-	00-
1	5		22.85	13.4	-	30.4	15.2	30.4	-	-	00-
1	6		27.43	14.9	-	30.4	17.0	30.4	-	-	00-
1	7		32.00	16.2	-	30.4	18.4	30.4	-	-	00-
1	8		36.57	17.0	-	30.4	19.4	30.4	-	-	00-
1	9		41.14	17.5	-	30.4	19.9	30.4	-	-	00-
1	10		45.71	17.7	-	30.4	20.0	30.4	-	-	00-
1	11		50.28	17.5	-	30.4	19.9	30.4	-	-	00-
1	12		54.85	17.0	-	30.4	19.4	30.4	-	-	00-
1	13		59.42	16.2	-	30.4	18.4	30.4	-	-	00-
1	14		63.99	14.9	-	30.4	17.0	30.4	-	-	00-
1	15		68.56	13.4	-	30.4	15.2	30.4	-	-	00-
1	16		73.13	12.1	-	30.4	19.0	30.4	-	-	00-
1	17		77.70	9.6	-	30.4	15.2	30.4	-	-	00-
1	18		82.28	6.8	-	30.4	10.7	30.4	-	-	00-
1	19		86.85	3.6	-	30.4	5.7	30.4	-	-	00-
1	20		91.42	0.0	-	30.4	0.0	30.4	-	-	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 = II -- Urban Interstate

SP NO	IN NO	D L (ft)	FROM SUPT	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	4.57		TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	2	9.14		TR	-1.0	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	3	13.71		TR	-1.4	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	4	18.28		TR	-1.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1		19.21	SCG						
1	5	22.85		TR	-2.0	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	6	27.43		TR	-2.2	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	7	32.00		TR	-2.3	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	8	36.57		TR	-2.4	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	9	41.14		TR	-2.4	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	10	45.71		TR	-2.4	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	11	50.28		TR	-2.4	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	12	54.85		TR	-2.4	A B B^C^C D E E^	TR	4.6	A B B^C^C D
1	13	59.42		TR	-2.3	A B B^C^C D E E^	TR	4.5	A B B^C^C D E
1	14	63.99		TR	-2.2	A B B^C^C D E E^	TR	4.2	A B B^C^C D E
1	15	68.56		TR	-2.0	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1		72.21	SCG						
1	16	73.13		TR	-1.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	17	77.70		TR	-1.4	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	18	82.28		TR	-1.0	A B B^C^C D E E^	TR	2.8	A B B^C^C D E
1	19	86.85		TR	-0.5	A B B^C^C D E E^	TR	1.5	A B B^C^C D E E^
1	20	91.42		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 = II -- Urban 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		4.57	TR	-0.3	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	2		9.14	TR	-0.5	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	3		13.71	TR	-0.7	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	4		18.28	TR	-0.9	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1			19.21	SCG					
1	5		22.85	TR	-1.0	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	6		27.43	TR	-1.1	A B B^C^C D E E^	TR	2.1	A B B^C^C D E
1	7		32.00	TR	-1.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E
1	8		36.57	TR	-1.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	9		41.14	TR	-1.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	10		45.71	TR	-1.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	11		50.28	TR	-1.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	12		54.85	TR	-1.2	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	13		59.42	TR	-1.2	A B B^C^C D E E^	TR	2.2	A B B^C^C D E
1	14		63.99	TR	-1.1	A B B^C^C D E E^	TR	2.1	A B B^C^C D E
1	15		68.56	TR	-1.0	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1			72.21	SCG					
1	16		73.13	TR	-0.9	A B B^C^C D E E^	TR	2.4	A B B^C^C D E
1	17		77.70	TR	-0.7	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	18		82.28	TR	-0.5	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	19		86.85	TR	-0.3	A B B^C^C D E E^	TR	0.7	A B B^C^C D E E^
1	20		91.42	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= 2038 & No. of Cycles= 55790250
* 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	54.4	.21127E-01	1.1	TR	3.09	3	8.08
1	1	4.57	52.4	.21127E-01	1.1	TR	3.09	3	8.38
1	2	9.14	50.6	.21127E-01	1.1	TR	3.09	3	8.68
1	3	13.71	48.8	.21127E-01	1.0	TR	3.09	3	8.99
1	4	18.28	47.2	.21127E-01	1.0	TR	3.09	3	9.31
1	5	22.85	45.6	.19563E-01	0.9	TR	3.09	3	10.40
1	6	27.43	44.2	.19563E-01	0.9	TR	3.09	3	10.74
1	7	32.00	42.5	.19563E-01	0.8	TR	3.09	3	11.15
1	8	36.57	41.5	.19563E-01	0.8	TR	3.09	3	11.45
1	9	41.14	41.0	.19563E-01	0.8	TR	3.09	3	11.57
1	10	45.71	40.8	.19563E-01	0.8	TR	3.09	3	11.62
1	11	50.28	41.0	.19563E-01	0.8	TR	3.09	3	11.57
1	12	54.85	41.5	.19563E-01	0.8	TR	3.09	3	11.45
1	13	59.42	42.5	.19563E-01	0.8	TR	3.09	3	11.15
1	14	63.99	44.2	.19563E-01	0.9	TR	3.09	3	10.74
1	15	68.56	45.6	.19563E-01	0.9	TR	3.09	3	10.40
1	16	73.13	47.2	.21127E-01	1.0	TR	3.09	3	9.31
1	17	77.70	48.8	.21127E-01	1.0	TR	3.09	3	8.99
1	18	82.28	50.6	.21127E-01	1.1	TR	3.09	3	8.68
1	19	86.85	52.4	.21127E-01	1.1	TR	3.09	3	8.38
1	20	91.42	54.4	.21127E-01	1.1	TR	3.09	3	8.08

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= 2038
*** 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]	AASHTO LRFD Eqn. 6.10.10.2-2 Vsr [3]	[4],Zr <--Input	SHEAR CONN. PER TRANS. SECTION [5], input		MAX. ALLOW. SHEAR CONN. ---PITCH---- (in),[6]
1	0	0.00	27.2	.21127E-01	0.6	TR	0.76	3	3.95
1	1	4.57	26.2	.21127E-01	0.6	TR	0.76	3	4.10
1	2	9.14	25.3	.21127E-01	0.5	TR	0.76	3	4.25
1	3	13.71	24.4	.21127E-01	0.5	TR	0.76	3	4.40
1	4	18.28	23.6	.21127E-01	0.5	TR	0.76	3	4.55
1	5	22.85	22.8	.19563E-01	0.4	TR	0.76	3	5.08
1	6	27.43	22.1	.19563E-01	0.4	TR	0.76	3	5.25
1	7	32.00	21.3	.19563E-01	0.4	TR	0.76	3	5.45
1	8	36.57	20.7	.19563E-01	0.4	TR	0.76	3	5.60
1	9	41.14	20.5	.19563E-01	0.4	TR	0.76	3	5.66
1	10	45.71	20.4	.19563E-01	0.4	TR	0.76	3	5.68
1	11	50.28	20.5	.19563E-01	0.4	TR	0.76	3	5.66
1	12	54.85	20.7	.19563E-01	0.4	TR	0.76	3	5.60
1	13	59.42	21.3	.19563E-01	0.4	TR	0.76	3	5.45
1	14	63.99	22.1	.19563E-01	0.4	TR	0.76	3	5.25
1	15	68.56	22.8	.19563E-01	0.4	TR	0.76	3	5.08
1	16	73.13	23.6	.21127E-01	0.5	TR	0.76	3	4.55
1	17	77.70	24.4	.21127E-01	0.5	TR	0.76	3	4.40
1	18	82.28	25.3	.21127E-01	0.5	TR	0.76	3	4.25
1	19	86.85	26.2	.21127E-01	0.6	TR	0.76	3	4.10
1	20	91.42	27.2	.21127E-01	0.6	TR	0.76	3	3.95

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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= 2038 & No. of Cycles= 55790250

*** If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)
****

SP NO	IN D FROM L SUPT (ft)	MOMENT REGION 1=POS. 0=NEG.	P2p,1n (k) LRFD 6.10.10.4.2	P1p (k)	P2n (k) FATIG. CRI.	NO OF SHEAR CONNECTOR N(PER ZONE)	<=>	SEE NOTE N1	SEE NOTE N2	STATUS BLANK=OK CHECK=**
1 10	45.71	1	3181.6	1115.6		207	>	67		
1 20	91.42	1	3181.6	1115.6		207	>	67		

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.

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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	45.708	8.000
1	45.708	91.417	8.000

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	91.4	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
NG	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	OPERAT. RATING FACTOR [3] POS. MAX. STRESS	OPERAT. RATING FACTOR [3] NEG.	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0	0.00	3936.8	0.0	0.0	0.0	0.0	HL-93	9.99	0
1	1	4.57	3936.8	179.4	72.0	234.6	0.0	HL-93	8.79	0
1	2	9.14	3936.8	340.0	136.5	442.4	0.0	HL-93	4.29	0
1	3	13.71	3936.8	481.9	193.4	623.2	0.0	HL-93	2.81	0
1	4	18.28	3936.8	605.1	242.7	777.0	0.0	HL-93	2.09	0
1	5	22.85	4774.7	709.1	284.4	902.3	0.0	HL-93	2.21	0
1	6	27.43	4774.7	794.1	318.5	1004.1	0.0	HL-93	1.90	0
1	7	32.00	4774.7	860.5	345.1	1083.4	0.0	HL-93	1.69	0
1	8	36.57	4774.7	908.0	364.0	1137.5	0.0	HL-93	1.57	0
1	9	41.14	4774.7	936.2	375.4	1162.1	0.0	HL-93	1.51	0
1	10	45.71	4774.7	945.6	379.2	1170.3	0.0	HL-93	1.49	0
1	11	50.28	4774.7	936.4	375.4	1162.1	0.0	HL-93	1.51	0
1	12	54.85	4774.7	908.0	364.0	1137.5	0.0	HL-93	1.57	0
1	13	59.42	4774.7	860.6	345.1	1083.4	0.0	HL-93	1.69	0
1	14	63.99	4774.7	794.5	318.5	1004.1	0.0	HL-93	1.90	0
1	15	68.56	4774.7	709.7	284.4	902.3	0.0	HL-93	2.21	0
1	16	73.13	3936.8	605.4	242.7	777.0	0.0	HL-93	2.09	0
1	17	77.70	3936.8	482.5	193.4	623.2	0.0	HL-93	2.81	0
1	18	82.28	3936.8	340.8	136.5	442.4	0.0	HL-93	4.29	0
1	19	86.85	3936.8	180.0	72.0	234.6	0.0	HL-93	8.79	0
1	20	91.42	3936.8	0.0	0.0	0.0	0.0	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	UNFACTORED L+I STRESS MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	30.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	4.57	30.4	-2.56 2.82	-0.80 2.19	0.00 0.00	9.99	HL-93	7.68
1	2	9.14	30.4	-4.85 5.34	-1.50 4.14	0.00 0.00	6.06	HL-93	4.66
1	3	13.71	30.4	-6.88 7.57	-2.12 5.83	0.00 0.00	3.92	HL-93	3.01
1	4	18.28	30.4	-8.64 9.50	-2.64 7.27	0.00 0.00	2.88	HL-93	2.21
1	5	22.85	30.4	-9.42 7.69	-3.02 5.81	0.00 0.00	3.91	HL-93	3.01
1	6	27.43	30.4	-10.55 8.61	-3.36 6.47	0.00 0.00	3.37	HL-93	2.59
1	7	32.00	30.4	-11.43 9.33	-3.63 6.98	0.00 0.00	3.02	HL-93	2.32
1	8	36.57	30.4	-12.07 9.84	-3.81 7.33	0.00 0.00	2.81	HL-93	2.16
1	9	41.14	30.4	-12.44 10.15	-3.89 7.48	0.00 0.00	2.71	HL-93	2.08
1	10	45.71	30.4	-12.57 10.25	-3.92 7.54	0.00 0.00	2.67	HL-93	2.06
1	11	50.28	30.4	-12.44 10.15	-3.89 7.48	0.00 0.00	2.71	HL-93	2.08
1	12	54.85	30.4	-12.07 9.84	-3.81 7.33	0.00 0.00	2.81	HL-93	2.16
1	13	59.42	30.4	-11.44 9.33	-3.63 6.98	0.00 0.00	3.02	HL-93	2.32
1	14	63.99	30.4	-10.56 8.61	-3.36 6.47	0.00 0.00	3.37	HL-93	2.59
1	15	68.56	30.4	-9.43 7.69	-3.02 5.81	0.00 0.00	3.91	HL-93	3.01
1	16	73.13	30.4	-8.64 9.51	-2.64 7.27	0.00 0.00	2.87	HL-93	2.21
1	17	77.70	30.4	-6.89 7.58	-2.12 5.83	0.00 0.00	3.92	HL-93	3.01
1	18	82.28	30.4	-4.86 5.35	-1.50 4.14	0.00 0.00	6.05	HL-93	4.66
1	19	86.85	30.4	-2.57 2.83	-0.80 2.19	0.00 0.00	9.99	HL-93	7.68
1	20	91.42	30.4	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	91.42	45.71	STRENGTH L.S.	1.93	1.49	HL-93
1	91.42	45.71	SERVICEBILITY	2.67	2.06	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 1.935
Inventory= 1.492

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	91.42	45.71	STRENGTH L.S.	2.56	1195.08
1	91.42	45.71	SERVICEBILITY	3.46	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 2.558

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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 DEAD LOAD SHEAR D2	UNFACTORED L+I SHEAR MAX. POS.	UNFACTORED L+I SHEAR MAX. NEG.	OPERAT. RATING FACTOR [3]	OPERAT. RATING LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	547.7	41.6	16.6	80.2	0.0	4.36	HL-93	3.36
1	1	4.57	547.7	37.2	14.9	74.9	-1.7	4.75	HL-93	3.66
1	2	9.14	547.7	33.1	13.3	69.7	-3.8	5.18	HL-93	4.00

1	3	13.71	547.7	29.0	11.6	64.5	-6.4	5.68	HL-93	4.38
1	4	18.28	547.7	24.9	10.0	59.5	-9.1	6.25	HL-93	4.82
1	5	22.85	547.7	20.6	8.3	54.6	-11.8	6.92	HL-93	5.34
1	6	27.43	547.7	16.6	6.6	49.8	-15.1	7.70	HL-93	5.94
1	7	32.00	547.7	12.5	5.0	45.1	-19.0	8.62	HL-93	6.65
1	8	36.57	547.7	8.2	3.3	40.5	-23.1	9.74	HL-93	7.52
1	9	41.14	547.7	4.1	1.7	36.0	-27.3	9.99	HL-93	8.57
1	10	45.71	547.7	0.0	0.0	31.6	-31.6	9.99	HL-93	9.90
1	11	50.28	547.7	-4.1	-1.7	27.3	-36.0	9.99	HL-93	8.57
1	12	54.85	547.7	-8.3	-3.3	23.1	-40.5	9.74	HL-93	7.51
1	13	59.42	547.7	-12.4	-5.0	19.0	-45.1	8.62	HL-93	6.65
1	14	63.99	547.7	-16.5	-6.6	15.1	-49.8	7.70	HL-93	5.94
1	15	68.56	547.7	-20.8	-8.3	11.8	-54.6	6.92	HL-93	5.33
1	16	73.13	547.7	-24.9	-10.0	9.1	-59.5	6.25	HL-93	4.82
1	17	77.70	547.7	-28.9	-11.6	6.4	-64.5	5.68	HL-93	4.38
1	18	82.28	547.7	-33.0	-13.3	3.8	-69.7	5.18	HL-93	4.00
1	19	86.85	547.7	-37.3	-14.9	1.7	-74.9	4.75	HL-93	3.66
1	20	91.42	547.7	-41.8	-16.6	0.0	-80.2	4.36	HL-93	3.36

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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	91.42	91.42	STRENGTH L.S.	4.36	3.36	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 4.360
Inventory= 3.363

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	91.42	91.42	STRENGTH L.S.	5.14	91.75

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 5.144

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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	45.71	2	0.400	-0.76	HL-93	1.80
		3	0.510	-0.97		1.41

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. 5800 Span 3 Exterior Beam	7/17/2013

CONTRACT NUMBER	STR NO	STR UNIT	DES -BY-	CHK -BY-	SPECS. USED
CKE410A4 5800	Span 3(6)	IAS/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	2	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)	DEPTH (ft)	WIDTH. (in)	(in)	REGION (%)	FACTOR >= 1.0
5	2	31.33	2.50	1.83	0.00	0.00	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
41.62	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
7.50	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 DEPTH ID.	WEIGHT (lb/ft)	STANDARD SECTN 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	36	150		0.00	0.0000	0.00	0.0000		

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	MEMB END	MEMBER DESCRIPTN	PARAMETERS FOR	YIELD STRESS

NUMBER (IN ORDER)	SECT LEFT	SECT RIGHT	ID	LNGTH (ft)	NONPRISMATIC MEMB TYPE 0=PRISMAT	S(0)	S(1)	(KSI) WEB	TOP	BOT
1	1	1		41.62				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	2	2398	2038	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 2398

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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 IMP FCTR	DF M-	DF M+	DF M-	IMP F	A	D	M	G	C	DF M+ LOADING TYPES

NO STR/SER (%)	ST/SE	FA	FA	FA (%)	NO=0 ;YES= 1	ST/SE	A D M G C
1 33.00	0	0.00	0.00	0.00	0.00	1 1 1 1 1	0.56	2 2 2 2 2
1 0.00	0	0.00	0.00	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.

TABLE 0.0.10.1 SLAB LOAD DEFINITION
****

LOAD NO	IDENTIFICATION NO	DESCRIPTION	SLAB DESIGN DEPTH INITIAL (in)	SLAB DESIGN DEPTH FINAL (in)	POUR DAY	MODULAR RATIO N1=3n	MODULAR RATIO N2=n	SLAB LOAD DATA INTENSITY (k/ft)	SLAB LOAD DATA POSITION FROM (ft)	SLAB LOAD DATA POSITION TO (ft)
1	1	DECK+haunch	0.0	7.0	0	30.0	10.0	0.60	0.00	41.62

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.

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

LOAD IDENTIFICATION NO.	LOAD TYPE	DESCRIPTION	UNIFROM LOAD DATA INTENSITY (k/ft)	UNIFROM LOAD DATA POSITION FROM (ft)	UNIFROM LOAD DATA POSITION TO (ft)	CONCENTRATED LOAD DATA INTENSITY (Kips)	CONCENTRATED LOAD DATA DISTANCE FROM L SUPT (ft)

1	0	wearing surf	0.201	0.00	41.62	0.00	0.00
2	0	rail	0.025	0.00	41.62	0.00	0.00
3	0	curb	0.287	0.00	41.62	0.00	0.00
4	0	end diaphrag	0.000	0.00	0.00	0.38	0.45
5	0	int diaphrag	0.000	0.00	0.00	0.18	13.45
6	0	int diaphrag	0.000	0.00	0.00	0.18	28.45
7	0	end diaphrag	0.000	0.00	0.00	0.38	41.17

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 PER TRAN. SEC (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 YIELD Fy (ksi)	BAR AREA PER FOOT STRESS OF SLAB (in**2)	DIST. FROM TOP (in)	COMP. STRENG. AT 28 DAYS (ksi)	COMP. ALLOW -ABLE (ksi)
0	0.000	0	0	0.00	0.0	0.0	0.0	0.00	0.00	2.50	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)

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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	SPACING

DISTANCE FROM (ft)	DISTANCE TO (ft)	STRESS Fy (ksi)	STRESS Fy (WEB) (ksi)	OF LATERAL BRACING Lb (ft)
0.00	0.45	33.0	33.0	0.45
0.45	13.45	33.0	33.0	12.99
13.45	28.45	33.0	33.0	15.00
28.45	41.17	33.0	33.0	12.73

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 2.2.1.1=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

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TABLE 2.2.2.1=LOADING INFORMATION
****

AVERAGE DEAD LOAD INTENSITIES
****

SPAN NO.	SLAB (K/FT)	+	STEEL (K/FT)	=	TOTAL (K/FT)

1	0.603	0.1504	0.7534

NONCOMPOSITE SUPERIMPOSED DEAD LOADS

LOAD	INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	0.201	0.000	41.625
2	0.025	0.000	41.625
3	0.287	0.000	41.625

TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES

CONCRETE DECK				STEEL	SUPERSTRUCTURE TOTAL WEIGHT
	VOLUME		TOTAL	TOTAL	
UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	WEIGHT (kip)	WEIGHT (kip)	(kip)
150.00	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.380	0.452
2	0.176	13.447
3	0.176	28.447
4	0.380	41.171

TABLE 2.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)

1	0.560	0.560	0.649	0.649	FOR STRENGTH POSITIVE MOMENT
	0.560				FOR STRENGTH POSITIVE SHEAR
1	0.484				FOR FATIGUE POSITIVE MOMENT
	0.484				FOR FATIGUE POSITIVE SHEAR

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TABLE 2.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY

SP NO	IN NO	D FROM L SUPT	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)	STEEL TOP. (in**3)
1	0	0.00	9040.0	35.9	17.95	503.6	503.6
1	1	2.08	9040.0	35.9	17.95	503.6	503.6
1	2	4.16	9040.0	35.9	17.95	503.6	503.6
1	3	6.24	9040.0	35.9	17.95	503.6	503.6
1	4	8.32	9040.0	35.9	17.95	503.6	503.6
1	5	10.41	9040.0	35.9	17.95	503.6	503.6
1	6	12.49	9040.0	35.9	17.95	503.6	503.6
1	7	14.57	9040.0	35.9	17.95	503.6	503.6
1	8	16.65	9040.0	35.9	17.95	503.6	503.6
1	9	18.73	9040.0	35.9	17.95	503.6	503.6
1	10	20.81	9040.0	35.9	17.95	503.6	503.6
1	11	22.89	9040.0	35.9	17.95	503.6	503.6
1	12	24.97	9040.0	35.9	17.95	503.6	503.6
1	13	27.06	9040.0	35.9	17.95	503.6	503.6
1	14	29.14	9040.0	35.9	17.95	503.6	503.6
1	15	31.22	9040.0	35.9	17.95	503.6	503.6
1	16	33.30	9040.0	35.9	17.95	503.6	503.6
1	17	35.38	9040.0	35.9	17.95	503.6	503.6
1	18	37.46	9040.0	35.9	17.95	503.6	503.6
1	19	39.54	9040.0	35.9	17.95	503.6	503.6
1	20	41.62	9040.0	35.9	17.95	503.6	503.6

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TABLE 2.2.5.1=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) LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0	
1	1	2.08	6.2	24.8	0.5	21.1	52.6	
1	2	4.16	11.7	47.0	0.9	40.0	99.6	
1	3	6.24	16.6	66.6	1.3	56.7	141.1	
1	4	8.32	20.8	83.6	1.6	71.1	177.2	
1	5	10.41	24.4	97.9	2.0	83.3	207.7	
1	6	12.49	27.4	109.7	2.4	93.3	232.7	
1	7	14.57	29.6	118.8	2.5	101.1	252.1	
1	8	16.65	31.3	125.4	2.5	106.7	265.8	
1	9	18.73	32.2	129.3	2.5	110.0	274.1	
1	10	20.81	32.6	130.6	2.5	111.1	276.8	
1	11	22.89	32.2	129.3	2.5	110.0	274.0	

1	12	24.97	31.3	125.4	2.5	106.7	265.8
1	13	27.06	29.6	118.8	2.5	101.1	252.1
1	14	29.14	27.4	109.7	2.4	93.3	232.8
1	15	31.22	24.4	97.9	2.0	83.3	207.7
1	16	33.30	20.8	83.6	1.6	71.1	177.2
1	17	35.38	16.6	66.6	1.3	56.7	141.2
1	18	37.46	11.7	47.0	0.9	40.0	99.6
1	19	39.54	6.2	24.8	0.5	21.1	52.7
1	20	41.62	0.0	0.0	0.0	0.0	0.0

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TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (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	2.08	0.0	0.0	95.8	HL-93	0.0	HL-93
1	2	4.16	0.0	0.0	178.8	HL-93	0.0	HL-93
1	3	6.24	0.0	0.0	249.1	HL-93	0.0	HL-93
1	4	8.32	0.0	0.0	306.7	HL-93	0.0	HL-93
1	5	10.41	0.0	0.0	349.5	HL-93	0.0	HL-93
1	6	12.49	0.0	0.0	383.8	HL-93	0.0	HL-93
1	7	14.57	0.0	0.0	410.6	HL-93	0.0	HL-93
1	8	16.65	0.0	0.0	426.7	HL-93	0.0	HL-93
1	9	18.73	0.0	0.0	427.0	HL-93	0.0	HL-93
1	10	20.81	0.0	0.0	427.9	HL-93	0.0	HL-93
1	11	22.89	0.0	0.0	427.0	HL-93	0.0	HL-93
1	12	24.97	0.0	0.0	426.7	HL-93	0.0	HL-93
1	13	27.06	0.0	0.0	410.6	HL-93	0.0	HL-93
1	14	29.14	0.0	0.0	383.8	HL-93	0.0	HL-93
1	15	31.22	0.0	0.0	349.5	HL-93	0.0	HL-93
1	16	33.30	0.0	0.0	306.7	HL-93	0.0	HL-93
1	17	35.38	0.0	0.0	249.1	HL-93	0.0	HL-93
1	18	37.46	0.0	0.0	178.8	HL-93	0.0	HL-93
1	19	39.54	0.0	0.0	95.8	HL-93	0.0	HL-93
1	20	41.62	0.0	0.0	0.0	HL-93	0.0	HL-93

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TABLE 2.2.5.2A=FATIGUE LIVE LOAD MOMENT RANGE (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	2.08	44.	0.	44.
1	2	4.16	80.	0.	80.
1	3	6.24	106.	0.	106.
1	4	8.32	130.	0.	130.
1	5	10.41	156.	0.	156.
1	6	12.49	176.	0.	176.
1	7	14.57	189.	0.	189.
1	8	16.65	198.	0.	198.

1	9	18.73	200.	0.	200.
1	10	20.81	201.	0.	201.
1	11	22.89	200.	0.	200.
1	12	24.97	198.	0.	198.
1	13	27.06	189.	0.	189.
1	14	29.14	176.	0.	176.
1	15	31.22	156.	0.	156.
1	16	33.30	130.	0.	130.
1	17	35.38	106.	0.	106.
1	18	37.46	80.	0.	80.
1	19	39.54	44.	0.	44.
1	20	41.62	0.	0.	0.

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TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0	0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1	2.08	52.6	95.8	HL-93	0.0	HL-93	148.4	52.6
1	2	4.16	99.6	178.8	HL-93	0.0	HL-93	278.4	99.6
1	3	6.24	141.1	249.1	HL-93	0.0	HL-93	390.3	141.1
1	4	8.32	177.2	306.7	HL-93	0.0	HL-93	483.9	177.2
1	5	10.41	207.7	349.5	HL-93	0.0	HL-93	557.2	207.7
1	6	12.49	232.7	383.8	HL-93	0.0	HL-93	616.5	232.7
1	7	14.57	252.1	410.6	HL-93	0.0	HL-93	662.7	252.1
1	8	16.65	265.8	426.7	HL-93	0.0	HL-93	692.5	265.8
1	9	18.73	274.1	427.0	HL-93	0.0	HL-93	701.1	274.1
1	10	20.81	276.8	427.9	HL-93	0.0	HL-93	704.7	276.8
1	11	22.89	274.0	427.0	HL-93	0.0	HL-93	701.1	274.0
1	12	24.97	265.8	426.7	HL-93	0.0	HL-93	692.5	265.8
1	13	27.06	252.1	410.6	HL-93	0.0	HL-93	662.7	252.1
1	14	29.14	232.8	383.8	HL-93	0.0	HL-93	616.6	232.8
1	15	31.22	207.7	349.5	HL-93	0.0	HL-93	557.3	207.7
1	16	33.30	177.2	306.7	HL-93	0.0	HL-93	483.9	177.2
1	17	35.38	141.2	249.1	HL-93	0.0	HL-93	390.3	141.2
1	18	37.46	99.6	178.8	HL-93	0.0	HL-93	278.4	99.6
1	19	39.54	52.7	95.8	HL-93	0.0	HL-93	148.4	52.7
1	20	41.62	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE 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	2.08	148.4	177.1	233.4	195.1	79.0	32.8
1	2	4.16	278.4	332.1	437.4	365.9	149.5	60.1
1	3	6.24	390.3	465.0	612.4	512.7	211.7	79.4
1	4	8.32	483.9	575.9	758.2	635.5	265.7	97.9

1	5	10.41	557.2	662.1	871.3	731.5	311.5	117.2
1	6	12.49	616.5	731.7	962.6	809.1	349.1	132.0
1	7	14.57	662.7	785.9	1033.6	869.4	378.2	141.9
1	8	16.65	692.5	820.6	1079.0	908.3	398.7	148.2
1	9	18.73	701.1	829.2	1089.8	919.0	411.1	149.9
1	10	20.81	704.7	833.1	1094.8	923.7	415.2	150.5
1	11	22.89	701.1	829.2	1089.8	919.0	411.1	149.9
1	12	24.97	692.5	820.5	1079.0	908.3	398.7	148.2
1	13	27.06	662.7	785.8	1033.6	869.4	378.2	141.9
1	14	29.14	616.6	731.7	962.6	809.1	349.2	132.0
1	15	31.22	557.3	662.1	871.3	731.5	311.6	117.2
1	16	33.30	483.9	575.9	758.2	635.6	265.8	97.9
1	17	35.38	390.3	465.0	612.4	512.7	211.7	79.4
1	18	37.46	278.4	332.1	437.4	365.9	149.5	60.1
1	19	39.54	148.4	177.1	233.4	195.1	79.0	32.8
1	20	41.62	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 2.2.6.1=NONCOMPOSITE DEAD LOAD SHEARS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k)	DEAD LOAD SLAB (k)	CONCENTRATED LOADS (k)	UNIFORM LOADS (k)	TOTAL NONCOMPOSITE DEAD LOAD (k)
1	0	0.00	3.13	12.55	0.6	10.68	26.91
1	1	2.08	2.82	11.29	0.2	9.61	23.90
1	2	4.16	2.50	10.04	0.2	8.54	21.26
1	3	6.24	2.19	8.78	0.2	7.47	18.62
1	4	8.32	1.88	7.53	0.2	6.41	15.99
1	5	10.41	1.57	6.27	0.2	5.34	13.35
1	6	12.49	1.25	5.02	0.2	4.27	10.72
1	7	14.57	0.94	3.76	0.0	3.20	7.91
1	8	16.65	0.63	2.51	0.0	2.14	5.27
1	9	18.73	0.31	1.25	0.0	1.07	2.63
1	10	20.81	0.00	0.00	0.0	0.00	0.00
1	11	22.89	-0.31	-1.25	0.0	-1.07	-2.64
1	12	24.97	-0.63	-2.51	0.0	-2.14	-5.27
1	13	27.06	-0.94	-3.76	0.0	-3.20	-7.91
1	14	29.14	-1.25	-5.02	-0.2	-4.27	-10.72
1	15	31.22	-1.57	-6.27	-0.2	-5.34	-13.36
1	16	33.30	-1.88	-7.53	-0.2	-6.41	-15.99
1	17	35.38	-2.19	-8.78	-0.2	-7.47	-18.63
1	18	37.46	-2.50	-10.04	-0.2	-8.54	-21.26
1	19	39.54	-2.82	-11.29	-0.2	-9.61	-23.90
1	20	41.62	-3.13	-12.55	-0.6	-10.68	-26.91

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TABLE 2.2.6.2=LIVE LOAD SHEARS FOR NONCOMPOSITE CONSTRUCTION (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	49.05	HL-93	0.00	HL-93
1	1	2.08	0.0	0.0	45.64	HL-93	-1.21	HL-93

1	2	4.16	0.0	0.0	42.27	HL-93	-2.45	HL-93
1	3	6.24	0.0	0.0	38.93	HL-93	-3.96	HL-93
1	4	8.32	0.0	0.0	35.64	HL-93	-5.95	HL-93
1	5	10.41	0.0	0.0	32.32	HL-93	-7.98	HL-93
1	6	12.49	0.0	0.0	29.16	HL-93	-10.05	HL-93
1	7	14.57	0.0	0.0	26.17	HL-93	-12.15	HL-93
1	8	16.65	0.0	0.0	23.26	HL-93	-14.29	HL-93
1	9	18.73	0.0	0.0	20.94	HL-93	-16.47	HL-93
1	10	20.81	0.0	0.0	18.69	HL-93	-18.69	HL-93
1	11	22.89	0.0	0.0	16.47	HL-93	-20.94	HL-93
1	12	24.97	0.0	0.0	14.29	HL-93	-23.26	HL-93
1	13	27.06	0.0	0.0	12.15	HL-93	-26.17	HL-93
1	14	29.14	0.0	0.0	10.05	HL-93	-29.16	HL-93
1	15	31.22	0.0	0.0	7.98	HL-93	-32.32	HL-93
1	16	33.30	0.0	0.0	5.95	HL-93	-35.64	HL-93
1	17	35.38	0.0	0.0	3.96	HL-93	-38.93	HL-93
1	18	37.46	0.0	0.0	2.45	HL-93	-42.27	HL-93
1	19	39.54	0.0	0.0	1.21	HL-93	-45.64	HL-93
1	20	41.62	0.0	0.0	0.00	HL-93	-49.05	HL-93

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TABLE 2.2.6.2A=LIVE LOAD SHEAR RANGE (in kips) (UNFACTORED)
****

SP NO	IN D NO L	FROM SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	23.	0.	23.
1	1	2.08	21.	-1.	22.
1	2	4.16	19.	-2.	21.
1	3	6.24	17.	-3.	20.
1	4	8.32	16.	-4.	19.
1	5	10.41	15.	-4.	19.
1	6	12.49	14.	-5.	19.
1	7	14.57	13.	-6.	19.
1	8	16.65	12.	-7.	19.
1	9	18.73	11.	-9.	19.
1	10	20.81	10.	-10.	19.
1	11	22.89	9.	-11.	19.
1	12	24.97	7.	-12.	19.
1	13	27.06	6.	-13.	19.
1	14	29.14	5.	-14.	19.
1	15	31.22	4.	-15.	19.
1	16	33.30	4.	-16.	19.
1	17	35.38	3.	-17.	20.
1	18	37.46	2.	-19.	21.
1	19	39.54	1.	-21.	22.
1	20	41.62	0.	-23.	23.

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TABLE 2.2.6.2B=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	22.8	.20618E-01	0.24	0.3125*.20618E-01	0.24	0.3125*
1	1	2.08	21.9	.20618E-01	0.23	0.3125*.20618E-01	0.23	0.3125*
1	2	4.16	21.0	.20618E-01	0.22	0.3125*.20618E-01	0.22	0.3125*
1	3	6.24	19.9	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	4	8.32	19.2	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	5	10.41	19.4	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	6	12.49	19.4	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	7	14.57	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	8	16.65	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	9	18.73	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	10	20.81	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	11	22.89	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	12	24.97	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	13	27.06	19.3	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	14	29.14	19.4	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	15	31.22	19.4	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	16	33.30	19.2	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	17	35.38	19.9	.20618E-01	0.20	0.3125*.20618E-01	0.20	0.3125*
1	18	37.46	21.0	.20618E-01	0.22	0.3125*.20618E-01	0.22	0.3125*
1	19	39.54	21.9	.20618E-01	0.23	0.3125*.20618E-01	0.23	0.3125*
1	20	41.62	22.8	.20618E-01	0.24	0.3125*.20618E-01	0.24	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 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L (ft)	FROM SUPP	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0	0.00		26.9	49.0	HL-93	0.0	HL-93	76.0	26.9
1	1	2.08		23.9	45.6	HL-93	-1.2	HL-93	69.5	22.7
1	2	4.16		21.3	42.3	HL-93	-2.5	HL-93	63.5	18.8
1	3	6.24		18.6	38.9	HL-93	-4.0	HL-93	57.6	14.7
1	4	8.32		16.0	35.6	HL-93	-5.9	HL-93	51.6	10.0
1	5	10.41		13.4	32.3	HL-93	-8.0	HL-93	45.7	5.4
1	6	12.49		10.7	29.2	HL-93	-10.0	HL-93	39.9	0.7
1	7	14.57		7.9	26.2	HL-93	-12.1	HL-93	34.1	-4.2
1	8	16.65		5.3	23.3	HL-93	-14.3	HL-93	28.5	-9.0
1	9	18.73		2.6	20.9	HL-93	-16.5	HL-93	23.6	-13.8
1	10	20.81		0.0	18.7	HL-93	-18.7	HL-93	-18.7	18.7
1	11	22.89		-2.6	16.5	HL-93	-20.9	HL-93	-23.6	13.8

1	12	24.97	-5.3	14.3	HL-93	-23.3	HL-93	-28.5	9.0
1	13	27.06	-7.9	12.1	HL-93	-26.2	HL-93	-34.1	4.2
1	14	29.14	-10.7	10.0	HL-93	-29.2	HL-93	-39.9	-0.7
1	15	31.22	-13.4	8.0	HL-93	-32.3	HL-93	-45.7	-5.4
1	16	33.30	-16.0	5.9	HL-93	-35.6	HL-93	-51.6	-10.0
1	17	35.38	-18.6	4.0	HL-93	-38.9	HL-93	-57.6	-14.7
1	18	37.46	-21.3	2.5	HL-93	-42.3	HL-93	-63.5	-18.8
1	19	39.54	-23.9	1.2	HL-93	-45.6	HL-93	-69.5	-22.7
1	20	41.62	-26.9	0.0	HL-93	-49.0	HL-93	-76.0	-26.9

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TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE 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	76.0	90.7	119.5	99.9	40.4	17.1
1	1	2.08	69.5	83.2	109.7	91.5	35.8	16.4
1	2	4.16	63.5	76.2	100.5	83.6	31.9	15.8
1	3	6.24	57.6	69.2	91.4	75.8	27.9	14.9
1	4	8.32	51.6	62.3	82.4	68.1	24.0	14.4
1	5	10.41	45.7	55.4	73.3	60.3	20.0	14.5
1	6	12.49	39.9	48.6	64.4	52.8	16.1	14.6
1	7	14.57	34.1	41.9	55.7	45.2	11.9	14.5
1	8	16.65	28.5	35.5	47.3	38.0	7.9	14.5
1	9	18.73	23.6	29.9	39.9	31.6	4.0	14.5
1	10	20.81	-18.7	-24.3	-32.7	-25.2	0.0	14.5
1	11	22.89	-23.6	-29.9	-39.9	-31.6	-4.0	14.5
1	12	24.97	-28.5	-35.5	-47.3	-38.0	-7.9	14.5
1	13	27.06	-34.1	-41.9	-55.7	-45.2	-11.9	14.5
1	14	29.14	-39.9	-48.6	-64.4	-52.8	-16.1	14.6
1	15	31.22	-45.7	-55.4	-73.3	-60.3	-20.0	14.5
1	16	33.30	-51.6	-62.3	-82.4	-68.1	-24.0	14.4
1	17	35.38	-57.6	-69.2	-91.4	-75.8	-27.9	14.9
1	18	37.46	-63.5	-76.2	-100.5	-83.6	-31.9	15.8
1	19	39.54	-69.5	-83.2	-109.7	-91.5	-35.8	16.4
1	20	41.62	-76.0	-90.7	-119.5	-99.9	-40.4	17.1

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TABLE 2.2.7.2=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	26.91	0.00	HL-93	49.05	HL-93	ST1	24.22	119.47
						ST2	24.22	99.85
						ST4	40.37	40.37
						SE1	26.91	75.96
						SE2	26.91	90.67
2	26.91	0.00	HL-93	49.05	HL-93	ST1	24.22	119.47
						ST2	24.22	99.86
						ST4	40.37	40.37

