

# Bridge Load Rating

*Prepared for*

## Maine Department of Transportation

**Bridge No. 5799**

**BANGOR**

**I-395**

**OVER**

**ROUTES US 1A & 9**

**Date of Inspection: November 18, 2011**

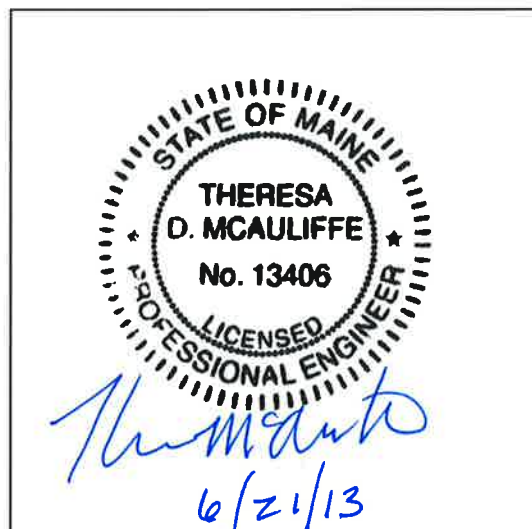
**Date of Rating: June 18, 2013**

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



**THE Louis Berger Group, INC.**

482 Congress Street, Suite 401 | Portland, ME 04101 | [www.louisberger.com](http://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: November 18, 2011**

**Date of Rating: July 31, 2019**

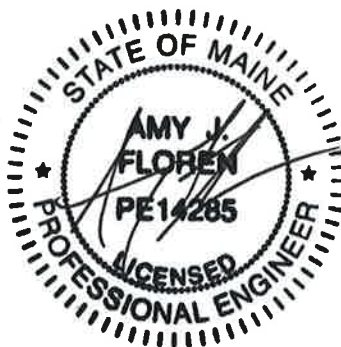
**Rating Factors are determined for the FAST Act's Emergency Vehicle Configurations Types EV2 and EV3 at the legal load rating level in accordance with AASHTO MBE.**

**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.: 5799  
 Town / City: Bangor  
 Route Carried: I-395  
 Crosses: Routes US 1A & 9

Owner: MaineDOT  
 Maintainer: MaineDOT  
 Year Built: 1986  
 Year(s) Rebuilt / Rehab: 2011

## SUMMARY OF BRIDGE RATING

VEHICLE TYPE		RF	RT (TONS)	POSTING LOAD (TONS)
HL-93	INVENTORY	1.15	41.40	
	OPERATING	1.49	53.64	
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		2.67	76.76	
(Updated 7/31/19) TYPE EV3		1.76	75.68	

### 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: 0.807  
 Live Load DF Routine Commercial: 0.807  
 Live Load DF Special Hauling: 0.807  
 Impact Factor: 1.33  
 Governing Condition Factor,  $\Phi_c$ : 1.00  
 System Factor,  $\Phi_s$ : 1.00  
 ADTT (one-way): 1190

### *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 \_\_\_\_\_ years

## **BREAKDOWN OF BRIDGE RATING**

Town / City: Bangor  
 Bridge No.: 5799

Route Carried: I-395  
 Crosses: Routes US 1A & 9

## **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
Interior Girder Positive Moment 0.5L of Span Left Bridge	1.21	1.57											2.82	1.86
Interior Girder Shear 0.05L of Span Left Bridge	1.83	2.38											4.14	2.78
<b>Exterior Girder Positive Moment 0.5L of Span Left Bridge</b>	<b>1.15</b>	<b>1.49</b>											<b>2.67</b>	<b>1.76</b>
Exterior Girder Shear 0.05L of Span Left Bridge	2.07	2.68											4.67	3.14
<b>CONTROLLING RATING FACTORS</b>	<b>1.15</b>	<b>1.49</b>											<b>2.67</b>	<b>1.76</b>

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## **DESCRIPTION OF BRIDGE**