SE1	26.91	75.96
SE2	26.91	90.68

NOTE: [1] " - " Indicates Uplift

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

TABLE 2.2.8.1=DEAD LOAD DEFLECTIONS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D NO L	FROM SUPT (ft)	DEAD LOAD BEAM (in)	DEAD LOAD SLAB (in)	CONCENTRATED LOADS (in)	UNIFORM LOADS (in)	T O T A L +=UP , -=DOWN
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	2.08	-0.0062	-0.0247	-0.0005	-0.0210	-0.0525
1	2	4.16	-0.0122	-0.0488	-0.0010	-0.0415	-0.1034
1	3	6.24	-0.0178	-0.0715	-0.0014	-0.0608	-0.1515
1	4	8.32	-0.0230	-0.0923	-0.0019	-0.0785	-0.1957
1	5	10.41	-0.0276	-0.1107	-0.0022	-0.0942	-0.2347
1	6	12.49	-0.0315	-0.1263	-0.0026	-0.1075	-0.2679
1	7	14.57	-0.0346	-0.1388	-0.0028	-0.1181	-0.2944
1	8	16.65	-0.0369	-0.1480	-0.0030	-0.1259	-0.3137
1	9	18.73	-0.0383	-0.1535	-0.0031	-0.1306	-0.3255
1	10	20.81	-0.0388	-0.1554	-0.0031	-0.1322	-0.3294
1	11	22.89	-0.0383	-0.1535	-0.0031	-0.1306	-0.3255
1	12	24.97	-0.0369	-0.1480	-0.0030	-0.1259	-0.3137
1	13	27.06	-0.0346	-0.1388	-0.0028	-0.1181	-0.2944
1	14	29.14	-0.0315	-0.1263	-0.0026	-0.1075	-0.2679
1	15	31.22	-0.0276	-0.1107	-0.0022	-0.0942	-0.2347
1	16	33.30	-0.0230	-0.0923	-0.0019	-0.0785	-0.1957
1	17	35.38	-0.0178	-0.0715	-0.0014	-0.0608	-0.1515
1	18	37.46	-0.0122	-0.0488	-0.0010	-0.0415	-0.1034
1	19	39.54	-0.0062	-0.0247	-0.0005	-0.0210	-0.0525
1	20	41.62	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " indicates downward deflection

TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)

SP NO	IN D NO L	FROM SUPT (FT)	NONCOMPOSITE DEAD LOADS (IN) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	OTHER 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	2.1	-0.006	0 1/16	-0.025	0 1/16	-0.022	0 1/16	-0.052	0 1/16
1	2	4.2	-0.012	0 1/16	-0.049	0 1/16	-0.042	0 1/16	-0.103	0 1/ 8
1	3	6.2	-0.018	0 1/16	-0.071	0 1/ 8	-0.062	0 1/16	-0.152	0 3/16
1	4	8.3	-0.023	0 1/16	-0.092	0 1/ 8	-0.080	0 1/ 8	-0.196	0 1/ 4
1	5	10.4	-0.028	0 1/16	-0.111	0 1/ 8	-0.096	0 1/ 8	-0.235	0 1/ 4
1	6	12.5	-0.032	0 1/16	-0.126	0 3/16	-0.110	0 1/ 8	-0.268	0 5/16
1	7	14.6	-0.035	0 1/16	-0.139	0 3/16	-0.121	0 1/ 8	-0.294	0 5/16
1	8	16.6	-0.037	0 1/16	-0.148	0 3/16	-0.129	0 3/16	-0.314	0 3/ 8
1	9	18.7	-0.038	0 1/16	-0.154	0 3/16	-0.134	0 3/16	-0.325	0 3/ 8
1	10	20.8	-0.039	0 1/16	-0.155	0 3/16	-0.135	0 3/16	-0.329	0 3/ 8
1	11	22.9	-0.038	0 1/16	-0.154	0 3/16	-0.134	0 3/16	-0.325	0 3/ 8
1	12	25.0	-0.037	0 1/16	-0.148	0 3/16	-0.129	0 3/16	-0.314	0 3/ 8
1	13	27.1	-0.035	0 1/16	-0.139	0 3/16	-0.121	0 1/ 8	-0.294	0 5/16
1	14	29.1	-0.032	0 1/16	-0.126	0 3/16	-0.110	0 1/ 8	-0.268	0 5/16

1	15	31.2	-0.028	0	1/16	-0.111	0	1/ 8	-0.096	0	1/ 8	-0.235	0	1/ 4
1	16	33.3	-0.023	0	1/16	-0.092	0	1/ 8	-0.080	0	1/ 8	-0.196	0	1/ 4
1	17	35.4	-0.018	0	1/16	-0.071	0	1/ 8	-0.062	0	1/16	-0.152	0	3/16
1	18	37.5	-0.012	0	1/16	-0.049	0	1/16	-0.042	0	1/16	-0.103	0	1/ 8
1	19	39.5	-0.006	0	1/16	-0.025	0	1/16	-0.022	0	1/16	-0.052	0	1/16
1	20	41.6	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 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****

(UNFACTORED)

SPAN NO.	D FROM SUPT L (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	20.81	2 0.400	-0.288 MAX	HL-93	0.62	0.00350
			0.038 MIN			
			-0.066 MAX	LANE		
			0.038 MIN	LANE		
		3 0.510	-0.368 MAX	HL-93	0.62	0.00350
			0.048 MIN			
			-0.084 MAX	LANE		
			0.048 MIN	LANE		
		2 0.400	-0.189 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 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	N.COMP DEAD LOAD STEEL BEAM TOP (ksi)	N.COMP 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	2.08	-0.74	0.74	-0.52	0.52	-1.25	1.25
1	2	4.16	-1.40	1.40	-0.97	0.97	-2.37	2.37
1	3	6.24	-1.98	1.98	-1.38	1.38	-3.36	3.36
1	4	8.32	-2.49	2.49	-1.73	1.73	-4.22	4.22
1	5	10.41	-2.92	2.92	-2.03	2.03	-4.95	4.95
1	6	12.49	-3.27	3.27	-2.28	2.28	-5.55	5.55
1	7	14.57	-3.54	3.54	-2.47	2.47	-6.01	6.01
1	8	16.65	-3.73	3.73	-2.60	2.60	-6.33	6.33
1	9	18.73	-3.85	3.85	-2.68	2.68	-6.53	6.53
1	10	20.81	-3.89	3.89	-2.71	2.71	-6.60	6.60
1	11	22.89	-3.85	3.85	-2.68	2.68	-6.53	6.53
1	12	24.97	-3.73	3.73	-2.60	2.60	-6.33	6.33
1	13	27.06	-3.54	3.54	-2.47	2.47	-6.01	6.01
1	14	29.14	-3.27	3.27	-2.28	2.28	-5.55	5.55
1	15	31.22	-2.92	2.92	-2.03	2.03	-4.95	4.95
1	16	33.30	-2.49	2.49	-1.73	1.73	-4.22	4.22
1	17	35.38	-1.98	1.98	-1.38	1.38	-3.36	3.36
1	18	37.46	-1.40	1.40	-0.97	0.97	-2.37	2.37
1	19	39.54	-0.74	0.74	-0.52	0.52	-1.25	1.25
1	20	41.62	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 2.2.9.2=LIVE + I STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	MAXIMUM POSITIVE,(ksi) STEEL TOP	MAXIMUM POSITIVE,(ksi) STEEL BOT	MAXIMUM NEGATIVE,(ksi) STEEL TOP	MAXIMUM NEGATIVE,(ksi) STEEL BOT
1	0	0.00	0.00	0.00	0.00	0.00
1	1	2.08	-2.28	2.28	0.00	0.00
1	2	4.16	-4.26	4.26	0.00	0.00
1	3	6.24	-5.94	5.94	0.00	0.00
1	4	8.32	-7.31	7.31	0.00	0.00
1	5	10.41	-8.33	8.33	0.00	0.00
1	6	12.49	-9.14	9.14	0.00	0.00
1	7	14.57	-9.78	9.78	0.00	0.00
1	8	16.65	-10.17	10.17	0.00	0.00
1	9	18.73	-10.17	10.17	0.00	0.00
1	10	20.81	-10.20	10.20	0.00	0.00
1	11	22.89	-10.17	10.17	0.00	0.00
1	12	24.97	-10.17	10.17	0.00	0.00
1	13	27.06	-9.78	9.78	0.00	0.00
1	14	29.14	-9.14	9.14	0.00	0.00
1	15	31.22	-8.33	8.33	0.00	0.00
1	16	33.30	-7.31	7.31	0.00	0.00
1	17	35.38	-5.94	5.94	0.00	0.00
1	18	37.46	-4.26	4.26	0.00	0.00
1	19	39.54	-2.28	2.28	0.00	0.00
1	20	41.62	0.00	0.00	0.00	0.00

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TABLE 2.2.9.2A=FATIGUE LIVE LOAD STRESS RANGE (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	LOAD TP	LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE 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	2.08	1.0	0.0	1.0	C	HL	1.0	0.0	1.0	T	HL
1	2	4.16	1.9	0.0	1.9	C	HL	1.9	0.0	1.9	T	HL
1	3	6.24	2.5	0.0	2.5	C	HL	2.5	0.0	2.5	T	HL
1	4	8.32	3.1	0.0	3.1	C	HL	3.1	0.0	3.1	T	HL
1	5	10.41	3.7	0.0	3.7	C	HL	3.7	0.0	3.7	T	HL
1	6	12.49	4.2	0.0	4.2	C	HL	4.2	0.0	4.2	T	HL
1	7	14.57	4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	8	16.65	4.7	0.0	4.7	C	HL	4.7	0.0	4.7	T	HL
1	9	18.73	4.8	0.0	4.8	C	HL	4.8	0.0	4.8	T	HL
1	10	20.81	4.8	0.0	4.8	C	HL	4.8	0.0	4.8	T	HL
1	11	22.89	4.8	0.0	4.8	C	HL	4.8	0.0	4.8	T	HL
1	12	24.97	4.7	0.0	4.7	C	HL	4.7	0.0	4.7	T	HL
1	13	27.06	4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	14	29.14	4.2	0.0	4.2	C	HL	4.2	0.0	4.2	T	HL
1	15	31.22	3.7	0.0	3.7	C	HL	3.7	0.0	3.7	T	HL
1	16	33.30	3.1	0.0	3.1	C	HL	3.1	0.0	3.1	T	HL
1	17	35.38	2.5	0.0	2.5	C	HL	2.5	0.0	2.5	T	HL
1	18	37.46	1.9	0.0	1.9	C	HL	1.9	0.0	1.9	T	HL
1	19	39.54	1.0	0.0	1.0	C	HL	1.0	0.0	1.0	T	HL
1	20	41.62	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 2.2.9.3A=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-3.54	3.54	-1.25	1.25
1	2	4.16	33.	33.	-6.63	6.63	-2.37	2.37
1	3	6.24	33.	33.	-9.30	9.30	-3.36	3.36
1	4	8.32	33.	33.	-11.53	11.53	-4.22	4.22
1	5	10.41	33.	33.	-13.28	13.28	-4.95	4.95
1	6	12.49	33.	33.	-14.69	14.69	-5.55	5.55
1	7	14.57	33.	33.	-15.79	15.79	-6.01	6.01
1	8	16.65	33.	33.	-16.50	16.50	-6.33	6.33
1	9	18.73	33.	33.	-16.70	16.70	-6.53	6.53
1	10	20.81	33.	33.	-16.79	16.79	-6.60	6.60
1	11	22.89	33.	33.	-16.70	16.70	-6.53	6.53
1	12	24.97	33.	33.	-16.50	16.50	-6.33	6.33
1	13	27.06	33.	33.	-15.79	15.79	-6.01	6.01
1	14	29.14	33.	33.	-14.69	14.69	-5.55	5.55
1	15	31.22	33.	33.	-13.28	13.28	-4.95	4.95
1	16	33.30	33.	33.	-11.53	11.53	-4.22	4.22
1	17	35.38	33.	33.	-9.30	9.30	-3.36	3.36
1	18	37.46	33.	33.	-6.63	6.63	-2.37	2.37
1	19	39.54	33.	33.	-3.54	3.54	-1.25	1.25
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=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	MAXIMUM POSITIVE,(ksi) STEEL TOP	MAXIMUM POSITIVE,(ksi) STEEL BOT	MAXIMUM NEGATIVE,(ksi) STEEL TOP	MAXIMUM NEGATIVE,(ksi) STEEL BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-4.22	4.22	-1.25	1.25
1	2	4.16	33.	33.	-7.91	7.91	-2.37	2.37
1	3	6.24	33.	33.	-11.08	11.08	-3.36	3.36
1	4	8.32	33.	33.	-13.72	13.72	-4.22	4.22
1	5	10.41	33.	33.	-15.78	15.78	-4.95	4.95
1	6	12.49	33.	33.	-17.43	17.43	-5.55	5.55
1	7	14.57	33.	33.	-18.72	18.72	-6.01	6.01
1	8	16.65	33.	33.	-19.55	19.55	-6.33	6.33
1	9	18.73	33.	33.	-19.76	19.76	-6.53	6.53
1	10	20.81	33.	33.	-19.85	19.85	-6.60	6.60
1	11	22.89	33.	33.	-19.76	19.76	-6.53	6.53
1	12	24.97	33.	33.	-19.55	19.55	-6.33	6.33
1	13	27.06	33.	33.	-18.72	18.72	-6.01	6.01
1	14	29.14	33.	33.	-17.43	17.43	-5.55	5.55
1	15	31.22	33.	33.	-15.78	15.78	-4.95	4.95
1	16	33.30	33.	33.	-13.72	13.72	-4.22	4.22
1	17	35.38	33.	33.	-11.08	11.08	-3.36	3.36
1	18	37.46	33.	33.	-7.91	7.91	-2.37	2.37
1	19	39.54	33.	33.	-4.22	4.22	-1.25	1.25
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=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	MAXIMUM POSITIVE,(ksi) STEEL TOP	MAXIMUM POSITIVE,(ksi) STEEL BOT	MAXIMUM NEGATIVE,(ksi) STEEL TOP	MAXIMUM NEGATIVE,(ksi) STEEL BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-5.56	5.56	-1.57	1.57
1	2	4.16	33.	33.	-10.42	10.42	-2.97	2.97
1	3	6.24	33.	33.	-14.59	14.59	-4.20	4.20
1	4	8.32	33.	33.	-18.07	18.07	-5.28	5.28
1	5	10.41	33.	33.	-20.76	20.76	-6.19	6.19
1	6	12.49	33.	33.	-22.94	22.94	-6.93	6.93
1	7	14.57	33.	33.	-24.63	24.63	-7.51	7.51
1	8	16.65	33.	33.	-25.71	25.71	-7.92	7.92
1	9	18.73	33.	33.	-25.97	25.97	-8.16	8.16
1	10	20.81	33.	33.	-26.09	26.09	-8.24	8.24
1	11	22.89	33.	33.	-25.97	25.97	-8.16	8.16
1	12	24.97	33.	33.	-25.71	25.71	-7.92	7.92
1	13	27.06	33.	33.	-24.63	24.63	-7.51	7.51
1	14	29.14	33.	33.	-22.94	22.94	-6.93	6.93
1	15	31.22	33.	33.	-20.76	20.76	-6.19	6.19
1	16	33.30	33.	33.	-18.07	18.07	-5.28	5.28
1	17	35.38	33.	33.	-14.59	14.59	-4.20	4.20
1	18	37.46	33.	33.	-10.42	10.42	-2.97	2.97
1	19	39.54	33.	33.	-5.56	5.56	-1.57	1.57
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=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	MAXIMUM POSITIVE,(ksi) STEEL TOP	MAXIMUM POSITIVE,(ksi) STEEL BOT	MAXIMUM NEGATIVE,(ksi) STEEL TOP	MAXIMUM NEGATIVE,(ksi) STEEL BOT

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi) TOP	YIELD STRESS Fy (ksi) BOT	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-5.12	5.12	-1.13	1.13
1	2	4.16	33.	33.	-9.59	9.59	-2.14	2.14
1	3	6.24	33.	33.	-13.41	13.41	-3.03	3.03
1	4	8.32	33.	33.	-16.59	16.59	-3.80	3.80
1	5	10.41	33.	33.	-19.03	19.03	-4.45	4.45
1	6	12.49	33.	33.	-20.99	20.99	-4.99	4.99
1	7	14.57	33.	33.	-22.53	22.53	-5.41	5.41
1	8	16.65	33.	33.	-23.49	23.49	-5.70	5.70
1	9	18.73	33.	33.	-23.68	23.68	-5.88	5.88
1	10	20.81	33.	33.	-23.78	23.78	-5.94	5.94
1	11	22.89	33.	33.	-23.68	23.68	-5.88	5.88
1	12	24.97	33.	33.	-23.49	23.49	-5.70	5.70
1	13	27.06	33.	33.	-22.53	22.53	-5.41	5.41
1	14	29.14	33.	33.	-21.00	21.00	-4.99	4.99
1	15	31.22	33.	33.	-19.03	19.03	-4.45	4.45
1	16	33.30	33.	33.	-16.59	16.59	-3.80	3.80
1	17	35.38	33.	33.	-13.41	13.41	-3.03	3.03
1	18	37.46	33.	33.	-9.59	9.59	-2.14	2.14
1	19	39.54	33.	33.	-5.12	5.12	-1.13	1.13
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-4.65	4.65	-1.57	1.57
1	2	4.16	33.	33.	-8.72	8.72	-2.97	2.97
1	3	6.24	33.	33.	-12.22	12.22	-4.20	4.20
1	4	8.32	33.	33.	-15.14	15.14	-5.28	5.28
1	5	10.41	33.	33.	-17.43	17.43	-6.19	6.19
1	6	12.49	33.	33.	-19.28	19.28	-6.93	6.93
1	7	14.57	33.	33.	-20.72	20.72	-7.51	7.51
1	8	16.65	33.	33.	-21.64	21.64	-7.92	7.92
1	9	18.73	33.	33.	-21.90	21.90	-8.16	8.16
1	10	20.81	33.	33.	-22.01	22.01	-8.24	8.24
1	11	22.89	33.	33.	-21.90	21.90	-8.16	8.16
1	12	24.97	33.	33.	-21.64	21.64	-7.92	7.92
1	13	27.06	33.	33.	-20.72	20.72	-7.51	7.51
1	14	29.14	33.	33.	-19.28	19.28	-6.93	6.93
1	15	31.22	33.	33.	-17.43	17.43	-6.19	6.19
1	16	33.30	33.	33.	-15.14	15.14	-5.28	5.28
1	17	35.38	33.	33.	-12.22	12.22	-4.20	4.20
1	18	37.46	33.	33.	-8.72	8.72	-2.97	2.97
1	19	39.54	33.	33.	-4.65	4.65	-1.57	1.57
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=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	MAXIMUM POSITIVE,(ksi) ------STEEL----- TOP	MAXIMUM POSITIVE,(ksi) ------STEEL----- BOT	MAXIMUM NEGATIVE,(ksi) -----STEEL------ TOP	MAXIMUM NEGATIVE,(ksi) -----STEEL------ BOT
1	0	0.00	33.	33.	0.00	0.00	0.00	0.00
1	1	2.08	33.	33.	-4.21	4.21	-1.13	1.13
1	2	4.16	33.	33.	-7.89	7.89	-2.14	2.14
1	3	6.24	33.	33.	-11.04	11.04	-3.03	3.03
1	4	8.32	33.	33.	-13.67	13.67	-3.80	3.80
1	5	10.41	33.	33.	-15.70	15.70	-4.45	4.45
1	6	12.49	33.	33.	-17.34	17.34	-4.99	4.99
1	7	14.57	33.	33.	-18.61	18.61	-5.41	5.41
1	8	16.65	33.	33.	-19.43	19.43	-5.70	5.70
1	9	18.73	33.	33.	-19.61	19.61	-5.88	5.88
1	10	20.81	33.	33.	-19.70	19.70	-5.94	5.94

1	11	22.89	33.	33.	-19.61	19.61	-5.88	5.88
1	12	24.97	33.	33.	-19.43	19.43	-5.70	5.70
1	13	27.06	33.	33.	-18.61	18.61	-5.41	5.41
1	14	29.14	33.	33.	-17.34	17.34	-4.99	4.99
1	15	31.22	33.	33.	-15.70	15.70	-4.45	4.45
1	16	33.30	33.	33.	-13.67	13.67	-3.80	3.80
1	17	35.38	33.	33.	-11.04	11.04	-3.03	3.03
1	18	37.46	33.	33.	-7.89	7.89	-2.14	2.14
1	19	39.54	33.	33.	-4.21	4.21	-1.13	1.13
1	20	41.62	33.	33.	0.00	0.00	0.00	0.00

1

TABLE 2.2.22.3=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	41.62	W	12.0	0.9390	34.0	0.6250	12.0	0.9390	0.00	0.0000	0.00	0.0000

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

1

TABLE 2.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 41.6

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	41.62		35.90	[2] does not apply			
				[3]= 35.90	0.07187	> 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 L	FROM 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	34.02	0.625	54.4	150.0	0.00	111.46			0
1	1		2.08	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	2		4.16	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	3		6.24	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	4		8.32	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	5		10.41	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	6		12.49	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	7		14.57	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	8		16.65	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	9		18.73	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	10		20.81	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	11		22.89	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	12		24.97	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	13		27.06	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	14		29.14	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	15		31.22	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	16		33.30	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	17		35.38	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	18		37.46	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	19		39.54	0	1	34.02	0.625	54.4	150.0	0.00	111.46			0
1	20		41.62	0	1	34.02	0.625	54.4	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

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	34.02	0.625	54.44	168.97	2
1	1		2.08	0	1	34.02	0.625	54.44	168.97	2
1	2		4.16	0	1	34.02	0.625	54.44	168.97	2
1	3		6.24	0	1	34.02	0.625	54.44	168.97	2
1	4		8.32	0	1	34.02	0.625	54.44	168.97	2
1	5		10.41	0	1	34.02	0.625	54.44	168.97	2
1	6		12.49	0	1	34.02	0.625	54.44	168.97	2
1	7		14.57	0	1	34.02	0.625	54.44	168.97	2
1	8		16.65	0	1	34.02	0.625	54.44	168.97	2
1	9		18.73	0	1	34.02	0.625	54.44	168.97	2
1	10		20.81	0	1	34.02	0.625	54.44	168.97	2
1	11		22.89	0	1	34.02	0.625	54.44	168.97	2
1	12		24.97	0	1	34.02	0.625	54.44	168.97	2

1	13	27.06	0	1	34.02	0.625	54.44	168.97	2
1	14	29.14	0	1	34.02	0.625	54.44	168.97	2
1	15	31.22	0	1	34.02	0.625	54.44	168.97	2
1	16	33.30	0	1	34.02	0.625	54.44	168.97	2
1	17	35.38	0	1	34.02	0.625	54.44	168.97	2
1	18	37.46	0	1	34.02	0.625	54.44	168.97	2
1	19	39.54	0	1	34.02	0.625	54.44	168.97	2
1	20	41.62	0	1	34.02	0.625	54.44	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

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING
UNIVERSITY OF MARYLAND

MERLIN V 10.4
NONCOMPOSITE
LRF -- 2010
CODE CHECK

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 39\5800\Rating\MD\5

TABLE 2.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D FROM L SUPT (ft)	bf/2tf [1]	bf (in)	[2]	tf (in)	[3]	Iyc/Iyt	FLAG
1 0	0.00	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 1	2.08	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 2	4.16	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 3	6.24	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 4	8.32	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 5	10.41	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 6	12.49	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 7	14.57	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 8	16.65	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 9	18.73	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 10	20.81	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 11	22.89	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 12	24.97	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 13	27.06	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 14	29.14	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 15	31.22	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 16	33.30	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 17	35.38	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 18	37.46	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 19	39.54	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	1.00	0
1 20	41.62	6.39	12.0	5.7	0.939	0.688		
		6.39	12.0	5.7	0.939	0.688	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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MERLIN V 10.4
NONCOMPOSITE
LRF -- 2010
CODE CHECK

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 40\5800\Rating\MD\5

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

---

Note: f0, f2, fmid, f1, and Cb are defined in Art. 6.10.8.2.3

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NONCOMPOSITE
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CODE CHECK

FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 41\5800\Rating\MD\5

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	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	LTB CAT	GOV CAT
1	0		0.00	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	1		2.08	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	2		4.16	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	3		6.24	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	4		8.32	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	5		10.41	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	6		12.49	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	7		14.57	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	8		16.65	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	9		18.73	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	10		20.81	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	11		22.89	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	12		24.97	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	13		27.06	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	14		29.14	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	15		31.22	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	16		33.30	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	17		35.38	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	18		37.46	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	19		39.54	1	6.39	11.26	19.84	0	0.00	7.46	28.03	0	0
1	20		41.62	1	6.39	11.26	19.84	0	0.00	7.46	28.03	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	33.00	33.00
1	1		2.08	1	1.000	1.000	1.00	33.00	33.00	33.00
1	2		4.16	1	1.000	1.000	1.00	33.00	33.00	33.00
1	3		6.24	1	1.000	1.000	1.00	33.00	33.00	33.00
1	4		8.32	1	1.000	1.000	1.00	33.00	33.00	33.00
1	5		10.41	1	1.000	1.000	1.00	33.00	33.00	33.00
1	6		12.49	1	1.000	1.000	1.00	33.00	33.00	33.00
1	7		14.57	1	1.000	1.000	1.00	33.00	33.00	33.00
1	8		16.65	1	1.000	1.000	1.00	33.00	33.00	33.00
1	9		18.73	1	1.000	1.000	1.00	33.00	33.00	33.00
1	10		20.81	1	1.000	1.000	1.00	33.00	33.00	33.00
1	11		22.89	1	1.000	1.000	1.00	33.00	33.00	33.00
1	12		24.97	1	1.000	1.000	1.00	33.00	33.00	33.00
1	13		27.06	1	1.000	1.000	1.00	33.00	33.00	33.00
1	14		29.14	1	1.000	1.000	1.00	33.00	33.00	33.00
1	15		31.22	1	1.000	1.000	1.00	33.00	33.00	33.00
1	16		33.30	1	1.000	1.000	1.00	33.00	33.00	33.00
1	17		35.38	1	1.000	1.000	1.00	33.00	33.00	33.00
1	18		37.46	1	1.000	1.000	1.00	33.00	33.00	33.00
1	19		39.54	1	1.000	1.000	1.00	33.00	33.00	33.00
1	20		41.62	1	1.000	1.000	1.00	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 2.2.22.9=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	2	0			2	
1	1		2.08	1	2	0			2	
1	2		4.16	1	2	0			2	
1	3		6.24	1	2	0			2	
1	4		8.32	1	2	0			2	

1	5	10.41	1	2	0	2
1	6	12.49	1	2	0	2
1	7	14.57	1	2	0	2
1	8	16.65	1	2	0	2
1	9	18.73	1	2	0	2
1	10	20.81	1	2	0	2
1	11	22.89	1	2	0	2
1	12	24.97	1	2	0	2
1	13	27.06	1	2	0	2
1	14	29.14	1	2	0	2
1	15	31.22	1	2	0	2
1	16	33.30	1	2	0	2
1	17	35.38	1	2	0	2
1	18	37.46	1	2	0	2
1	19	39.54	1	2	0	2
1	20	41.62	1	2	0	2

Please read NOTE on the following page

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)

TABLE 2.2.22.10=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		2.08	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	2		4.16	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	3		6.24	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	4		8.32	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	5		10.41	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	6	12.49	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	7	14.57	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	8	16.65	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	9	18.73	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	10	20.81	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	11	22.89	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	12	24.97	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	13	27.06	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	14	29.14	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
1	15	31.22	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	16	33.30	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	17	35.38	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	18	37.46	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	19	39.54	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	20	41.62	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 2.2.22.10A=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		2.08	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	2		4.16	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	3		6.24	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	4		8.32	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	5		10.41	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	6		12.49	0.000	0.000	0.000	0.000	0.000	1

					0.000		0.000		3
1	7	14.57	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	8	16.65	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	9	18.73	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	10	20.81	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	11	22.89	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	12	24.97	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	13	27.06	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	14	29.14	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	15	31.22	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	16	33.30	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	17	35.38	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	18	37.46	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	19	39.54	0.000	0.000	0.000	0.000	0.000		1
					0.000		0.000		3
1	20	41.62	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 2.2.22.14=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	-	0.0	19.8	0.0	33.0	-0--
					-	-	0.0	-	0.0	19.8	0.0	33.0	--0-
1	1		2.08	0	-	-	5.6	-	0.0	19.8	5.6	33.0	-0--
					-	-	5.6	-	0.0	19.8	5.6	33.0	--0-
1	2		4.16	0	-	-	10.4	-	0.0	19.8	10.4	33.0	-0--
					-	-	10.4	-	0.0	19.8	10.4	33.0	--0-
1	3		6.24	0	-	-	14.6	-	0.0	19.8	14.6	33.0	-0--
					-	-	14.6	-	0.0	19.8	14.6	33.0	--0-
1	4		8.32	0	-	-	18.1	-	0.0	19.8	18.1	33.0	-0--
					-	-	18.1	-	0.0	19.8	18.1	33.0	--0-
1	5		10.41	0	-	-	20.8	-	0.0	19.8	20.8	33.0	-0--
					-	-	20.8	-	0.0	19.8	20.8	33.0	--0-
1	6		12.49	0	-	-	22.9	-	0.0	19.8	22.9	33.0	-0--
					-	-	22.9	-	0.0	19.8	22.9	33.0	--0-
1	7		14.57	0	-	-	24.6	-	0.0	19.8	24.6	33.0	-0--
					-	-	24.6	-	0.0	19.8	24.6	33.0	--0-
1	8		16.65	0	-	-	25.7	-	0.0	19.8	25.7	33.0	-0--
					-	-	25.7	-	0.0	19.8	25.7	33.0	--0-
1	9		18.73	0	-	-	26.0	-	0.0	19.8	26.0	33.0	-0--
					-	-	26.0	-	0.0	19.8	26.0	33.0	--0-

1	10	20.81	0	-	-	26.1	-	0.0	19.8	26.1	33.0	-0--
				-	-	26.1	-	0.0	19.8	26.1	33.0	--0-
1	11	22.89	0	-	-	26.0	-	0.0	19.8	26.0	33.0	-0--
				-	-	26.0	-	0.0	19.8	26.0	33.0	--0-
1	12	24.97	0	-	-	25.7	-	0.0	19.8	25.7	33.0	-0--
				-	-	25.7	-	0.0	19.8	25.7	33.0	--0-
1	13	27.06	0	-	-	24.6	-	0.0	19.8	24.6	33.0	-0--
				-	-	24.6	-	0.0	19.8	24.6	33.0	--0-
1	14	29.14	0	-	-	22.9	-	0.0	19.8	22.9	33.0	-0--
				-	-	22.9	-	0.0	19.8	22.9	33.0	--0-
1	15	31.22	0	-	-	20.8	-	0.0	19.8	20.8	33.0	-0--
				-	-	20.8	-	0.0	19.8	20.8	33.0	--0-
1	16	33.30	0	-	-	18.1	-	0.0	19.8	18.1	33.0	-0--
				-	-	18.1	-	0.0	19.8	18.1	33.0	--0-
1	17	35.38	0	-	-	14.6	-	0.0	19.8	14.6	33.0	-0--
				-	-	14.6	-	0.0	19.8	14.6	33.0	--0-
1	18	37.46	0	-	-	10.4	-	0.0	19.8	10.4	33.0	-0--
				-	-	10.4	-	0.0	19.8	10.4	33.0	--0-
1	19	39.54	0	-	-	5.6	-	0.0	19.8	5.6	33.0	-0--
				-	-	5.6	-	0.0	19.8	5.6	33.0	--0-
1	20	41.62	0	-	-	0.0	-	0.0	19.8	0.0	33.0	-0--
				-	-	0.0	-	0.0	19.8	0.0	33.0	--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 2.2.22.14A=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	0.000		4
					0.000	0.000	0.000		4
1	1	2.08	0		0.000	0.169	0.169		4
					0.000	0.169	0.169		4
1	2	4.16	0		0.000	0.316	0.316		4
					0.000	0.316	0.316		4
1	3	6.24	0		0.000	0.442	0.442		4

						0.000	0.442	0.442	4
1	4	8.32	0			0.000	0.547	0.547	4
						0.000	0.547	0.547	4
1	5	10.41	0			0.000	0.629	0.629	4
						0.000	0.629	0.629	4
1	6	12.49	0			0.000	0.695	0.695	4
						0.000	0.695	0.695	4
1	7	14.57	0			0.000	0.746	0.746	4
						0.000	0.746	0.746	4
1	8	16.65	0			0.000	0.779	0.779	4
						0.000	0.779	0.779	4
1	9	18.73	0			0.000	0.787	0.787	4
						0.000	0.787	0.787	4
1	10	20.81	0			0.000	0.791	0.791	4
						0.000	0.791	0.791	4
1	11	22.89	0			0.000	0.787	0.787	4
						0.000	0.787	0.787	4
1	12	24.97	0			0.000	0.779	0.779	4
						0.000	0.779	0.779	4
1	13	27.06	0			0.000	0.746	0.746	4
						0.000	0.746	0.746	4
1	14	29.14	0			0.000	0.695	0.695	4
						0.000	0.695	0.695	4
1	15	31.22	0			0.000	0.629	0.629	4
						0.000	0.629	0.629	4
1	16	33.30	0			0.000	0.547	0.547	4
						0.000	0.547	0.547	4
1	17	35.38	0			0.000	0.442	0.442	4
						0.000	0.442	0.442	4
1	18	37.46	0			0.000	0.316	0.316	4
						0.000	0.316	0.316	4
1	19	39.54	0			0.000	0.169	0.169	4
						0.000	0.169	0.169	4
1	20	41.62	0			0.000	0.000	0.000	4
						0.000	0.000	0.000	4

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

TABLE 2.2.22.15=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	54.4	1.000	407.0	407.0
1	1		2.08	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	2		4.16	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	3		6.24	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0

1	4	8.32	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	5	10.41	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	6	12.49	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	7	14.57	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	8	16.65	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	9	18.73	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	10	20.81	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	11	22.89	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	12	24.97	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	13	27.06	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	14	29.14	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	15	31.22	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	16	33.30	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	17	35.38	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	18	37.46	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	19	39.54	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0
1	20	41.62	33.0	5.0	74.2	92.8	54.4	1.000	407.0	407.0

Please read NOTE on the following page

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRF -- 2010
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FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 50\5800\Rating\MD\5

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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FILE NAME = I:\Projects\CKE410A4_MaineDOT Bridge Load Ratings\DesigPAGE 51\5800\Rating\MD\5

TABLE 2.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
****

SP NO	IN D NO	FROM WEB L (ft)	STABILITY SUPT UNSTIFFENED,[1] ---- D/tw -----	LFD/LRFD MAX. SHEAR <=> (kip)	UNSTIFFENED SHEAR CAPA- CITY, [2]	STRENGTH CATEGORY	OVERALL WEB

1	0	0.00	2	119.5	<	407.0	2	2
1	1	2.08	2	109.7	<	407.0	2	2
1	2	4.16	2	100.5	<	407.0	2	2
1	3	6.24	2	91.4	<	407.0	2	2
1	4	8.32	2	82.4	<	407.0	2	2
1	5	10.41	2	73.3	<	407.0	2	2
1	6	12.49	2	64.4	<	407.0	2	2
1	7	14.57	2	55.7	<	407.0	2	2
1	8	16.65	2	47.3	<	407.0	2	2
1	9	18.73	2	39.9	<	407.0	2	2
1	10	20.81	2	32.7	<	407.0	2	2
1	11	22.89	2	39.9	<	407.0	2	2
1	12	24.97	2	47.3	<	407.0	2	2
1	13	27.06	2	55.7	<	407.0	2	2
1	14	29.14	2	64.4	<	407.0	2	2
1	15	31.22	2	73.3	<	407.0	2	2
1	16	33.30	2	82.4	<	407.0	2	2
1	17	35.38	2	91.4	<	407.0	2	2
1	18	37.46	2	100.5	<	407.0	2	2
1	19	39.54	2	109.7	<	407.0	2	2
1	20	41.62	2	119.5	<	407.0	2	2

Please read NOTE on the following page

1

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 2.2.22.17=TRANSVERSE STIFFENER SPACING
****

SP NO	IN D L NO	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 STIFFENERS SPACING (ft-in)
1	0	0.00	33.	119.47	406.99	0	
1	1	2.08	33.	109.74	406.99	0	
1	2	4.16	33.	100.54	406.99	0	
1	3	6.24	33.	91.41	406.99	0	
1	4	8.32	33.	82.35	406.99	0	
1	5	10.41	33.	73.26	406.99	0	
1	6	12.49	33.	64.42	406.99	0	
1	7	14.57	33.	55.68	406.99	0	
1	8	16.65	33.	47.29	406.99	0	
1	9	18.73	33.	39.94	406.99	0	
1	10	20.81	33.	32.70	406.99	0	
1	11	22.89	33.	39.94	406.99	0	
1	12	24.97	33.	47.29	406.99	0	
1	13	27.06	33.	55.68	406.99	0	
1	14	29.14	33.	64.42	406.99	0	
1	15	31.22	33.	73.26	406.99	0	
1	16	33.30	33.	82.35	406.99	0	
1	17	35.38	33.	91.42	406.99	0	
1	18	37.46	33.	100.55	406.99	0	
1	19	39.54	33.	109.74	406.99	0	
1	20	41.62	33.	119.47	406.99	0	

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TABLE 2.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN D L NO	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	26.4	0.0	26.4	0.0	33.0	000
1	1	2.08	-	4.2	26.4	4.2	26.4	4.2	33.0	000
1	2	4.16	-	7.9	26.4	7.9	26.4	7.9	33.0	000
1	3	6.24	-	11.1	26.4	11.1	26.4	11.1	33.0	000
1	4	8.32	-	13.7	26.4	13.7	26.4	13.7	33.0	000
1	5	10.41	-	15.8	26.4	15.8	26.4	15.8	33.0	000
1	6	12.49	-	17.4	26.4	17.4	26.4	17.4	33.0	000
1	7	14.57	-	18.7	26.4	18.7	26.4	18.7	33.0	000
1	8	16.65	-	19.6	26.4	19.6	26.4	19.6	33.0	000
1	9	18.73	-	19.8	26.4	19.8	26.4	19.8	33.0	000
1	10	20.81	-	19.9	26.4	19.9	26.4	19.9	33.0	000
1	11	22.89	-	19.8	26.4	19.8	26.4	19.8	33.0	000
1	12	24.97	-	19.6	26.4	19.6	26.4	19.6	33.0	000
1	13	27.06	-	18.7	26.4	18.7	26.4	18.7	33.0	000
1	14	29.14	-	17.4	26.4	17.4	26.4	17.4	33.0	000
1	15	31.22	-	15.8	26.4	15.8	26.4	15.8	33.0	000
1	16	33.30	-	13.7	26.4	13.7	26.4	13.7	33.0	000
1	17	35.38	-	11.1	26.4	11.1	26.4	11.1	33.0	000
1	18	37.46	-	7.9	26.4	7.9	26.4	7.9	33.0	000
1	19	39.54	-	4.2	26.4	4.2	26.4	4.2	33.0	000
1	20	41.62	-	0.0	26.4	0.0	26.4	0.0	33.0	000

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = II -- Urban 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		2.08	TR	-1.6	A B B^C^C D E E^	TR	1.6	A B B^C^C D E E^
1	2		4.16	TR	-2.9	A B B^C^C D E E^	TR	2.9	A B B^C^C D E
1	3		6.24	TR	-3.8	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	4		8.32	TR	-4.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	5		10.41	TR	-5.6	A B B^C^C D E E^	TR	5.6	A B B^C^C D
1	6		12.49	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B B^C^C D
1	7		14.57	TR	-6.8	A B B^C^C D E E^	TR	6.8	A B B^C^C D
1	8		16.65	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B B^C^C
1	9		18.73	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B B^C^C
1	10		20.81	TR	-7.2	A B B^C^C D E E^	TR	7.2	A B B^C^C
1	11		22.89	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B B^C^C
1	12		24.97	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B B^C^C
1	13		27.06	TR	-6.8	A B B^C^C D E E^	TR	6.8	A B B^C^C D
1	14		29.14	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B B^C^C D
1	15		31.22	TR	-5.6	A B B^C^C D E E^	TR	5.6	A B B^C^C D
1	16		33.30	TR	-4.7	A B B^C^C D E E^	TR	4.7	A B B^C^C D
1	17		35.38	TR	-3.8	A B B^C^C D E E^	TR	3.8	A B B^C^C D E
1	18		37.46	TR	-2.9	A B B^C^C D E E^	TR	2.9	A B B^C^C D E
1	19		39.54	TR	-1.6	A B B^C^C D E E^	TR	1.6	A B B^C^C D E E^
1	20		41.62	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 2.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = II -- Urban 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		2.08	TR	-0.8	A B B^C^C D E E^	TR	0.8	A B B^C^C D E E^
1	2		4.16	TR	-1.4	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	3		6.24	TR	-1.9	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	4		8.32	TR	-2.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	5		10.41	TR	-2.8	A B B^C^C D E E^	TR	2.8	A B B^C^C D
1	6		12.49	TR	-3.1	A B B^C^C D E E^	TR	3.1	A B B^C^C D
1	7		14.57	TR	-3.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D
1	8		16.65	TR	-3.5	A B B^C^C D E E^	TR	3.5	A B B^C^C
1	9		18.73	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C
1	10		20.81	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C
1	11		22.89	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C
1	12		24.97	TR	-3.5	A B B^C^C D E E^	TR	3.5	A B B^C^C
1	13		27.06	TR	-3.4	A B B^C^C D E E^	TR	3.4	A B B^C^C D
1	14		29.14	TR	-3.1	A B B^C^C D E E^	TR	3.1	A B B^C^C D
1	15		31.22	TR	-2.8	A B B^C^C D E E^	TR	2.8	A B B^C^C D
1	16		33.30	TR	-2.3	A B B^C^C D E E^	TR	2.3	A B B^C^C D E
1	17		35.38	TR	-1.9	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	18		37.46	TR	-1.4	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1	19		39.54	TR	-0.8	A B B^C^C D E E^	TR	0.8	A B B^C^C D E E^
1	20		41.62	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= 2038 & No. of Cycles= 55790250
* 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 1.2.30.1=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 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT

SP NO	IN NO	D L	FROM SUPT (ft) or STRESS	MOMENT CAP. (k-ft)		UNFACTORED DEAD LOAD MOMENT D1; k-ft or ksi	UNFACTORED L+I MAX. MOMENT D2 POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	33.00	TOP	0.00	0.00	0.00	9.99	HL-93	9.99	0
				33.00	BOT	0.00	0.00	0.00	9.99	HL-93	9.99	0
1	1		2.08	33.00	TOP	-1.25	-2.28	0.00	9.99	HL-93	7.87	0
				33.00	BOT	1.25	2.28	0.00	9.99	HL-93	7.87	0
1	2		4.16	33.00	TOP	-2.37	-4.26	0.00	5.22	HL-93	4.03	0
				33.00	BOT	2.37	4.26	0.00	5.22	HL-93	4.03	0
1	3		6.24	33.00	TOP	-3.36	-5.94	0.00	3.59	HL-93	2.77	0
				33.00	BOT	3.36	5.94	0.00	3.59	HL-93	2.77	0
1	4		8.32	33.00	TOP	-4.22	-7.31	0.00	2.81	HL-93	2.17	0
				33.00	BOT	4.22	7.31	0.00	2.81	HL-93	2.17	0
1	5		10.41	33.00	TOP	-4.95	-8.33	0.00	2.38	HL-93	1.84	0
				33.00	BOT	4.95	8.33	0.00	2.38	HL-93	1.84	0
1	6		12.49	33.00	TOP	-5.55	-9.14	0.00	2.11	HL-93	1.63	0
				33.00	BOT	5.55	9.14	0.00	2.11	HL-93	1.63	0
1	7		14.57	33.00	TOP	-6.01	-9.78	0.00	1.93	HL-93	1.49	0
				33.00	BOT	6.01	9.78	0.00	1.93	HL-93	1.49	0
1	8		16.65	33.00	TOP	-6.33	-10.17	0.00	1.83	HL-93	1.41	0
				33.00	BOT	6.33	10.17	0.00	1.83	HL-93	1.41	0
1	9		18.73	33.00	TOP	-6.53	-10.17	0.00	1.81	HL-93	1.39	0
				33.00	BOT	6.53	10.17	0.00	1.81	HL-93	1.39	0
1	10		20.81	33.00	TOP	-6.60	-10.20	0.00	1.80	HL-93	1.39	0
				33.00	BOT	6.60	10.20	0.00	1.80	HL-93	1.39	0
1	11		22.89	33.00	TOP	-6.53	-10.17	0.00	1.81	HL-93	1.39	0
				33.00	BOT	6.53	10.17	0.00	1.81	HL-93	1.39	0
1	12		24.97	33.00	TOP	-6.33	-10.17	0.00	1.83	HL-93	1.41	0
				33.00	BOT	6.33	10.17	0.00	1.83	HL-93	1.41	0
1	13		27.06	33.00	TOP	-6.01	-9.78	0.00	1.93	HL-93	1.49	0
				33.00	BOT	6.01	9.78	0.00	1.93	HL-93	1.49	0
1	14		29.14	33.00	TOP	-5.55	-9.14	0.00	2.11	HL-93	1.63	0
				33.00	BOT	5.55	9.14	0.00	2.11	HL-93	1.63	0
1	15		31.22	33.00	TOP	-4.95	-8.33	0.00	2.38	HL-93	1.84	0
				33.00	BOT	4.95	8.33	0.00	2.38	HL-93	1.84	0
1	16		33.30	33.00	TOP	-4.22	-7.31	0.00	2.81	HL-93	2.17	0

		33.00	BOT	4.22	7.31	0.00	2.81	HL-93	2.17	0
1 17	35.38	33.00	TOP	-3.36	-5.94	0.00	3.59	HL-93	2.77	0
		33.00	BOT	3.36	5.94	0.00	3.59	HL-93	2.77	0
1 18	37.46	33.00	TOP	-2.37	-4.26	0.00	5.22	HL-93	4.03	0
		33.00	BOT	2.37	4.26	0.00	5.22	HL-93	4.03	0
1 19	39.54	33.00	TOP	-1.25	-2.28	0.00	9.99	HL-93	7.87	0
		33.00	BOT	1.25	2.28	0.00	9.99	HL-93	7.87	0
1 20	41.62	33.00	TOP	0.00	0.00	0.00	9.99	HL-93	9.99	0
		33.00	BOT	0.00	0.00	0.00	9.99	HL-93	9.99	0

Please read NOTES on the following page

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 2.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D FROM 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 TYPE	INVENT. RATING FACTOR
1	0	0.00	26.4	0.00 0.00	0.00 0.00	0.00 0.00	9.99	HL-93	7.68
1	1	2.08	26.4	-1.25 1.25	-2.28 2.28	0.00 0.00	9.99	HL-93	7.68
1	2	4.16	26.4	-2.37 2.37	-4.26 4.26	0.00 0.00	5.64	HL-93	4.34
1	3	6.24	26.4	-3.36 3.36	-5.94 5.94	0.00 0.00	3.88	HL-93	2.99
1	4	8.32	26.4	-4.22 4.22	-7.31 7.31	0.00 0.00	3.03	HL-93	2.33
1	5	10.41	26.4	-4.95 4.95	-8.33 8.33	0.00 0.00	2.58	HL-93	1.98
1	6	12.49	26.4	-5.55 5.55	-9.14 9.14	0.00 0.00	2.28	HL-93	1.75
1	7	14.57	26.4	-6.01 6.01	-9.78 9.78	0.00 0.00	2.08	HL-93	1.60
1	8	16.65	26.4	-6.33 6.33	-10.17 10.17	0.00 0.00	1.97	HL-93	1.52
1	9	18.73	26.4	-6.53 6.53	-10.17 10.17	0.00 0.00	1.95	HL-93	1.50
1	10	20.81	26.4	-6.60 6.60	-10.20 10.20	0.00 0.00	1.94	HL-93	1.49
1	11	22.89	26.4	-6.53 6.53	-10.17 10.17	0.00 0.00	1.95	HL-93	1.50
1	12	24.97	26.4	-6.33 6.33	-10.17 10.17	0.00 0.00	1.97	HL-93	1.52
1	13	27.06	26.4	-6.01 6.01	-9.78 9.78	0.00 0.00	2.08	HL-93	1.60
1	14	29.14	26.4	-5.55 5.55	-9.14 9.14	0.00 0.00	2.28	HL-93	1.75
1	15	31.22	26.4	-4.95 4.95	-8.33 8.33	0.00 0.00	2.58	HL-93	1.98
1	16	33.30	26.4	-4.22 4.22	-7.31 7.31	0.00 0.00	3.03	HL-93	2.33

1	17	35.38	26.4	-3.36	3.36	-5.94	5.94	0.00	0.00	3.88	HL-93	2.99
1	18	37.46	26.4	-2.37	2.37	-4.26	4.26	0.00	0.00	5.64	HL-93	4.34
1	19	39.54	26.4	-1.25	1.25	-2.28	2.28	0.00	0.00	9.99	HL-93	7.68
1	20	41.62	26.4	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

TABLE 2.2.32.3A=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	41.62	20.81	STRENGTH L.S.	1.80	1.39	HL-93
1	41.62	20.81	SERVICEBILITY	1.94	1.49	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 1.798
Inventory= 1.387

TABLE 2.2.32.3B=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	41.62	20.81	STRENGTH L.S.	2.21	461.40
1	41.62	20.81	SERVICEBILITY	2.38	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 2.213

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TABLE 2.2.33.1=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 MAX. SHEAR D2	OPERAT. RATING FACTOR [3] POS.	OPERAT. RATING FACTOR [3] NEG.	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	407.0	0.0	26.9	49.0	0.0	HL-93	4.27
1	1	2.08	407.0	0.0	23.9	45.6	-1.2	HL-93	4.65
1	2	4.16	407.0	0.0	21.3	42.3	-2.5	HL-93	5.07
1	3	6.24	407.0	0.0	18.6	38.9	-4.0	HL-93	5.56
1	4	8.32	407.0	0.0	16.0	35.6	-5.9	HL-93	6.14
1	5	10.41	407.0	0.0	13.4	32.3	-8.0	HL-93	6.84
1	6	12.49	407.0	0.0	10.7	29.2	-10.0	HL-93	7.66
1	7	14.57	407.0	0.0	7.9	26.2	-12.1	HL-93	8.63
1	8	16.65	407.0	0.0	5.3	23.3	-14.3	HL-93	9.81
1	9	18.73	407.0	0.0	2.6	20.9	-16.5	HL-93	9.99
1	10	20.81	407.0	0.0	0.0	18.7	-18.7	HL-93	9.99
1	11	22.89	407.0	0.0	-2.6	16.5	-20.9	HL-93	9.99
1	12	24.97	407.0	0.0	-5.3	14.3	-23.3	HL-93	9.81
1	13	27.06	407.0	0.0	-7.9	12.1	-26.2	HL-93	8.63
1	14	29.14	407.0	0.0	-10.7	10.0	-29.2	HL-93	7.66
1	15	31.22	407.0	0.0	-13.4	8.0	-32.3	HL-93	6.84
1	16	33.30	407.0	0.0	-16.0	5.9	-35.6	HL-93	6.14
1	17	35.38	407.0	0.0	-18.6	4.0	-38.9	HL-93	5.56
1	18	37.46	407.0	0.0	-21.3	2.5	-42.3	HL-93	5.07
1	19	39.54	407.0	0.0	-23.9	1.2	-45.6	HL-93	4.65
1	20	41.62	407.0	0.0	-26.9	0.0	-49.0	HL-93	4.27

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TABLE 2.2.33.3A=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	41.62	41.62	STRENGTH L.S.	5.54	4.27	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 5.537
Inventory= 4.271

TABLE 2.2.33.3B=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	41.62	41.62	STRENGTH L.S.	6.99	52.42

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.720 & ADTT = 2398
Legal Load Rating= 6.994

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TABLE 2.2.34.1=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	20.81	2 0.400	-0.29	HL-93	2.16
		3 0.510	-0.37		1.70

NOTE: [1] "-" indicates downward deflection

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

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

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.39	0.0	0.0	156.7		0.0	
1	2	8.78	0.0	0.0	291.8		0.0	
1	3	13.18	0.0	0.0	405.5		0.0	
1	4	17.57	0.0	0.0	497.6		0.0	
1	5	21.96	0.0	0.0	568.3		0.0	
1	6	26.35	0.0	0.0	617.5		0.0	
1	7	30.75	0.0	0.0	642.4		0.0	
1	8	35.14	0.0	0.0	645.9		0.0	
1	9	39.53	0.0	0.0	645.9		0.0	
1	10	43.92	0.0	0.0	642.4		0.0	
1	11	48.31	0.0	0.0	617.5		0.0	
1	12	52.71	0.0	0.0	568.3		0.0	
1	13	57.10	0.0	0.0	497.6		0.0	
1	14	61.49	0.0	0.0	405.5		0.0	
1	15	65.88	0.0	0.0	291.8		0.0	
1	16	70.27	0.0	0.0	156.7		0.0	
1	17	74.67	0.0	0.0	0.0		0.0	

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (UNFACTORED)
****

SP NO	IN D NO L	FROM 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	68.6		0.0	
1	1	4.39	0.0	0.0	64.2		-2.6	
1	2	8.78	0.0	0.0	59.8		-4.9	
1	3	13.18	0.0	0.0	55.4		-7.4	
1	4	17.57	0.0	0.0	51.0		-11.3	
1	5	21.96	0.0	0.0	46.6		-15.7	
1	6	26.35	0.0	0.0	42.2		-20.1	
1	7	30.75	0.0	0.0	37.8		-24.5	
1	8	35.14	0.0	0.0	33.3		-28.9	
1	9	39.53	0.0	0.0	28.9		-33.3	
1	10	43.92	0.0	0.0	24.5		-37.8	
1	11	48.31	0.0	0.0	20.1		-42.2	
1	12	52.71	0.0	0.0	15.7		-46.6	
1	13	57.10	0.0	0.0	11.3		-51.0	
1	14	61.49	0.0	0.0	7.4		-55.4	
1	15	65.88	0.0	0.0	4.9		-59.8	
1	16	70.27	0.0	0.0	2.6		-64.2	
1	17	74.67	0.0	0.0	0.0		-68.6	

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	49.11	0.00	HL-93	68.58	EV2	ST1	41.98	63.61
						ST2	41.98	156.19
						ST4	60.33	73.66
						SE1	49.11	49.11
						SE2	49.11	138.26
2	48.97	0.00	HL-93	68.58	EV2	ST1	41.85	63.43
						ST2	41.85	156.02
						ST4	60.13	73.45
						SE1	48.97	48.97
						SE2	48.97	138.12

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.030 MIN			
			0.000 MAX	LANE		
			0.030 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.045 MIN			
			0.000 MAX	LANE		
			0.045 MIN	LANE		
		2 0.400	-0.395 MAX	EV2		
			0.030 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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 DEAD LOAD 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	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		4.39	2986.6	144.3	56.0	156.7	0.0	12.90	EV2	9.95	0
1	2		8.78	2986.6	269.9	105.0	291.8	0.0	6.35	EV2	4.90	0
1	3		13.18	2986.6	378.0	147.1	405.5	0.0	4.21	EV2	3.25	0
1	4		17.57	4129.4	468.6	182.1	497.6	0.0	4.89	EV2	3.77	0
1	5		21.96	4129.4	540.3	210.1	568.3	0.0	4.11	EV2	3.17	0
1	6		26.35	4129.4	594.2	231.1	617.5	0.0	3.67	EV2	2.83	0
1	7		30.75	4129.4	630.4	245.1	642.4	0.0	3.45	EV2	2.66	0
1	8		35.14	4129.4	648.5	252.1	645.9	0.0	3.40	EV2	2.62	0
1	9		39.53	4129.4	648.1	252.1	645.9	0.0	3.40	EV2	2.62	0
1	10		43.92	4129.4	630.0	245.1	642.4	0.0	3.45	EV2	2.66	0
1	11		48.31	4129.4	594.4	231.1	617.5	0.0	3.67	EV2	2.83	0
1	12		52.71	4129.4	539.8	210.1	568.3	0.0	4.12	EV2	3.18	0
1	13		57.10	4129.4	467.4	182.1	497.6	0.0	4.89	EV2	3.78	0
1	14		61.49	2986.6	377.5	147.1	405.5	0.0	4.21	EV2	3.25	0
1	15		65.88	2986.6	269.6	105.0	291.8	0.0	6.35	EV2	4.90	0
1	16		70.27	2986.6	143.8	56.0	156.7	0.0	12.90	EV2	9.95	0
1	17		74.67	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00 0.00	0.00	0.00	0.00 0.00	99.99		76.92
1	1		4.39	30.4	-3.03 3.44	-0.46	2.08	0.00 0.00	12.96		9.97
1	2		8.78	30.4	-5.67 6.43	-0.85	3.88	0.00 0.00	6.18		4.76
1	3		13.18	30.4	-7.93 9.01	-1.18	5.38	0.00 0.00	3.97		3.06
1	4		17.57	30.4	-8.96 7.30	-1.50	4.34	0.00 0.00	5.32		4.09
1	5		21.96	30.4	-10.33 8.42	-1.71	4.96	0.00 0.00	4.43		3.41
1	6		26.35	30.4	-11.36 9.26	-1.86	5.39	0.00 0.00	3.92		3.02
1	7		30.75	30.4	-12.05 9.82	-1.93	5.61	0.00 0.00	3.67		2.82
1	8		35.14	30.4	-12.40 10.11	-1.94	5.64	0.00 0.00	3.60		2.77
1	9		39.53	30.4	-12.39 10.10	-1.94	5.64	0.00 0.00	3.60		2.77
1	10		43.92	30.4	-12.04 9.82	-1.93	5.61	0.00 0.00	3.67		2.82
1	11		48.31	30.4	-11.36 9.26	-1.86	5.39	0.00 0.00	3.92		3.02
1	12		52.71	30.4	-10.32 8.41	-1.71	4.96	0.00 0.00	4.43		3.41
1	13		57.10	30.4	-8.94 7.29	-1.50	4.34	0.00 0.00	5.32		4.09
1	14		61.49	30.4	-7.92 9.00	-1.18	5.38	0.00 0.00	3.97		3.06
1	15		65.88	30.4	-5.66 6.43	-0.85	3.88	0.00 0.00	6.19		4.76
1	16		70.27	30.4	-3.02 3.43	-0.46	2.08	0.00 0.00	12.97		9.97
1	17		74.67	30.4	0.00 0.00	0.00	0.00	0.00 0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	35.14	STRENGTH L.S.	3.39	873.54
1	74.67	35.14	SERVICEBILITY	2.77	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 2.770

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

SP NO	IN NO	D L	FROM 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	481.9	35.6	13.6	68.6	0.0	4.52	EV2	3.48
1	1		4.39	481.9	30.6	12.0	64.2	-2.6	4.92	EV2	3.80
1	2		8.78	481.9	26.6	10.4	59.8	-4.9	5.38	EV2	4.15
1	3		13.18	481.9	22.6	8.8	55.4	-7.4	5.90	EV2	4.55
1	4		17.57	481.9	18.6	7.2	51.0	-11.3	6.52	EV2	5.03
1	5		21.96	481.9	14.3	5.6	46.6	-15.7	7.26	EV2	5.60
1	6		26.35	481.9	10.3	4.0	42.2	-20.1	8.14	EV2	6.28
1	7		30.75	481.9	6.2	2.4	37.8	-24.5	9.23	EV2	7.12
1	8		35.14	481.9	1.9	0.8	33.3	-28.9	10.63	EV2	8.20
1	9		39.53	481.9	-2.1	-0.8	28.9	-33.3	10.62	EV2	8.19
1	10		43.92	481.9	-6.1	-2.4	24.5	-37.8	9.24	EV2	7.13
1	11		48.31	481.9	-10.1	-4.0	20.1	-42.2	8.15	EV2	6.28
1	12		52.71	481.9	-14.5	-5.6	15.7	-46.6	7.25	EV2	5.60
1	13		57.10	481.9	-18.5	-7.2	11.3	-51.0	6.52	EV2	5.03
1	14		61.49	481.9	-22.5	-8.8	7.4	-55.4	5.90	EV2	4.56
1	15		65.88	481.9	-26.7	-10.4	4.9	-59.8	5.38	EV2	4.15
1	16		70.27	481.9	-30.7	-12.0	2.6	-64.2	4.92	EV2	3.80
1	17		74.67	481.9	-35.4	-13.6	0.0	-68.6	4.52	EV2	3.49

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	0.00	STRENGTH L.S.	4.51	92.75

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 4.509

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	3.73	0.0	0.0	134.5		0.0	
1	2	7.47	0.0	0.0	253.5		0.0	
1	3	11.20	0.0	0.0	357.0		0.0	
1	4	14.93	0.0	0.0	444.9		0.0	
1	5	18.67	0.0	0.0	517.3		0.0	
1	6	22.40	0.0	0.0	574.2		0.0	
1	7	26.13	0.0	0.0	615.5		0.0	
1	8	29.87	0.0	0.0	641.3		0.0	
1	9	33.60	0.0	0.0	645.1		0.0	
1	10	37.33	0.0	0.0	646.3		0.0	
1	11	41.07	0.0	0.0	645.1		0.0	
1	12	44.80	0.0	0.0	641.3		0.0	
1	13	48.53	0.0	0.0	615.5		0.0	
1	14	52.27	0.0	0.0	574.2		0.0	
1	15	56.00	0.0	0.0	517.3		0.0	
1	16	59.73	0.0	0.0	444.9		0.0	
1	17	63.47	0.0	0.0	357.0		0.0	
1	18	67.20	0.0	0.0	253.5		0.0	
1	19	70.93	0.0	0.0	134.5		0.0	
1	20	74.67	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	68.6		0.0	
1	1	3.73	0.0	0.0	64.8		-2.2	
1	2	7.47	0.0	0.0	61.1		-4.4	
1	3	11.20	0.0	0.0	57.4		-6.2	
1	4	14.93	0.0	0.0	53.6		-8.7	
1	5	18.67	0.0	0.0	49.9		-12.4	
1	6	22.40	0.0	0.0	46.1		-16.2	
1	7	26.13	0.0	0.0	42.4		-19.9	
1	8	29.87	0.0	0.0	38.6		-23.7	
1	9	33.60	0.0	0.0	34.9		-27.4	
1	10	37.33	0.0	0.0	31.1		-31.1	
1	11	41.07	0.0	0.0	27.4		-34.9	
1	12	44.80	0.0	0.0	23.7		-38.6	
1	13	48.53	0.0	0.0	19.9		-42.4	
1	14	52.27	0.0	0.0	16.2		-46.1	
1	15	56.00	0.0	0.0	12.4		-49.9	
1	16	59.73	0.0	0.0	8.7		-53.6	
1	17	63.47	0.0	0.0	6.2		-57.4	
1	18	67.20	0.0	0.0	4.4		-61.1	
1	19	70.93	0.0	0.0	2.2		-64.8	
1	20	74.67	0.0	0.0	0.0		-68.6	

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	49.11	0.00	HL-93	68.58	EV2	ST1	41.98	63.61
						ST2	41.98	156.19
						ST4	60.33	73.66
						SE1	49.11	49.11
						SE2	49.11	138.26
2	48.97	0.00	HL-93	68.58	EV2	ST1	41.85	63.43
						ST2	41.85	156.02
						ST4	60.13	73.45
						SE1	48.97	48.97
						SE2	48.97	138.12

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.030 MIN			
			0.000 MAX	LANE		
			0.030 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.045 MIN			
			0.000 MAX	LANE		
			0.045 MIN	LANE		
		2 0.400	-0.395 MAX	EV2		
			0.030 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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	D2 ksi	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	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		3.73	2986.6	123.9	48.1	134.5	0.0	15.22	EV2	11.74	0
1	2		7.47	2986.6	234.1	91.1	253.5	0.0	7.50	EV2	5.78	0
1	3		11.20	2986.6	331.6	129.0	357.0	0.0	4.96	EV2	3.83	0
1	4		14.93	4129.4	416.4	161.9	444.9	0.0	5.63	EV2	4.34	0
1	5		18.67	4129.4	488.4	189.7	517.3	0.0	4.65	EV2	3.59	0
1	6		22.40	4129.4	546.5	212.5	574.2	0.0	4.06	EV2	3.13	0
1	7		26.13	4129.4	591.9	230.2	615.5	0.0	3.69	EV2	2.84	0
1	8		29.87	4129.4	624.6	242.9	641.3	0.0	3.47	EV2	2.68	0
1	9		33.60	4129.4	644.5	250.4	645.1	0.0	3.41	EV2	2.63	0
1	10		37.33	4129.4	650.5	253.0	646.3	0.0	3.39	EV2	2.62	0
1	11		41.07	4129.4	643.7	250.4	645.1	0.0	3.41	EV2	2.63	0
1	12		44.80	4129.4	624.3	242.9	641.3	0.0	3.47	EV2	2.68	0
1	13		48.53	4129.4	592.1	230.2	615.5	0.0	3.69	EV2	2.84	0
1	14		52.27	4129.4	546.0	212.5	574.2	0.0	4.06	EV2	3.13	0
1	15		56.00	4129.4	487.2	189.7	517.3	0.0	4.66	EV2	3.59	0
1	16		59.73	4129.4	415.6	161.9	444.9	0.0	5.63	EV2	4.34	0
1	17		63.47	2986.6	331.3	129.0	357.0	0.0	4.96	EV2	3.83	0
1	18		67.20	2986.6	233.7	91.1	253.5	0.0	7.50	EV2	5.78	0
1	19		70.93	2986.6	123.4	48.1	134.5	0.0	15.22	EV2	11.74	0
1	20		74.67	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		3.73	30.4	-2.60	2.95	-0.39	1.79	0.00	0.00	15.37	11.82
1	2		7.47	30.4	-4.91	5.58	-0.74	3.37	0.00	0.00	7.37	5.67
1	3		11.20	30.4	-6.96	7.90	-1.04	4.74	0.00	0.00	4.75	3.65
1	4		14.93	30.4	-7.96	6.49	-1.34	3.88	0.00	0.00	6.16	4.74
1	5		18.67	30.4	-9.34	7.61	-1.56	4.51	0.00	0.00	5.05	3.88
1	6		22.40	30.4	-10.45	8.52	-1.73	5.01	0.00	0.00	4.37	3.36
1	7		26.13	30.4	-11.32	9.23	-1.85	5.37	0.00	0.00	3.94	3.03
1	8		29.87	30.4	-11.94	9.73	-1.93	5.60	0.00	0.00	3.69	2.84
1	9		33.60	30.4	-12.32	10.04	-1.94	5.63	0.00	0.00	3.62	2.78
1	10		37.33	30.4	-12.44	10.14	-1.94	5.64	0.00	0.00	3.59	2.76
1	11		41.07	30.4	-12.31	10.03	-1.94	5.63	0.00	0.00	3.62	2.78
1	12		44.80	30.4	-11.93	9.73	-1.93	5.60	0.00	0.00	3.69	2.84
1	13		48.53	30.4	-11.32	9.23	-1.85	5.37	0.00	0.00	3.94	3.03
1	14		52.27	30.4	-10.44	8.51	-1.73	5.01	0.00	0.00	4.37	3.36
1	15		56.00	30.4	-9.31	7.60	-1.56	4.51	0.00	0.00	5.05	3.89
1	16		59.73	30.4	-7.95	6.48	-1.34	3.88	0.00	0.00	6.16	4.74
1	17		63.47	30.4	-6.95	7.90	-1.04	4.74	0.00	0.00	4.75	3.65
1	18		67.20	30.4	-4.91	5.57	-0.74	3.37	0.00	0.00	7.37	5.67
1	19		70.93	30.4	-2.59	2.94	-0.39	1.79	0.00	0.00	15.37	11.82
1	20		74.67	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	37.33	STRENGTH L.S.	3.38	874.13
1	74.67	37.33	SERVICEBILITY	2.76	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 2.763

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

SP NO	IN NO	D L	FROM 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	481.9	35.6	13.6	68.6	0.0	4.52	EV2	3.48
1	1		3.73	481.9	31.2	12.2	64.8	-2.2	4.86	EV2	3.75
1	2		7.47	481.9	27.8	10.8	61.1	-4.4	5.23	EV2	4.04
1	3		11.20	481.9	24.4	9.5	57.4	-6.2	5.66	EV2	4.36
1	4		14.93	481.9	21.0	8.1	53.6	-8.7	6.14	EV2	4.73
1	5		18.67	481.9	17.3	6.8	49.9	-12.4	6.70	EV2	5.17
1	6		22.40	481.9	13.9	5.4	46.1	-16.2	7.34	EV2	5.66
1	7		26.13	481.9	10.5	4.1	42.4	-19.9	8.09	EV2	6.24
1	8		29.87	481.9	7.0	2.7	38.6	-23.7	9.00	EV2	6.94
1	9		33.60	481.9	3.6	1.4	34.9	-27.4	10.09	EV2	7.79
1	10		37.33	481.9	-0.1	0.0	31.1	-31.1	11.46	EV2	8.84
1	11		41.07	481.9	-3.5	-1.4	27.4	-34.9	10.10	EV2	7.79
1	12		44.80	481.9	-6.9	-2.7	23.7	-38.6	9.00	EV2	6.94
1	13		48.53	481.9	-10.3	-4.1	19.9	-42.4	8.10	EV2	6.25
1	14		52.27	481.9	-14.1	-5.4	16.2	-46.1	7.33	EV2	5.66
1	15		56.00	481.9	-17.5	-6.8	12.4	-49.9	6.69	EV2	5.16
1	16		59.73	481.9	-20.9	-8.1	8.7	-53.6	6.14	EV2	4.74
1	17		63.47	481.9	-24.3	-9.5	6.2	-57.4	5.66	EV2	4.37
1	18		67.20	481.9	-27.9	-10.8	4.4	-61.1	5.23	EV2	4.04
1	19		70.93	481.9	-31.3	-12.2	2.2	-64.8	4.86	EV2	3.75
1	20		74.67	481.9	-35.4	-13.6	0.0	-68.6	4.52	EV2	3.49

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	0.00	STRENGTH L.S.	4.51	92.75

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 4.509

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	75.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.67	0.0	0.0	142.0		0.0	
1	2	9.33	0.0	0.0	263.1		0.0	
1	3	14.00	0.0	0.0	363.5		0.0	
1	4	18.67	0.0	0.0	443.1		0.0	
1	5	23.33	0.0	0.0	502.0		0.0	
1	6	28.00	0.0	0.0	540.0		0.0	
1	7	32.67	0.0	0.0	552.0		0.0	
1	8	37.33	0.0	0.0	553.7		0.0	
1	9	42.00	0.0	0.0	552.0		0.0	
1	10	46.67	0.0	0.0	540.0		0.0	
1	11	51.33	0.0	0.0	502.0		0.0	
1	12	56.00	0.0	0.0	443.1		0.0	
1	13	60.67	0.0	0.0	363.5		0.0	
1	14	65.33	0.0	0.0	263.1		0.0	
1	15	70.00	0.0	0.0	142.0		0.0	
1	16	74.67	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	49.3		0.0	
1	1	4.67	0.0	0.0	46.0		-2.0	
1	2	9.33	0.0	0.0	42.6		-3.7	
1	3	14.00	0.0	0.0	39.2		-5.8	
1	4	18.67	0.0	0.0	35.9		-8.9	
1	5	23.33	0.0	0.0	32.5		-12.3	
1	6	28.00	0.0	0.0	29.1		-15.7	
1	7	32.67	0.0	0.0	25.8		-19.0	
1	8	37.33	0.0	0.0	22.4		-22.4	
1	9	42.00	0.0	0.0	19.0		-25.8	
1	10	46.67	0.0	0.0	15.7		-29.1	
1	11	51.33	0.0	0.0	12.3		-32.5	
1	12	56.00	0.0	0.0	8.9		-35.9	
1	13	60.67	0.0	0.0	5.8		-39.2	
1	14	65.33	0.0	0.0	3.7		-42.6	
1	15	70.00	0.0	0.0	2.0		-46.0	
1	16	74.67	0.0	0.0	0.0		-49.3	

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	44.78	0.00	HL-93	49.32	EV2	ST1	38.08	58.20
						ST2	38.08	124.77
						ST4	53.84	67.17
						SE1	44.78	44.78
						SE2	44.78	108.89
2	44.78	0.00	HL-93	49.32	EV2	ST1	38.08	58.20
						ST2	38.08	124.78
						ST4	53.85	67.17
						SE1	44.78	44.78
						SE2	44.78	108.89

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.033 MIN			
			0.000 MAX	LANE		
			0.033 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.050 MIN			
			0.000 MAX	LANE		
			0.050 MIN	LANE		
		2 0.400	-0.437 MAX	EV2		
			0.033 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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RATING

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	D2 ksi	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	2891.9	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		4.67	2891.9	135.2	59.3	142.0	0.0	13.77	EV2	10.62	0
1	2		9.33	2891.9	252.5	110.7	263.1	0.0	6.81	EV2	5.25	0
1	3		14.00	2891.9	352.0	154.2	363.5	0.0	4.55	EV2	3.51	0
1	4		18.67	3756.2	433.0	189.7	443.1	0.0	4.93	EV2	3.80	0
1	5		23.33	3756.2	496.1	217.4	502.0	0.0	4.17	EV2	3.22	0
1	6		28.00	3756.2	541.4	237.2	540.0	0.0	3.76	EV2	2.90	0
1	7		32.67	3756.2	568.4	249.0	552.0	0.0	3.61	EV2	2.79	0
1	8		37.33	3756.2	577.3	253.0	553.7	0.0	3.58	EV2	2.76	0
1	9		42.00	3756.2	568.4	249.0	552.0	0.0	3.61	EV2	2.79	0
1	10		46.67	3756.2	541.4	237.2	540.0	0.0	3.76	EV2	2.90	0
1	11		51.33	3756.2	496.1	217.4	502.0	0.0	4.17	EV2	3.22	0
1	12		56.00	3756.2	433.0	189.7	443.1	0.0	4.93	EV2	3.80	0
1	13		60.67	2891.9	352.0	154.2	363.5	0.0	4.55	EV2	3.51	0
1	14		65.33	2891.9	252.5	110.7	263.1	0.0	6.81	EV2	5.25	0
1	15		70.00	2891.9	135.2	59.3	142.0	0.0	13.77	EV2	10.62	0
1	16		74.67	2891.9	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92
1	1		4.67	30.4	-2.94	3.34	-0.50	1.91	0.00	0.00	14.13		10.87
1	2		9.33	30.4	-5.49	6.23	-0.93	3.55	0.00	0.00	6.81		5.24
1	3		14.00	30.4	-7.65	8.69	-1.28	4.90	0.00	0.00	4.43		3.41
1	4		18.67	30.4	-8.67	7.36	-1.58	4.14	0.00	0.00	5.57		4.28
1	5		23.33	30.4	-9.93	8.43	-1.78	4.69	0.00	0.00	4.69		3.61
1	6		28.00	30.4	-10.84	9.20	-1.92	5.04	0.00	0.00	4.21		3.24
1	7		32.67	30.4	-11.38	9.66	-1.96	5.15	0.00	0.00	4.03		3.10
1	8		37.33	30.4	-11.56	9.81	-1.97	5.17	0.00	0.00	3.98		3.06
1	9		42.00	30.4	-11.38	9.66	-1.96	5.15	0.00	0.00	4.03		3.10
1	10		46.67	30.4	-10.84	9.20	-1.92	5.04	0.00	0.00	4.21		3.24
1	11		51.33	30.4	-9.93	8.43	-1.78	4.69	0.00	0.00	4.69		3.61
1	12		56.00	30.4	-8.67	7.36	-1.58	4.14	0.00	0.00	5.57		4.28
1	13		60.67	30.4	-7.65	8.69	-1.28	4.90	0.00	0.00	4.43		3.41
1	14		65.33	30.4	-5.49	6.23	-0.93	3.55	0.00	0.00	6.81		5.24
1	15		70.00	30.4	-2.94	3.34	-0.50	1.91	0.00	0.00	14.13		10.87
1	16		74.67	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	37.33	STRENGTH L.S.	3.57	748.79
1	74.67	37.33	SERVICEBILITY	3.06	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 3.064

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RATING

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

SP NO	IN NO	D L	FROM 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	481.9	31.2	13.6	49.3	0.0	6.36	EV2	4.91
1	1		4.67	481.9	27.0	11.9	46.0	-2.0	6.95	EV2	5.36
1	2		9.33	481.9	23.2	10.2	42.6	-3.7	7.63	EV2	5.88
1	3		14.00	481.9	19.4	8.5	39.2	-5.8	8.42	EV2	6.49
1	4		18.67	481.9	15.4	6.8	35.9	-8.9	9.36	EV2	7.22
1	5		23.33	481.9	11.6	5.1	32.5	-12.3	10.49	EV2	8.09
1	6		28.00	481.9	7.8	3.4	29.1	-15.7	11.88	EV2	9.17
1	7		32.67	481.9	3.8	1.7	25.8	-19.0	13.65	EV2	10.53
1	8		37.33	481.9	0.0	0.0	22.4	-22.4	15.94	EV2	12.29
1	9		42.00	481.9	-3.8	-1.7	19.0	-25.8	13.65	EV2	10.53
1	10		46.67	481.9	-7.8	-3.4	15.7	-29.1	11.88	EV2	9.17
1	11		51.33	481.9	-11.6	-5.1	12.3	-32.5	10.49	EV2	8.09
1	12		56.00	481.9	-15.4	-6.8	8.9	-35.9	9.36	EV2	7.22
1	13		60.67	481.9	-19.4	-8.5	5.8	-39.2	8.42	EV2	6.49
1	14		65.33	481.9	-23.2	-10.2	3.7	-42.6	7.63	EV2	5.88
1	15		70.00	481.9	-27.0	-11.9	2.0	-46.0	6.95	EV2	5.36
1	16		74.67	481.9	-31.2	-13.6	0.0	-49.3	6.36	EV2	4.91

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	74.67	STRENGTH L.S.	6.35	66.70

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 6.352

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TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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)	REINFORCED CONCRETE SECTION Ix (in**4)
1	PG			33.4	0.8850	16.59	1.5700	16.59	1.5700		
2	PG			33.4	0.8850	16.59	1.5700	15.82	3.0700		

NOTE: [1] maximum allowable section number is 70

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

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user	
EV2		

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	5.71	0.0	0.0	204.7		0.0	
1	2	11.43	0.0	0.0	379.9		0.0	
1	3	17.14	0.0	0.0	525.7		0.0	
1	4	22.85	0.0	0.0	642.0		0.0	
1	5	28.57	0.0	0.0	728.9		0.0	
1	6	34.28	0.0	0.0	786.4		0.0	
1	7	39.99	0.0	0.0	808.4		0.0	
1	8	45.71	0.0	0.0	813.0		0.0	
1	9	51.42	0.0	0.0	808.4		0.0	
1	10	57.14	0.0	0.0	786.4		0.0	
1	11	62.85	0.0	0.0	728.9		0.0	
1	12	68.56	0.0	0.0	642.0		0.0	
1	13	74.28	0.0	0.0	525.7		0.0	
1	14	79.99	0.0	0.0	379.9		0.0	
1	15	85.70	0.0	0.0	204.7		0.0	
1	16	91.42	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	69.0		0.0	
1	1	5.71	0.0	0.0	64.4		-2.6	
1	2	11.43	0.0	0.0	59.8		-5.5	
1	3	17.14	0.0	0.0	55.1		-9.0	
1	4	22.85	0.0	0.0	50.5		-13.4	
1	5	28.57	0.0	0.0	45.9		-18.1	
1	6	34.28	0.0	0.0	41.2		-22.7	
1	7	39.99	0.0	0.0	36.6		-27.3	
1	8	45.71	0.0	0.0	32.0		-32.0	
1	9	51.42	0.0	0.0	27.3		-36.6	
1	10	57.14	0.0	0.0	22.7		-41.2	
1	11	62.85	0.0	0.0	18.1		-45.9	
1	12	68.56	0.0	0.0	13.4		-50.5	
1	13	74.28	0.0	0.0	9.0		-55.1	
1	14	79.99	0.0	0.0	5.5		-59.8	
1	15	85.70	0.0	0.0	2.6		-64.4	
1	16	91.42	0.0	0.0	0.0		-69.0	

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	64.52	0.00	HL-93	69.02	EV2	ST1	55.34	83.36
						ST2	55.34	176.54
						ST4	80.45	96.77
						SE1	64.52	64.52
						SE2	64.52	154.24
2	64.52	0.00	HL-93	69.02	EV2	ST1	55.34	83.36
						ST2	55.34	176.54
						ST4	80.45	96.77
						SE1	64.52	64.52
						SE2	64.52	154.24