Bridge Number: 5799  
Owner: MaineDOT  
Maintained By: MaineDOT  
Location: Bangor  
Route Carried: I-395  
Feature Intersected: Routes US 1A & 9  
Latest NBI Inspection Date: November 18, 2011  
Field Verification Date: March 25, 2013  
Date of Construction: 1986  
Bridge Type: Simple span steel girders.  
Material Properties: Steel: Girder flanges specified as ASTM A-572 ( $f_y = 50$  ksi);  
Girder web and other steel specified as ASTM A36 ( $f_y = 36$  ksi).  
Concrete: Unknown, based on year of construction and MBE Table 6A.5.2.1-1 ( $f'_c = 3.0$  ksi).  
Original Design Loading: HS25 (Modified for Interstate)  
Date(s) of Rebuild/Rehab: 2011  
Description of Rebuild/Rehab: New asphalt wearing surface, compression seals and new joint headers based on MaineDOT bridge inspection photos.  
Posting: Open, no restrictions  
Superstructure: Two Bridges:  
Right Bridge: Nine steel plate girders with composite cast in place composite concrete deck.  
Left Bridge: Six steel plate girders with a composite cast in place composite concrete deck.  
Substructure: Cast in place concrete abutments on piles/spread footings.  
Bearings: Steel pedestals  
Bridge Spans: 111'-6"  
Bridge Skew: 7°-54'-02" Back left  
Bridge Width: 140' - 0 1/4" ± out-to-out at mid-span (Varies due to curved fascia on left bridge)  
Roadway Width: Right Bridge: 82'-10 1/2" curb-to-curb  
Left Bridge: 50'-10 1/4" curb-to-curb at mid-span (Varies due to curved fascia)  
Roadway Surface: Bituminous Concrete  
Curbs: Granite  
Sidewalk/Walkway/Median: Raised Median on Right Bridge  
Utilities: Aerial utilities supported by the outside exterior girders on the Left and Right Bridges.  
Bridge Railing: Two bar aluminum rail with snow fence and concrete barrier Type IIIA  
Approach Railing: Concrete endposts with bridge transition Type I with guardrail

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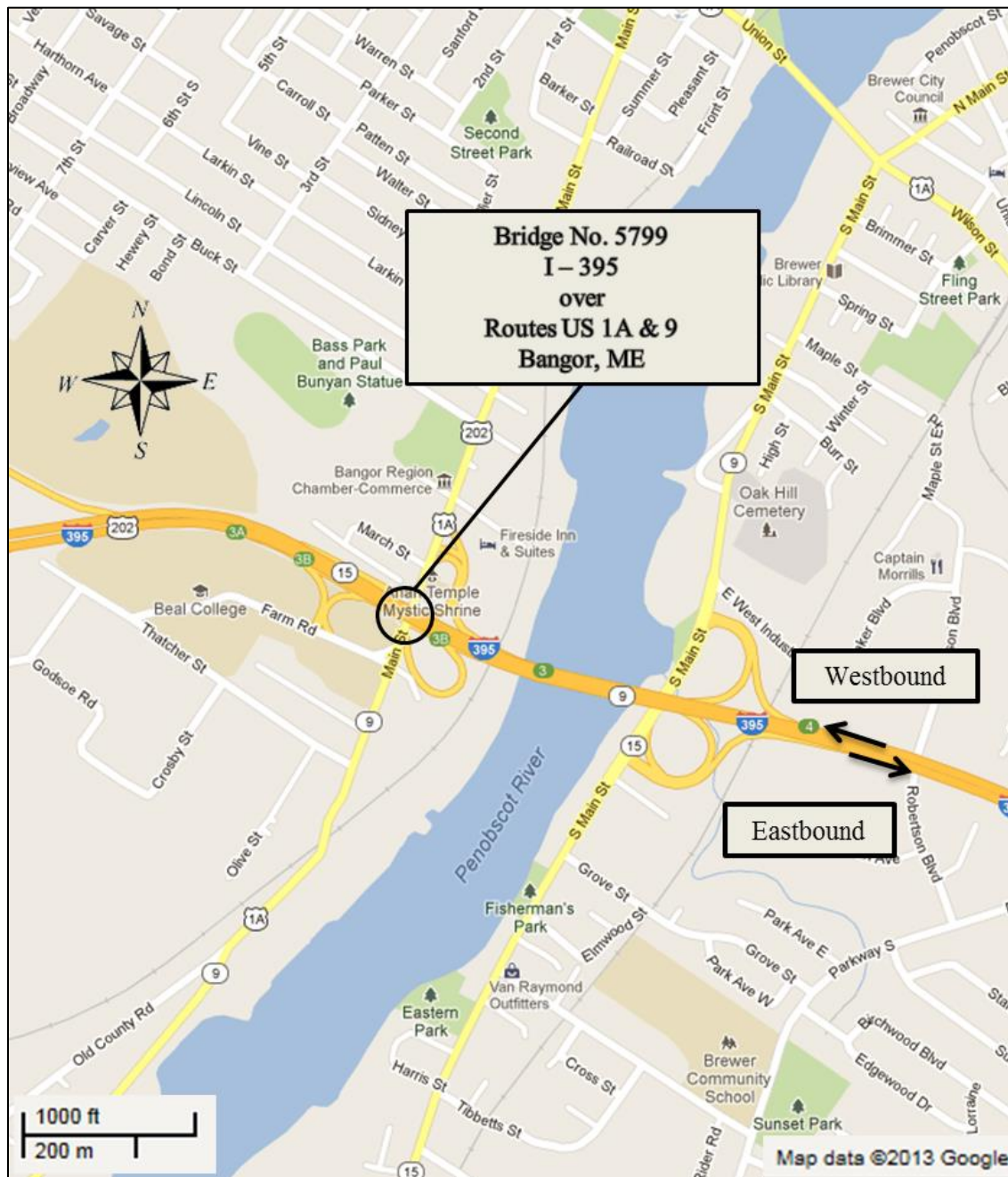
Wearing Surface Condition:	Good (Field Observation)
Bridge Railing Condition:	Satisfactory (Field Observation)
Deck Condition:	Good (SI&A)
Beam Condition:	Good (SI&A)
Bearing Condition:	Satisfactory (Field Observation)
Abutment Condition:	Satisfactory (SI&A)
Pier Condition:	N/A