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.37	0.00000
			0.034 MIN			
			0.000 MAX	LANE		
			0.034 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.37	0.00000
			0.051 MIN			
			0.000 MAX	LANE		
			0.051 MIN	LANE		
		2 0.400	-0.554 MAX	EV2		
			0.034 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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 DEAD LOAD 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	4033.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		5.71	4033.3	253.3	88.9	204.7	0.0	13.00	EV2	10.03	0
1	2		11.43	4033.3	473.2	165.9	379.9	0.0	6.25	EV2	4.82	0
1	3		17.14	4033.3	658.8	231.1	525.7	0.0	4.06	EV2	3.13	0
1	4		22.85	5399.2	810.5	284.4	642.0	0.0	4.60	EV2	3.55	0
1	5		28.57	5399.2	929.3	325.9	728.9	0.0	3.84	EV2	2.96	0
1	6		34.28	5399.2	1013.3	355.5	786.4	0.0	3.42	EV2	2.64	0
1	7		39.99	5399.2	1063.9	373.3	808.4	0.0	3.25	EV2	2.50	0
1	8		45.71	5399.2	1081.4	379.2	813.0	0.0	3.20	EV2	2.47	0
1	9		51.42	5399.2	1063.9	373.3	808.4	0.0	3.25	EV2	2.50	0
1	10		57.14	5399.2	1013.3	355.5	786.4	0.0	3.42	EV2	2.64	0
1	11		62.85	5399.2	929.3	325.9	728.9	0.0	3.84	EV2	2.96	0
1	12		68.56	5399.2	810.5	284.4	642.0	0.0	4.60	EV2	3.55	0
1	13		74.28	4033.3	658.8	231.1	525.7	0.0	4.06	EV2	3.13	0
1	14		79.99	4033.3	473.2	165.9	379.9	0.0	6.25	EV2	4.82	0
1	15		85.70	4033.3	253.3	88.9	204.7	0.0	13.00	EV2	10.03	0
1	16		91.42	4033.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92
1	1		5.71	30.4	-3.53	3.88	-0.60	1.91	0.00	0.00	13.91		10.70
1	2		11.43	30.4	-6.58	7.25	-1.11	3.54	0.00	0.00	6.54		5.03
1	3		17.14	30.4	-9.17	10.10	-1.54	4.90	0.00	0.00	4.15		3.19
1	4		22.85	30.4	-10.34	7.91	-1.87	3.79	0.00	0.00	5.93		4.57
1	5		28.57	30.4	-11.86	9.07	-2.12	4.30	0.00	0.00	4.96		3.81
1	6		34.28	30.4	-12.93	9.89	-2.29	4.64	0.00	0.00	4.42		3.40
1	7		39.99	30.4	-13.58	10.38	-2.35	4.77	0.00	0.00	4.20		3.23
1	8		45.71	30.4	-13.80	10.55	-2.37	4.80	0.00	0.00	4.14		3.18
1	9		51.42	30.4	-13.58	10.38	-2.35	4.77	0.00	0.00	4.20		3.23
1	10		57.14	30.4	-12.93	9.89	-2.29	4.64	0.00	0.00	4.42		3.40
1	11		62.85	30.4	-11.86	9.07	-2.12	4.30	0.00	0.00	4.96		3.81
1	12		68.56	30.4	-10.34	7.91	-1.87	3.79	0.00	0.00	5.93		4.57
1	13		74.28	30.4	-9.17	10.10	-1.54	4.90	0.00	0.00	4.15		3.19
1	14		79.99	30.4	-6.58	7.25	-1.11	3.54	0.00	0.00	6.54		5.03
1	15		85.70	30.4	-3.53	3.88	-0.60	1.91	0.00	0.00	13.91		10.70
1	16		91.42	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	91.42	45.71	STRENGTH L.S.	3.19	1099.53
1	91.42	45.71	SERVICEBILITY	3.18	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 3.181

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

SP NO	IN NO	D L	FROM 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	548.0	47.9	16.6	69.0	0.0	4.99	EV2	3.85
1	1		5.71	548.0	41.4	14.5	64.4	-2.6	5.47	EV2	4.22
1	2		11.43	548.0	35.6	12.4	59.8	-5.5	6.02	EV2	4.65
1	3		17.14	548.0	29.5	10.4	55.1	-9.0	6.67	EV2	5.15
1	4		22.85	548.0	23.7	8.3	50.5	-13.4	7.43	EV2	5.73
1	5		28.57	548.0	17.9	6.2	45.9	-18.1	8.35	EV2	6.44
1	6		34.28	548.0	11.7	4.1	41.2	-22.7	9.48	EV2	7.31
1	7		39.99	548.0	6.0	2.1	36.6	-27.3	10.88	EV2	8.39
1	8		45.71	548.0	0.2	0.0	32.0	-32.0	12.69	EV2	9.79
1	9		51.42	548.0	-6.0	-2.1	27.3	-36.6	10.88	EV2	8.39
1	10		57.14	548.0	-11.7	-4.1	22.7	-41.2	9.48	EV2	7.31
1	11		62.85	548.0	-17.9	-6.2	18.1	-45.9	8.35	EV2	6.44
1	12		68.56	548.0	-23.7	-8.3	13.4	-50.5	7.43	EV2	5.73
1	13		74.28	548.0	-29.5	-10.4	9.0	-55.1	6.67	EV2	5.15
1	14		79.99	548.0	-35.6	-12.4	5.5	-59.8	6.02	EV2	4.65
1	15		85.70	548.0	-41.4	-14.5	2.6	-64.4	5.47	EV2	4.22
1	16		91.42	548.0	-47.9	-16.6	0.0	-69.0	4.99	EV2	3.85

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	91.42	91.42	STRENGTH L.S.	4.98	93.35

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 4.977

TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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)	REINFORCED CONCRETE SECTION Ix (in**4)
1	PG			33.4	0.8850	16.59	1.5700	16.59	1.5700		
2	PG			33.4	0.8850	16.59	1.5700	15.91	2.7580		

NOTE: [1] maximum allowable section number is 70

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

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	75.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.57	0.0	0.0	145.1		0.0	
1	2	9.14	0.0	0.0	273.8		0.0	
1	3	13.71	0.0	0.0	386.0		0.0	
1	4	18.28	0.0	0.0	481.7		0.0	
1	5	22.85	0.0	0.0	561.0		0.0	
1	6	27.43	0.0	0.0	623.8		0.0	
1	7	32.00	0.0	0.0	670.2		0.0	
1	8	36.57	0.0	0.0	700.1		0.0	
1	9	41.14	0.0	0.0	707.8		0.0	
1	10	45.71	0.0	0.0	710.4		0.0	
1	11	50.28	0.0	0.0	707.8		0.0	
1	12	54.85	0.0	0.0	700.1		0.0	
1	13	59.42	0.0	0.0	670.2		0.0	
1	14	63.99	0.0	0.0	623.8		0.0	
1	15	68.56	0.0	0.0	561.0		0.0	
1	16	73.13	0.0	0.0	481.7		0.0	
1	17	77.70	0.0	0.0	386.0		0.0	
1	18	82.28	0.0	0.0	273.8		0.0	
1	19	86.85	0.0	0.0	145.1		0.0	
1	20	91.42	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	49.6		0.0	
1	1	4.57	0.0	0.0	46.9		-1.4	
1	2	9.14	0.0	0.0	44.3		-3.1	
1	3	13.71	0.0	0.0	41.6		-4.9	
1	4	18.28	0.0	0.0	38.9		-7.0	
1	5	22.85	0.0	0.0	36.3		-9.7	
1	6	27.43	0.0	0.0	33.6		-12.3	
1	7	32.00	0.0	0.0	30.9		-15.0	
1	8	36.57	0.0	0.0	28.3		-17.6	
1	9	41.14	0.0	0.0	25.6		-20.3	
1	10	45.71	0.0	0.0	23.0		-23.0	
1	11	50.28	0.0	0.0	20.3		-25.6	
1	12	54.85	0.0	0.0	17.6		-28.3	
1	13	59.42	0.0	0.0	15.0		-30.9	
1	14	63.99	0.0	0.0	12.3		-33.6	
1	15	68.56	0.0	0.0	9.7		-36.3	
1	16	73.13	0.0	0.0	7.0		-38.9	
1	17	77.70	0.0	0.0	4.9		-41.6	
1	18	82.28	0.0	0.0	3.1		-44.3	
1	19	86.85	0.0	0.0	1.4		-46.9	
1	20	91.42	0.0	0.0	0.0		-49.6	

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	58.11	0.00	HL-93	49.58	EV2	ST1	49.58	75.36
						ST2	49.58	142.29
						ST4	70.85	87.17
						SE1	58.11	58.11
						SE2	58.11	122.56
2	58.24	0.00	HL-93	49.58	EV2	ST1	49.70	75.52
						ST2	49.70	142.45
						ST4	71.05	87.36
						SE1	58.24	58.24
						SE2	58.24	122.69

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.37	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.37	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.624 MAX	EV2		
			0.038 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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 DEAD LOAD MOMENT D2 ksi	UNFACTORED L+I MAX. MOMENT POS. MAX. STRESS	UNFACTORED L+I MAX. MOMENT NEG. MAX. STRESS	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	3935.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		4.57	3935.3	178.9	72.0	145.1	0.0	18.42	EV2	14.21	0
1	2		9.14	3935.3	339.0	136.5	273.8	0.0	8.98	EV2	6.93	0
1	3		13.71	3935.3	480.5	193.4	386.0	0.0	5.87	EV2	4.53	0
1	4		18.28	3935.3	603.3	242.7	481.7	0.0	4.36	EV2	3.37	0
1	5		22.85	4775.4	707.1	284.4	561.0	0.0	4.61	EV2	3.55	0
1	6		27.43	4775.4	791.9	318.5	623.8	0.0	3.96	EV2	3.06	0
1	7		32.00	4775.4	858.0	345.1	670.2	0.0	3.55	EV2	2.74	0
1	8		36.57	4775.4	905.4	364.0	700.1	0.0	3.31	EV2	2.55	0
1	9		41.14	4775.4	933.5	375.4	707.8	0.0	3.22	EV2	2.48	0
1	10		45.71	4775.4	942.9	379.2	710.4	0.0	3.19	EV2	2.46	0
1	11		50.28	4775.4	933.7	375.4	707.8	0.0	3.22	EV2	2.48	0
1	12		54.85	4775.4	905.4	364.0	700.1	0.0	3.31	EV2	2.55	0
1	13		59.42	4775.4	858.2	345.1	670.2	0.0	3.55	EV2	2.74	0
1	14		63.99	4775.4	792.3	318.5	623.8	0.0	3.96	EV2	3.05	0
1	15		68.56	4775.4	707.7	284.4	561.0	0.0	4.61	EV2	3.55	0
1	16		73.13	3935.3	603.7	242.7	481.7	0.0	4.36	EV2	3.37	0
1	17		77.70	3935.3	481.1	193.4	386.0	0.0	5.87	EV2	4.53	0
1	18		82.28	3935.3	339.8	136.5	273.8	0.0	8.98	EV2	6.92	0
1	19		86.85	3935.3	179.5	72.0	145.1	0.0	18.42	EV2	14.21	0
1	20		91.42	3935.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00	99.99		76.92
1	1		4.57	30.4	-2.59 2.85	-0.49 1.37	0.00 0.00	0.00	20.08		15.44
1	2		9.14	30.4	-4.90 5.40	-0.93 2.59	0.00 0.00	0.00	9.66		7.43
1	3		13.71	30.4	-6.95 7.65	-1.32 3.65	0.00 0.00	0.00	6.23		4.79
1	4		18.28	30.4	-8.72 9.61	-1.64 4.55	0.00 0.00	0.00	4.57		3.51
1	5		22.85	30.4	-9.51 7.73	-1.89 3.63	0.00 0.00	0.00	6.24		4.80
1	6		27.43	30.4	-10.65 8.65	-2.10 4.04	0.00 0.00	0.00	5.38		4.14
1	7		32.00	30.4	-11.54 9.38	-2.25 4.34	0.00 0.00	0.00	4.84		3.73
1	8		36.57	30.4	-12.17 9.89	-2.35 4.53	0.00 0.00	0.00	4.52		3.48
1	9		41.14	30.4	-12.55 10.20	-2.38 4.59	0.00 0.00	0.00	4.41		3.39
1	10		45.71	30.4	-12.68 10.30	-2.39 4.60	0.00 0.00	0.00	4.37		3.36
1	11		50.28	30.4	-12.55 10.20	-2.38 4.59	0.00 0.00	0.00	4.40		3.39
1	12		54.85	30.4	-12.17 9.89	-2.35 4.53	0.00 0.00	0.00	4.52		3.48
1	13		59.42	30.4	-11.54 9.38	-2.25 4.34	0.00 0.00	0.00	4.84		3.73
1	14		63.99	30.4	-10.65 8.66	-2.10 4.04	0.00 0.00	0.00	5.38		4.14
1	15		68.56	30.4	-9.51 7.73	-1.89 3.63	0.00 0.00	0.00	6.24		4.80
1	16		73.13	30.4	-8.73 9.61	-1.64 4.55	0.00 0.00	0.00	4.56		3.51
1	17		77.70	30.4	-6.95 7.66	-1.32 3.65	0.00 0.00	0.00	6.23		4.79
1	18		82.28	30.4	-4.91 5.41	-0.93 2.59	0.00 0.00	0.00	9.65		7.43
1	19		86.85	30.4	-2.59 2.86	-0.49 1.37	0.00 0.00	0.00	20.07		15.44
1	20		91.42	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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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MERLIN V10.10
COMPOSITE
LRFD- 8TH ED
RATING

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	91.42	45.71	STRENGTH L.S.	3.19	960.82
1	91.42	45.71	SERVICEBILITY	3.36	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 3.185

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MERLIN V10.10
COMPOSITE
LRFD- 8TH ED
RATING

FILE NAME = C:\Users\jtzoc\Desktop\TEMP\5800 Ext G S2Rev_ev2.dat

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

SP NO	IN NO	D L	FROM 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	548.0	41.5	16.6	49.6	0.0	7.06	EV2	5.45
1	1		4.57	548.0	37.1	14.9	46.9	-1.4	7.59	EV2	5.85
1	2		9.14	548.0	33.0	13.3	44.3	-3.1	8.17	EV2	6.30
1	3		13.71	548.0	28.9	11.6	41.6	-4.9	8.82	EV2	6.81
1	4		18.28	548.0	24.8	10.0	38.9	-7.0	9.57	EV2	7.38
1	5		22.85	548.0	20.6	8.3	36.3	-9.7	10.43	EV2	8.04
1	6		27.43	548.0	16.5	6.6	33.6	-12.3	11.42	EV2	8.81
1	7		32.00	548.0	12.4	5.0	30.9	-15.0	12.58	EV2	9.70
1	8		36.57	548.0	8.2	3.3	28.3	-17.6	13.96	EV2	10.77
1	9		41.14	548.0	4.1	1.7	25.6	-20.3	15.62	EV2	12.05
1	10		45.71	548.0	0.0	0.0	23.0	-23.0	17.68	EV2	13.64
1	11		50.28	548.0	-4.1	-1.7	20.3	-25.6	15.63	EV2	12.05
1	12		54.85	548.0	-8.3	-3.3	17.6	-28.3	13.96	EV2	10.77
1	13		59.42	548.0	-12.4	-5.0	15.0	-30.9	12.58	EV2	9.70
1	14		63.99	548.0	-16.5	-6.6	12.3	-33.6	11.42	EV2	8.81
1	15		68.56	548.0	-20.7	-8.3	9.7	-36.3	10.42	EV2	8.04
1	16		73.13	548.0	-24.8	-10.0	7.0	-38.9	9.57	EV2	7.38
1	17		77.70	548.0	-28.9	-11.6	4.9	-41.6	8.82	EV2	6.81
1	18		82.28	548.0	-32.9	-13.3	3.1	-44.3	8.17	EV2	6.30
1	19		86.85	548.0	-37.2	-14.9	1.4	-46.9	7.58	EV2	5.85
1	20		91.42	548.0	-41.6	-16.6	0.0	-49.6	7.06	EV2	5.45

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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MERLIN V10.10
COMPOSITE
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RATING

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	91.42	91.42	STRENGTH L.S.	7.05	67.05

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 7.046

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

TABLE 2.1.3.1=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	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.760	0.453
2	0.176	2.132
3	0.352	17.132
4	0.176	32.132
5	0.760	41.172

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE SPACING		AXLE SPACING		AXLE SPACING		AXLE SPACING	
WHT (k)	DIST (ft)	WHT (k)	DIST (ft)	WHT (k)	DIST (ft)	WHT (k)	DIST (ft)
24.00	15.00	33.50	0.00				

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE SPACING		AXLE SPACING		AXLE SPACING		AXLE SPACING	
WHT (k)	DIST (ft)	WHT (k)	DIST (ft)	WHT (k)	DIST (ft)	WHT (k)	DIST (ft)
24.00	15.00	33.50	0.00				

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0		0.0		0.0	0.0
1	1		2.08	49.9	95.4		0.0		145.3	49.9
1	2		4.16	93.9	179.0		0.0		272.9	93.9
1	3		6.24	132.8	250.5		0.0		383.3	132.8
1	4		8.32	166.6	310.2		0.0		476.8	166.6
1	5		10.41	195.3	357.9		0.0		553.1	195.3
1	6		12.49	218.8	393.6		0.0		612.4	218.8
1	7		14.57	237.2	417.5		0.0		654.7	237.2
1	8		16.65	250.5	429.3		0.0		679.9	250.5
1	9		18.73	258.1	420.3		0.0		678.5	258.1
1	10		20.81	260.5	417.3		0.0		677.8	260.5
1	11		22.89	257.7	420.3		0.0		678.0	257.7
1	12		24.97	249.8	429.3		0.0		679.1	249.8
1	13		27.06	236.7	417.5		0.0		654.2	236.7
1	14		29.14	218.5	393.6		0.0		612.2	218.5
1	15		31.22	195.3	357.9		0.0		553.1	195.3
1	16		33.30	166.7	310.2		0.0		476.8	166.7
1	17		35.38	132.8	250.5		0.0		383.3	132.8
1	18		37.46	93.7	179.0		0.0		272.7	93.7
1	19		39.54	49.6	95.4		0.0		145.1	49.6
1	20		41.62	0.0	0.0		0.0		0.0	0.0

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0		0.00	25.8	54.1		0.0		79.9	25.8
1	1		2.08	22.6	50.9		-1.9		73.5	20.7
1	2		4.16	19.9	47.7		-3.7		67.7	16.2
1	3		6.24	17.5	44.6		-5.6		62.0	11.9
1	4		8.32	15.0	41.4		-7.4		56.4	7.6
1	5		10.41	12.5	38.2		-9.1		50.7	3.4
1	6		12.49	10.1	35.0		-11.1		45.1	-1.0
1	7		14.57	7.6	31.8		-13.3		39.4	-5.7
1	8		16.65	5.2	28.6		-15.9		33.8	-10.7
1	9		18.73	2.3	25.4		-19.1		27.8	-16.7
1	10		20.81	-0.1	22.3		-22.3		-22.4	22.1
1	11		22.89	-2.6	19.1		-25.4		-28.0	16.5
1	12		24.97	-5.0	15.9		-28.6		-33.7	10.8
1	13		27.06	-7.5	13.3		-31.8		-39.3	5.8
1	14		29.14	-10.0	11.1		-35.0		-45.0	1.2
1	15		31.22	-12.4	9.1		-38.2		-50.6	-3.3
1	16		33.30	-15.1	7.4		-41.4		-56.4	-7.6
1	17		35.38	-17.5	5.6		-44.6		-62.1	-12.0
1	18		37.46	-20.0	3.7		-47.7		-67.7	-16.3
1	19		39.54	-22.4	1.9		-50.9		-73.4	-20.6
1	20		41.62	-25.7	0.0		-54.1		-79.8	-25.7

TABLE 2.2.7.2=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	25.78	0.00	HL-93	54.11	MAX.V	ST1	23.20	32.23
						ST2	23.20	105.27
						ST4	38.67	38.67
						SE1	25.78	25.78
						SE2	25.78	96.12
2	25.66	0.00	HL-93	54.11	MAX.V	ST1	23.09	32.07
						ST2	23.09	105.12
						ST4	38.49	38.49
						SE1	25.66	25.66
						SE2	25.66	96.00

NOTE: [1] " - " Indicates Uplift

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

TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****
(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	0.00	2 0.400	0.000 MAX	HL-93	0.62	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	0.62	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.252 MAX	EV2		
			0.038 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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.

TABLE 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft) or STRESS	MOMENT CAP. (k-ft)		UNFACTORED DEAD LOAD MOMENT D1; k-ft ksi	UNFACTORED L+I MAX. MOMENT D2 POS.	UNFACTORED L+I MAX. MOMENT NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0
1	1		2.08	33.00	TOP	-1.19	-2.27	0.00	10.26	EV2	7.92	0
				33.00	BOT	1.19	2.27	0.00	10.26	EV2	7.92	0
1	2		4.16	33.00	TOP	-2.24	-4.26	0.00	5.25	EV2	4.05	0
				33.00	BOT	2.24	4.26	0.00	5.25	EV2	4.05	0
1	3		6.24	33.00	TOP	-3.16	-5.97	0.00	3.60	EV2	2.78	0
				33.00	BOT	3.16	5.97	0.00	3.60	EV2	2.78	0
1	4		8.32	33.00	TOP	-3.97	-7.39	0.00	2.81	EV2	2.17	0
				33.00	BOT	3.97	7.39	0.00	2.81	EV2	2.17	0
1	5		10.41	33.00	TOP	-4.65	-8.53	0.00	2.36	EV2	1.82	0
				33.00	BOT	4.65	8.53	0.00	2.36	EV2	1.82	0
1	6		12.49	33.00	TOP	-5.21	-9.38	0.00	2.09	EV2	1.61	0
				33.00	BOT	5.21	9.38	0.00	2.09	EV2	1.61	0
1	7		14.57	33.00	TOP	-5.65	-9.95	0.00	1.93	EV2	1.49	0
				33.00	BOT	5.65	9.95	0.00	1.93	EV2	1.49	0
1	8		16.65	33.00	TOP	-5.97	-10.23	0.00	1.85	EV2	1.43	0
				33.00	BOT	5.97	10.23	0.00	1.85	EV2	1.43	0
1	9		18.73	29.38	TOP	-6.15	-10.01	0.00	1.60	EV2	1.24	0
				33.00	BOT	6.15	10.01	0.00	1.87	EV2	1.44	0
1	10		20.81	29.38	TOP	-6.21	-9.94	0.00	1.61	EV2	1.24	0
				33.00	BOT	6.21	9.94	0.00	1.88	EV2	1.45	0
1	11		22.89	29.38	TOP	-6.14	-10.01	0.00	1.61	EV2	1.24	0
				33.00	BOT	6.14	10.01	0.00	1.87	EV2	1.45	0
1	12		24.97	29.38	TOP	-5.95	-10.23	0.00	1.59	EV2	1.23	0
				33.00	BOT	5.95	10.23	0.00	1.85	EV2	1.43	0
1	13		27.06	29.38	TOP	-5.64	-9.95	0.00	1.66	EV2	1.28	0
				33.00	BOT	5.64	9.95	0.00	1.93	EV2	1.49	0
1	14		29.14	29.38	TOP	-5.21	-9.38	0.00	1.81	EV2	1.39	0
				33.00	BOT	5.21	9.38	0.00	2.09	EV2	1.61	0
1	15		31.22	29.38	TOP	-4.65	-8.53	0.00	2.05	EV2	1.58	0
				33.00	BOT	4.65	8.53	0.00	2.36	EV2	1.82	0
1	16		33.30	33.00	TOP	-3.97	-7.39	0.00	2.81	EV2	2.17	0
				33.00	BOT	3.97	7.39	0.00	2.81	EV2	2.17	0
1	17		35.38	33.00	TOP	-3.16	-5.97	0.00	3.60	EV2	2.78	0
				33.00	BOT	3.16	5.97	0.00	3.60	EV2	2.78	0
1	18		37.46	33.00	TOP	-2.23	-4.26	0.00	5.25	EV2	4.05	0
				33.00	BOT	2.23	4.26	0.00	5.25	EV2	4.05	0
1	19		39.54	33.00	TOP	-1.18	-2.27	0.00	10.27	EV2	7.92	0
				33.00	BOT	1.18	2.27	0.00	10.27	EV2	7.92	0
1	20		41.62	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

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

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TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D L	FROM 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 TYPE	INVENT. RATING FACTOR
1	0		0.00	26.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		2.08	26.4	-1.19	1.19	-2.27	2.27	0.00	0.00	11.09	8.53
1	2		4.16	26.4	-2.24	2.24	-4.26	4.26	0.00	0.00	5.67	4.36
1	3		6.24	26.4	-3.16	3.16	-5.97	5.97	0.00	0.00	3.89	2.99
1	4		8.32	26.4	-3.97	3.97	-7.39	7.39	0.00	0.00	3.04	2.33
1	5		10.41	26.4	-4.65	4.65	-8.53	8.53	0.00	0.00	2.55	1.96
1	6		12.49	26.4	-5.21	5.21	-9.38	9.38	0.00	0.00	2.26	1.74
1	7		14.57	26.4	-5.65	5.65	-9.95	9.95	0.00	0.00	2.09	1.60
1	8		16.65	26.4	-5.97	5.97	-10.23	10.23	0.00	0.00	2.00	1.54
1	9		18.73	26.4	-6.15	6.15	-10.01	10.01	0.00	0.00	2.02	1.56
1	10		20.81	26.4	-6.21	6.21	-9.94	9.94	0.00	0.00	2.03	1.56
1	11		22.89	26.4	-6.14	6.14	-10.01	10.01	0.00	0.00	2.02	1.56
1	12		24.97	26.4	-5.95	5.95	-10.23	10.23	0.00	0.00	2.00	1.54
1	13		27.06	26.4	-5.64	5.64	-9.95	9.95	0.00	0.00	2.09	1.61
1	14		29.14	26.4	-5.21	5.21	-9.38	9.38	0.00	0.00	2.26	1.74
1	15		31.22	26.4	-4.65	4.65	-8.53	8.53	0.00	0.00	2.55	1.96
1	16		33.30	26.4	-3.97	3.97	-7.39	7.39	0.00	0.00	3.03	2.33
1	17		35.38	26.4	-3.16	3.16	-5.97	5.97	0.00	0.00	3.89	2.99
1	18		37.46	26.4	-2.23	2.23	-4.26	4.26	0.00	0.00	5.67	4.36
1	19		39.54	26.4	-1.18	1.18	-2.27	2.27	0.00	0.00	11.09	8.53
1	20		41.62	26.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 2.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	41.62	24.97	STRENGTH L.S.	1.59	580.65
1	41.62	16.65	SERVICEBILITY	1.54	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 1.536

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TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D L	FROM 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	407.0	0.0	25.8	54.1	0.0	5.04	EV2	3.89
1	1		2.08	407.0	0.0	22.6	50.9	-1.9	5.43	EV2	4.19
1	2		4.16	407.0	0.0	19.9	47.7	-3.7	5.85	EV2	4.51
1	3		6.24	407.0	0.0	17.5	44.6	-5.6	6.33	EV2	4.88
1	4		8.32	407.0	0.0	15.0	41.4	-7.4	6.89	EV2	5.31
1	5		10.41	407.0	0.0	12.5	38.2	-9.1	7.53	EV2	5.81
1	6		12.49	407.0	0.0	10.1	35.0	-11.1	8.29	EV2	6.40
1	7		14.57	407.0	0.0	7.6	31.8	-13.3	9.21	EV2	7.11
1	8		16.65	407.0	0.0	5.2	28.6	-15.9	10.33	EV2	7.97
1	9		18.73	407.0	0.0	2.3	25.4	-19.1	11.75	EV2	9.06
1	10		20.81	407.0	0.0	-0.1	22.3	-22.3	13.54	EV2	10.45
1	11		22.89	407.0	0.0	-2.6	19.1	-25.4	11.74	EV2	9.06
1	12		24.97	407.0	0.0	-5.0	15.9	-28.6	10.34	EV2	7.97
1	13		27.06	407.0	0.0	-7.5	13.3	-31.8	9.22	EV2	7.11
1	14		29.14	407.0	0.0	-10.0	11.1	-35.0	8.30	EV2	6.40
1	15		31.22	407.0	0.0	-12.4	9.1	-38.2	7.53	EV2	5.81
1	16		33.30	407.0	0.0	-15.1	7.4	-41.4	6.88	EV2	5.31
1	17		35.38	407.0	0.0	-17.5	5.6	-44.6	6.33	EV2	4.88
1	18		37.46	407.0	0.0	-20.0	3.7	-47.7	5.85	EV2	4.51
1	19		39.54	407.0	0.0	-22.4	1.9	-50.9	5.43	EV2	4.19
1	20		41.62	407.0	0.0	-25.7	0.0	-54.1	5.04	EV2	3.89

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 2.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	41.62	0.00	STRENGTH L.S.	5.03	73.17

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 5.033

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV2	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

TABLE 2.1.3.1=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)	V O L U M E (yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.380	0.453
2	0.176	13.448
3	0.176	28.448
4	0.380	41.172

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)

(k)	(ft)	(k)	(ft)	(k)	(ft)	(k)	(ft)
24.00	15.00	33.50	0.00				

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2
****

NO. OF AXLES = 2

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	33.50	0.00				

TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0		0.0		0.0	0.0
1	1		2.08	52.6	71.3		0.0		123.9	52.6
1	2		4.16	99.6	133.6		0.0		233.3	99.6
1	3		6.24	141.1	187.1		0.0		328.2	141.1
1	4		8.32	177.2	231.6		0.0		408.8	177.2
1	5		10.41	207.7	267.2		0.0		474.9	207.7
1	6		12.49	232.7	293.9		0.0		526.7	232.7
1	7		14.57	252.1	311.7		0.0		563.8	252.1
1	8		16.65	265.8	320.6		0.0		586.4	265.8
1	9		18.73	274.1	313.8		0.0		587.9	274.1
1	10		20.81	276.8	311.6		0.0		588.4	276.8
1	11		22.89	274.0	313.8		0.0		587.9	274.0
1	12		24.97	265.8	320.6		0.0		586.4	265.8
1	13		27.06	252.1	311.7		0.0		563.8	252.1
1	14		29.14	232.8	293.9		0.0		526.7	232.8
1	15		31.22	207.7	267.2		0.0		474.9	207.7
1	16		33.30	177.2	231.6		0.0		408.8	177.2
1	17		35.38	141.2	187.1		0.0		328.2	141.2
1	18		37.46	99.6	133.6		0.0		233.3	99.6
1	19		39.54	52.7	71.3		0.0		123.9	52.7
1	20		41.62	0.0	0.0		0.0		0.0	0.0

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0		0.00	26.9	36.4		0.0		63.3	26.9
1	1		2.08	23.9	34.2		-1.2		58.1	22.7
1	2		4.16	21.3	32.1		-2.5		53.4	18.8
1	3		6.24	18.6	29.9		-3.7		48.6	14.9
1	4		8.32	16.0	27.8		-5.0		43.8	11.0
1	5		10.41	13.4	25.7		-6.1		39.0	7.2
1	6		12.49	10.7	23.5		-7.5		34.2	3.2
1	7		14.57	7.9	21.4		-8.9		29.3	-1.0
1	8		16.65	5.3	19.2		-10.7		24.5	-5.4
1	9		18.73	2.6	17.1		-12.8		19.7	-10.2
1	10		20.81	0.0	15.0		-15.0		-15.0	15.0
1	11		22.89	-2.6	12.8		-17.1		-19.7	10.2
1	12		24.97	-5.3	10.7		-19.2		-24.5	5.4
1	13		27.06	-7.9	8.9		-21.4		-29.3	1.0
1	14		29.14	-10.7	7.5		-23.5		-34.2	-3.2
1	15		31.22	-13.4	6.1		-25.7		-39.0	-7.2
1	16		33.30	-16.0	5.0		-27.8		-43.8	-11.0
1	17		35.38	-18.6	3.7		-29.9		-48.6	-14.9
1	18		37.46	-21.3	2.5		-32.1		-53.4	-18.8
1	19		39.54	-23.9	1.2		-34.2		-58.1	-22.7
1	20		41.62	-26.9	0.0		-36.4		-63.3	-26.9

TABLE 2.2.7.2=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	26.91	0.00	HL-93	36.37	MAX.V	ST1	24.22	33.64
						ST2	24.22	82.74
						ST4	40.37	40.37
						SE1	26.91	26.91
						SE2	26.91	74.20
2	26.91	0.00	HL-93	36.37	MAX.V	ST1	24.22	33.64
						ST2	24.22	82.75
						ST4	40.37	40.37
						SE1	26.91	26.91
						SE2	26.91	74.20

NOTE: [1] " - " Indicates Uplift

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

TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****
(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	0.00	2 0.400	0.000 MAX	HL-93	0.62	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	0.62	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.252 MAX	EV2		
			0.038 MIN	EV2		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft) or STRESS	MOMENT CAP. (k-ft)		UNFACTORED DEAD LOAD MOMENT D1; k-ft D2 ksi	UNFACTORED L+I MAX. MOMENT POS. MAX. STRESS	UNFACTORED L+I MAX. MOMENT NEG. MAX. STRESS	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
1	0		0.00	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0
1	1		2.08	33.00	TOP	-1.25	-1.70	0.00	13.71	EV2	10.58	0
				33.00	BOT	1.25	1.70	0.00	13.71	EV2	10.58	0
1	2		4.16	33.00	TOP	-2.37	-3.18	0.00	6.99	EV2	5.39	0
				33.00	BOT	2.37	3.18	0.00	6.99	EV2	5.39	0
1	3		6.24	33.00	TOP	-3.36	-4.46	0.00	4.79	EV2	3.69	0
				33.00	BOT	3.36	4.46	0.00	4.79	EV2	3.69	0
1	4		8.32	33.00	TOP	-4.22	-5.52	0.00	3.72	EV2	2.87	0
				33.00	BOT	4.22	5.52	0.00	3.72	EV2	2.87	0
1	5		10.41	33.00	TOP	-4.95	-6.37	0.00	3.12	EV2	2.41	0
				33.00	BOT	4.95	6.37	0.00	3.12	EV2	2.41	0
1	6		12.49	33.00	TOP	-5.55	-7.00	0.00	2.76	EV2	2.13	0
				33.00	BOT	5.55	7.00	0.00	2.76	EV2	2.13	0
1	7		14.57	29.37	TOP	-6.01	-7.43	0.00	2.18	EV2	1.68	0
				33.00	BOT	6.01	7.43	0.00	2.54	EV2	1.96	0
1	8		16.65	29.37	TOP	-6.33	-7.64	0.00	2.08	EV2	1.60	0
				33.00	BOT	6.33	7.64	0.00	2.43	EV2	1.88	0
1	9		18.73	29.37	TOP	-6.53	-7.48	0.00	2.10	EV2	1.62	0
				33.00	BOT	6.53	7.48	0.00	2.46	EV2	1.90	0
1	10		20.81	29.37	TOP	-6.60	-7.42	0.00	2.11	EV2	1.63	0
				33.00	BOT	6.60	7.42	0.00	2.47	EV2	1.91	0
1	11		22.89	29.37	TOP	-6.53	-7.48	0.00	2.10	EV2	1.62	0
				33.00	BOT	6.53	7.48	0.00	2.46	EV2	1.90	0
1	12		24.97	29.37	TOP	-6.33	-7.64	0.00	2.08	EV2	1.61	0
				33.00	BOT	6.33	7.64	0.00	2.43	EV2	1.88	0
1	13		27.06	29.37	TOP	-6.01	-7.43	0.00	2.18	EV2	1.68	0
				33.00	BOT	6.01	7.43	0.00	2.54	EV2	1.96	0
1	14		29.14	33.00	TOP	-5.55	-7.00	0.00	2.76	EV2	2.13	0
				33.00	BOT	5.55	7.00	0.00	2.76	EV2	2.13	0
1	15		31.22	33.00	TOP	-4.95	-6.37	0.00	3.12	EV2	2.41	0
				33.00	BOT	4.95	6.37	0.00	3.12	EV2	2.41	0
1	16		33.30	33.00	TOP	-4.22	-5.52	0.00	3.72	EV2	2.87	0
				33.00	BOT	4.22	5.52	0.00	3.72	EV2	2.87	0
1	17		35.38	33.00	TOP	-3.36	-4.46	0.00	4.79	EV2	3.69	0
				33.00	BOT	3.36	4.46	0.00	4.79	EV2	3.69	0
1	18		37.46	33.00	TOP	-2.37	-3.18	0.00	6.99	EV2	5.39	0
				33.00	BOT	2.37	3.18	0.00	6.99	EV2	5.39	0
1	19		39.54	33.00	TOP	-1.25	-1.70	0.00	13.71	EV2	10.58	0
				33.00	BOT	1.25	1.70	0.00	13.71	EV2	10.58	0
1	20		41.62	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

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

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TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D L	FROM 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	26.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92
1	1		2.08	26.4	-1.25	1.25	-1.70	1.70	0.00	0.00	14.81		11.39
1	2		4.16	26.4	-2.37	2.37	-3.18	3.18	0.00	0.00	7.55		5.80
1	3		6.24	26.4	-3.36	3.36	-4.46	4.46	0.00	0.00	5.17		3.98
1	4		8.32	26.4	-4.22	4.22	-5.52	5.52	0.00	0.00	4.02		3.09
1	5		10.41	26.4	-4.95	4.95	-6.37	6.37	0.00	0.00	3.37		2.59
1	6		12.49	26.4	-5.55	5.55	-7.00	7.00	0.00	0.00	2.98		2.29
1	7		14.57	26.4	-6.01	6.01	-7.43	7.43	0.00	0.00	2.75		2.11
1	8		16.65	26.4	-6.33	6.33	-7.64	7.64	0.00	0.00	2.63		2.02
1	9		18.73	26.4	-6.53	6.53	-7.48	7.48	0.00	0.00	2.66		2.04
1	10		20.81	26.4	-6.60	6.60	-7.42	7.42	0.00	0.00	2.67		2.05
1	11		22.89	26.4	-6.53	6.53	-7.48	7.48	0.00	0.00	2.66		2.04
1	12		24.97	26.4	-6.33	6.33	-7.64	7.64	0.00	0.00	2.63		2.02
1	13		27.06	26.4	-6.01	6.01	-7.43	7.43	0.00	0.00	2.75		2.11
1	14		29.14	26.4	-5.55	5.55	-7.00	7.00	0.00	0.00	2.98		2.29
1	15		31.22	26.4	-4.95	4.95	-6.37	6.37	0.00	0.00	3.37		2.59
1	16		33.30	26.4	-4.22	4.22	-5.52	5.52	0.00	0.00	4.02		3.09
1	17		35.38	26.4	-3.36	3.36	-4.46	4.46	0.00	0.00	5.17		3.98
1	18		37.46	26.4	-2.37	2.37	-3.18	3.18	0.00	0.00	7.55		5.80
1	19		39.54	26.4	-1.25	1.25	-1.70	1.70	0.00	0.00	14.81		11.39
1	20		41.62	26.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 2.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	41.62	16.65	STRENGTH L.S.	2.08	433.55
1	41.62	16.65	SERVICEBILITY	2.02	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 2.021

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TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D L	FROM 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	407.0	0.0	26.9	36.4	0.0	7.47	EV2	5.76
1	1		2.08	407.0	0.0	23.9	34.2	-1.2	8.03	EV2	6.20
1	2		4.16	407.0	0.0	21.3	32.1	-2.5	8.66	EV2	6.68
1	3		6.24	407.0	0.0	18.6	29.9	-3.7	9.37	EV2	7.23
1	4		8.32	407.0	0.0	16.0	27.8	-5.0	10.20	EV2	7.87
1	5		10.41	407.0	0.0	13.4	25.7	-6.1	11.17	EV2	8.61
1	6		12.49	407.0	0.0	10.7	23.5	-7.5	12.31	EV2	9.49
1	7		14.57	407.0	0.0	7.9	21.4	-8.9	13.69	EV2	10.56
1	8		16.65	407.0	0.0	5.3	19.2	-10.7	15.36	EV2	11.85
1	9		18.73	407.0	0.0	2.6	17.1	-12.8	17.46	EV2	13.47
1	10		20.81	407.0	0.0	0.0	15.0	-15.0	20.15	EV2	15.54
1	11		22.89	407.0	0.0	-2.6	12.8	-17.1	17.46	EV2	13.47
1	12		24.97	407.0	0.0	-5.3	10.7	-19.2	15.36	EV2	11.85
1	13		27.06	407.0	0.0	-7.9	8.9	-21.4	13.69	EV2	10.56
1	14		29.14	407.0	0.0	-10.7	7.5	-23.5	12.31	EV2	9.49
1	15		31.22	407.0	0.0	-13.4	6.1	-25.7	11.17	EV2	8.61
1	16		33.30	407.0	0.0	-16.0	5.0	-27.8	10.20	EV2	7.87
1	17		35.38	407.0	0.0	-18.6	3.7	-29.9	9.37	EV2	7.23
1	18		37.46	407.0	0.0	-21.3	2.5	-32.1	8.66	EV2	6.68
1	19		39.54	407.0	0.0	-23.9	1.2	-34.2	8.03	EV2	6.20
1	20		41.62	407.0	0.0	-26.9	0.0	-36.4	7.47	EV2	5.76

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

TABLE 2.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	41.62	41.62	STRENGTH L.S.	7.45	49.19

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 7.453

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user	
	EV3	

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.39	0.0	0.0	232.5		0.0	
1	2	8.78	0.0	0.0	432.9		0.0	
1	3	13.18	0.0	0.0	601.1		0.0	
1	4	17.57	0.0	0.0	736.1		0.0	
1	5	21.96	0.0	0.0	840.9		0.0	
1	6	26.35	0.0	0.0	916.2		0.0	
1	7	30.75	0.0	0.0	963.5		0.0	
1	8	35.14	0.0	0.0	983.8		0.0	
1	9	39.53	0.0	0.0	983.8		0.0	
1	10	43.92	0.0	0.0	963.5		0.0	
1	11	48.31	0.0	0.0	916.2		0.0	
1	12	52.71	0.0	0.0	840.9		0.0	
1	13	57.10	0.0	0.0	736.1		0.0	
1	14	61.49	0.0	0.0	601.1		0.0	
1	15	65.88	0.0	0.0	432.9		0.0	
1	16	70.27	0.0	0.0	232.5		0.0	
1	17	74.67	0.0	0.0	0.0		0.0	

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TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (UNFACTORED)
****

SP NO	IN D NO L	FROM 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	101.8		0.0	
1	1	4.39	0.0	0.0	95.3		-3.2	
1	2	8.78	0.0	0.0	88.7		-7.2	
1	3	13.18	0.0	0.0	82.1		-11.9	
1	4	17.57	0.0	0.0	75.5		-17.1	
1	5	21.96	0.0	0.0	68.9		-22.9	
1	6	26.35	0.0	0.0	62.3		-29.4	
1	7	30.75	0.0	0.0	55.7		-36.0	
1	8	35.14	0.0	0.0	49.2		-42.6	
1	9	39.53	0.0	0.0	42.6		-49.2	
1	10	43.92	0.0	0.0	36.0		-55.7	
1	11	48.31	0.0	0.0	29.4		-62.3	
1	12	52.71	0.0	0.0	22.9		-68.9	
1	13	57.10	0.0	0.0	17.1		-75.5	
1	14	61.49	0.0	0.0	11.9		-82.1	
1	15	65.88	0.0	0.0	7.2		-88.7	
1	16	70.27	0.0	0.0	3.2		-95.3	
1	17	74.67	0.0	0.0	0.0		-101.8	

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	49.11	0.00	HL-93	101.85	EV3	ST1	41.98	63.61
						ST2	41.98	201.10
						ST4	60.33	73.66
						SE1	49.11	49.11
						SE2	49.11	181.51
2	48.97	0.00	HL-93	101.85	EV3	ST1	41.85	63.43
						ST2	41.85	200.93
						ST4	60.13	73.45
						SE1	48.97	48.97
						SE2	48.97	181.37

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.030 MIN			
			0.000 MAX	LANE		
			0.030 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.045 MIN			
			0.000 MAX	LANE		
			0.045 MIN	LANE		
		2 0.400	-0.589 MAX	EV3		
			0.030 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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 DEAD LOAD 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	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		4.39	2986.6	144.3	56.0	232.5	0.0	8.69	EV3	6.70	0
1	2		8.78	2986.6	269.9	105.0	432.9	0.0	4.28	EV3	3.30	0
1	3		13.18	2986.6	378.0	147.1	601.1	0.0	2.84	EV3	2.19	0
1	4		17.57	4129.4	468.6	182.1	736.1	0.0	3.31	EV3	2.55	0
1	5		21.96	4129.4	540.3	210.1	840.9	0.0	2.78	EV3	2.15	0
1	6		26.35	4129.4	594.2	231.1	916.2	0.0	2.47	EV3	1.91	0
1	7		30.75	4129.4	630.4	245.1	963.5	0.0	2.30	EV3	1.78	0
1	8		35.14	4129.4	648.5	252.1	983.8	0.0	2.23	EV3	1.72	0
1	9		39.53	4129.4	648.1	252.1	983.8	0.0	2.23	EV3	1.72	0
1	10		43.92	4129.4	630.0	245.1	963.5	0.0	2.30	EV3	1.78	0
1	11		48.31	4129.4	594.4	231.1	916.2	0.0	2.47	EV3	1.91	0
1	12		52.71	4129.4	539.8	210.1	840.9	0.0	2.78	EV3	2.15	0
1	13		57.10	4129.4	467.4	182.1	736.1	0.0	3.31	EV3	2.55	0
1	14		61.49	2986.6	377.5	147.1	601.1	0.0	2.84	EV3	2.19	0
1	15		65.88	2986.6	269.6	105.0	432.9	0.0	4.28	EV3	3.30	0
1	16		70.27	2986.6	143.8	56.0	232.5	0.0	8.69	EV3	6.70	0
1	17		74.67	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		4.39	30.4	-3.03	3.44	-0.68	3.09	0.00	0.00	8.73	6.72
1	2		8.78	30.4	-5.67	6.43	-1.26	5.75	0.00	0.00	4.17	3.21
1	3		13.18	30.4	-7.93	9.01	-1.75	7.98	0.00	0.00	2.68	2.06
1	4		17.57	30.4	-8.96	7.30	-2.21	6.42	0.00	0.00	3.60	2.77
1	5		21.96	30.4	-10.33	8.42	-2.53	7.34	0.00	0.00	3.00	2.30
1	6		26.35	30.4	-11.36	9.26	-2.75	7.99	0.00	0.00	2.64	2.03
1	7		30.75	30.4	-12.05	9.82	-2.90	8.41	0.00	0.00	2.45	1.88
1	8		35.14	30.4	-12.40	10.11	-2.96	8.58	0.00	0.00	2.36	1.82
1	9		39.53	30.4	-12.39	10.10	-2.96	8.58	0.00	0.00	2.36	1.82
1	10		43.92	30.4	-12.04	9.82	-2.90	8.41	0.00	0.00	2.45	1.88
1	11		48.31	30.4	-11.36	9.26	-2.75	7.99	0.00	0.00	2.64	2.03
1	12		52.71	30.4	-10.32	8.41	-2.53	7.34	0.00	0.00	3.00	2.30
1	13		57.10	30.4	-8.94	7.29	-2.21	6.42	0.00	0.00	3.60	2.77
1	14		61.49	30.4	-7.92	9.00	-1.75	7.98	0.00	0.00	2.68	2.06
1	15		65.88	30.4	-5.66	6.43	-1.26	5.75	0.00	0.00	4.17	3.21
1	16		70.27	30.4	-3.02	3.43	-0.68	3.09	0.00	0.00	8.73	6.72
1	17		74.67	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	35.14	STRENGTH L.S.	2.23	1330.57
1	74.67	35.14	SERVICEBILITY	1.82	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 1.818

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

SP NO	IN NO	D L	FROM 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	481.9	35.6	13.6	101.8	0.0	3.04	EV3	2.35
1	1		4.39	481.9	30.6	12.0	95.3	-3.2	3.32	EV3	2.56
1	2		8.78	481.9	26.6	10.4	88.7	-7.2	3.62	EV3	2.80
1	3		13.18	481.9	22.6	8.8	82.1	-11.9	3.98	EV3	3.07
1	4		17.57	481.9	18.6	7.2	75.5	-17.1	4.40	EV3	3.39
1	5		21.96	481.9	14.3	5.6	68.9	-22.9	4.90	EV3	3.78
1	6		26.35	481.9	10.3	4.0	62.3	-29.4	5.51	EV3	4.25
1	7		30.75	481.9	6.2	2.4	55.7	-36.0	6.25	EV3	4.83
1	8		35.14	481.9	1.9	0.8	49.2	-42.6	7.21	EV3	5.56
1	9		39.53	481.9	-2.1	-0.8	42.6	-49.2	7.21	EV3	5.56
1	10		43.92	481.9	-6.1	-2.4	36.0	-55.7	6.26	EV3	4.83
1	11		48.31	481.9	-10.1	-4.0	29.4	-62.3	5.51	EV3	4.25
1	12		52.71	481.9	-14.5	-5.6	22.9	-68.9	4.90	EV3	3.78
1	13		57.10	481.9	-18.5	-7.2	17.1	-75.5	4.40	EV3	3.40
1	14		61.49	481.9	-22.5	-8.8	11.9	-82.1	3.98	EV3	3.07
1	15		65.88	481.9	-26.7	-10.4	7.2	-88.7	3.62	EV3	2.80
1	16		70.27	481.9	-30.7	-12.0	3.2	-95.3	3.32	EV3	2.56
1	17		74.67	481.9	-35.4	-13.6	0.0	-101.8	3.04	EV3	2.35

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	0.00	STRENGTH L.S.	3.04	137.74

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 3.037

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

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TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	3.73	0.0	0.0	199.7		0.0	
1	2	7.47	0.0	0.0	376.2		0.0	
1	3	11.20	0.0	0.0	529.4		0.0	
1	4	14.93	0.0	0.0	659.4		0.0	
1	5	18.67	0.0	0.0	765.0		0.0	
1	6	22.40	0.0	0.0	849.7		0.0	
1	7	26.13	0.0	0.0	913.1		0.0	
1	8	29.87	0.0	0.0	957.0		0.0	
1	9	33.60	0.0	0.0	979.0		0.0	
1	10	37.33	0.0	0.0	986.4		0.0	
1	11	41.07	0.0	0.0	979.0		0.0	
1	12	44.80	0.0	0.0	957.0		0.0	
1	13	48.53	0.0	0.0	913.1		0.0	
1	14	52.27	0.0	0.0	849.7		0.0	
1	15	56.00	0.0	0.0	765.0		0.0	
1	16	59.73	0.0	0.0	659.4		0.0	
1	17	63.47	0.0	0.0	529.4		0.0	
1	18	67.20	0.0	0.0	376.2		0.0	
1	19	70.93	0.0	0.0	199.7		0.0	
1	20	74.67	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	101.8		0.0	
1	1	3.73	0.0	0.0	96.2		-2.7	
1	2	7.47	0.0	0.0	90.6		-5.9	
1	3	11.20	0.0	0.0	85.1		-9.8	
1	4	14.93	0.0	0.0	79.5		-14.0	
1	5	18.67	0.0	0.0	73.9		-18.5	
1	6	22.40	0.0	0.0	68.3		-23.5	
1	7	26.13	0.0	0.0	62.7		-29.1	
1	8	29.87	0.0	0.0	57.1		-34.7	
1	9	33.60	0.0	0.0	51.5		-40.3	
1	10	37.33	0.0	0.0	45.9		-45.9	
1	11	41.07	0.0	0.0	40.3		-51.5	
1	12	44.80	0.0	0.0	34.7		-57.1	
1	13	48.53	0.0	0.0	29.1		-62.7	
1	14	52.27	0.0	0.0	23.5		-68.3	
1	15	56.00	0.0	0.0	18.5		-73.9	
1	16	59.73	0.0	0.0	14.0		-79.5	
1	17	63.47	0.0	0.0	9.8		-85.1	
1	18	67.20	0.0	0.0	5.9		-90.6	
1	19	70.93	0.0	0.0	2.7		-96.2	
1	20	74.67	0.0	0.0	0.0		-101.8	

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	49.11	0.00	HL-93	101.85	EV3	ST1	41.98	63.61
						ST2	41.98	201.10
						ST4	60.33	73.66
						SE1	49.11	49.11
						SE2	49.11	181.51
2	48.97	0.00	HL-93	101.85	EV3	ST1	41.85	63.43
						ST2	41.85	200.93
						ST4	60.13	73.45
						SE1	48.97	48.97
						SE2	48.97	181.37

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.030 MIN			
			0.000 MAX	LANE		
			0.030 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.045 MIN			
			0.000 MAX	LANE		
			0.045 MIN	LANE		
		2 0.400	-0.589 MAX	EV3		
			0.030 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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	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	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		3.73	2986.6	123.9	48.1	199.7	0.0	10.25	EV3	7.91	0
1	2		7.47	2986.6	234.1	91.1	376.2	0.0	5.05	EV3	3.90	0
1	3		11.20	2986.6	331.6	129.0	529.4	0.0	3.34	EV3	2.58	0
1	4		14.93	4129.4	416.4	161.9	659.4	0.0	3.80	EV3	2.93	0
1	5		18.67	4129.4	488.4	189.7	765.0	0.0	3.15	EV3	2.43	0
1	6		22.40	4129.4	546.5	212.5	849.7	0.0	2.74	EV3	2.12	0
1	7		26.13	4129.4	591.9	230.2	913.1	0.0	2.49	EV3	1.92	0
1	8		29.87	4129.4	624.6	242.9	957.0	0.0	2.33	EV3	1.79	0
1	9		33.60	4129.4	644.5	250.4	979.0	0.0	2.25	EV3	1.73	0
1	10		37.33	4129.4	650.5	253.0	986.4	0.0	2.22	EV3	1.71	0
1	11		41.07	4129.4	643.7	250.4	979.0	0.0	2.25	EV3	1.73	0
1	12		44.80	4129.4	624.3	242.9	957.0	0.0	2.33	EV3	1.79	0
1	13		48.53	4129.4	592.1	230.2	913.1	0.0	2.49	EV3	1.92	0
1	14		52.27	4129.4	546.0	212.5	849.7	0.0	2.74	EV3	2.12	0
1	15		56.00	4129.4	487.2	189.7	765.0	0.0	3.15	EV3	2.43	0
1	16		59.73	4129.4	415.6	161.9	659.4	0.0	3.80	EV3	2.93	0
1	17		63.47	2986.6	331.3	129.0	529.4	0.0	3.34	EV3	2.58	0
1	18		67.20	2986.6	233.7	91.1	376.2	0.0	5.05	EV3	3.90	0
1	19		70.93	2986.6	123.4	48.1	199.7	0.0	10.25	EV3	7.91	0
1	20		74.67	2986.6	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		3.73	30.4	-2.60	2.95	-0.58	2.65	0.00	0.00	10.35	7.96
1	2		7.47	30.4	-4.91	5.58	-1.10	5.00	0.00	0.00	4.97	3.82
1	3		11.20	30.4	-6.96	7.90	-1.54	7.03	0.00	0.00	3.20	2.46
1	4		14.93	30.4	-7.96	6.49	-1.98	5.75	0.00	0.00	4.16	3.20
1	5		18.67	30.4	-9.34	7.61	-2.30	6.68	0.00	0.00	3.41	2.63
1	6		22.40	30.4	-10.45	8.52	-2.55	7.41	0.00	0.00	2.95	2.27
1	7		26.13	30.4	-11.32	9.23	-2.75	7.97	0.00	0.00	2.66	2.04
1	8		29.87	30.4	-11.94	9.73	-2.88	8.35	0.00	0.00	2.47	1.90
1	9		33.60	30.4	-12.32	10.04	-2.94	8.54	0.00	0.00	2.38	1.83
1	10		37.33	30.4	-12.44	10.14	-2.97	8.61	0.00	0.00	2.35	1.81
1	11		41.07	30.4	-12.31	10.03	-2.94	8.54	0.00	0.00	2.38	1.83
1	12		44.80	30.4	-11.93	9.73	-2.88	8.35	0.00	0.00	2.48	1.90
1	13		48.53	30.4	-11.32	9.23	-2.75	7.97	0.00	0.00	2.66	2.04
1	14		52.27	30.4	-10.44	8.51	-2.55	7.41	0.00	0.00	2.95	2.27
1	15		56.00	30.4	-9.31	7.60	-2.30	6.68	0.00	0.00	3.42	2.63
1	16		59.73	30.4	-7.95	6.48	-1.98	5.75	0.00	0.00	4.16	3.20
1	17		63.47	30.4	-6.95	7.90	-1.54	7.03	0.00	0.00	3.20	2.46
1	18		67.20	30.4	-4.91	5.57	-1.10	5.00	0.00	0.00	4.97	3.82
1	19		70.93	30.4	-2.59	2.94	-0.58	2.65	0.00	0.00	10.35	7.96
1	20		74.67	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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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 99.99 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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	37.33	STRENGTH L.S.	2.22	1334.01
1	74.67	37.33	SERVICEBILITY	1.81	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 1.811

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

SP NO	IN NO	D L	FROM 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	481.9	35.6	13.6	101.8	0.0	3.04	EV3	2.35
1	1		3.73	481.9	31.2	12.2	96.2	-2.7	3.28	EV3	2.53
1	2		7.47	481.9	27.8	10.8	90.6	-5.9	3.53	EV3	2.72
1	3		11.20	481.9	24.4	9.5	85.1	-9.8	3.81	EV3	2.94
1	4		14.93	481.9	21.0	8.1	79.5	-14.0	4.14	EV3	3.19
1	5		18.67	481.9	17.3	6.8	73.9	-18.5	4.52	EV3	3.49
1	6		22.40	481.9	13.9	5.4	68.3	-23.5	4.96	EV3	3.83
1	7		26.13	481.9	10.5	4.1	62.7	-29.1	5.47	EV3	4.22
1	8		29.87	481.9	7.0	2.7	57.1	-34.7	6.09	EV3	4.70
1	9		33.60	481.9	3.6	1.4	51.5	-40.3	6.84	EV3	5.28
1	10		37.33	481.9	-0.1	0.0	45.9	-45.9	7.78	EV3	6.00
1	11		41.07	481.9	-3.5	-1.4	40.3	-51.5	6.85	EV3	5.28
1	12		44.80	481.9	-6.9	-2.7	34.7	-57.1	6.09	EV3	4.70
1	13		48.53	481.9	-10.3	-4.1	29.1	-62.7	5.48	EV3	4.22
1	14		52.27	481.9	-14.1	-5.4	23.5	-68.3	4.96	EV3	3.82
1	15		56.00	481.9	-17.5	-6.8	18.5	-73.9	4.52	EV3	3.49
1	16		59.73	481.9	-20.9	-8.1	14.0	-79.5	4.14	EV3	3.20
1	17		63.47	481.9	-24.3	-9.5	9.8	-85.1	3.82	EV3	2.94
1	18		67.20	481.9	-27.9	-10.8	5.9	-90.6	3.53	EV3	2.72
1	19		70.93	481.9	-31.3	-12.2	2.7	-96.2	3.28	EV3	2.53
1	20		74.67	481.9	-35.4	-13.6	0.0	-101.8	3.04	EV3	2.35

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	0.00	STRENGTH L.S.	3.04	137.74

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 3.037

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	75.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.67	0.0	0.0	210.7		0.0	
1	2	9.33	0.0	0.0	390.4		0.0	
1	3	14.00	0.0	0.0	538.9		0.0	
1	4	18.67	0.0	0.0	655.3		0.0	
1	5	23.33	0.0	0.0	743.0		0.0	
1	6	28.00	0.0	0.0	803.1		0.0	
1	7	32.67	0.0	0.0	835.1		0.0	
1	8	37.33	0.0	0.0	845.0		0.0	
1	9	42.00	0.0	0.0	835.1		0.0	
1	10	46.67	0.0	0.0	803.1		0.0	
1	11	51.33	0.0	0.0	743.0		0.0	
1	12	56.00	0.0	0.0	655.3		0.0	
1	13	60.67	0.0	0.0	538.9		0.0	
1	14	65.33	0.0	0.0	390.4		0.0	
1	15	70.00	0.0	0.0	210.7		0.0	
1	16	74.67	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	73.2		0.0	
1	1	4.67	0.0	0.0	68.2		-2.5	
1	2	9.33	0.0	0.0	63.2		-5.6	
1	3	14.00	0.0	0.0	58.1		-9.3	
1	4	18.67	0.0	0.0	53.1		-13.3	
1	5	23.33	0.0	0.0	48.1		-17.9	
1	6	28.00	0.0	0.0	43.0		-22.9	
1	7	32.67	0.0	0.0	38.0		-27.9	
1	8	37.33	0.0	0.0	33.0		-33.0	
1	9	42.00	0.0	0.0	27.9		-38.0	
1	10	46.67	0.0	0.0	22.9		-43.0	
1	11	51.33	0.0	0.0	17.9		-48.1	
1	12	56.00	0.0	0.0	13.3		-53.1	
1	13	60.67	0.0	0.0	9.3		-58.1	
1	14	65.33	0.0	0.0	5.6		-63.2	
1	15	70.00	0.0	0.0	2.5		-68.2	
1	16	74.67	0.0	0.0	0.0		-73.2	

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	44.78	0.00	HL-93	73.24	EV3	ST1	38.08	58.20
						ST2	38.08	157.07
						ST4	53.84	67.17
						SE1	44.78	44.78
						SE2	44.78	139.99
2	44.78	0.00	HL-93	73.24	EV3	ST1	38.08	58.20
						ST2	38.08	157.07
						ST4	53.85	67.17
						SE1	44.78	44.78
						SE2	44.78	139.99

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.12	0.00000
			0.033 MIN			
			0.000 MAX	LANE		
			0.033 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.12	0.00000
			0.050 MIN			
			0.000 MAX	LANE		
			0.050 MIN	LANE		
		2 0.400	-0.650 MAX	EV3		
			0.033 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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 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	2891.9	0.0	0.0	0.0	0.0 99.99	HL-93	99.99	0
1	1		4.67	2891.9	135.2	59.3	210.7	0.0 9.28	EV3	7.16	0
1	2		9.33	2891.9	252.5	110.7	390.4	0.0 4.59	EV3	3.54	0
1	3		14.00	2891.9	352.0	154.2	538.9	0.0 3.07	EV3	2.37	0
1	4		18.67	3756.2	433.0	189.7	655.3	0.0 3.33	EV3	2.57	0
1	5		23.33	3756.2	496.1	217.4	743.0	0.0 2.82	EV3	2.18	0
1	6		28.00	3756.2	541.4	237.2	803.1	0.0 2.53	EV3	1.95	0
1	7		32.67	3756.2	568.4	249.0	835.1	0.0 2.39	EV3	1.84	0
1	8		37.33	3756.2	577.3	253.0	845.0	0.0 2.35	EV3	1.81	0
1	9		42.00	3756.2	568.4	249.0	835.1	0.0 2.39	EV3	1.84	0
1	10		46.67	3756.2	541.4	237.2	803.1	0.0 2.53	EV3	1.95	0
1	11		51.33	3756.2	496.1	217.4	743.0	0.0 2.82	EV3	2.18	0
1	12		56.00	3756.2	433.0	189.7	655.3	0.0 3.33	EV3	2.57	0
1	13		60.67	2891.9	352.0	154.2	538.9	0.0 3.07	EV3	2.37	0
1	14		65.33	2891.9	252.5	110.7	390.4	0.0 4.59	EV3	3.54	0
1	15		70.00	2891.9	135.2	59.3	210.7	0.0 9.28	EV3	7.16	0
1	16		74.67	2891.9	0.0	0.0	0.0	0.0 99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00	99.99		76.92
1	1		4.67	30.4	-2.94 3.34	-0.74 2.84	0.00 0.00	0.00	9.52		7.33
1	2		9.33	30.4	-5.49 6.23	-1.37 5.26	0.00 0.00	0.00	4.59		3.53
1	3		14.00	30.4	-7.65 8.69	-1.90 7.27	0.00 0.00	0.00	2.99		2.30
1	4		18.67	30.4	-8.67 7.36	-2.33 6.12	0.00 0.00	0.00	3.77		2.90
1	5		23.33	30.4	-9.93 8.43	-2.64 6.94	0.00 0.00	0.00	3.17		2.44
1	6		28.00	30.4	-10.84 9.20	-2.86 7.50	0.00 0.00	0.00	2.83		2.18
1	7		32.67	30.4	-11.38 9.66	-2.97 7.80	0.00 0.00	0.00	2.66		2.05
1	8		37.33	30.4	-11.56 9.81	-3.00 7.89	0.00 0.00	0.00	2.61		2.01
1	9		42.00	30.4	-11.38 9.66	-2.97 7.80	0.00 0.00	0.00	2.66		2.05
1	10		46.67	30.4	-10.84 9.20	-2.86 7.50	0.00 0.00	0.00	2.83		2.18
1	11		51.33	30.4	-9.93 8.43	-2.64 6.94	0.00 0.00	0.00	3.17		2.44
1	12		56.00	30.4	-8.67 7.36	-2.33 6.12	0.00 0.00	0.00	3.77		2.90
1	13		60.67	30.4	-7.65 8.69	-1.90 7.27	0.00 0.00	0.00	2.99		2.30
1	14		65.33	30.4	-5.49 6.23	-1.37 5.26	0.00 0.00	0.00	4.59		3.53
1	15		70.00	30.4	-2.94 3.34	-0.74 2.84	0.00 0.00	0.00	9.52		7.33
1	16		74.67	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	74.67	37.33	STRENGTH L.S.	2.34	1142.74
1	74.67	37.33	SERVICEBILITY	2.01	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 2.008

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

SP NO	IN NO	D L	FROM 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	481.9	31.2	13.6	73.2	0.0	4.28	EV3	3.31
1	1		4.67	481.9	27.0	11.9	68.2	-2.5	4.68	EV3	3.61
1	2		9.33	481.9	23.2	10.2	63.2	-5.6	5.14	EV3	3.97
1	3		14.00	481.9	19.4	8.5	58.1	-9.3	5.68	EV3	4.38
1	4		18.67	481.9	15.4	6.8	53.1	-13.3	6.32	EV3	4.87
1	5		23.33	481.9	11.6	5.1	48.1	-17.9	7.09	EV3	5.47
1	6		28.00	481.9	7.8	3.4	43.0	-22.9	8.04	EV3	6.20
1	7		32.67	481.9	3.8	1.7	38.0	-27.9	9.25	EV3	7.14
1	8		37.33	481.9	0.0	0.0	33.0	-33.0	10.82	EV3	8.35
1	9		42.00	481.9	-3.8	-1.7	27.9	-38.0	9.25	EV3	7.14
1	10		46.67	481.9	-7.8	-3.4	22.9	-43.0	8.04	EV3	6.20
1	11		51.33	481.9	-11.6	-5.1	17.9	-48.1	7.09	EV3	5.47
1	12		56.00	481.9	-15.4	-6.8	13.3	-53.1	6.32	EV3	4.87
1	13		60.67	481.9	-19.4	-8.5	9.3	-58.1	5.68	EV3	4.38
1	14		65.33	481.9	-23.2	-10.2	5.6	-63.2	5.14	EV3	3.97
1	15		70.00	481.9	-27.0	-11.9	2.5	-68.2	4.68	EV3	3.61
1	16		74.67	481.9	-31.2	-13.6	0.0	-73.2	4.28	EV3	3.31

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	74.67	74.67	STRENGTH L.S.	4.28	99.05

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 4.277

TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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)	REINFORCED CONCRETE SECTION Ix (in**4)
1	PG			33.4	0.8850	16.59	1.5700	16.59	1.5700		
2	PG			33.4	0.8850	16.59	1.5700	15.82	3.0700		

NOTE: [1] maximum allowable section number is 70

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

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	90.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	5.71	0.0	0.0	304.2		0.0	
1	2	11.43	0.0	0.0	564.4		0.0	
1	3	17.14	0.0	0.0	780.6		0.0	
1	4	22.85	0.0	0.0	951.6		0.0	
1	5	28.57	0.0	0.0	1081.3		0.0	
1	6	34.28	0.0	0.0	1170.9		0.0	
1	7	39.99	0.0	0.0	1220.1		0.0	
1	8	45.71	0.0	0.0	1235.5		0.0	
1	9	51.42	0.0	0.0	1220.1		0.0	
1	10	57.14	0.0	0.0	1170.9		0.0	
1	11	62.85	0.0	0.0	1081.3		0.0	
1	12	68.56	0.0	0.0	951.6		0.0	
1	13	74.28	0.0	0.0	780.6		0.0	
1	14	79.99	0.0	0.0	564.5		0.0	
1	15	85.70	0.0	0.0	304.2		0.0	
1	16	91.42	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (UNFACTORED)
****

SP NO	IN D NO L	FROM 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	102.6		0.0	
1	1	5.71	0.0	0.0	95.7		-3.7	
1	2	11.43	0.0	0.0	88.8		-8.0	
1	3	17.14	0.0	0.0	81.9		-13.1	
1	4	22.85	0.0	0.0	74.9		-19.6	
1	5	28.57	0.0	0.0	68.0		-26.4	
1	6	34.28	0.0	0.0	61.1		-33.4	
1	7	39.99	0.0	0.0	54.2		-40.3	
1	8	45.71	0.0	0.0	47.2		-47.2	
1	9	51.42	0.0	0.0	40.3		-54.2	
1	10	57.14	0.0	0.0	33.4		-61.1	
1	11	62.85	0.0	0.0	26.4		-68.0	
1	12	68.56	0.0	0.0	19.6		-74.9	
1	13	74.28	0.0	0.0	13.1		-81.9	
1	14	79.99	0.0	0.0	8.0		-88.8	
1	15	85.70	0.0	0.0	3.7		-95.7	
1	16	91.42	0.0	0.0	0.0		-102.6	

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	64.52	0.00	HL-93	102.64	EV3	ST1	55.34	83.36
						ST2	55.34	221.93
						ST4	80.45	96.77
						SE1	64.52	64.52
						SE2	64.52	197.95
2	64.52	0.00	HL-93	102.64	EV3	ST1	55.34	83.36
						ST2	55.34	221.93
						ST4	80.45	96.77
						SE1	64.52	64.52
						SE2	64.52	197.95

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.37	0.00000
			0.034 MIN			
			0.000 MAX	LANE		
			0.034 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.37	0.00000
			0.051 MIN			
			0.000 MAX	LANE		
			0.051 MIN	LANE		
		2 0.400	-0.823 MAX	EV3		
			0.034 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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	D2 ksi	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	4033.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		5.71	4033.3	253.3	88.9	304.2	0.0	8.74	EV3	6.74	0
1	2		11.43	4033.3	473.2	165.9	564.4	0.0	4.21	EV3	3.25	0
1	3		17.14	4033.3	658.8	231.1	780.6	0.0	2.74	EV3	2.11	0
1	4		22.85	5399.2	810.5	284.4	951.6	0.0	3.10	EV3	2.39	0
1	5		28.57	5399.2	929.3	325.9	1081.3	0.0	2.59	EV3	2.00	0
1	6		34.28	5399.2	1013.3	355.5	1170.9	0.0	2.30	EV3	1.77	0
1	7		39.99	5399.2	1063.9	373.3	1220.1	0.0	2.15	EV3	1.66	0
1	8		45.71	5399.2	1081.4	379.2	1235.5	0.0	2.11	EV3	1.62	0
1	9		51.42	5399.2	1063.9	373.3	1220.1	0.0	2.15	EV3	1.66	0
1	10		57.14	5399.2	1013.3	355.5	1170.9	0.0	2.30	EV3	1.77	0
1	11		62.85	5399.2	929.3	325.9	1081.3	0.0	2.59	EV3	2.00	0
1	12		68.56	5399.2	810.5	284.4	951.6	0.0	3.10	EV3	2.39	0
1	13		74.28	4033.3	658.8	231.1	780.6	0.0	2.74	EV3	2.11	0
1	14		79.99	4033.3	473.2	165.9	564.5	0.0	4.21	EV3	3.25	0
1	15		85.70	4033.3	253.3	88.9	304.2	0.0	8.74	EV3	6.74	0
1	16		91.42	4033.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

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

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MERLIN V10.10
COMPOSITE
LRFD- 8TH ED
RATING

TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D L	FROM 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	30.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		5.71	30.4	-3.53	3.88	-0.89	2.83	0.00	0.00	9.36	7.20
1	2		11.43	30.4	-6.58	7.25	-1.65	5.26	0.00	0.00	4.40	3.39
1	3		17.14	30.4	-9.17	10.10	-2.28	7.27	0.00	0.00	2.79	2.15
1	4		22.85	30.4	-10.34	7.91	-2.77	5.62	0.00	0.00	4.00	3.08
1	5		28.57	30.4	-11.86	9.07	-3.15	6.38	0.00	0.00	3.34	2.57
1	6		34.28	30.4	-12.93	9.89	-3.41	6.91	0.00	0.00	2.97	2.28
1	7		39.99	30.4	-13.58	10.38	-3.55	7.20	0.00	0.00	2.78	2.14
1	8		45.71	30.4	-13.80	10.55	-3.60	7.29	0.00	0.00	2.72	2.09
1	9		51.42	30.4	-13.58	10.38	-3.55	7.20	0.00	0.00	2.78	2.14
1	10		57.14	30.4	-12.93	9.89	-3.41	6.91	0.00	0.00	2.97	2.28
1	11		62.85	30.4	-11.86	9.07	-3.15	6.38	0.00	0.00	3.34	2.57
1	12		68.56	30.4	-10.34	7.91	-2.77	5.62	0.00	0.00	4.00	3.08
1	13		74.28	30.4	-9.17	10.10	-2.28	7.27	0.00	0.00	2.79	2.15
1	14		79.99	30.4	-6.58	7.25	-1.65	5.26	0.00	0.00	4.40	3.39
1	15		85.70	30.4	-3.53	3.88	-0.89	2.83	0.00	0.00	9.36	7.20
1	16		91.42	30.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	91.42	45.71	STRENGTH L.S.	2.10	1670.90
1	91.42	45.71	SERVICEBILITY	2.09	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 2.094

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

SP NO	IN NO	D L	FROM 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	548.0	47.9	16.6	102.6	0.0	3.35	EV3	2.59
1	1		5.71	548.0	41.4	14.5	95.7	-3.7	3.68	EV3	2.84
1	2		11.43	548.0	35.6	12.4	88.8	-8.0	4.05	EV3	3.13
1	3		17.14	548.0	29.5	10.4	81.9	-13.1	4.49	EV3	3.47
1	4		22.85	548.0	23.7	8.3	74.9	-19.6	5.01	EV3	3.86
1	5		28.57	548.0	17.9	6.2	68.0	-26.4	5.63	EV3	4.34
1	6		34.28	548.0	11.7	4.1	61.1	-33.4	6.40	EV3	4.93
1	7		39.99	548.0	6.0	2.1	54.2	-40.3	7.35	EV3	5.67
1	8		45.71	548.0	0.2	0.0	47.2	-47.2	8.59	EV3	6.63
1	9		51.42	548.0	-6.0	-2.1	40.3	-54.2	7.35	EV3	5.67
1	10		57.14	548.0	-11.7	-4.1	33.4	-61.1	6.40	EV3	4.93
1	11		62.85	548.0	-17.9	-6.2	26.4	-68.0	5.63	EV3	4.34
1	12		68.56	548.0	-23.7	-8.3	19.6	-74.9	5.01	EV3	3.86
1	13		74.28	548.0	-29.5	-10.4	13.1	-81.9	4.49	EV3	3.47
1	14		79.99	548.0	-35.6	-12.4	8.0	-88.8	4.05	EV3	3.13
1	15		85.70	548.0	-41.4	-14.5	3.7	-95.7	3.68	EV3	2.84
1	16		91.42	548.0	-47.9	-16.6	0.0	-102.6	3.35	EV3	2.59

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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RATING

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	91.42	91.42	STRENGTH L.S.	3.35	138.82

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 3.347

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INPUT
FILE NAME = C:\Users\jtzoc\Desktop\TEMP\5800 Ext G S2Rev_ev3.dat — PAGE 3

TABLE 0.0.4.1 DEFINITION OF SECTIONS
****

SECTION NO.	ID.	STANDARD SECTN NOMINAL DEPTH (in)	STANDARD SECTN WEIGHT (lb/ft)	PLATE GIRDER WEB DEPTH (in)	PLATE GIRDER 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)	REINFORCED CONCRETE SECTION Ix (in**4)
1	PG			33.4	0.8850	16.59	1.5700	16.59	1.5700		
2	PG			33.4	0.8850	16.59	1.5700	15.91	2.7580		

NOTE: [1] maximum allowable section number is 70

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

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

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

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	7.50	75.00

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

TABLE 1.2.5.3=COMPOSITE LIVE LOAD MOMENTS FOR N = 10.00 (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	4.57	0.0	0.0	215.8		0.0	
1	2	9.14	0.0	0.0	406.9		0.0	
1	3	13.71	0.0	0.0	573.4		0.0	
1	4	18.28	0.0	0.0	715.3		0.0	
1	5	22.85	0.0	0.0	831.5		0.0	
1	6	27.43	0.0	0.0	925.2		0.0	
1	7	32.00	0.0	0.0	995.9		0.0	
1	8	36.57	0.0	0.0	1045.1		0.0	
1	9	41.14	0.0	0.0	1071.0		0.0	
1	10	45.71	0.0	0.0	1079.6		0.0	
1	11	50.28	0.0	0.0	1071.0		0.0	
1	12	54.85	0.0	0.0	1045.1		0.0	
1	13	59.42	0.0	0.0	995.9		0.0	
1	14	63.99	0.0	0.0	925.2		0.0	
1	15	68.56	0.0	0.0	831.5		0.0	
1	16	73.13	0.0	0.0	715.3		0.0	
1	17	77.70	0.0	0.0	573.4		0.0	
1	18	82.28	0.0	0.0	406.9		0.0	
1	19	86.85	0.0	0.0	215.8		0.0	
1	20	91.42	0.0	0.0	0.0		0.0	

TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 10.0 (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	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	73.7		0.0	
1	1	4.57	0.0	0.0	69.7		-2.1	
1	2	9.14	0.0	0.0	65.8		-4.5	
1	3	13.71	0.0	0.0	61.8		-7.1	
1	4	18.28	0.0	0.0	57.8		-10.2	
1	5	22.85	0.0	0.0	53.8		-14.1	
1	6	27.43	0.0	0.0	49.8		-18.0	
1	7	32.00	0.0	0.0	45.9		-22.0	
1	8	36.57	0.0	0.0	41.9		-26.0	
1	9	41.14	0.0	0.0	37.9		-29.9	
1	10	45.71	0.0	0.0	33.9		-33.9	
1	11	50.28	0.0	0.0	29.9		-37.9	
1	12	54.85	0.0	0.0	26.0		-41.9	
1	13	59.42	0.0	0.0	22.0		-45.9	
1	14	63.99	0.0	0.0	18.0		-49.8	
1	15	68.56	0.0	0.0	14.1		-53.8	
1	16	73.13	0.0	0.0	10.2		-57.8	
1	17	77.70	0.0	0.0	7.1		-61.8	
1	18	82.28	0.0	0.0	4.5		-65.8	
1	19	86.85	0.0	0.0	2.1		-69.7	
1	20	91.42	0.0	0.0	0.0		-73.7	

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	58.11	0.00	HL-93	73.73	EV3	ST1	49.58	75.36
						ST2	49.58	174.89
						ST4	70.85	87.17
						SE1	58.11	58.11
						SE2	58.11	153.96
2	58.24	0.00	HL-93	73.73	EV3	ST1	49.70	75.52
						ST2	49.70	175.05
						ST4	71.05	87.36
						SE1	58.24	58.24
						SE2	58.24	154.09

NOTE: [1] " - " Indicates Uplift

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

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	0.00	2 0.400	0.000 MAX	HL-93	1.37	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	1.37	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.927 MAX	EV3		
			0.038 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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] By AASHTO 3.6.1.3.2 live load deflection is the larger of (design truck alone) and (25% design truck + design lane).

In the Load Type column, the former is called "HL-93" and the latter is called "LANE" where MAX means the highest downward deflection and MIN means the highest upward deflection.