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APPENDIX C – SECTION LOSS SKETCHES	

## LOCATION MAP





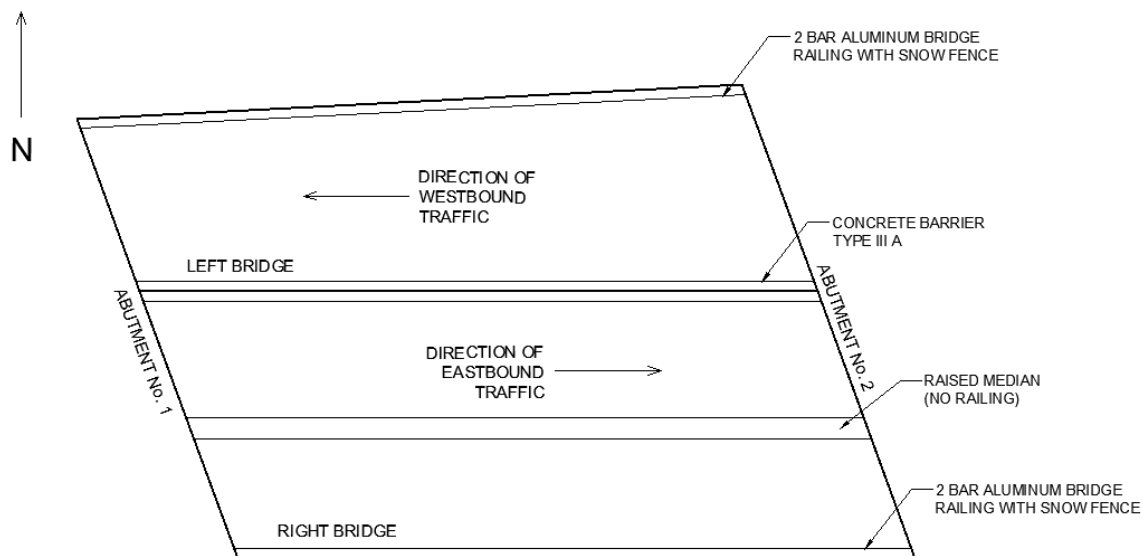
## **EXISTING BRIDGE CONDITION**

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

For the purposes of this report, please refer to Figure 1 for the terminology used. The terms Left Bridge and Right Bridge are utilized to distinguish between the two independent bridge superstructures located at the site. The Left and Right terms refer to the stationing on the 1984 bridge construction plans.