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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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	D2 ksi	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	3935.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0
1	1		4.57	3935.3	178.9	72.0	215.8	0.0	12.39	EV3	9.56	0
1	2		9.14	3935.3	339.0	136.5	406.9	0.0	6.04	EV3	4.66	0
1	3		13.71	3935.3	480.5	193.4	573.4	0.0	3.95	EV3	3.05	0
1	4		18.28	3935.3	603.3	242.7	715.3	0.0	2.94	EV3	2.27	0
1	5		22.85	4775.4	707.1	284.4	831.5	0.0	3.11	EV3	2.40	0
1	6		27.43	4775.4	791.9	318.5	925.2	0.0	2.67	EV3	2.06	0
1	7		32.00	4775.4	858.0	345.1	995.9	0.0	2.39	EV3	1.84	0
1	8		36.57	4775.4	905.4	364.0	1045.1	0.0	2.22	EV3	1.71	0
1	9		41.14	4775.4	933.5	375.4	1071.0	0.0	2.13	EV3	1.64	0
1	10		45.71	4775.4	942.9	379.2	1079.6	0.0	2.10	EV3	1.62	0
1	11		50.28	4775.4	933.7	375.4	1071.0	0.0	2.13	EV3	1.64	0
1	12		54.85	4775.4	905.4	364.0	1045.1	0.0	2.22	EV3	1.71	0
1	13		59.42	4775.4	858.2	345.1	995.9	0.0	2.39	EV3	1.84	0
1	14		63.99	4775.4	792.3	318.5	925.2	0.0	2.67	EV3	2.06	0
1	15		68.56	4775.4	707.7	284.4	831.5	0.0	3.11	EV3	2.40	0
1	16		73.13	3935.3	603.7	242.7	715.3	0.0	2.94	EV3	2.27	0
1	17		77.70	3935.3	481.1	193.4	573.4	0.0	3.95	EV3	3.05	0
1	18		82.28	3935.3	339.8	136.5	406.9	0.0	6.04	EV3	4.66	0
1	19		86.85	3935.3	179.5	72.0	215.8	0.0	12.39	EV3	9.56	0
1	20		91.42	3935.3	0.0	0.0	0.0	0.0	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

[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 L	FROM 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	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	99.99		76.92
1	1		4.57	30.4	-2.59 2.85	-0.74 2.04	0.00 0.00	0.00 0.00	13.51		10.39
1	2		9.14	30.4	-4.90 5.40	-1.39 3.85	0.00 0.00	0.00 0.00	6.50		5.00
1	3		13.71	30.4	-6.95 7.65	-1.96 5.42	0.00 0.00	0.00 0.00	4.20		3.23
1	4		18.28	30.4	-8.72 9.61	-2.44 6.76	0.00 0.00	0.00 0.00	3.07		2.37
1	5		22.85	30.4	-9.51 7.73	-2.80 5.39	0.00 0.00	0.00 0.00	4.21		3.24
1	6		27.43	30.4	-10.65 8.65	-3.11 5.99	0.00 0.00	0.00 0.00	3.63		2.79
1	7		32.00	30.4	-11.54 9.38	-3.35 6.45	0.00 0.00	0.00 0.00	3.26		2.51
1	8		36.57	30.4	-12.17 9.89	-3.52 6.77	0.00 0.00	0.00 0.00	3.03		2.33
1	9		41.14	30.4	-12.55 10.20	-3.60 6.94	0.00 0.00	0.00 0.00	2.91		2.24
1	10		45.71	30.4	-12.68 10.30	-3.63 6.99	0.00 0.00	0.00 0.00	2.87		2.21
1	11		50.28	30.4	-12.55 10.20	-3.60 6.94	0.00 0.00	0.00 0.00	2.91		2.24
1	12		54.85	30.4	-12.17 9.89	-3.52 6.77	0.00 0.00	0.00 0.00	3.03		2.33
1	13		59.42	30.4	-11.54 9.38	-3.35 6.45	0.00 0.00	0.00 0.00	3.26		2.51
1	14		63.99	30.4	-10.65 8.66	-3.11 5.99	0.00 0.00	0.00 0.00	3.63		2.79
1	15		68.56	30.4	-9.51 7.73	-2.80 5.39	0.00 0.00	0.00 0.00	4.21		3.24
1	16		73.13	30.4	-8.73 9.61	-2.44 6.76	0.00 0.00	0.00 0.00	3.07		2.36
1	17		77.70	30.4	-6.95 7.66	-1.96 5.42	0.00 0.00	0.00 0.00	4.19		3.23
1	18		82.28	30.4	-4.91 5.41	-1.39 3.85	0.00 0.00	0.00 0.00	6.50		5.00
1	19		86.85	30.4	-2.59 2.86	-0.74 2.04	0.00 0.00	0.00 0.00	13.50		10.39
1	20		91.42	30.4	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	99.99		76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3
****

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	91.42	45.71	STRENGTH L.S.	2.10	1460.10
1	91.42	45.71	SERVICEBILITY	2.21	

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 2.096

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

SP NO	IN NO	D L	FROM 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	548.0	41.5	16.6	73.7	0.0	4.75	EV3	3.66
1	1		4.57	548.0	37.1	14.9	69.7	-2.1	5.10	EV3	3.94
1	2		9.14	548.0	33.0	13.3	65.8	-4.5	5.50	EV3	4.24
1	3		13.71	548.0	28.9	11.6	61.8	-7.1	5.94	EV3	4.58
1	4		18.28	548.0	24.8	10.0	57.8	-10.2	6.44	EV3	4.97
1	5		22.85	548.0	20.6	8.3	53.8	-14.1	7.03	EV3	5.42
1	6		27.43	548.0	16.5	6.6	49.8	-18.0	7.70	EV3	5.94
1	7		32.00	548.0	12.4	5.0	45.9	-22.0	8.49	EV3	6.55
1	8		36.57	548.0	8.2	3.3	41.9	-26.0	9.43	EV3	7.27
1	9		41.14	548.0	4.1	1.7	37.9	-29.9	10.56	EV3	8.15
1	10		45.71	548.0	0.0	0.0	33.9	-33.9	11.96	EV3	9.23
1	11		50.28	548.0	-4.1	-1.7	29.9	-37.9	10.56	EV3	8.15
1	12		54.85	548.0	-8.3	-3.3	26.0	-41.9	9.42	EV3	7.27
1	13		59.42	548.0	-12.4	-5.0	22.0	-45.9	8.49	EV3	6.55
1	14		63.99	548.0	-16.5	-6.6	18.0	-49.8	7.70	EV3	5.94
1	15		68.56	548.0	-20.7	-8.3	14.1	-53.8	7.02	EV3	5.42
1	16		73.13	548.0	-24.8	-10.0	10.2	-57.8	6.44	EV3	4.97
1	17		77.70	548.0	-28.9	-11.6	7.1	-61.8	5.94	EV3	4.58
1	18		82.28	548.0	-32.9	-13.3	4.5	-65.8	5.50	EV3	4.24
1	19		86.85	548.0	-37.2	-14.9	2.1	-69.7	5.10	EV3	3.94
1	20		91.42	548.0	-41.6	-16.6	0.0	-73.7	4.75	EV3	3.66

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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FILE NAME = C:\Users\jtzoc\Desktop\TEMP\5800 Ext G S2Rev_ev3.dat — PAGE 87

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	91.42	91.42	STRENGTH L.S.	4.74	99.71

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 4.738

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

TABLE 2.1.3.1=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	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.760	0.453
2	0.176	2.132
3	0.352	17.132
4	0.176	32.132
5	0.760	41.172

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor *** Kg < 10,000 In4 (4 x 10^9 mm4) OR
Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)

TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0		0.0		0.0	0.0
1	1		2.08	49.9	140.7		0.0		190.5	49.9
1	2		4.16	93.9	263.5		0.0		357.4	93.9
1	3		6.24	132.8	368.5		0.0		501.3	132.8
1	4		8.32	166.6	455.6		0.0		622.2	166.6
1	5		10.41	195.3	523.2		0.0		718.4	195.3
1	6		12.49	218.8	576.3		0.0		795.1	218.8
1	7		14.57	237.2	614.0		0.0		851.3	237.2
1	8		16.65	250.5	639.1		0.0		889.6	250.5
1	9		18.73	258.1	648.2		0.0		906.4	258.1
1	10		20.81	260.5	651.3		0.0		911.7	260.5
1	11		22.89	257.7	648.2		0.0		905.9	257.7
1	12		24.97	249.8	639.1		0.0		888.9	249.8
1	13		27.06	236.7	614.0		0.0		850.7	236.7
1	14		29.14	218.5	576.3		0.0		794.8	218.5
1	15		31.22	195.3	523.2		0.0		718.4	195.3
1	16		33.30	166.7	455.6		0.0		622.3	166.7
1	17		35.38	132.8	368.5		0.0		501.2	132.8
1	18		37.46	93.7	263.5		0.0		357.2	93.7
1	19		39.54	49.6	140.7		0.0		190.3	49.6
1	20		41.62	0.0	0.0		0.0		0.0	0.0

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0		0.00	25.8	79.8		0.0		105.6	25.8
1	1		2.08	22.6	75.1		-1.4		97.6	21.2
1	2		4.16	19.9	70.3		-3.6		90.2	16.4
1	3		6.24	17.5	65.5		-7.0		83.0	10.5
1	4		8.32	15.0	60.8		-10.4		75.8	4.6
1	5		10.41	12.5	56.0		-13.9		68.5	-1.3
1	6		12.49	10.1	51.2		-17.3		61.3	-7.2
1	7		14.57	7.6	46.5		-20.6		54.1	-13.0
1	8		16.65	5.2	41.7		-24.2		46.9	-19.0
1	9		18.73	2.3	36.8		-28.0		39.1	-25.6
1	10		20.81	-0.1	32.2		-32.2		-32.3	32.1
1	11		22.89	-2.6	28.0		-36.8		-39.3	25.4
1	12		24.97	-5.0	24.2		-41.7		-46.7	19.1
1	13		27.06	-7.5	20.6		-46.5		-54.0	13.1
1	14		29.14	-10.0	17.3		-51.2		-61.2	7.3
1	15		31.22	-12.4	13.9		-56.0		-68.4	1.4
1	16		33.30	-15.1	10.4		-60.8		-75.8	-4.6
1	17		35.38	-17.5	7.0		-65.5		-83.0	-10.5
1	18		37.46	-20.0	3.6		-70.3		-90.3	-16.4
1	19		39.54	-22.4	1.4		-75.1		-97.5	-21.1
1	20		41.62	-25.7	0.0		-79.8		-105.5	-25.7

TABLE 2.2.7.2=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	25.78	0.00	HL-93	79.82	MAX.V	ST1	23.20	32.23
						ST2	23.20	139.98
						ST4	38.67	38.67
						SE1	25.78	25.78
						SE2	25.78	129.54
2	25.66	0.00	HL-93	79.82	MAX.V	ST1	23.09	32.07
						ST2	23.09	139.83
						ST4	38.49	38.49
						SE1	25.66	25.66
						SE2	25.66	129.42

NOTE: [1] " - " Indicates Uplift

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

TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
(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	0.00	2 0.400	0.000 MAX	HL-93	0.62	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	0.62	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.374 MAX	EV3		
			0.038 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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 2.2.32.1=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 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	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0
1	1		2.08	33.00	TOP	-1.19	-3.35	0.00	6.96	EV3	5.37	0
				33.00	BOT	1.19	3.35	0.00	6.96	EV3	5.37	0
1	2		4.16	33.00	TOP	-2.24	-6.28	0.00	3.56	EV3	2.75	0
				33.00	BOT	2.24	6.28	0.00	3.56	EV3	2.75	0
1	3		6.24	33.00	TOP	-3.16	-8.78	0.00	2.45	EV3	1.89	0
				33.00	BOT	3.16	8.78	0.00	2.45	EV3	1.89	0
1	4		8.32	33.00	TOP	-3.97	-10.86	0.00	1.91	EV3	1.48	0
				33.00	BOT	3.97	10.86	0.00	1.91	EV3	1.48	0
1	5		10.41	33.00	TOP	-4.65	-12.47	0.00	1.62	EV3	1.25	0
				33.00	BOT	4.65	12.47	0.00	1.62	EV3	1.25	0
1	6		12.49	33.00	TOP	-5.21	-13.73	0.00	1.43	EV3	1.10	0
				33.00	BOT	5.21	13.73	0.00	1.43	EV3	1.10	0
1	7		14.57	33.00	TOP	-5.65	-14.63	0.00	1.31	EV3	1.01	0
				33.00	BOT	5.65	14.63	0.00	1.31	EV3	1.01	0
1	8		16.65	32.99	TOP	-5.97	-15.23	0.00	1.24	EV3	0.96	0
				32.99	BOT	5.97	15.23	0.00	1.24	EV3	0.96	0
1	9		18.73	29.40	TOP	-6.15	-15.45	0.00	1.04	EV3	0.80	0
				32.97	BOT	6.15	15.45	0.00	1.21	EV3	0.94	0
1	10		20.81	29.40	TOP	-6.21	-15.52	0.00	1.03	EV3	0.80	0
				32.97	BOT	6.21	15.52	0.00	1.20	EV3	0.93	0
1	11		22.89	29.40	TOP	-6.14	-15.45	0.00	1.04	EV3	0.80	0
				32.97	BOT	6.14	15.45	0.00	1.21	EV3	0.94	0
1	12		24.97	29.41	TOP	-5.95	-15.23	0.00	1.07	EV3	0.82	0
				32.99	BOT	5.95	15.23	0.00	1.24	EV3	0.96	0
1	13		27.06	29.42	TOP	-5.64	-14.63	0.00	1.13	EV3	0.87	0
				33.00	BOT	5.64	14.63	0.00	1.31	EV3	1.01	0
1	14		29.14	29.42	TOP	-5.21	-13.73	0.00	1.24	EV3	0.95	0
				33.00	BOT	5.21	13.73	0.00	1.43	EV3	1.10	0
1	15		31.22	29.42	TOP	-4.65	-12.47	0.00	1.40	EV3	1.08	0
				33.00	BOT	4.65	12.47	0.00	1.62	EV3	1.25	0
1	16		33.30	33.00	TOP	-3.97	-10.86	0.00	1.91	EV3	1.48	0
				33.00	BOT	3.97	10.86	0.00	1.91	EV3	1.48	0
1	17		35.38	33.00	TOP	-3.16	-8.78	0.00	2.45	EV3	1.89	0
				33.00	BOT	3.16	8.78	0.00	2.45	EV3	1.89	0
1	18		37.46	33.00	TOP	-2.23	-6.28	0.00	3.56	EV3	2.75	0
				33.00	BOT	2.23	6.28	0.00	3.56	EV3	2.75	0
1	19		39.54	33.00	TOP	-1.18	-3.35	0.00	6.97	EV3	5.37	0
				33.00	BOT	1.18	3.35	0.00	6.97	EV3	5.37	0
1	20		41.62	33.00	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

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

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TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D L	FROM 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 TYPE	INVENT. RATING FACTOR
1	0		0.00	26.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		2.08	26.4	-1.19	1.19	-3.35	3.35	0.00	0.00	7.52	5.79
1	2		4.16	26.4	-2.24	2.24	-6.28	6.28	0.00	0.00	3.85	2.96
1	3		6.24	26.4	-3.16	3.16	-8.78	8.78	0.00	0.00	2.65	2.04
1	4		8.32	26.4	-3.97	3.97	-10.86	10.86	0.00	0.00	2.07	1.59
1	5		10.41	26.4	-4.65	4.65	-12.47	12.47	0.00	0.00	1.74	1.34
1	6		12.49	26.4	-5.21	5.21	-13.73	13.73	0.00	0.00	1.54	1.19
1	7		14.57	26.4	-5.65	5.65	-14.63	14.63	0.00	0.00	1.42	1.09
1	8		16.65	26.4	-5.97	5.97	-15.23	15.23	0.00	0.00	1.34	1.03
1	9		18.73	26.4	-6.15	6.15	-15.45	15.45	0.00	0.00	1.31	1.01
1	10		20.81	26.4	-6.21	6.21	-15.52	15.52	0.00	0.00	1.30	1.00
1	11		22.89	26.4	-6.14	6.14	-15.45	15.45	0.00	0.00	1.31	1.01
1	12		24.97	26.4	-5.95	5.95	-15.23	15.23	0.00	0.00	1.34	1.03
1	13		27.06	26.4	-5.64	5.64	-14.63	14.63	0.00	0.00	1.42	1.09
1	14		29.14	26.4	-5.21	5.21	-13.73	13.73	0.00	0.00	1.54	1.19
1	15		31.22	26.4	-4.65	4.65	-12.47	12.47	0.00	0.00	1.74	1.34
1	16		33.30	26.4	-3.97	3.97	-10.86	10.86	0.00	0.00	2.07	1.59
1	17		35.38	26.4	-3.16	3.16	-8.78	8.78	0.00	0.00	2.65	2.04
1	18		37.46	26.4	-2.23	2.23	-6.28	6.28	0.00	0.00	3.85	2.96
1	19		39.54	26.4	-1.18	1.18	-3.35	3.35	0.00	0.00	7.52	5.79
1	20		41.62	26.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 2.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	41.62	20.81	STRENGTH L.S.	1.03	880.80
1	41.62	20.81	SERVICEBILITY	1.00	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 1.001

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TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D L	FROM 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	407.0	0.0	25.8	79.8	0.0	3.42	EV3	2.64
1	1		2.08	407.0	0.0	22.6	75.1	-1.4	3.68	EV3	2.84
1	2		4.16	407.0	0.0	19.9	70.3	-3.6	3.97	EV3	3.07
1	3		6.24	407.0	0.0	17.5	65.5	-7.0	4.30	EV3	3.32
1	4		8.32	407.0	0.0	15.0	60.8	-10.4	4.69	EV3	3.62
1	5		10.41	407.0	0.0	12.5	56.0	-13.9	5.13	EV3	3.96
1	6		12.49	407.0	0.0	10.1	51.2	-17.3	5.67	EV3	4.37
1	7		14.57	407.0	0.0	7.6	46.5	-20.6	6.31	EV3	4.86
1	8		16.65	407.0	0.0	5.2	41.7	-24.2	7.09	EV3	5.47
1	9		18.73	407.0	0.0	2.3	36.8	-28.0	8.13	EV3	6.27
1	10		20.81	407.0	0.0	-0.1	32.2	-32.2	9.37	EV3	7.22
1	11		22.89	407.0	0.0	-2.6	28.0	-36.8	8.12	EV3	6.27
1	12		24.97	407.0	0.0	-5.0	24.2	-41.7	7.09	EV3	5.47
1	13		27.06	407.0	0.0	-7.5	20.6	-46.5	6.31	EV3	4.87
1	14		29.14	407.0	0.0	-10.0	17.3	-51.2	5.67	EV3	4.37
1	15		31.22	407.0	0.0	-12.4	13.9	-56.0	5.14	EV3	3.96
1	16		33.30	407.0	0.0	-15.1	10.4	-60.8	4.69	EV3	3.62
1	17		35.38	407.0	0.0	-17.5	7.0	-65.5	4.30	EV3	3.32
1	18		37.46	407.0	0.0	-20.0	3.6	-70.3	3.97	EV3	3.07
1	19		39.54	407.0	0.0	-22.4	1.4	-75.1	3.68	EV3	2.84
1	20		41.62	407.0	0.0	-25.7	0.0	-79.8	3.42	EV3	2.64

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 2.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	41.62	0.00	STRENGTH L.S.	3.41	107.95

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 3.412

TABLE 0.6.2.2 STATE VEHICLE LOADING -- Load Type D & M

	LOAD TYPE D	LOAD TYPE M
STATE	DUMP TRUCK	MAXIMUM ALLOWABLE TRUCK
EV3	2D, 3D or defined by user	Types 3, 3S2, 3-3 or defined by user

NOTE: [1] Load type D is used for the state dump truck loading

[2] The loading configurations for 2D (2 axles) and 3D (3 axles) are shown in the MERLIN DASH USER'S MANUAL. Default dump truck = 2D if the input truck type is not defined.

[3] If you wish to build up your own dump truck data, please contact BEST Center (Bridge Engineering Software Center) at (301) 405-2011.

[4] Load type M is used for the state maximum allowable truck

[5] Typical legal load types (for rating) are shown in the Plate 11 of "Manual for Condition Evaluation of Bridges, AASHTO (1994)"

[6] For creating your own maximum allowable truck data please refer to Note [3]

TABLE 2.1.3.1=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)	V O L U M E (yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	849.8	31.5	127.5	31.3	158.78

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

[2] Superimposed dead load not included

1	0.380	0.453
2	0.176	13.448
3	0.176	28.448
4	0.380	41.172

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT	SPACING DIST	AXLE WHT	SPACING DIST	AXLE WHT	SPACING DIST	AXLE WHT	SPACING DIST

(k)	(ft)	(k)	(ft)	(k)	(ft)	(k)	(ft)
24.00	15.00	31.00	4.00	31.00	0.00		

SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3
****

NO. OF AXLES = 3

AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)	AXLE WHT (k)	SPACING DIST (ft)
24.00	15.00	31.00	4.00	31.00	0.00		

TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0		0.0		0.0	0.0
1	1		2.08	52.6	105.0		0.0		157.7	52.6
1	2		4.16	99.6	196.8		0.0		296.4	99.6
1	3		6.24	141.1	275.1		0.0		416.3	141.1
1	4		8.32	177.2	340.2		0.0		517.3	177.2
1	5		10.41	207.7	390.6		0.0		598.3	207.7
1	6		12.49	232.7	430.3		0.0		663.0	232.7
1	7		14.57	252.1	458.5		0.0		710.6	252.1
1	8		16.65	265.8	477.2		0.0		743.0	265.8
1	9		18.73	274.1	484.0		0.0		758.1	274.1
1	10		20.81	276.8	486.3		0.0		763.1	276.8
1	11		22.89	274.0	484.0		0.0		758.1	274.0
1	12		24.97	265.8	477.2		0.0		743.0	265.8
1	13		27.06	252.1	458.5		0.0		710.6	252.1
1	14		29.14	232.8	430.3		0.0		663.1	232.8
1	15		31.22	207.7	390.6		0.0		598.4	207.7
1	16		33.30	177.2	340.2		0.0		517.4	177.2
1	17		35.38	141.2	275.1		0.0		416.3	141.2
1	18		37.46	99.6	196.8		0.0		296.4	99.6
1	19		39.54	52.7	105.0		0.0		157.7	52.7
1	20		41.62	0.0	0.0		0.0		0.0	0.0

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0		0.00	26.9	53.7		0.0		80.6	26.9
1	1		2.08	23.9	50.5		-0.9		74.4	23.0
1	2		4.16	21.3	47.3		-2.4		68.5	18.9
1	3		6.24	18.6	44.1		-4.7		62.7	13.9
1	4		8.32	16.0	40.8		-7.0		56.8	9.0
1	5		10.41	13.4	37.6		-9.3		51.0	4.0
1	6		12.49	10.7	34.4		-11.6		45.2	-0.9
1	7		14.57	7.9	31.2		-13.8		39.1	-5.9
1	8		16.65	5.3	28.0		-16.2		33.3	-11.0
1	9		18.73	2.6	24.7		-18.8		27.3	-16.2
1	10		20.81	0.0	21.6		-21.6		-21.6	21.6
1	11		22.89	-2.6	18.8		-24.7		-27.3	16.2
1	12		24.97	-5.3	16.2		-28.0		-33.3	11.0
1	13		27.06	-7.9	13.8		-31.2		-39.1	5.9
1	14		29.14	-10.7	11.6		-34.4		-45.2	0.9
1	15		31.22	-13.4	9.3		-37.6		-51.0	-4.0
1	16		33.30	-16.0	7.0		-40.8		-56.8	-9.0
1	17		35.38	-18.6	4.7		-44.1		-62.7	-13.9
1	18		37.46	-21.3	2.4		-47.3		-68.5	-18.9
1	19		39.54	-23.9	0.9		-50.5		-74.4	-23.0
1	20		41.62	-26.9	0.0		-53.7		-80.6	-26.9

TABLE 2.2.7.2=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	26.91	0.00	HL-93	53.66	MAX.V	ST1	24.22	33.64
						ST2	24.22	106.08
						ST4	40.37	40.37
						SE1	26.91	26.91
						SE2	26.91	96.67
2	26.91	0.00	HL-93	53.66	MAX.V	ST1	24.22	33.64
						ST2	24.22	106.08
						ST4	40.37	40.37
						SE1	26.91	26.91
						SE2	26.91	96.67

NOTE: [1] " - " Indicates Uplift

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

TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****
(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	0.00	2 0.400	0.000 MAX	HL-93	0.62	0.00000
			0.038 MIN			
			0.000 MAX	LANE		
			0.038 MIN	LANE		
		3 0.600	0.000 MAX	HL-93	0.62	0.00000
			0.057 MIN			
			0.000 MAX	LANE		
			0.057 MIN	LANE		
		2 0.400	-0.374 MAX	EV3		
			0.038 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

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

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

[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 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft) or STRESS	MOMENT CAP. (k-ft)		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	33.00	TOP	0.00		0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00		0.00	0.00	99.99	HL-93	99.99	0
1	1		2.08	33.00	TOP	-1.25		-2.50	0.00	9.30	EV3	7.18	0
				33.00	BOT	1.25		2.50	0.00	9.30	EV3	7.18	0
1	2		4.16	33.00	TOP	-2.37		-4.69	0.00	4.75	EV3	3.66	0
				33.00	BOT	2.37		4.69	0.00	4.75	EV3	3.66	0
1	3		6.24	33.00	TOP	-3.36		-6.56	0.00	3.25	EV3	2.51	0
				33.00	BOT	3.36		6.56	0.00	3.25	EV3	2.51	0
1	4		8.32	33.00	TOP	-4.22		-8.11	0.00	2.53	EV3	1.95	0
				33.00	BOT	4.22		8.11	0.00	2.53	EV3	1.95	0
1	5		10.41	33.00	TOP	-4.95		-9.31	0.00	2.13	EV3	1.65	0
				33.00	BOT	4.95		9.31	0.00	2.13	EV3	1.65	0
1	6		12.49	33.00	TOP	-5.55		-10.25	0.00	1.88	EV3	1.45	0
				33.00	BOT	5.55		10.25	0.00	1.88	EV3	1.45	0
1	7		14.57	29.37	TOP	-6.01		-10.92	0.00	1.48	EV3	1.14	0
				33.00	BOT	6.01		10.92	0.00	1.73	EV3	1.33	0
1	8		16.65	29.37	TOP	-6.33		-11.37	0.00	1.40	EV3	1.08	0
				33.00	BOT	6.33		11.37	0.00	1.63	EV3	1.26	0
1	9		18.73	29.37	TOP	-6.53		-11.53	0.00	1.36	EV3	1.05	0
				33.00	BOT	6.53		11.53	0.00	1.60	EV3	1.23	0
1	10		20.81	29.37	TOP	-6.60		-11.59	0.00	1.35	EV3	1.04	0
				33.00	BOT	6.60		11.59	0.00	1.58	EV3	1.22	0
1	11		22.89	29.37	TOP	-6.53		-11.53	0.00	1.36	EV3	1.05	0
				33.00	BOT	6.53		11.53	0.00	1.60	EV3	1.23	0
1	12		24.97	29.37	TOP	-6.33		-11.37	0.00	1.40	EV3	1.08	0
				33.00	BOT	6.33		11.37	0.00	1.63	EV3	1.26	0
1	13		27.06	29.37	TOP	-6.01		-10.92	0.00	1.48	EV3	1.14	0
				33.00	BOT	6.01		10.92	0.00	1.73	EV3	1.33	0
1	14		29.14	33.00	TOP	-5.55		-10.25	0.00	1.88	EV3	1.45	0
				33.00	BOT	5.55		10.25	0.00	1.88	EV3	1.45	0
1	15		31.22	33.00	TOP	-4.95		-9.31	0.00	2.13	EV3	1.65	0
				33.00	BOT	4.95		9.31	0.00	2.13	EV3	1.65	0
1	16		33.30	33.00	TOP	-4.22		-8.11	0.00	2.53	EV3	1.95	0
				33.00	BOT	4.22		8.11	0.00	2.53	EV3	1.95	0
1	17		35.38	33.00	TOP	-3.36		-6.56	0.00	3.25	EV3	2.51	0
				33.00	BOT	3.36		6.56	0.00	3.25	EV3	2.51	0
1	18		37.46	33.00	TOP	-2.37		-4.69	0.00	4.75	EV3	3.66	0
				33.00	BOT	2.37		4.69	0.00	4.75	EV3	3.66	0
1	19		39.54	33.00	TOP	-1.25		-2.50	0.00	9.30	EV3	7.18	0
				33.00	BOT	1.25		2.50	0.00	9.30	EV3	7.18	0
1	20		41.62	33.00	TOP	0.00		0.00	0.00	99.99	HL-93	99.99	0
				33.00	BOT	0.00		0.00	0.00	99.99	HL-93	99.99	0

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
DRI: Factor related to Ductility, Redundancy and
operational Importance
Maximum for Strength and Minimum for Other Limit State
(AASHTO LRFD Art. 1.3.2.1)

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

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TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
****

SP NO	IN NO	D L	FROM 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	26.4	0.00	0.00	0.00	0.00	0.00	99.99	76.92
1	1		2.08	26.4	-1.25	1.25	-2.50	2.50	0.00	0.00	10.05	7.73
1	2		4.16	26.4	-2.37	2.37	-4.69	4.69	0.00	0.00	5.12	3.94
1	3		6.24	26.4	-3.36	3.36	-6.56	6.56	0.00	0.00	3.51	2.70
1	4		8.32	26.4	-4.22	4.22	-8.11	8.11	0.00	0.00	2.74	2.10
1	5		10.41	26.4	-4.95	4.95	-9.31	9.31	0.00	0.00	2.30	1.77
1	6		12.49	26.4	-5.55	5.55	-10.25	10.25	0.00	0.00	2.03	1.56
1	7		14.57	26.4	-6.01	6.01	-10.92	10.92	0.00	0.00	1.87	1.44
1	8		16.65	26.4	-6.33	6.33	-11.37	11.37	0.00	0.00	1.76	1.36
1	9		18.73	26.4	-6.53	6.53	-11.53	11.53	0.00	0.00	1.72	1.33
1	10		20.81	26.4	-6.60	6.60	-11.59	11.59	0.00	0.00	1.71	1.31
1	11		22.89	26.4	-6.53	6.53	-11.53	11.53	0.00	0.00	1.72	1.33
1	12		24.97	26.4	-6.33	6.33	-11.37	11.37	0.00	0.00	1.76	1.36
1	13		27.06	26.4	-6.01	6.01	-10.92	10.92	0.00	0.00	1.87	1.44
1	14		29.14	26.4	-5.55	5.55	-10.25	10.25	0.00	0.00	2.03	1.56
1	15		31.22	26.4	-4.95	4.95	-9.31	9.31	0.00	0.00	2.30	1.77
1	16		33.30	26.4	-4.22	4.22	-8.11	8.11	0.00	0.00	2.74	2.10
1	17		35.38	26.4	-3.36	3.36	-6.56	6.56	0.00	0.00	3.51	2.70
1	18		37.46	26.4	-2.37	2.37	-4.69	4.69	0.00	0.00	5.12	3.94
1	19		39.54	26.4	-1.25	1.25	-2.50	2.50	0.00	0.00	10.05	7.73
1	20		41.62	26.4	0.00	0.00	0.00	0.00	0.00	0.00	99.99	76.92

Please read NOTES on the following page

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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 99.99 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

TABLE 2.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	41.62	20.81	STRENGTH L.S.	1.35	657.67
1	41.62	20.81	SERVICEBILITY	1.31	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 2398
Legal Load Rating= 1.315

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TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
****

SP NO	IN NO	D L	FROM 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	407.0	0.0	26.9	53.7	0.0	5.06	EV3	3.90
1	1		2.08	407.0	0.0	23.9	50.5	-0.9	5.45	EV3	4.20
1	2		4.16	407.0	0.0	21.3	47.3	-2.4	5.88	EV3	4.54
1	3		6.24	407.0	0.0	18.6	44.1	-4.7	6.37	EV3	4.92
1	4		8.32	407.0	0.0	16.0	40.8	-7.0	6.95	EV3	5.36
1	5		10.41	407.0	0.0	13.4	37.6	-9.3	7.61	EV3	5.87
1	6		12.49	407.0	0.0	10.7	34.4	-11.6	8.41	EV3	6.49
1	7		14.57	407.0	0.0	7.9	31.2	-13.8	9.37	EV3	7.23
1	8		16.65	407.0	0.0	5.3	28.0	-16.2	10.54	EV3	8.13
1	9		18.73	407.0	0.0	2.6	24.7	-18.8	12.08	EV3	9.32
1	10		20.81	407.0	0.0	0.0	21.6	-21.6	13.94	EV3	10.75
1	11		22.89	407.0	0.0	-2.6	18.8	-24.7	12.08	EV3	9.32
1	12		24.97	407.0	0.0	-5.3	16.2	-28.0	10.54	EV3	8.13
1	13		27.06	407.0	0.0	-7.9	13.8	-31.2	9.37	EV3	7.23
1	14		29.14	407.0	0.0	-10.7	11.6	-34.4	8.41	EV3	6.49
1	15		31.22	407.0	0.0	-13.4	9.3	-37.6	7.61	EV3	5.87
1	16		33.30	407.0	0.0	-16.0	7.0	-40.8	6.95	EV3	5.36
1	17		35.38	407.0	0.0	-18.6	4.7	-44.1	6.37	EV3	4.92
1	18		37.46	407.0	0.0	-21.3	2.4	-47.3	5.88	EV3	4.54
1	19		39.54	407.0	0.0	-23.9	0.9	-50.5	5.45	EV3	4.20
1	20		41.62	407.0	0.0	-26.9	0.0	-53.7	5.06	EV3	3.90

The above rating factors are incorrectly calculated using the HL-93 Operating & Inventory live load factors instead of the appropriate live load factor for the EV live load. See post-processed EV rating spreadsheet for correct EV rating factors.

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TABLE 2.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

SPAN NO.	SPAN LENGTH (ft)	CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	41.62	41.62	STRENGTH L.S.	5.05	72.57

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.352 & ADTT = 2398
Legal Load Rating= 5.052

Span 1 (4) WF 36 x 194

- haunch - field measured & averaged

$(2\frac{1}{2} + 2\frac{1}{4})\frac{1}{2} = 2.375"$ (int.) & 2.5" (ext.)

For Dead Load $h = 2.5"$, $b_{e_{int}} = b_f + 2\times 3" = 12.12 + 6 = 18.12"$, $h_{up} = \frac{2.5}{1.625} = 0.875"$

- assumed water proofing membrane 0.250 after rehab (7 of 12)
- curb (+soffit for Spans 1 & 3, calculation on page 8 of 9)

slope $1/4"/12" = 0.0208$

$\text{Area}_{curb} = \frac{1}{2}\left(12\frac{3}{4} + 9 + 3.25\right)(22)\frac{1}{144} = 1.91\ ft^2$

- Diaphragm

Concrete end diaphragm: Top of channel elev same as top of bf.

$\text{Weight}_{concrete} = 3.75 \times (2.5 - 1.26)\,0.150\,(13.08') = 0.064$ kips (6 of C15x50, L of C15x50)

18 L 42.7, $L = 7.5' - 2\left(\frac{3}{12}\right) = 7.0'$ (intermediate)

15 L 50, $L = 13' - 6\frac{15}{16}" - 2\left(\frac{3}{12}\right) = 13.08'$ (end)

PL (Intermed.), $H = (36.48" - 2\times 1.26") - 2\cdot(3) = 28"$, 30" - field measured

$t_p = 0.5"$, $W_p = 6"$ (Sht 9 of 10 for #5800 & #1427)

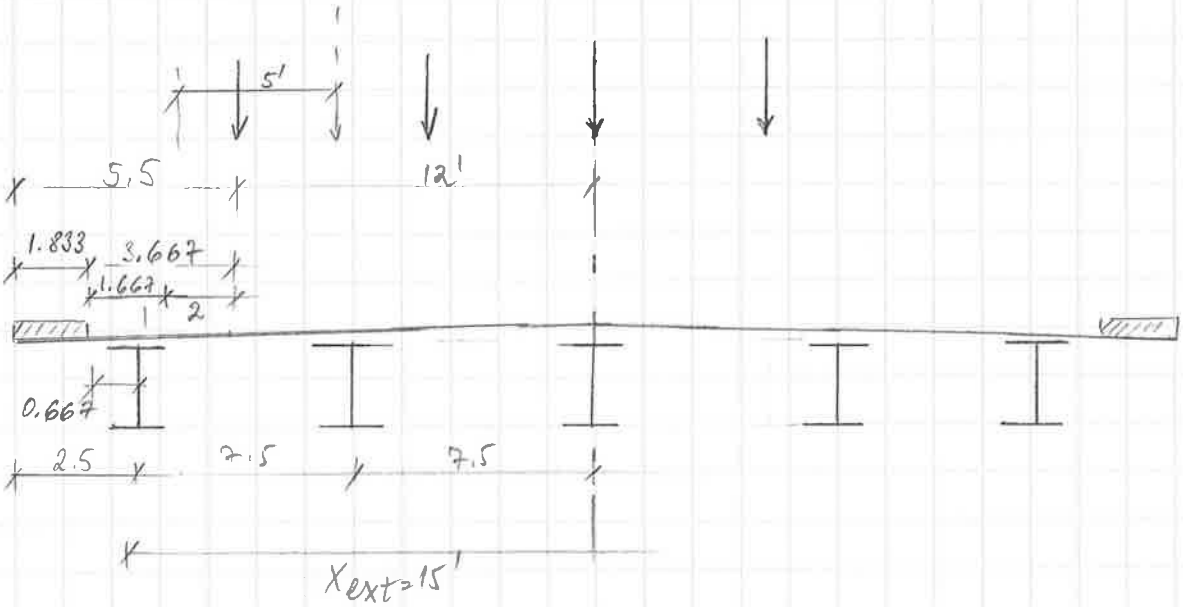
PL (end) $H = 20"$ assumed conservatively based on ↑

$t_p = 0.5"$, $W_p = 6"$

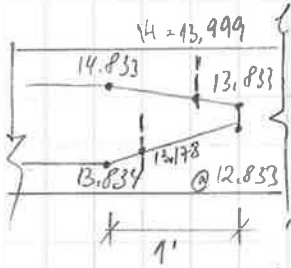
$\text{Weight}_{PL} = 0.5(6)(20)\frac{1}{1728}(0.49)(2) = 0.084$ k

$W_{Diagh.} = 0.696 + 0.064 = 0.76$ k

Rigid body Rotation and Lever Rule diagram



Span 1(4) W 36x194 + Cover PL 11"x1 1/16" Ext., L = 47 ft
+ Cover PL 11"x1 1/4" Int., L = 49 ft



ext. beam @ points of interest

Lever R ext = 47 - 2 = 45', S=16 for fatigue & strength

Lever R int = 49 - 2 = 47', S=17 for fatigue, S=20 for strength

- Av. depth = $\frac{1}{5(74.67)}\{2[36.48(74.667-45)+37.54(45)]+3[36.48(74.667-47)+37.73(47)]\}$
  $= 37.21$ in
- Av. Area = $\frac{1}{5(74.67)}\{2[57.11(74.67-45)+68.80(45)]+3[57.4(74.67-47)+70.86(47)]\} = (9580.56+14731.45)\frac{1}{5.74.67} = 65.12$ in^2
- Av. $I_o = \frac{1}{5(74.67)}\{2[12103(74.67-45)+15522.7(45)]+3[12103(74.67-47)+16[illegible]]\}$ = 4417.99 in^4

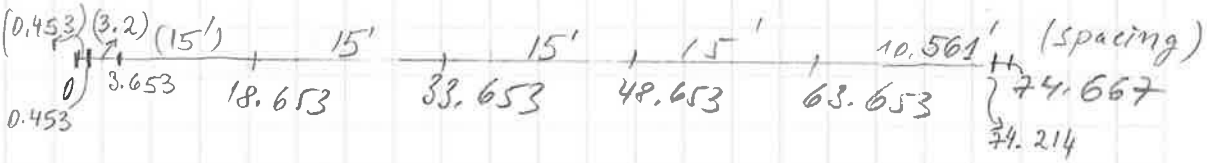
• Av. $\bar{y} = \frac{1}{5(74.67)}\{2[18.24(74.67-45)+16.11(45)]+3[18.24(74.67-47)+15.83(47)]\} = 16.824$

• $e_g = \frac{7}{2}'' + (h - OCT - t_f) + (d_{av} - \bar{y}) = \frac{7}{2} + (2.5 - 0.875 - 1.26) + 37.21 - 16.82 =$