### **Site Observations**

The overall geometry of the left and right bridges was measured. Dimensions were recorded to verify the provided 1984 bridge construction plans. The original asphalt wearing surface and joints were replaced in 2011 based on the latest bridge inspection photos. The bridge deck was observed to be in good condition. The bridge superstructure was observed to be in good condition with minor rust and paint failure on the primary members. This does not impact the member capacity for purposes of this load rating. The bearings are in satisfactory condition. Two bridge mounted signs are bolted to the outside exterior girder and curb of both the Left and Right bridges.



**Figure 1: Bridge No. 5799 Terminology**

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# **ASSUMPTIONS, CRITERIA, AND REFERENCES**

## **Assumptions and Criteria**

The analysis of Bridge No. 5799 was performed per 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 girders for the Left Bridge; shear at the ends and moment at mid-span. The rating for the Left Bridge governs over the Right Bridge due to the following: The interior girders have a larger spacing. Therefore, there is more dead load and live load. The exterior girder on the outside of the Left Bridge does not carry any live load except for a very minor amount at the flared end for the on-ramp due to the layout of the striped lanes. All of the other overhangs are the same. Since the Left Bridge has the larger girder spacing, the inside exterior girder on the Left Bridge controls over the other exterior girders due to the larger tributary width and relative location of the girder to the edge of the striped lane.

### ***Structural Analysis:***

- Deterioration: None noted. Reduction of steel girder capacity was not considered.
- Composite: Yes. Shear studs shown on plans.
- Unbraced Length: Fully braced; composite construction and top flange is embedded in the concrete.
- Haunch / Blocking: The steel plate girders are cambered for all dead load. From the plans, the blocking is 2 inches at the supports and is utilized to account for dead load deflection. The overcamber tolerance is subtracted when calculating composite section properties.
- Stiffeners: 10 stiffeners and diaphragm connection plates.
- Girder Section Properties: Calculated based upon steel plate girder dimensions given on the 1984 bridge construction plans.

### ***Dead Load:***

- Factors:  $\gamma_{DC} = 1.25$  per MBE 6A.2.2.1  
 $\gamma_{DW} = 1.50$  per MBE 6A.2.2.1
- Railing: The 2 bar aluminum rail was assumed 25 plf based on NH DOT standard details and the Type III rail was calculated 400 plf.
- Deck: 9.5 inches thick based on plans.
- Blocking: Applied as distributed load. Assumed constant thickness utilizing the maximum thickness at the bearings.
- Wearing Surface: 3 inch bituminous concrete wearing surface with ¼ inch membrane based upon the 1984 bridge construction plans.
- Curb: Granite – per plans (confirmed in field).
- Fence: Snow fence on the outside of each bridge.
- Cross-Frames: K diaphragms with WT sections, applied as point loads.

**Dead Load Distribution:**

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

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

- Factor: The load factors are summarized below:

	Strength	Service
HL-93	1.75	1.30

- Dynamic Allowance: IM = 1.33 for all AASHTO and Legal Loads (Rating Guide Section 3.2)
- Loads: HL-93, Maine Legal Loads, Fast Act Emergency Vehicles – See loading figures on pages 6 through 9.
- Number of Lanes: Seven lanes based on current striping. Three and four lanes for the Left and Right Bridges, respectively,

**Live Load Distribution:**

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

The live load distribution factors for the exterior girders were determined in accordance with AASHTO LRFD Section 4.6.2.2d. For the rigid body rotation and lever rule calculations, the position of the vehicles and the number of lanes are based on the existing striped lanes per Rating Guide 3.3.1.7. The first wheel is assumed to be on the white stripe.

The live load distribution factors corrected for skew = 7°-54'-02" are summarized below:

Element	Moment	Shear
Interior Girders – Left Bridge	0.734	0.969
Exterior Girders – Left Bridge	0.807	0.872

**Condition Factor:** (Rating Guide Section 3.5.1)

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

**System Factor:** (Rating Guide Section 3.5.2)

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

**Material Properties:**

Steel: Based on the 1984 bridge construction plans, the girder flanges are specified to be ASTM A-572 ( $f_y = 50$  ksi) and the webs and all other components are

specified to be ASTM A36 ( $f_y = 36$  ksi). Since the top flange of the girder is embedded in the concrete deck, it is considered to be continuously braced. Lateral torsional buckling does not control the girder capacity.

Concrete: Deck is assumed to be normal weight concrete with a compressive strength  $f'_c = 3.0$  ksi based on MBE Table 6A.5.2.1-1 and the year constructed.

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

No serviceability related issues were noted. There are no category C or lower fatigue-prone details noted on the bridge, therefore, fatigue life was not evaluated in this analysis.

**References**

MBE	AASHTO Manual for Bridge Evaluation, 2nd Edition, with 2011 and 2013 Interims.
AASHTO LRFD	AASHTO <i>LRFD Bridge Design Specifications</i> , 5th Edition, 2010, with 2010 Interim Revisions.
BDG	MaineDOT <i>Bridge Design Guide</i> , 2003, with revisions through August 2008.
Rating Guide	MaineDOT <i>Draft Load Rating Guide</i> , November 2011.
SI&A	MaineDOT Structure Inventory and Appraisal Sheet, November 18, 2011

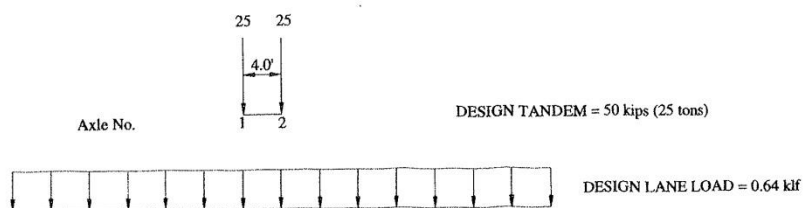
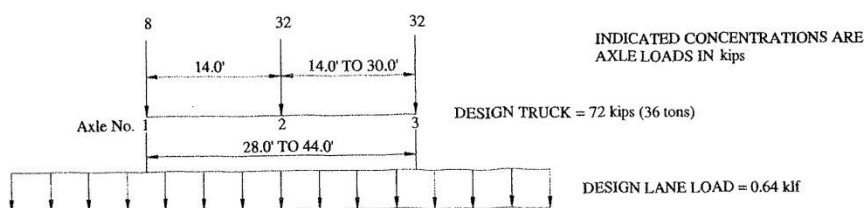
# LOADINGS USED FOR BRIDGE RATINGS

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

6-66

THE MANUAL FOR BRIDGE EVALUATION

## APPENDIX 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%)

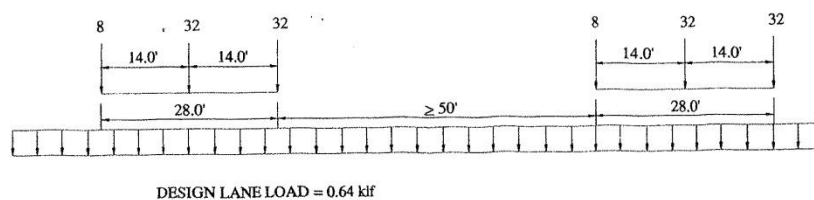
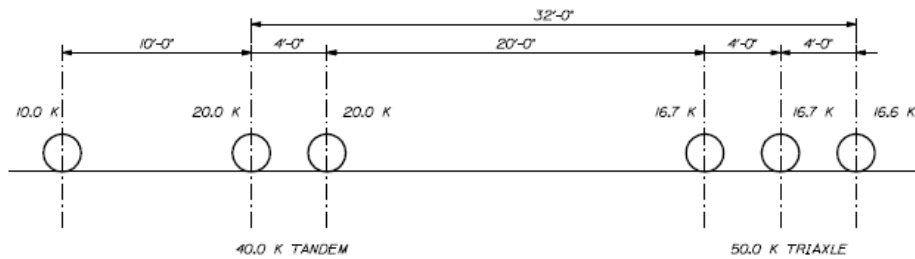


Figure C6A-1—LRFD Design Live Load (HL-93)