$= 24.26$ in

Interior Beam

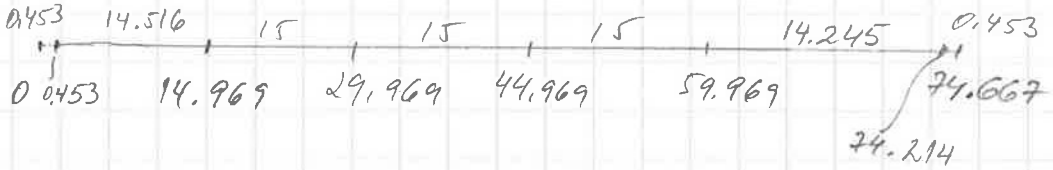
No. 19



$14' - 11\frac{5}{8}'' - 7.5' \, tg\, 56.464 = 14.969 - 11.316 = 3.653$

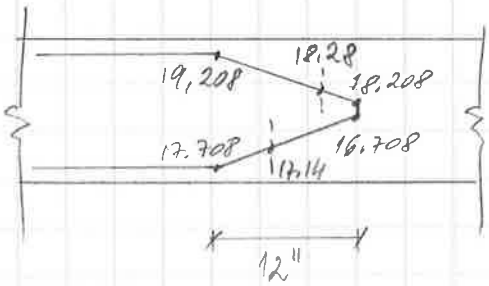
Exterior Beam

No. 20



Span 2 - W36x280 + Cover Pl $15'' \times 1\frac{3}{16}''$ - Ext. Beam, L = 55 ft

+ Cover Pl $15'' \times 1\frac{1}{2}''$ - Int. Beam, L = 58 ft



Ext. Beam points of interest from bearings to S = 20, ok for fatigue & IR for Mom. @ midspan & Cover Pl transition

Int. Beam points S = 16, ok for fatigue & IR for Moment @ midspan & Cover Pl transition

$L_{\text{cover Pl}}^{\text{ext.}} = 55' - 2' = 53'$

$L_{\text{cover Pl}}^{\text{Int}} = 58' - 2' = 56'$

$91.4167/20 = 4.57'$ (Ext), $4.57' \times 4 = 18.28'$ - check point for Ext. Beam (10 y. remaining) for fatigue

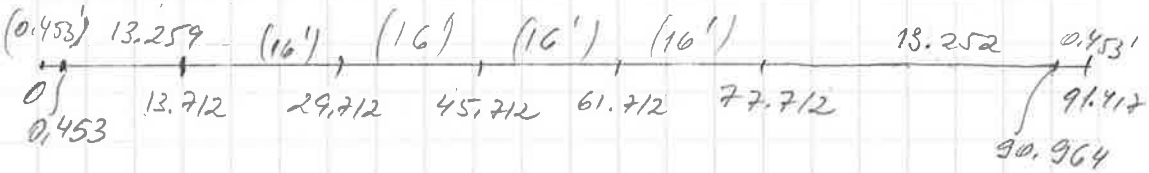
$91.4167/16 = 5.71'$ (Int.), $5.71 \times 3 = 17.14'$ - check point for Int. Beam (14 y. remaining) for fatigue

- Av. depth $= \frac{1}{(91.417)5}\{2[36.50(91.417-53)+37.69(53)]+3[36.50(91.417-56)+38.00(56)]\} = 37.33$ in
- Av. Area $= \frac{1}{(91.417)5}\{2[82.32(91.42-53)+100.13(53)]+3[82.32(91.42-56)+104.82(56)]\} = 94.72$ in^2
- Av. $I_o = \frac{1}{91.417(5)}\{2[18819(91.42-53)+24020.93(53)]+3[18819(91.42-56)+25202.14(56)]\} = 22\,371.48$ in^4
- Av. $\bar{y} = \frac{1}{91.417(5)}\{2[18.25(91.42-53)+16.09(53)]+3[18.25(91.42-56)+5(15.67$

= 16.88 in

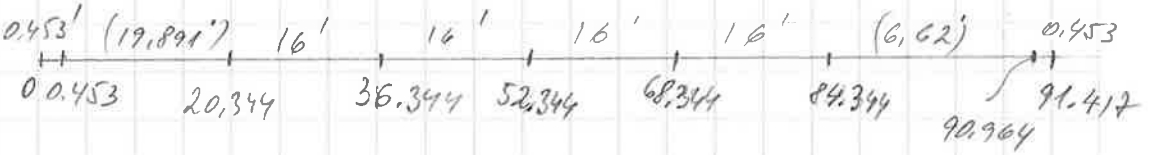
Interior Beam

No. 23



Exterior Beam

No. 25



$$e_g = \frac{7}{2} + (37.33 - 16.8) + (2.5 - 1.57 - 1.125)$$

$$e_g = \frac{7}{2} + (37.33 - 16.8) + 0$$

$e_g = 24.03''$

2 = Over Camber Toe.

t_f = but not less than zero (0)

$h_{Mercer} = 2.5 - 1.125 = 1.375$ but not less than t_f

Over Camber Toe

Therefore use 1.57"

Span 3(6) - non-composite

W36x150

Lever Rule & open steel deck case in calculating Distribution Factors:

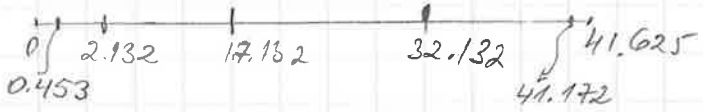
Lever Rule , Lever Rule formulas

Interior Beam, S=7.5', $d_c \geq 0$

No. 29

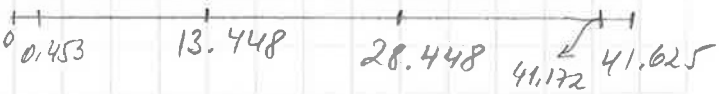
1 Lane Loaded (1 - 3/5)

2 Lanes Loaded ($\frac{3}{2}$ - 5/5)



Exterior Beam, Lever Rule, RBR / section "a", Tables 4.6.2.2.2

No. 30



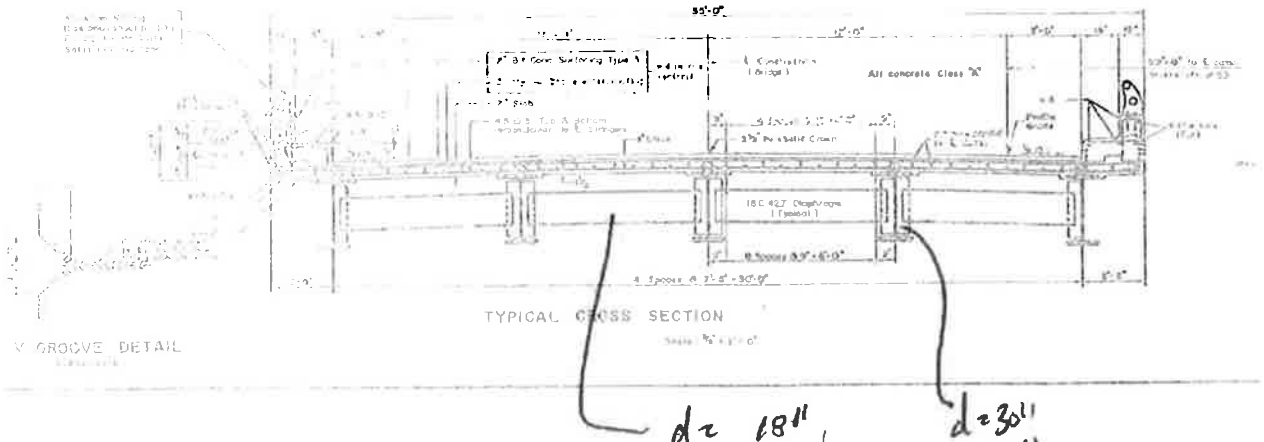
Haunch

h=2.5" for DLoad

Job #: 5800 - NB

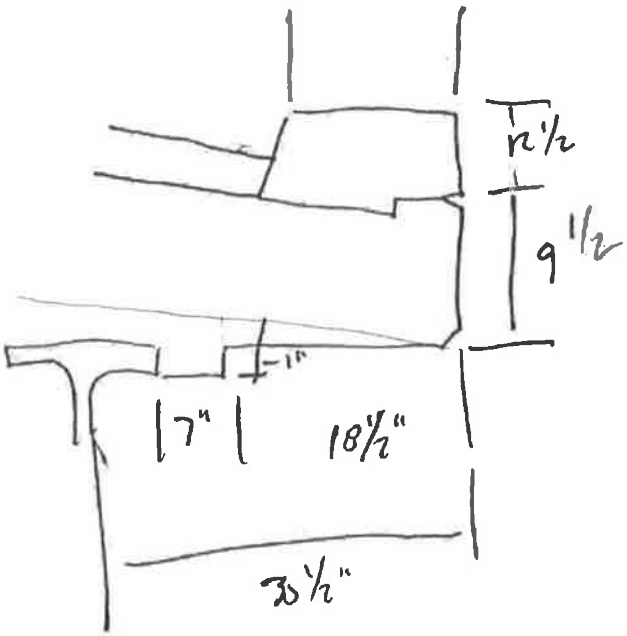
Field Sketches Span 1, Ext.

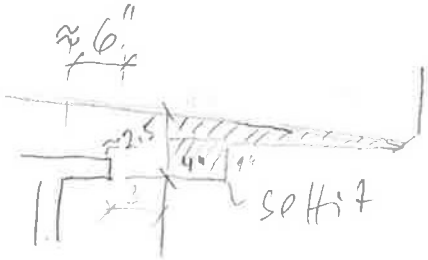
Before Going to Field:



d = 18"
w = 3¾ ✓
tf = .547"

d = 30"
w = 6"
t = .519, .516





$A_{soffit} = 4"(1") + (11.5")(30" - 6" - 3")\frac{1}{2} =$

$= 19.75 in^2$

$19.75 in^2 = 2.5"(x)$

$x = 7.9"$ → input in DL calc. sheet

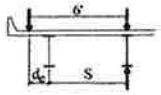
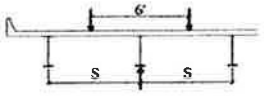
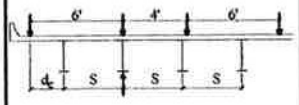
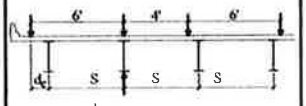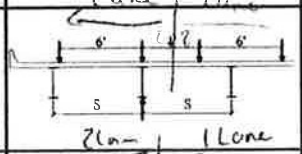
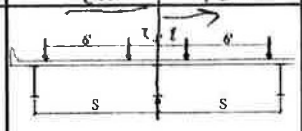
B4 LEVER RULE FORMULA

Tables B4-1 and B4-2 may be used to perform lever rule computations. The following assumptions were used in developing these tables and should be considered when using the listed values for analysis:

- two-foot spacing between the outer wheel centroid and the curb/barrier
- six-foot axle gage
- four-foot between vehicles.

The limitations of the equations are illustrated in the tables. Where the limitations are not met, conventional lever rule computations apply.

Table B4-1. Lever Rule Distribution of Live Loads per Lane Longitudinal Beams for Slab-on-Beam and Box-Beam Bridges for Interior Girders

Girder Location	Number of Loaded Lanes	Distribution Factor	Range of Application	Loading Diagram	Number of Wheels to Beam
Interior	1	$\frac{1}{2}$	$S \le 6\,ft$ $d_e \ge 0$		1
		$1-\frac{3}{S}$	$S > 6\,ft$ $d_e \ge 0$		2
	2 or more	$\frac{1}{2}$	$S \le 4\,ft$ $d_e \ge 0$		1
		$1-\frac{2}{S}$	$4 < S \le 6\,ft$ $d_e \ge 0$		2
		$\frac{3}{2}-\frac{5}{S}$	$6 < S \le 10\,ft$ $d_e \ge 0$		3
		$2-\frac{10}{S}$	$10 < S \le 16\,ft$ $d_e \ge 0$		4
	$d_e < 0$	Use lever rule and manually place the vehicle for critical effect on the first interior beam			varies

1960

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
ASD5	WF	WF 36x12	36 x	12	57.11	36.48	0.77	12.12	1.26	194.00	12103	663.6	14.56	355.4	58.7	2.49
LRFD3	W	W36x194			57.00	36.50	0.77	12.10	1.26	194.00	12100	664	14.6	375	61.9	2.56
					-0.19%	0.05%	**-0.65%**	-0.14%	0.00%	0.00%	-0.03%	0.06%	0.27%	5.23%	5.17%	2.73%
ASD5	WF	WF 36x16.5	36 x	16 1/2	82.32	36.50	0.89	16.60	1.57	280.00	18819	1031.2	15.12	1128	135.9	3.70
LRFD3	W	W36x280			82.40	36.50	0.89	16.60	1.57	280.00	18900	1030	15.1	1200	144	3.81
					0.10%	0.00%	0.00%	0.03%	0.00%	0.00%	0.43%	-0.12%	-0.13%	6.04%	5.63%	2.89%
ASD5	WF	WF 36x12	36 x	12	44.16	35.84	0.63	11.97	0.94	150.00	9012.1	502.9	14.29	250.4	41.8	2.38
LRFD3	W	W36x150			44.20	35.90	0.63	12.00	0.94	150.00	9040	504	14.3	270	45.1	2.47
					0.09%	0.17%	0.00%	0.23%	0.00%	0.00%	0.31%	0.22%	0.07%	7.26%	7.32%	3.64%

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.

Span 2 - Equivalent Plate Girder Sections:

Objective:

The rolled beam shape WF36x380 for span 2 is not available in the current Merlin Dash v 10.10 section database, therefore equivalent plate girder sections have been developed and used as a substitute in the Merlin Dash LRFR rating software. Four equivalent plate girder sections are required as follows: the ends of the interior beam, the midspan of the interior beam, the ends of the exterior beam, and the midspan of the exterior beam. Since the midspan sections have bottom cover plates and the Merlin Dash software is limited to defining a plate girder section to only a top flange, web, and bottom flange, an equivalent bottom flange has been determined for each midspan section to represent a combined bottom flange and bottom cover plate.

INTERIOR BEAM WITH COVER PLATE AT MIDSPAN - SECTION PROPERTIES

WF36x380 W/ 15" x 1 1/2" COVER PLATE									
Section	b	h	Area (A)	Centriod (y)	Ay	I_0	d	Ad^2	I
	in	in	in^2	in	in^3	in^4	in	in^4	in^4
Top Flange	16.595	1.570	26.05	37.22	969.61	5.35	21.57	12122.50	12127.85
Web	0.885	33.360	29.52	19.75	583.09	2738.04	4.11	497.59	3235.63
Bottom Flange	16.595	1.570	26.05	2.29	59.53	5.35	13.36	4650.14	4655.49
Bottom Cover Plate	15.000	1.500	22.50	0.75	16.88	4.22	14.89	4991.62	4995.84
		ΣA	104.13	ΣAy	1629.11			ΣI	**25014.82**

Y = **15.64** in Neutral Axis
d_b = 38.00 in Depth of Beam

A_{eq} = 48.554 in^2 Total Area of Bottom Flange & Cover Plate
h_{eq} = 3.070 in Total Height of Bottom Flange & Cover Plate and Height of Equivalent Bottom Flange
$b_{eq} = A_{eq} / h_{eq}$ = 15.816 in Width of Equivalent Bottom Flange

EQUIVALENT PLATE GIRDER									
Section	b	h	Area (A)	Centriod (y)	Ay	I_0	d	Ad^2	I
	in	in	in^2	in	in^3	in^4	in	in^4	in^4
Top Flange	16.595	1.570	26.05	37.22	969.61	5.35	21.59	12142.93	12148.28
Web	0.885	33.360	29.52	19.75	583.09	2738.04	4.12	502.01	3240.05
Eq. Bottom Flange	15.816	3.070	48.56	1.54	74.53	38.14	14.09	9641.56	9679.69
		ΣA	104.13	ΣAy	1627.23			ΣI	**25068.02**

Y = **15.63** in Neutral Axis
d_b = 38.00 in Depth of Beam

EXTERIOR BEAM WITH COVER PLATE AT MIDSPAN - SECTION PROPERTIES

WF36x380 W/ 15" x 1 3/16" COVER PLATE									
Section	b	h	Area (A)	Centriod (y)	Ay	I_0	d	Ad^2	I
	in	in	in^2	in	in^3	in^4	in	in^4	in^4
Top Flange	16.595	1.570	26.05	36.90	961.46	5.35	20.84	11315.78	11321.14
Web	0.885	33.360	29.52	19.44	573.86	2738.04	3.38	336.35	3074.39
Bottom Flange	16.595	1.570	26.05	1.97	51.39	5.35	14.09	5172.26	5177.62
Bottom Cover Plate	15.000	1.188	17.81	0.59	10.58	2.09	15.47	4262.05	4264.14
		ΣA	99.44	ΣAy	1597.30			ΣI	**23837.29**

Y = **16.06** in Neutral Axis
d_b = 37.69 in Depth of Beam

A_{eq} = 43.867 in^2 Total Area of Bottom Flange & Cover Plate
h_{eq} = 2.758 in Total Height of Bottom Flange & Cover Plate and Height of Equivalent Bottom Flange
$b_{eq} = A_{eq} / h_{eq}$ = 15.908 in Width of Equivalent Bottom Flange

EQUIVALENT PLATE GIRDER									
Section	b	h	Area (A)	Centriod (y)	Ay	I_0	d	Ad^2	I
	in	in	in^2	in	in^3	in^4	in	in^4	in^4
Top Flange	16.595	1.570	26.05	36.90	961.48	5.35	20.86	11333.37	11338.72
Web	0.885	33.360	29.52	19.44	573.88	2738.04	3.39	339.58	3077.63
Eq. Bottom Flange	15.908	2.758	43.87	1.38	60.50	27.81	14.67	9438.94	9466.75
		ΣA	99.45	ΣAy	1595.86			ΣI	**23883.10**

Y = **16.05** in Neutral Axis
d_b = 37.69 in Depth of Beam

Objective, General Information, & POIs:

Objective:

EV rating factors are taken from MDash Tables 1.2.32.3D: Bridge Moment Rating for Max Truck and Table 1.2.33.3D: Bridge Shear Rating Information for Max Truck, however these tables only show the governing emergency vehicle Strength I & Service II load ratings per span, for moments and shear respectively. Therefore the load ratings for non-governing points of interest (POI) are taken from MDash Tables 1.2.32.1: Max Strength for Moment Rating, 1.2.32.2: Serviceability Strength for Moment Rating, and 1.2.33.1: Max Strength for Shear Rating, however these tables incorrectly use the live load factor for HL-93 Inventory & Operating to determine rating factors.

The unfactored dead loads and live loads and capacities from Tables 1.2.32.1, 1.2.32.2, and 1.2.33.1 and the correct live load factors for the emergency vehicles will be used in post-processing to determine the appropriate load rating factors for all POIs.

General Information:

Governing Condition Factor	φc	=	1	
System Factor	φs	=	1	
ADTT (one-way)		=	2398	
Resistance Factor for Strength Limit State - Flexure	φf	=	1	AASHTO 6.5.4.2
Resistance Factor for Strength Limit State - Shear	φv	=	1	AASHTO 6.5.4.2
Load Factors:				
For Strength I:				
Dead Load 1	γ_{DC_STR}	=	1.25	MBE Table 6A.4.2.2-1
Dead Load 2	γ_{DW_STR}	=	1.50	MBE Table 6A.4.2.2-1
EV Live Load for given ADTT using linear interploation	γ_{LL_STR}	=	1.35	MBE Table 6A.4.4.2.3b-1
For Service:				
Dead Load	γ_{D_SRV}	=	1.00	MBE Table 6A.4.2.2-1
EV Live Load	γ_{LL_SRV}	=	1.30	MBE Table 6A.4.2.2-1

Postprocessed EV Ratings for the following POIs:

1 Span 1 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
2 Span 1 Interior Beam Shear 0.0L of Span - EV2 Rating
3 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV2 Rating

4 Span 1 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
5 Span 1 Interior Beam Shear 0.0L of Span - EV3 Rating
6 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV3 Rating

7 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
8 Span 1 Exterior Beam Shear 0.0L of Span - EV2 Rating
9 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV2 Rating

10 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
11 Span 1 Exterior Beam Shear 0.0L of Span - EV3 Rating
12 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV3 Rating

13 Span 2 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
14 Span 2 Interior Beam Shear 0.0L of Span - EV2 Rating
15 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV2 Rating

16 Span 2 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
17 Span 2 Interior Beam Shear 0.0L of Span - EV3 Rating
18 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV3 Rating

19 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
20 Span 2 Exterior Beam Shear 0.0L of Span - EV2 Rating
21 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV2 Rating

22 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
23 Span 2 Exterior Beam Shear 0.0L of Span - EV3 Rating
24 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV3 Rating

25 Span 3 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
26 Span 3 Interior Beam Shear 0.0L of Span - EV2 Rating

27 Span 3 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
28 Span 3 Interior Beam Shear 0.0L of Span - EV3 Rating

29 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
30 Span 3 Exterior Beam Shear 0.0L of Span - EV2 Rating

31 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
32 Span 3 Exterior Beam Shear 0.0L of Span - EV3 Rating

1 Span 1 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
3 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= (φc*φs*φf*C - (γ_{DC_STR})*D1 - (γ_{DW_STR})*D2) / (γ_{LL_STR}*LL+I)

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
1	0	0	2986.6	0	0	0	--
1	1	4.39	2986.6	144.3	56	156.7	12.85
1	2	8.78	2986.6	269.9	105	291.8	6.31
1	**3**	**13.18**	**2986.6**	**378**	**147.1**	**405.5**	**4.18**
1	4	17.57	4129.4	468.6	182.1	497.6	4.86
1	5	21.96	4129.4	540.3	210.1	568.3	4.08
1	6	26.35	4129.4	594.2	231.1	617.5	3.64
1	7	30.75	4129.4	630.4	245.1	642.4	3.42
1	**8**	**35.14**	**4129.4**	**648.5**	**252.1**	**645.9**	**3.37**
1	**9**	**39.53**	**4129.4**	**648.1**	**252.1**	**645.9**	**3.37**
1	10	43.92	4129.4	630	245.1	642.4	3.42
1	11	48.31	4129.4	594.4	231.1	617.5	3.64
1	12	52.71	4129.4	539.8	210.1	568.3	4.08
1	13	57.1	4129.4	467.4	182.1	497.6	4.86
1	14	61.49	2986.6	377.5	147.1	405.5	4.18
1	15	65.88	2986.6	269.6	105	291.8	6.31
1	16	70.27	2986.6	143.8	56	156.7	12.85
1	17	74.67	2986.6	0	0	0	--
							--
							--
							--

1 Span 1 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
3 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= (φc*φs*φf*C - (γ_{D_SRV})*D) / (γ_{LL_SVR}*LL+I)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
1	0	0	30.4	0	0	--
1	1	4.39	30.4	3.44	2.08	9.97
1	2	8.78	30.4	6.43	3.88	4.75
1	**3**	**13.18**	**30.4**	**9.01**	**5.38**	**3.06**
1	4	17.57	30.4	7.3	4.34	4.09
1	5	21.96	30.4	8.42	4.96	3.41
1	6	26.35	30.4	9.26	5.39	3.02
1	7	30.75	30.4	9.82	5.61	2.82
1	**8**	**35.14**	**30.4**	**10.11**	**5.64**	**2.77**
1	**9**	**39.53**	**30.4**	**10.1**	**5.64**	**2.77**
1	10	43.92	30.4	9.82	5.61	2.82
1	11	48.31	30.4	9.26	5.39	3.02
1	12	52.71	30.4	8.41	4.96	3.41
1	13	57.1	30.4	7.29	4.34	4.10
1	14	61.49	30.4	9	5.38	3.06
1	15	65.88	30.4	6.43	3.88	4.75
1	16	70.27	30.4	3.43	2.08	9.97
1	17	74.67	30.4	0	0	--
						--
						--
						--

2 Span 1 Interior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	- (LL+I)	RF
-	-	FT	K	K	K	K	K	-
1	**0**	**0**	**481.9**	**35.6**	**13.6**	**68.6**	**0**	**4.49**
1	1	4.39	481.9	30.6	12	64.2	-2.6	4.90
1	2	8.78	481.9	26.6	10.4	59.8	-4.9	5.35
1	3	13.18	481.9	22.6	8.8	55.4	-7.4	5.88
1	4	17.57	481.9	18.6	7.2	51	-11.3	6.49
1	5	21.96	481.9	14.3	5.6	46.6	-15.7	7.23
1	6	26.35	481.9	10.3	4	42.2	-20.1	8.11
1	7	30.75	481.9	6.2	2.4	37.8	-24.5	9.20
1	8	35.14	481.9	1.9	0.8	33.3	-28.9	10.62
1	9	39.53	481.9	-2.1	-0.8	28.9	-33.3	10.62
1	10	43.92	481.9	-6.1	-2.4	24.5	-37.8	9.21
1	11	48.31	481.9	-10.1	-4	20.1	-42.2	8.12
1	12	52.71	481.9	-14.5	-5.6	15.7	-46.6	7.23
1	13	57.1	481.9	-18.5	-7.2	11.3	-51	6.49
1	14	61.49	481.9	-22.5	-8.8	7.4	-55.4	5.88
1	15	65.88	481.9	-26.7	-10.4	4.9	-59.8	5.35
1	16	70.27	481.9	-30.7	-12	2.6	-64.2	4.90
1	17	74.67	481.9	-35.4	-13.6	0	-68.6	4.50
								--
								--
								--

4 Span 1 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
6 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi f*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
1	0	0	2986.6	0	0	0	--
1	1	4.39	2986.6	144.3	56	232.5	8.66
1	2	8.78	2986.6	269.9	105	432.9	4.26
1	**3**	**13.18**	**2986.6**	**378**	**147.1**	**601.1**	**2.82**
1	4	17.57	4129.4	468.6	182.1	736.1	3.29
1	5	21.96	4129.4	540.3	210.1	840.9	2.76
1	6	26.35	4129.4	594.2	231.1	916.2	2.45
1	7	30.75	4129.4	630.4	245.1	963.5	2.28
1	**8**	**35.14**	**4129.4**	**648.5**	**252.1**	**983.8**	**2.21**
1	**9**	**39.53**	**4129.4**	**648.1**	**252.1**	**983.8**	**2.21**
1	10	43.92	4129.4	630	245.1	963.5	2.28
1	11	48.31	4129.4	594.4	231.1	916.2	2.45
1	12	52.71	4129.4	539.8	210.1	840.9	2.76
1	13	57.1	4129.4	467.4	182.1	736.1	3.29
1	14	61.49	2986.6	377.5	147.1	601.1	2.82
1	15	65.88	2986.6	269.6	105	432.9	4.26
1	16	70.27	2986.6	143.8	56	232.5	8.66
1	17	74.67	2986.6	0	0	0	--
							--
							--
							--

4 Span 1 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
6 Span 1 Interior Beam Moment at Cover Plate Transition (13.18') - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
1	0	0	30.4	0	0	--
1	1	4.39	30.4	3.44	3.09	6.71
1	2	8.78	30.4	6.43	5.75	3.21
1	**3**	**13.18**	**30.4**	**9.01**	**7.98**	**2.06**
1	4	17.57	30.4	7.3	6.42	2.77
1	5	21.96	30.4	8.42	7.34	2.30
1	6	26.35	30.4	9.26	7.99	2.04
1	7	30.75	30.4	9.82	8.41	1.88
1	**8**	**35.14**	**30.4**	**10.11**	**8.58**	**1.82**
1	**9**	**39.53**	**30.4**	**10.1**	**8.58**	**1.82**
1	10	43.92	30.4	9.82	8.41	1.88
1	11	48.31	30.4	9.26	7.99	2.04
1	12	52.71	30.4	8.41	7.34	2.30
1	13	57.1	30.4	7.29	6.42	2.77
1	14	61.49	30.4	9	7.98	2.06
1	15	65.88	30.4	6.43	5.75	3.21
1	16	70.27	30.4	3.43	3.09	6.71
1	17	74.67	30.4	0	0	--
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5 Span 1 Interior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
1	**0**	**0**	**481.9**	**35.6**	**13.6**	**101.8**	**0**	**3.03**
1	1	4.39	481.9	30.6	12	95.3	-3.2	3.30
1	2	8.78	481.9	26.6	10.4	88.7	-7.2	3.61
1	3	13.18	481.9	22.6	8.8	82.1	-11.9	3.97
1	4	17.57	481.9	18.6	7.2	75.5	-17.1	4.39
1	5	21.96	481.9	14.3	5.6	68.9	-22.9	4.89
1	6	26.35	481.9	10.3	4	62.3	-29.4	5.50
1	7	30.75	481.9	6.2	2.4	55.7	-36	6.25
1	8	35.14	481.9	1.9	0.8	49.2	-42.6	7.19
1	9	39.53	481.9	-2.1	-0.8	42.6	-49.2	7.18
1	10	43.92	481.9	-6.1	-2.4	36	-55.7	6.25
1	11	48.31	481.9	-10.1	-4	29.4	-62.3	5.50
1	12	52.71	481.9	-14.5	-5.6	22.9	-68.9	4.89
1	13	57.1	481.9	-18.5	-7.2	17.1	-75.5	4.39
1	14	61.49	481.9	-22.5	-8.8	11.9	-82.1	3.97
1	15	65.88	481.9	-26.7	-10.4	7.2	-88.7	3.61
1	16	70.27	481.9	-30.7	-12	3.2	-95.3	3.30
1	17	74.67	481.9	-35.4	-13.6	0	-101.8	3.03
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7 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
9 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi f*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
1	0	0	2891.9	0	0	0	--
1	1	4.67	2891.9	135.2	59.3	142	13.72
1	2	9.33	2891.9	252.5	110.7	263.1	6.77
1	**3**	**14**	**2891.9**	**352**	**154.2**	**363.5**	**4.52**
1	4	18.67	3756.2	433	189.7	443.1	4.89
1	5	23.33	3756.2	496.1	217.4	502	4.14
1	6	28	3756.2	541.4	237.2	540	3.73
1	7	32.67	3756.2	568.4	249	552	3.58
1	**8**	**37.33**	**3756.2**	**577.3**	**253**	**553.7**	**3.55**
1	9	42	3756.2	568.4	249	552	3.58
1	10	46.67	3756.2	541.4	237.2	540	3.73
1	11	51.33	3756.2	496.1	217.4	502	4.14
1	12	56	3756.2	433	189.7	443.1	4.89
1	13	60.67	2891.9	352	154.2	363.5	4.52
1	14	65.33	2891.9	252.5	110.7	263.1	6.77
1	15	70	2891.9	135.2	59.3	142	13.72
1	16	74.67	2891.9	0	0	0	--
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7 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
9 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= (φc*φs*φf*C - (γ_{D_SRV})*D) / (γ_{LL_SVR}*LL+I)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
1	0	0	30.4	0	0	--
1	1	4.67	30.4	3.34	1.91	10.90
1	2	9.33	30.4	6.23	3.55	5.24
1	**3**	**14**	**30.4**	**8.69**	**4.9**	**3.41**
1	4	18.67	30.4	7.36	4.14	4.28
1	5	23.33	30.4	8.43	4.69	3.60
1	6	28	30.4	9.2	5.04	3.24
1	7	32.67	30.4	9.66	5.15	3.10
1	**8**	**37.33**	**30.4**	**9.81**	**5.17**	**3.06**
1	9	42	30.4	9.66	5.15	3.10
1	10	46.67	30.4	9.2	5.04	3.24
1	11	51.33	30.4	8.43	4.69	3.60
1	12	56	30.4	7.36	4.14	4.28
1	13	60.67	30.4	8.69	4.9	3.41
1	14	65.33	30.4	6.23	3.55	5.24
1	15	70	30.4	3.34	1.91	10.90
1	16	74.67	30.4	0	0	--
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8 Span 1 Exterior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
1	**0**	**0**	**481.9**	**31.2**	**13.6**	**49.3**	**0**	**6.34**
1	1	4.67	481.9	27	11.9	46	-2	6.92
1	2	9.33	481.9	23.2	10.2	42.6	-3.7	7.60
1	3	14	481.9	19.4	8.5	39.2	-5.8	8.39
1	4	18.67	481.9	15.4	6.8	35.9	-8.9	9.32
1	5	23.33	481.9	11.6	5.1	32.5	-12.3	10.46
1	6	28	481.9	7.8	3.4	29.1	-15.7	11.87
1	7	32.67	481.9	3.8	1.7	25.8	-19	13.60
1	8	37.33	481.9	0	0	22.4	-22.4	15.91
1	9	42	481.9	-3.8	-1.7	19	-25.8	13.60
1	10	46.67	481.9	-7.8	-3.4	15.7	-29.1	11.87
1	11	51.33	481.9	-11.6	-5.1	12.3	-32.5	10.46
1	12	56	481.9	-15.4	-6.8	8.9	-35.9	9.32
1	13	60.67	481.9	-19.4	-8.5	5.8	-39.2	8.39
1	14	65.33	481.9	-23.2	-10.2	3.7	-42.6	7.60
1	15	70	481.9	-27	-11.9	2	-46	6.92
1	16	74.67	481.9	-31.2	-13.6	0	-49.3	6.34
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10 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
12 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi f*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
1	0	0	2891.9	0	0	0	--
1	1	4.67	2891.9	135.2	59.3	210.7	9.24
1	2	9.33	2891.9	252.5	110.7	390.4	4.56
1	**3**	**14**	**2891.9**	**352**	**154.2**	**538.9**	**3.05**
1	4	18.67	3756.2	433	189.7	655.3	3.31
1	5	23.33	3756.2	496.1	217.4	743	2.80
1	6	28	3756.2	541.4	237.2	803.1	2.51
1	7	32.67	3756.2	568.4	249	835.1	2.37
1	**8**	**37.33**	**3756.2**	**577.3**	**253**	**845**	**2.32**
1	9	42	3756.2	568.4	249	835.1	2.37
1	10	46.67	3756.2	541.4	237.2	803.1	2.51
1	11	51.33	3756.2	496.1	217.4	743	2.80
1	12	56	3756.2	433	189.7	655.3	3.31
1	13	60.67	2891.9	352	154.2	538.9	3.05
1	14	65.33	2891.9	252.5	110.7	390.4	4.56
1	15	70	2891.9	135.2	59.3	210.7	9.24
1	16	74.67	2891.9	0	0	0	--
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10 Span 1 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
12 Span 1 Exterior Beam Moment at Cover Plate Transition (14.0') - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= (φc*φs*φf*C - (γ_{D_SRV})*D) / (γ_{LL_SVR}*LL+I)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
1	0	0	30.4	0	0	--
1	1	4.67	30.4	3.34	2.84	7.33
1	2	9.33	30.4	6.23	5.26	3.53
1	**3**	**14**	**30.4**	**8.69**	**7.27**	**2.30**
1	4	18.67	30.4	7.36	6.12	2.90
1	5	23.33	30.4	8.43	6.94	2.44
1	6	28	30.4	9.2	7.5	2.17
1	7	32.67	30.4	9.66	7.8	2.05
1	**8**	**37.33**	**30.4**	**9.81**	**7.89**	**2.01**
1	9	42	30.4	9.66	7.8	2.05
1	10	46.67	30.4	9.2	7.5	2.17
1	11	51.33	30.4	8.43	6.94	2.44
1	12	56	30.4	7.36	6.12	2.90
1	13	60.67	30.4	8.69	7.27	2.30
1	14	65.33	30.4	6.23	5.26	3.53
1	15	70	30.4	3.34	2.84	7.33
1	16	74.67	30.4	0	0	--
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11 Span 1 Exterior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
1	**0**	**0**	**481.9**	**31.2**	**13.6**	**73.2**	**0**	**4.27**
1	1	4.67	481.9	27	11.9	68.2	-2.5	4.67
1	2	9.33	481.9	23.2	10.2	63.2	-5.6	5.12
1	3	14	481.9	19.4	8.5	58.1	-9.3	5.66
1	4	18.67	481.9	15.4	6.8	53.1	-13.3	6.30
1	5	23.33	481.9	11.6	5.1	48.1	-17.9	7.07
1	6	28	481.9	7.8	3.4	43	-22.9	8.03
1	7	32.67	481.9	3.8	1.7	38	-27.9	9.23
1	8	37.33	481.9	0	0	33	-33	10.80
1	9	42	481.9	-3.8	-1.7	27.9	-38	9.23
1	10	46.67	481.9	-7.8	-3.4	22.9	-43	8.03
1	11	51.33	481.9	-11.6	-5.1	17.9	-48.1	7.07
1	12	56	481.9	-15.4	-6.8	13.3	-53.1	6.30
1	13	60.67	481.9	-19.4	-8.5	9.3	-58.1	5.66
1	14	65.33	481.9	-23.2	-10.2	5.6	-63.2	5.12
1	15	70	481.9	-27	-11.9	2.5	-68.2	4.67
1	16	74.67	481.9	-31.2	-13.6	0	-73.2	4.27
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13 Span 2 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
15 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi f*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
2	0	0	4033.3	0	0	0	--
2	1	5.71	4033.3	253.3	88.9	204.7	12.94
2	2	11.43	4033.3	473.2	165.9	379.9	6.21
2	**3**	**17.14**	**4033.3**	**658.8**	**231.1**	**525.7**	**4.03**
2	4	22.85	5399.2	810.5	284.4	642	4.56
2	5	28.57	5399.2	929.3	325.9	728.9	3.80
2	6	34.28	5399.2	1013.3	355.5	786.4	3.38
2	7	39.99	5399.2	1063.9	373.3	808.4	3.21
2	**8**	**45.71**	**5399.2**	**1081.4**	**379.2**	**813**	**3.16**
2	9	51.42	5399.2	1063.9	373.3	808.4	3.21
2	10	57.14	5399.2	1013.3	355.5	786.4	3.38
2	11	62.85	5399.2	929.3	325.9	728.9	3.80
2	12	68.56	5399.2	810.5	284.4	642	4.56
2	13	74.28	4033.3	658.8	231.1	525.7	4.03
2	14	79.99	4033.3	473.2	165.9	379.9	6.21
2	15	85.7	4033.3	253.3	88.9	204.7	12.94
2	16	91.42	4033.3	0	0	0	--
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13 Span 2 Interior Beam Positive Moment 0.5L of Span - EV2 Rating
15 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= (φc*φs*φf*C - (γ_{D_SRV})*D) / (γ_{LL_SVR}*LL+I)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
2	0	0	30.4	0	0	--
2	1	5.71	30.4	3.88	1.91	10.68
2	2	11.43	30.4	7.25	3.54	5.03
2	**3**	**17.14**	**30.4**	**10.1**	**4.9**	**3.19**
2	4	22.85	30.4	7.91	3.79	4.56
2	5	28.57	30.4	9.07	4.3	3.82
2	6	34.28	30.4	9.89	4.64	3.40
2	7	39.99	30.4	10.38	4.77	3.23
2	**8**	**45.71**	**30.4**	**10.55**	**4.8**	**3.18**
2	9	51.42	30.4	10.38	4.77	3.23
2	10	57.14	30.4	9.89	4.64	3.40
2	11	62.85	30.4	9.07	4.3	3.82
2	12	68.56	30.4	7.91	3.79	4.56
2	13	74.28	30.4	10.1	4.9	3.19
2	14	79.99	30.4	7.25	3.54	5.03
2	15	85.7	30.4	3.88	1.91	10.68
2	16	91.42	30.4	0	0	--
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14 Span 2 Interior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
2	**0**	**0**	**548**	**47.9**	**16.6**	**69**	**0**	**4.96**
2	1	5.71	548	41.4	14.5	64.4	-2.6	5.45
2	2	11.43	548	35.6	12.4	59.8	-5.5	6.00
2	3	17.14	548	29.5	10.4	55.1	-9	6.65
2	4	22.85	548	23.7	8.3	50.5	-13.4	7.41
2	5	28.57	548	17.9	6.2	45.9	-18.1	8.32
2	6	34.28	548	11.7	4.1	41.2	-22.7	9.46
2	7	39.99	548	6	2.1	36.6	-27.3	10.86
2	8	45.71	548	0.2	0	32	-32	12.66
2	9	51.42	548	-6	-2.1	27.3	-36.6	10.86
2	10	57.14	548	-11.7	-4.1	22.7	-41.2	9.46
2	11	62.85	548	-17.9	-6.2	18.1	-45.9	8.32
2	12	68.56	548	-23.7	-8.3	13.4	-50.5	7.41
2	13	74.28	548	-29.5	-10.4	9	-55.1	6.65
2	14	79.99	548	-35.6	-12.4	5.5	-59.8	6.00
2	15	85.7	548	-41.4	-14.5	2.6	-64.4	5.45
2	16	91.42	548	-47.9	-16.6	0	-69	4.96
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16 Span 2 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
18 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= (φc*φs*φf*C - (γ_{DC_STR})*D1 - (γ_{DW_STR})*D2) / (γ_{LL_STR}*LL+I)