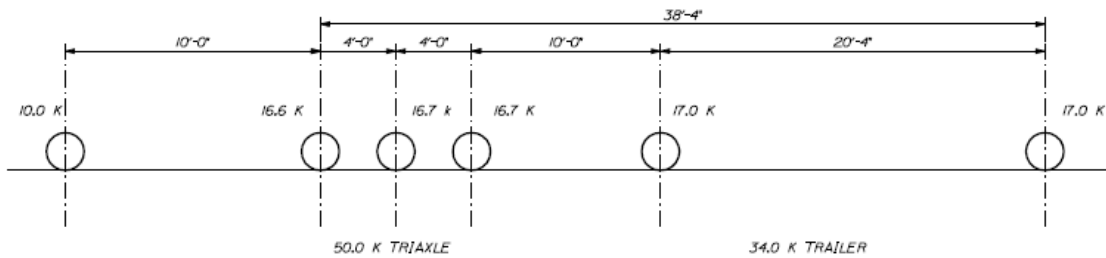
# LOADINGS USED FOR BRIDGE RATINGS, CONT'D

## Maine Legal Load Configurations:



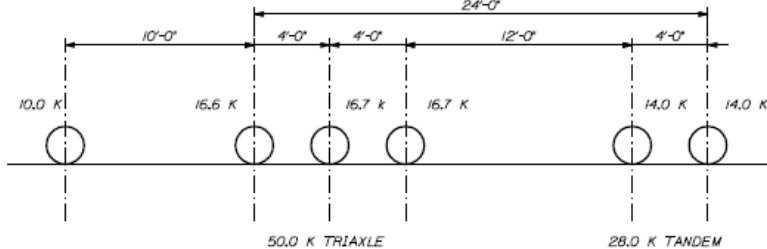
CONFIGURATON 1

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



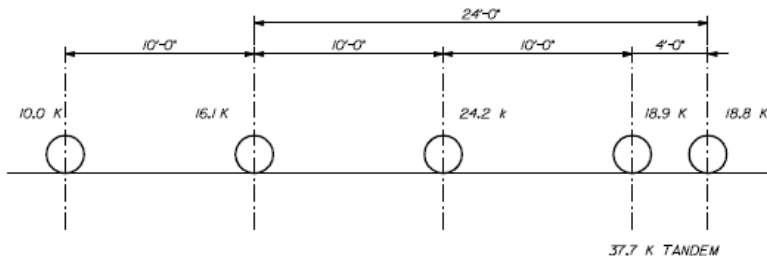
CONFIGURATON 2

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



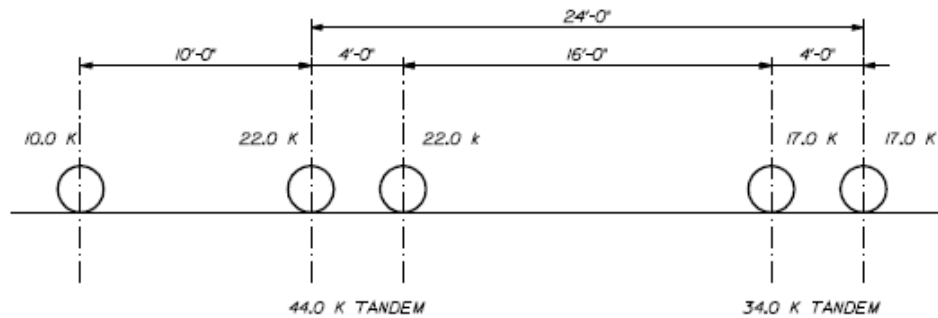
CONFIGURATON 4

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

REVISION 03-07-11

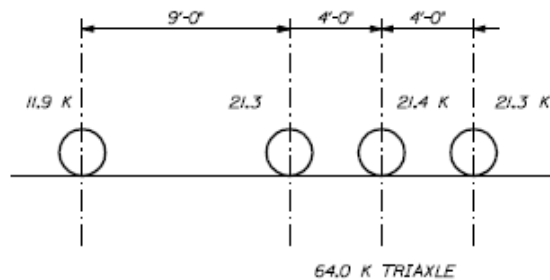
# LOADINGS USED FOR BRIDGE RATINGS, CONT'D

Maine Legal Load Configurations:



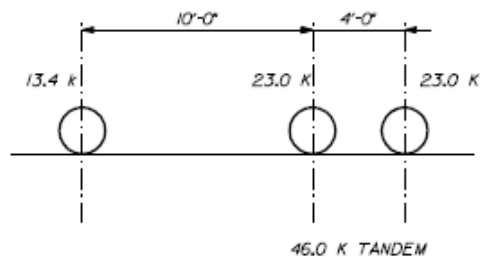
CONFIGURATON 5

FIVE AXLE  
TANDEM AXLE TRUCK  
OR TRACTOR SEMI-TRAILER  
88.0 K GVW



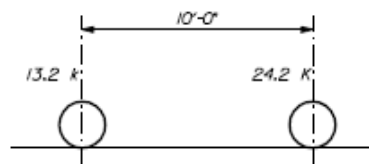
CONFIGURATON 6

FOUR AXLE TRUCK  
75.9 K GVW



CONFIGURATON 7

THREE AXLE TRUCK  
59.0 K GVW



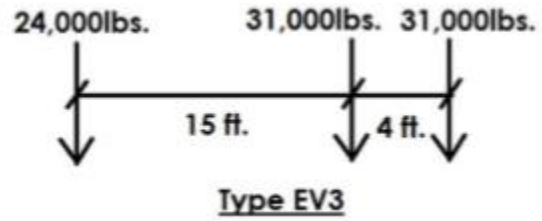
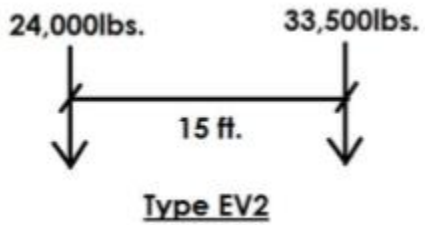
CONFIGURATON 8

TWO AXLE TRUCK  
37.4 K GVW

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## **LOADINGS USED FOR BRIDGE RATINGS, CONT'D**