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
2	0	0	4033.3	0	0	0	--
2	1	5.71	4033.3	253.3	88.9	304.2	8.71
2	2	11.43	4033.3	473.2	165.9	564.4	4.18
2	**3**	**17.14**	**4033.3**	**658.8**	**231.1**	**780.6**	**2.71**
2	4	22.85	5399.2	810.5	284.4	951.6	3.08
2	5	28.57	5399.2	929.3	325.9	1081.3	2.56
2	6	34.28	5399.2	1013.3	355.5	1170.9	2.27
2	7	39.99	5399.2	1063.9	373.3	1220.1	2.13
2	**8**	**45.71**	**5399.2**	**1081.4**	**379.2**	**1235.5**	**2.08**
2	9	51.42	5399.2	1063.9	373.3	1220.1	2.13
2	10	57.14	5399.2	1013.3	355.5	1170.9	2.27
2	11	62.85	5399.2	929.3	325.9	1081.3	2.56
2	12	68.56	5399.2	810.5	284.4	951.6	3.08
2	13	74.28	4033.3	658.8	231.1	780.6	2.71
2	14	79.99	4033.3	473.2	165.9	564.5	4.18
2	15	85.7	4033.3	253.3	88.9	304.2	8.71
2	16	91.42	4033.3	0	0	0	--
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16 Span 2 Interior Beam Positive Moment 0.5L of Span - EV3 Rating
18 Span 2 Interior Beam Moment at Cover Plate Transition (17.14') - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
2	0	0	30.4	0	0	--
2	1	5.71	30.4	3.88	2.83	7.21
2	2	11.43	30.4	7.25	5.26	3.39
2	**3**	**17.14**	**30.4**	**10.1**	**7.27**	**2.15**
2	4	22.85	30.4	7.91	5.62	3.08
2	5	28.57	30.4	9.07	6.38	2.57
2	6	34.28	30.4	9.89	6.91	2.28
2	7	39.99	30.4	10.38	7.2	2.14
2	**8**	**45.71**	**30.4**	**10.55**	**7.29**	**2.09**
2	9	51.42	30.4	10.38	7.2	2.14
2	10	57.14	30.4	9.89	6.91	2.28
2	11	62.85	30.4	9.07	6.38	2.57
2	12	68.56	30.4	7.91	5.62	3.08
2	13	74.28	30.4	10.1	7.27	2.15
2	14	79.99	30.4	7.25	5.26	3.39
2	15	85.7	30.4	3.88	2.83	7.21
2	16	91.42	30.4	0	0	--
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17 Span 2 Interior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
2	**0**	**0**	**548**	**47.9**	**16.6**	**102.6**	**0**	**3.34**
2	1	5.71	548	41.4	14.5	95.7	-3.7	3.67
2	2	11.43	548	35.6	12.4	88.8	-8	4.04
2	3	17.14	548	29.5	10.4	81.9	-13.1	4.47
2	4	22.85	548	23.7	8.3	74.9	-19.6	4.99
2	5	28.57	548	17.9	6.2	68	-26.4	5.61
2	6	34.28	548	11.7	4.1	61.1	-33.4	6.38
2	7	39.99	548	6	2.1	54.2	-40.3	7.33
2	8	45.71	548	0.2	0	47.2	-47.2	8.58
2	9	51.42	548	-6	-2.1	40.3	-54.2	7.33
2	10	57.14	548	-11.7	-4.1	33.4	-61.1	6.38
2	11	62.85	548	-17.9	-6.2	26.4	-68	5.61
2	12	68.56	548	-23.7	-8.3	19.6	-74.9	4.99
2	13	74.28	548	-29.5	-10.4	13.1	-81.9	4.47
2	14	79.99	548	-35.6	-12.4	8	-88.8	4.04
2	15	85.7	548	-41.4	-14.5	3.7	-95.7	3.67
2	16	91.42	548	-47.9	-16.6	0	-102.6	3.34
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19 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
21 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi f*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
2	0	0	3935.3	0	0	0	--
2	1	4.57	3935.3	178.9	72	145.1	18.36
2	2	9.14	3935.3	339	136.5	273.8	8.93
2	3	13.71	3935.3	480.5	193.4	386	5.83
2	**4**	**18.28**	**3935.3**	**603.3**	**242.7**	**481.7**	**4.32**
2	5	22.85	4775.4	707.1	284.4	561	4.57
2	6	27.43	4775.4	791.9	318.5	623.8	3.92
2	7	32	4775.4	858	345.1	670.2	3.51
2	8	36.57	4775.4	905.4	364	700.1	3.27
2	9	41.14	4775.4	933.5	375.4	707.8	3.18
2	**10**	**45.71**	**4775.4**	**942.9**	**379.2**	**710.4**	**3.15**
2	11	50.28	4775.4	933.7	375.4	707.8	3.18
2	12	54.85	4775.4	905.4	364	700.1	3.27
2	13	59.42	4775.4	858.2	345.1	670.2	3.51
2	14	63.99	4775.4	792.3	318.5	623.8	3.92
2	15	68.56	4775.4	707.7	284.4	561	4.57
2	16	73.13	3935.3	603.7	242.7	481.7	4.32
2	17	77.7	3935.3	481.1	193.4	386	5.83
2	18	82.28	3935.3	339.8	136.5	273.8	8.93
2	19	86.85	3935.3	179.5	72	145.1	18.36
2	20	91.42	3935.3	0	0	0	--

19 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating
21 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
2	0	0	30.4	0	0	--
2	1	4.57	30.4	2.85	1.37	15.47
2	2	9.14	30.4	5.4	2.59	7.43
2	3	13.71	30.4	7.65	3.65	4.79
2	**4**	**18.28**	**30.4**	**9.61**	**4.55**	**3.51**
2	5	22.85	30.4	7.73	3.63	4.80
2	6	27.43	30.4	8.65	4.04	4.14
2	7	32	30.4	9.38	4.34	3.73
2	8	36.57	30.4	9.89	4.53	3.48
2	9	41.14	30.4	10.2	4.59	3.39
2	**10**	**45.71**	**30.4**	**10.3**	**4.6**	**3.36**
2	11	50.28	30.4	10.2	4.59	3.39
2	12	54.85	30.4	9.89	4.53	3.48
2	13	59.42	30.4	9.38	4.34	3.73
2	14	63.99	30.4	8.66	4.04	4.14
2	15	68.56	30.4	7.73	3.63	4.80
2	16	73.13	30.4	9.61	4.55	3.51
2	17	77.7	30.4	7.66	3.65	4.79
2	18	82.28	30.4	5.41	2.59	7.42
2	19	86.85	30.4	2.86	1.37	15.46
2	20	91.42	30.4	0	0	--

20 Span 2 Exterior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
2	**0**	**0**	**548**	**41.5**	**16.6**	**49.6**	**0**	**7.02**
2	1	4.57	548	37.1	14.9	46.9	-1.4	7.56
2	2	9.14	548	33	13.3	44.3	-3.1	8.13
2	3	13.71	548	28.9	11.6	41.6	-4.9	8.79
2	4	18.28	548	24.8	10	38.9	-7	9.54
2	5	22.85	548	20.6	8.3	36.3	-9.7	10.38
2	6	27.43	548	16.5	6.6	33.6	-12.3	11.39
2	7	32	548	12.4	5	30.9	-15	12.56
2	8	36.57	548	8.2	3.3	28.3	-17.6	13.92
2	9	41.14	548	4.1	1.7	25.6	-20.3	15.61
2	10	45.71	548	0	0	23	-23	17.62
2	11	50.28	548	-4.1	-1.7	20.3	-25.6	15.61
2	12	54.85	548	-8.3	-3.3	17.6	-28.3	13.92
2	13	59.42	548	-12.4	-5	15	-30.9	12.56
2	14	63.99	548	-16.5	-6.6	12.3	-33.6	11.39
2	15	68.56	548	-20.7	-8.3	9.7	-36.3	10.38
2	16	73.13	548	-24.8	-10	7	-38.9	9.54
2	17	77.7	548	-28.9	-11.6	4.9	-41.6	8.79
2	18	82.28	548	-32.9	-13.3	3.1	-44.3	8.13
2	19	86.85	548	-37.2	-14.9	1.4	-46.9	7.55
2	20	91.42	548	-41.6	-16.6	0	-49.6	7.02

22 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
24 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Moment Capacity
D1 = Unfactored Dead Load Moment D1
D2 = Unfactored Dead Load Moment D2
LL+I = Unfactored Live Load Moment with Impact

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	RF
-	-	FT	K-FT	K-FT	K-FT	K-FT	-
2	0	0	3935.3	0	0	0	--
2	1	4.57	3935.3	178.9	72	215.8	12.35
2	2	9.14	3935.3	339	136.5	406.9	6.01
2	3	13.71	3935.3	480.5	193.4	573.4	3.93
2	**4**	**18.28**	**3935.3**	**603.3**	**242.7**	**715.3**	**2.91**
2	5	22.85	4775.4	707.1	284.4	831.5	3.08
2	6	27.43	4775.4	791.9	318.5	925.2	2.64
2	7	32	4775.4	858	345.1	995.9	2.36
2	8	36.57	4775.4	905.4	364	1045.1	2.19
2	9	41.14	4775.4	933.5	375.4	1071	2.10
2	**10**	**45.71**	**4775.4**	**942.9**	**379.2**	**1079.6**	**2.07**
2	11	50.28	4775.4	933.7	375.4	1071	2.10
2	12	54.85	4775.4	905.4	364	1045.1	2.19
2	13	59.42	4775.4	858.2	345.1	995.9	2.36
2	14	63.99	4775.4	792.3	318.5	925.2	2.64
2	15	68.56	4775.4	707.7	284.4	831.5	3.08
2	16	73.13	3935.3	603.7	242.7	715.3	2.91
2	17	77.7	3935.3	481.1	193.4	573.4	3.93
2	18	82.28	3935.3	339.8	136.5	406.9	6.01
2	19	86.85	3935.3	179.5	72	215.8	12.34
2	20	91.42	3935.3	0	0	0	--

22 Span 2 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating
24 Span 2 Exterior Beam Moment at Cover Plate Transition (18.28') - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D	LL+I	RF
-	-	FT	KSI	KSI	KSI	-
2	0	0	30.4	0	0	--
2	1	4.57	30.4	2.85	2.04	10.39
2	2	9.14	30.4	5.4	3.85	5.00
2	3	13.71	30.4	7.65	5.42	3.23
2	**4**	**18.28**	**30.4**	**9.61**	**6.76**	**2.37**
2	5	22.85	30.4	7.73	5.39	3.24
2	6	27.43	30.4	8.65	5.99	2.79
2	7	32	30.4	9.38	6.45	2.51
2	8	36.57	30.4	9.89	6.77	2.33
2	9	41.14	30.4	10.2	6.94	2.24
2	**10**	**45.71**	**30.4**	**10.3**	**6.99**	**2.21**
2	11	50.28	30.4	10.2	6.94	2.24
2	12	54.85	30.4	9.89	6.77	2.33
2	13	59.42	30.4	9.38	6.45	2.51
2	14	63.99	30.4	8.66	5.99	2.79
2	15	68.56	30.4	7.73	5.39	3.24
2	16	73.13	30.4	9.61	6.76	2.37
2	17	77.7	30.4	7.66	5.42	3.23
2	18	82.28	30.4	5.41	3.85	4.99
2	19	86.85	30.4	2.86	2.04	10.38
2	20	91.42	30.4	0	0	--

23 Span 2 Exterior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi v * C - (\gamma_{DC_STR}) * D1 - (\gamma_{DW_STR}) * D2) / (\gamma_{LL_STR} * LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	-(LL+I)	RF
-	-	FT	K	K	K	K	K	-
2	**0**	**0**	**548**	**41.5**	**16.6**	**73.7**	**0**	**4.73**
2	1	4.57	548	37.1	14.9	69.7	-2.1	5.08
2	2	9.14	548	33	13.3	65.8	-4.5	5.47
2	3	13.71	548	28.9	11.6	61.8	-7.1	5.92
2	4	18.28	548	24.8	10	57.8	-10.2	6.42
2	5	22.85	548	20.6	8.3	53.8	-14.1	7.01
2	6	27.43	548	16.5	6.6	49.8	-18	7.68
2	7	32	548	12.4	5	45.9	-22	8.46
2	8	36.57	548	8.2	3.3	41.9	-26	9.40
2	9	41.14	548	4.1	1.7	37.9	-29.9	10.54
2	10	45.71	548	0	0	33.9	-33.9	11.95
2	11	50.28	548	-4.1	-1.7	29.9	-37.9	10.54
2	12	54.85	548	-8.3	-3.3	26	-41.9	9.40
2	13	59.42	548	-12.4	-5	22	-45.9	8.46
2	14	63.99	548	-16.5	-6.6	18	-49.8	7.68
2	15	68.56	548	-20.7	-8.3	14.1	-53.8	7.00
2	16	73.13	548	-24.8	-10	10.2	-57.8	6.42
2	17	77.7	548	-28.9	-11.6	7.1	-61.8	5.92
2	18	82.28	548	-32.9	-13.3	4.5	-65.8	5.47
2	19	86.85	548	-37.2	-14.9	2.1	-69.7	5.08
2	20	91.42	548	-41.6	-16.6	0	-73.7	4.73

25 Span 3 Interior Beam Positive Moment 0.5L of Span - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Flexural Stress Capacity
D = Unfactored Dead Load Stress
LL+I = Unfactored Live Load Stress with Impact
RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{DC_STR}) * D$) / ($\gamma_{LL_STR} * LL+I$)

SP	INT	DIST	C		D	LL+I	RF
-	-	FT	KSI	-	KSI	KSI	-
3	0	0	33	TOP	0	0	--
			33	BOT	0	0	--
3	1	2.08	33	TOP	-1.19	-2.27	10.26
			33	BOT	1.19	2.27	10.26
3	2	4.16	33	TOP	-2.24	-4.26	5.24
			33	BOT	2.24	4.26	5.24
3	3	6.24	33	TOP	-3.16	-5.97	3.60
			33	BOT	3.16	5.97	3.60
3	4	8.32	33	TOP	-3.97	-7.39	2.81
			33	BOT	3.97	7.39	2.81
3	5	10.41	33	TOP	-4.65	-8.53	2.36
			33	BOT	4.65	8.53	2.36
3	6	12.49	33	TOP	-5.21	-9.38	2.09
			33	BOT	5.21	9.38	2.09
3	7	14.57	33	TOP	-5.65	-9.95	1.93
			33	BOT	5.65	9.95	1.93
3	8	16.65	33	TOP	-5.97	-10.23	1.85
			33	BOT	5.97	10.23	1.85
3	9	18.73	29.38	TOP	-6.15	-10.01	1.60
			33	BOT	6.15	10.01	1.87
3	**10**	**20.81**	**29.38**	**TOP**	**-6.21**	**-9.94**	**1.61**
			33	BOT	6.21	9.94	1.88
3	11	22.89	29.38	TOP	-6.14	-10.01	1.60
			33	BOT	6.14	10.01	1.87
3	**12**	**24.97**	**29.38**	**TOP**	**-5.95**	**-10.23**	**1.59**
			33	BOT	5.95	10.23	1.85
3	13	27.06	29.38	TOP	-5.64	-9.95	1.66
			33	BOT	5.64	9.95	1.93
3	14	29.14	29.38	TOP	-5.21	-9.38	1.80
			33	BOT	5.21	9.38	2.09
3	15	31.22	29.38	TOP	-4.65	-8.53	2.04
			33	BOT	4.65	8.53	2.36
3	16	33.3	33	TOP	-3.97	-7.39	2.81
			33	BOT	3.97	7.39	2.81
3	17	35.38	33	TOP	-3.16	-5.97	3.60
			33	BOT	3.16	5.97	3.60
3	18	37.46	33	TOP	-2.23	-4.26	5.24
			33	BOT	2.23	4.26	5.24
3	19	39.54	33	TOP	-1.18	-2.27	10.27
			33	BOT	1.18	2.27	10.27
3	20	41.62	33	TOP	0	0	--
			33	BOT	0	0	--

25 Span 3 Interior Beam Positive Moment 0.5L of Span - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D		LL+I		RF	
				TOP	BOT	TOP	BOT	TOP	BOT
-	-	FT	KSI	KSI	KSI	KSI	KSI	-	-
3	0	0	26.4	0	0	0	0	--	--
3	1	2.08	26.4	-1.19	1.19	-2.27	2.27	11.63	11.63
3	2	4.16	26.4	-2.24	2.24	-4.26	4.26	6.20	6.20
3	3	6.24	26.4	-3.16	3.16	-5.97	5.97	4.42	4.42
3	4	8.32	26.4	-3.97	3.97	-7.39	7.39	3.57	3.57
3	5	10.41	26.4	-4.65	4.65	-8.53	8.53	3.09	3.09
3	6	12.49	26.4	-5.21	5.21	-9.38	9.38	2.81	2.81
3	7	14.57	26.4	-5.65	5.65	-9.95	9.95	2.65	2.65
3	8	16.65	26.4	-5.97	5.97	-10.23	10.23	2.58	2.58
3	9	18.73	26.4	-6.15	6.15	-10.01	10.01	2.64	2.64
3	**10**	**20.81**	**26.4**	**-6.21**	**6.21**	**-9.94**	**9.94**	**2.66**	2.66
3	11	22.89	26.4	-6.14	6.14	-10.01	10.01	2.64	2.64
3	**12**	**24.97**	**26.4**	**-5.95**	**5.95**	**-10.23**	**10.23**	**2.58**	2.58
3	13	27.06	26.4	-5.64	5.64	-9.95	9.95	2.65	2.65
3	14	29.14	26.4	-5.21	5.21	-9.38	9.38	2.81	2.81
3	15	31.22	26.4	-4.65	4.65	-8.53	8.53	3.09	3.09
3	16	33.3	26.4	-3.97	3.97	-7.39	7.39	3.57	3.57
3	17	35.38	26.4	-3.16	3.16	-5.97	5.97	4.42	4.42
3	18	37.46	26.4	-2.23	2.23	-4.26	4.26	6.20	6.20
3	19	39.54	26.4	-1.18	1.18	-2.27	2.27	11.63	11.63
3	20	41.62	26.4	0	0	0	0	--	--

26 Span 3 Interior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	- (LL+I)	RF
-	-	FT	K	K	K	K	K	-
3	**0**	**0**	**407**	**0**	**25.8**	**54.1**	**0**	**5.03**
3	1	2.08	407	0	22.6	50.9	-1.9	5.42
3	2	4.16	407	0	19.9	47.7	-3.7	5.85
3	3	6.24	407	0	17.5	44.6	-5.6	6.31
3	4	8.32	407	0	15	41.4	-7.4	6.87
3	5	10.41	407	0	12.5	38.2	-9.1	7.52
3	6	12.49	407	0	10.1	35	-11.1	8.28
3	7	14.57	407	0	7.6	31.8	-13.3	9.20
3	8	16.65	407	0	5.2	28.6	-15.9	10.32
3	9	18.73	407	0	2.3	25.4	-19.1	11.75
3	10	20.81	407	0	-0.1	22.3	-22.3	13.49
3	11	22.89	407	0	-2.6	19.1	-25.4	11.73
3	12	24.97	407	0	-5	15.9	-28.6	10.33
3	13	27.06	407	0	-7.5	13.3	-31.8	9.20
3	14	29.14	407	0	-10	11.1	-35	8.28
3	15	31.22	407	0	-12.4	9.1	-38.2	7.52
3	16	33.3	407	0	-15.1	7.4	-41.4	6.86
3	17	35.38	407	0	-17.5	5.6	-44.6	6.31
3	18	37.46	407	0	-20	3.7	-47.7	5.84
3	19	39.54	407	0	-22.4	1.9	-50.9	5.42
3	20	41.62	407	0	-25.7	0	-54.1	5.04

27 Span 3 Interior Beam Positive Moment 0.5L of Span - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Flexural Stress Capacity
D = Unfactored Dead Load Stress
LL+I = Unfactored Live Load Stress with Impact
RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{DC_STR}) * D$) / ($\gamma_{LL_STR} * LL+I$)

SP	INT	DIST	C		D	LL+I	RF
-	-	FT	KSI	-	KSI	KSI	-
3	0	0	33	TOP	0	0	--
			33	BOT	0	0	--
3	1	2.08	33	TOP	-1.19	-3.35	6.96
			33	BOT	1.19	3.35	6.96
3	2	4.16	33	TOP	-2.24	-6.28	3.56
			33	BOT	2.24	6.28	3.56
3	3	6.24	33	TOP	-3.16	-8.78	2.45
			33	BOT	3.16	8.78	2.45
3	4	8.32	33	TOP	-3.97	-10.86	1.91
			33	BOT	3.97	10.86	1.91
3	5	10.41	33	TOP	-4.65	-12.47	1.61
			33	BOT	4.65	12.47	1.61
3	6	12.49	33	TOP	-5.21	-13.73	1.43
			33	BOT	5.21	13.73	1.43
3	7	14.57	33	TOP	-5.65	-14.63	1.31
			33	BOT	5.65	14.63	1.31
3	8	16.65	32.99	TOP	-5.97	-15.23	1.24
			32.99	BOT	5.97	15.23	1.24
3	9	18.73	29.4	TOP	-6.15	-15.45	1.04
			32.97	BOT	6.15	15.45	1.21
3	**10**	**20.81**	**29.4**	**TOP**	**-6.21**	**-15.52**	**1.03**
			32.97	BOT	6.21	15.52	1.20
3	11	22.89	29.4	TOP	-6.14	-15.45	1.04
			32.97	BOT	6.14	15.45	1.21
3	**12**	**24.97**	**29.41**	**TOP**	**-5.95**	**-15.23**	**1.07**
			32.99	BOT	5.95	15.23	1.24
3	13	27.06	29.42	TOP	-5.64	-14.63	1.13
			33	BOT	5.64	14.63	1.31
3	14	29.14	29.42	TOP	-5.21	-13.73	1.23
			33	BOT	5.21	13.73	1.43
3	15	31.22	29.42	TOP	-4.65	-12.47	1.40
			33	BOT	4.65	12.47	1.61
3	16	33.3	33	TOP	-3.97	-10.86	1.91
			33	BOT	3.97	10.86	1.91
3	17	35.38	33	TOP	-3.16	-8.78	2.45
			33	BOT	3.16	8.78	2.45
3	18	37.46	33	TOP	-2.23	-6.28	3.56
			33	BOT	2.23	6.28	3.56
3	19	39.54	33	TOP	-1.18	-3.35	6.96
			33	BOT	1.18	3.35	6.96
3	20	41.62	33	TOP	0	0	--
			33	BOT	0	0	--

27 Span 3 Interior Beam Positive Moment 0.5L of Span - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D) / (\gamma_{LL_SVR} * LL+I)$

SP	INT	DIST	C	D		LL+I		RF	
				TOP	BOT	TOP	BOT	TOP	BOT
-	-	FT	KSI	KSI	KSI	KSI	KSI	-	-
3	0	0	26.4	0	0	0	0	--	--
3	1	2.08	26.4	-1.19	1.19	-3.35	3.35	7.88	7.88
3	2	4.16	26.4	-2.24	2.24	-6.28	6.28	4.20	4.20
3	3	6.24	26.4	-3.16	3.16	-8.78	8.78	3.01	3.01
3	4	8.32	26.4	-3.97	3.97	-10.86	10.86	2.43	2.43
3	5	10.41	26.4	-4.65	4.65	-12.47	12.47	2.12	2.12
3	6	12.49	26.4	-5.21	5.21	-13.73	13.73	1.92	1.92
3	7	14.57	26.4	-5.65	5.65	-14.63	14.63	1.80	1.80
3	8	16.65	26.4	-5.97	5.97	-15.23	15.23	1.73	1.73
3	9	18.73	26.4	-6.15	6.15	-15.45	15.45	1.71	1.71
3	**10**	**20.81**	**26.4**	**-6.21**	**6.21**	**-15.52**	**15.52**	**1.70**	1.70
3	11	22.89	26.4	-6.14	6.14	-15.45	15.45	1.71	1.71
3	**12**	**24.97**	**26.4**	**-5.95**	**5.95**	**-15.23**	**15.23**	**1.73**	1.73
3	13	27.06	26.4	-5.64	5.64	-14.63	14.63	1.80	1.80
3	14	29.14	26.4	-5.21	5.21	-13.73	13.73	1.92	1.92
3	15	31.22	26.4	-4.65	4.65	-12.47	12.47	2.12	2.12
3	16	33.3	26.4	-3.97	3.97	-10.86	10.86	2.43	2.43
3	17	35.38	26.4	-3.16	3.16	-8.78	8.78	3.01	3.01
3	18	37.46	26.4	-2.23	2.23	-6.28	6.28	4.20	4.20
3	19	39.54	26.4	-1.18	1.18	-3.35	3.35	7.88	7.88
3	20	41.62	26.4	0	0	0	0	--	--

28 Span 3 Interior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	- (LL+I)	RF
-	-	FT	K	K	K	K	K	-
3	**0**	**0**	**407**	**0**	**25.8**	**79.8**	**0**	**3.41**
3	1	2.08	407	0	22.6	75.1	-1.4	3.67
3	2	4.16	407	0	19.9	70.3	-3.6	3.97
3	3	6.24	407	0	17.5	65.5	-7	4.30
3	4	8.32	407	0	15	60.8	-10.4	4.68
3	5	10.41	407	0	12.5	56	-13.9	5.13
3	6	12.49	407	0	10.1	51.2	-17.3	5.66
3	7	14.57	407	0	7.6	46.5	-20.6	6.29
3	8	16.65	407	0	5.2	41.7	-24.2	7.08
3	9	18.73	407	0	2.3	36.8	-28	8.11
3	10	20.81	407	0	-0.1	32.2	-32.2	9.34
3	11	22.89	407	0	-2.6	28	-36.8	8.10
3	12	24.97	407	0	-5	24.2	-41.7	7.08
3	13	27.06	407	0	-7.5	20.6	-46.5	6.29
3	14	29.14	407	0	-10	17.3	-51.2	5.66
3	15	31.22	407	0	-12.4	13.9	-56	5.13
3	16	33.3	407	0	-15.1	10.4	-60.8	4.67
3	17	35.38	407	0	-17.5	7	-65.5	4.30
3	18	37.46	407	0	-20	3.6	-70.3	3.97
3	19	39.54	407	0	-22.4	1.4	-75.1	3.68
3	20	41.62	407	0	-25.7	0	-79.8	3.41

29 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Flexural Stress Capacity
D = Unfactored Dead Load Stress
LL+I = Unfactored Live Load Stress with Impact
RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{DC_STR}) * D$) / ($\gamma_{LL_STR} * LL+I$)

SP	INT	DIST	C		D	LL+I	RF
-	-	FT	KSI	-	KSI	KSI	-
3	0	0	33	TOP	0	0	--
			33	BOT	0	0	--
3	1	2.08	33	TOP	-1.25	-1.7	13.67
			33	BOT	1.25	1.7	13.67
3	2	4.16	33	TOP	-2.37	-3.18	6.98
			33	BOT	2.37	3.18	6.98
3	3	6.24	33	TOP	-3.36	-4.46	4.77
			33	BOT	3.36	4.46	4.77
3	4	8.32	33	TOP	-4.22	-5.52	3.71
			33	BOT	4.22	5.52	3.71
3	5	10.41	33	TOP	-4.95	-6.37	3.11
			33	BOT	4.95	6.37	3.11
3	6	12.49	33	TOP	-5.55	-7	2.75
			33	BOT	5.55	7	2.75
3	7	14.57	29.37	TOP	-6.01	-7.43	2.18
			33	BOT	6.01	7.43	2.54
3	**8**	**16.65**	**29.37**	**TOP**	**-6.33**	**-7.64**	**2.08**
			33	BOT	6.33	7.64	2.43
3	9	18.73	29.37	TOP	-6.53	-7.48	2.10
			33	BOT	6.53	7.48	2.46
3	**10**	**20.81**	**29.37**	**TOP**	**-6.6**	**-7.42**	**2.10**
			33	BOT	6.6	7.42	2.47
3	11	22.89	29.37	TOP	-6.53	-7.48	2.10
			33	BOT	6.53	7.48	2.46
3	**12**	**24.97**	**29.37**	**TOP**	**-6.33**	**-7.64**	**2.08**
			33	BOT	6.33	7.64	2.43
3	13	27.06	29.37	TOP	-6.01	-7.43	2.18
			33	BOT	6.01	7.43	2.54
3	14	29.14	33	TOP	-5.55	-7	2.75
			33	BOT	5.55	7	2.75
3	15	31.22	33	TOP	-4.95	-6.37	3.11
			33	BOT	4.95	6.37	3.11
3	16	33.3	33	TOP	-4.22	-5.52	3.71
			33	BOT	4.22	5.52	3.71
3	17	35.38	33	TOP	-3.36	-4.46	4.77
			33	BOT	3.36	4.46	4.77
3	18	37.46	33	TOP	-2.37	-3.18	6.98
			33	BOT	2.37	3.18	6.98
3	19	39.54	33	TOP	-1.25	-1.7	13.67
			33	BOT	1.25	1.7	13.67
3	20	41.62	33	TOP	0	0	--
			33	BOT	0	0	--

29 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV2 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D		LL+I		RF	
				TOP	BOT	TOP	BOT	TOP	BOT
-	-	FT	KSI	KSI	KSI	KSI	KSI	-	-
3	0	0	26.4	0	0	0	0	--	--
3	1	2.08	26.4	-1.25	1.25	-1.7	1.7	15.53	15.53
3	2	4.16	26.4	-2.37	2.37	-3.18	3.18	8.30	8.30
3	3	6.24	26.4	-3.36	3.36	-4.46	4.46	5.92	5.92
3	4	8.32	26.4	-4.22	4.22	-5.52	5.52	4.78	4.78
3	5	10.41	26.4	-4.95	4.95	-6.37	6.37	4.14	4.14
3	6	12.49	26.4	-5.55	5.55	-7	7	3.77	3.77
3	7	14.57	26.4	-6.01	6.01	-7.43	7.43	3.55	3.55
3	**8**	**16.65**	**26.4**	**-6.33**	**6.33**	**-7.64**	**7.64**	**3.46**	3.46
3	9	18.73	26.4	-6.53	6.53	-7.48	7.48	3.53	3.53
3	**10**	**20.81**	**26.4**	**-6.6**	**6.6**	**-7.42**	**7.42**	**3.56**	3.56
3	11	22.89	26.4	-6.53	6.53	-7.48	7.48	3.53	3.53
3	**12**	**24.97**	**26.4**	**-6.33**	**6.33**	**-7.64**	**7.64**	**3.46**	3.46
3	13	27.06	26.4	-6.01	6.01	-7.43	7.43	3.55	3.55
3	14	29.14	26.4	-5.55	5.55	-7	7	3.77	3.77
3	15	31.22	26.4	-4.95	4.95	-6.37	6.37	4.14	4.14
3	16	33.3	26.4	-4.22	4.22	-5.52	5.52	4.78	4.78
3	17	35.38	26.4	-3.36	3.36	-4.46	4.46	5.92	5.92
3	18	37.46	26.4	-2.37	2.37	-3.18	3.18	8.30	8.30
3	19	39.54	26.4	-1.25	1.25	-1.7	1.7	15.53	15.53
3	20	41.62	26.4	0	0	0	0	--	--

30 Span 3 Exterior Beam Shear 0.0L of Span - EV2 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	- (LL+I)	RF
-	-	FT	K	K	K	K	K	-
3	**0**	**0**	**407**	**0**	**26.9**	**36.4**	**0**	**7.45**
3	1	2.08	407	0	23.9	34.2	-1.2	8.02
3	2	4.16	407	0	21.3	32.1	-2.5	8.64
3	3	6.24	407	0	18.6	29.9	-3.7	9.37
3	4	8.32	407	0	16	27.8	-5	10.19
3	5	10.41	407	0	13.4	25.7	-6.1	11.13
3	6	12.49	407	0	10.7	23.5	-7.5	12.30
3	7	14.57	407	0	7.9	21.4	-8.9	13.65
3	8	16.65	407	0	5.3	19.2	-10.7	15.37
3	9	18.73	407	0	2.6	17.1	-12.8	17.43
3	10	20.81	407	0	0	15	-15	20.06
3	11	22.89	407	0	-2.6	12.8	-17.1	17.43
3	12	24.97	407	0	-5.3	10.7	-19.2	15.37
3	13	27.06	407	0	-7.9	8.9	-21.4	13.65
3	14	29.14	407	0	-10.7	7.5	-23.5	12.30
3	15	31.22	407	0	-13.4	6.1	-25.7	11.13
3	16	33.3	407	0	-16	5	-27.8	10.19
3	17	35.38	407	0	-18.6	3.7	-29.9	9.37
3	18	37.46	407	0	-21.3	2.5	-32.1	8.64
3	19	39.54	407	0	-23.9	1.2	-34.2	8.02
3	20	41.62	407	0	-26.9	0	-36.4	7.45

31 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating

MDash Table 1.2.32.1 - Maximum Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Flexural Stress Capacity
D = Unfactored Dead Load Stress
LL+I = Unfactored Live Load Stress with Impact
RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{DC_STR}) * D$) / ($\gamma_{LL_STR} * LL+I$)

SP	INT	DIST	C		D	LL+I	RF
-	-	FT	KSI	-	KSI	KSI	-
3	0	0	33	TOP	0	0	--
			33	BOT	0	0	--
3	1	2.08	33	TOP	-1.25	-2.5	9.30
			33	BOT	1.25	2.5	9.30
3	2	4.16	33	TOP	-2.37	-4.69	4.74
			33	BOT	2.37	4.69	4.74
3	3	6.24	33	TOP	-3.36	-6.56	3.25
			33	BOT	3.36	6.56	3.25
3	4	8.32	33	TOP	-4.22	-8.11	2.53
			33	BOT	4.22	8.11	2.53
3	5	10.41	33	TOP	-4.95	-9.31	2.13
			33	BOT	4.95	9.31	2.13
3	6	12.49	33	TOP	-5.55	-10.25	1.88
			33	BOT	5.55	10.25	1.88
3	7	14.57	29.37	TOP	-6.01	-10.92	1.48
			33	BOT	6.01	10.92	1.73
3	**8**	**16.65**	**29.37**	**TOP**	**-6.33**	**-11.37**	**1.40**
			33	BOT	6.33	11.37	1.63
3	9	18.73	29.37	TOP	-6.53	-11.53	1.36
			33	BOT	6.53	11.53	1.59
3	**10**	**20.81**	**29.37**	**TOP**	**-6.6**	**-11.59**	**1.35**
			33	BOT	6.6	11.59	1.58
3	11	22.89	29.37	TOP	-6.53	-11.53	1.36
			33	BOT	6.53	11.53	1.59
3	**12**	**24.97**	**29.37**	**TOP**	**-6.33**	**-11.37**	**1.40**
			33	BOT	6.33	11.37	1.63
3	13	27.06	29.37	TOP	-6.01	-10.92	1.48
			33	BOT	6.01	10.92	1.73
3	14	29.14	33	TOP	-5.55	-10.25	1.88
			33	BOT	5.55	10.25	1.88
3	15	31.22	33	TOP	-4.95	-9.31	2.13
			33	BOT	4.95	9.31	2.13
3	16	33.3	33	TOP	-4.22	-8.11	2.53
			33	BOT	4.22	8.11	2.53
3	17	35.38	33	TOP	-3.36	-6.56	3.25
			33	BOT	3.36	6.56	3.25
3	18	37.46	33	TOP	-2.37	-4.69	4.74
			33	BOT	2.37	4.69	4.74
3	19	39.54	33	TOP	-1.25	-2.5	9.30
			33	BOT	1.25	2.5	9.30
3	20	41.62	33	TOP	0	0	--
			33	BOT	0	0	--

31 Span 3 Exterior Beam Positive Moment 0.5L of Span - EV3 Rating

MDash Table 1.2.32.2 - Serviceability Strength for Moment Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = 0.95Fy or 0.80Fy
D = Unfactored Dead Load Stress D1 at Bottom
LL+I = Unfactored Live Load Stress with Impact at Bottom

RF = Rating Factor
= ($\varphi c * \varphi s * \varphi f * C - (\gamma_{D_SRV}) * D$) / ($\gamma_{LL_SVR} * LL+I$)

SP	INT	DIST	C	D		LL+I		RF	
				TOP	BOT	TOP	BOT	TOP	BOT
-	-	FT	KSI	KSI	KSI	KSI	KSI	-	-
3	0	0	26.4	0	0	0	0	--	--
3	1	2.08	26.4	-1.25	1.25	-2.5	2.5	10.56	10.56
3	2	4.16	26.4	-2.37	2.37	-4.69	4.69	5.63	5.63
3	3	6.24	26.4	-3.36	3.36	-6.56	6.56	4.02	4.02
3	4	8.32	26.4	-4.22	4.22	-8.11	8.11	3.26	3.26
3	5	10.41	26.4	-4.95	4.95	-9.31	9.31	2.84	2.84
3	6	12.49	26.4	-5.55	5.55	-10.25	10.25	2.58	2.58
3	7	14.57	26.4	-6.01	6.01	-10.92	10.92	2.42	2.42
3	**8**	**16.65**	**26.4**	**-6.33**	**6.33**	**-11.37**	**11.37**	**2.32**	2.32
3	9	18.73	26.4	-6.53	6.53	-11.53	11.53	2.29	2.29
3	**10**	**20.81**	**26.4**	**-6.6**	**6.6**	**-11.59**	**11.59**	**2.28**	2.28
3	11	22.89	26.4	-6.53	6.53	-11.53	11.53	2.29	2.29
3	**12**	**24.97**	**26.4**	**-6.33**	**6.33**	**-11.37**	**11.37**	**2.32**	2.32
3	13	27.06	26.4	-6.01	6.01	-10.92	10.92	2.42	2.42
3	14	29.14	26.4	-5.55	5.55	-10.25	10.25	2.58	2.58
3	15	31.22	26.4	-4.95	4.95	-9.31	9.31	2.84	2.84
3	16	33.3	26.4	-4.22	4.22	-8.11	8.11	3.26	3.26
3	17	35.38	26.4	-3.36	3.36	-6.56	6.56	4.02	4.02
3	18	37.46	26.4	-2.37	2.37	-4.69	4.69	5.63	5.63
3	19	39.54	26.4	-1.25	1.25	-2.5	2.5	10.56	10.56
3	20	41.62	26.4	0	0	0	0	--	--

32 Span 3 Exterior Beam Shear 0.0L of Span - EV3 Rating

MDash Table 1.2.33.1 - Maximum Strength for Shear Rating

SP = Span Number
INT = Interval Number
DIST = Distance from Left Support
C = Shear Capacity
D1 = Unfactored Dead Load Shear D1
D2 = Unfactored Dead Load Shear D2
LL+I = Unfactored Live Load Shear with Impact (Positive)
-(LL+I) = Unfactored Live Load Shear with Impact (Negative)

RF = Rating Factor
= ($\varphi c*\varphi s*\varphi v*C - (\gamma_{DC_STR})*D1 - (\gamma_{DW_STR})*D2) / (\gamma_{LL_STR}*LL+I)$

SP	INT	DIST	C	D1	D2	LL+I	- (LL+I)	RF
-	-	FT	K	K	K	K	K	-
3	**0**	**0**	**407**	**0**	**26.9**	**53.7**	**0**	**5.05**
3	1	2.08	407	0	23.9	50.5	-0.9	5.43
3	2	4.16	407	0	21.3	47.3	-2.4	5.86
3	3	6.24	407	0	18.6	44.1	-4.7	6.36
3	4	8.32	407	0	16	40.8	-7	6.94
3	5	10.41	407	0	13.4	37.6	-9.3	7.61
3	6	12.49	407	0	10.7	34.4	-11.6	8.40
3	7	14.57	407	0	7.9	31.2	-13.8	9.36
3	8	16.65	407	0	5.3	28	-16.2	10.54
3	9	18.73	407	0	2.6	24.7	-18.8	12.07
3	10	20.81	407	0	0	21.6	-21.6	13.93
3	11	22.89	407	0	-2.6	18.8	-24.7	12.07
3	12	24.97	407	0	-5.3	16.2	-28	10.54
3	13	27.06	407	0	-7.9	13.8	-31.2	9.36
3	14	29.14	407	0	-10.7	11.6	-34.4	8.40
3	15	31.22	407	0	-13.4	9.3	-37.6	7.61
3	16	33.3	407	0	-16	7	-40.8	6.94
3	17	35.38	407	0	-18.6	4.7	-44.1	6.36
3	18	37.46	407	0	-21.3	2.4	-47.3	5.86
3	19	39.54	407	0	-23.9	0.9	-50.5	5.43
3	20	41.62	407	0	-26.9	0	-53.7	5.05

APPENDIX C

SECTION LOSSES SKETCHES

(See CD)

(Not APPLICABLE)