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





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# **PHOTOGRAPHS**

**Photo No. 1**

**Location:**  
Bridge Approach

**Description:**  
Eastbound  
Approach (Right  
Bridge)



**Photo No. 2**

**Location:**  
Northerly  
Elevation

**Description:**  
Northerly  
Elevation



**Photo No. 3**

**Location:**  
Southerly  
Elevation

**Description:**  
Southerly  
Elevation



**Photo No. 4**

**Location:**  
Route 1A & 9

**Description:**  
Abutment No. 1  
Elevation



**Photo No. 5**

**Location:**  
Route 1A & 9

**Description:**  
Abutment No. 2  
Elevation



**Photo No. 6**

**Location:**  
Underside of  
Bridge

**Description:**  
Typical Girder Bay  
(Right Bridge)





**Photo No. 7**

**Location:**  
Underside of  
Bridge

**Description:**  
Typical Girder Bay  
(Left Bridge)



**Photo No. 8**

**Location:**  
Underside of  
Bridge

**Description:**  
Joint between Left  
and Right Bridges



**Photo No. 9**

**Location:**  
Underside of  
Bridge

**Description:**  
Typical Transverse  
Stiffeners



**Photo No. 10**

**Location:**  
Abutment

**Description:**  
Typical Expansion  
Bearing



**Photo No. 11**

**Location:**  
Underside of  
Bridge

**Description:**  
Typical Interior  
Diaphragm

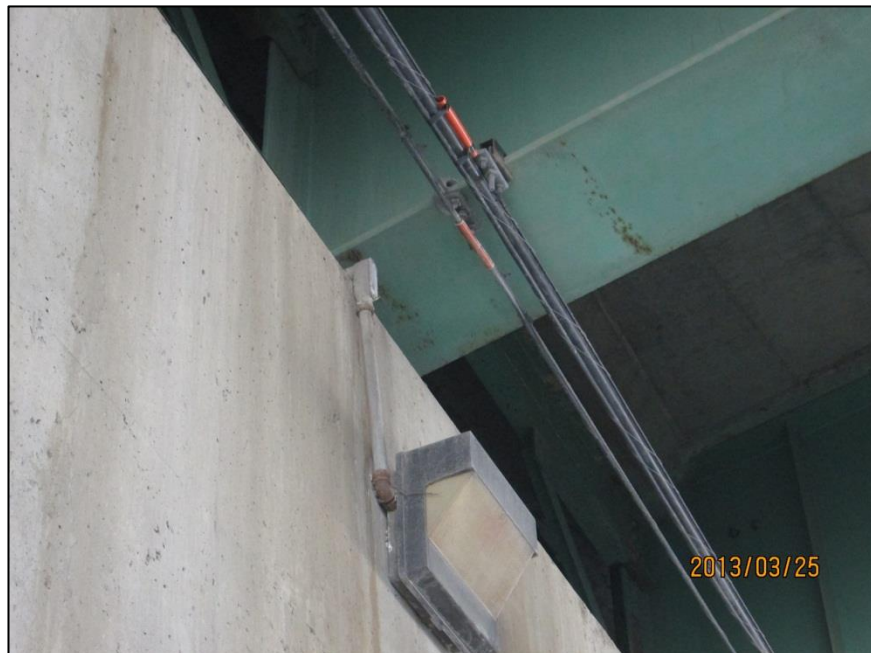


**Photo No. 12**

**Location:**  
Underside of  
Bridge

**Description:**  
Typical Utility  
Line Supported by  
Outside Girder

Note: Utility line  
runs transversely.





**Photo No. 13**

**Location:**  
Bridge Deck

**Description:**  
Abutment No. 2  
Joint



**Photo No. 14**

**Location:**  
Bridge Elevation

**Description:**  
Typical Snow  
Fence





**Photo No. 15**

**Location:**  
Bridge Approach

**Description:**  
Bridge Rail  
Transition



**Photo No. 16**

**Location:**  
Bridge Approach

**Description:**  
Type IIIA Bridge  
Rail Endpost and  
Transition



**Photo No. 17**

**Location:**  
Southerly  
Elevation

**Description:**  
Road Signs  
Attached to Bridge  
Fascia on Right  
Bridge



**Photo No. 18**

**Location:**  
Northerly  
Elevation

**Description:**  
Road Signs  
Attached to Bridge  
Fascia on Left  
Bridge



## **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: 5799	Agency ID: 5799	SR: 96.5	SD/FO: ND
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### IDENTIFICATION

State 1: 23 Maine Struc Num 8: 5799  
Facility Carried 7: INTERSTATE 395 Location 9: I395 INTERCHANGE #3  
Rte.(On/Under)5A: Route On Structure Rte. Signing Prefix 5B: 3 State Hwy  
Level of Service 5C: 1 Mainline Rte. Number 5D: 00395  
Directional Suffix 5E: 0 N/A (NBI) % Responsibility : 0  
SHD District 2: 04 Eastern County Code 3: 019 Penobscot  
Place Code 4: 19020 Bangor Kilometer Post 11: 02.8 km  
Feature Intersected 6: ROUTES US 1A & 9  
Latitude 16: 44d 47' 06" Longitude 17: 068d 46' 54"  
Border Bridge Code 98: Not Applicable (P)  
Border Bridge Number 99: n/a

### INSPECTION

Frequency 91: 24 months Inspection Date 90: 11/18/2011 Next Inspection: 11/18/2013  
FC Frequency 92A: NA FC Inspection Date 93A: NA Next FC Inspection: NA  
UW Frequency 92B: NA UW Inspection Date 93B: NA Next UW Inspection: NA  
SI Frequency 92C: NA SI Date 93C: NA Next SI: NA  
Element Frequency: 24 months Element Inspection Date: 11/18/2011 Next Elem. Insp. Due: 11/18/2013

### STRUCTURE TYPE AND MATERIALS

Number of Approach Spans 46: 0 Number of Spans Main Unit 45: 1  
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

### CLASSIFICATION

Defense Highway 100: 1 On Interstate STRAHNET Parallel Structure 101: No || bridge exists  
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

### AGE AND SERVICE

Year Built 27: 1986 Year Reconstructed 106: -4  
Type of Service on 42A: 1 Highway  
Type of Service under 42B: 1 Highway  
Lanes on 28A: 7 Lanes Under 28B: 6 Detour Length 19: 2.6 km  
ADT 29: 11,900 Truck ADT 109: 10 % Year of ADT 30: 2011

### CONDITION

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

### GEOMETRIC DATA

Length Max Span 48: 34.14 m Structure Length 49: 35.66 m  
Curb/Sdwk Width L 50A: 0.18 m Curb/Sidewalk Width R 50B: 0.18 m  
Width Curb to Curb 51: 38.83 m Width Out to Out 52: 43.16 m  
Approach Roadway Width 32:(w/ shoulders) 11.58 m Median 33: 3 Closed Med w/Barriers  
Deck Area: 1,539.17 m²  
Skew 34: 8.00 ° Structure Flared 35: 0 No flare  
Vertical Clearance 10: 99.99 m Horiz. Clearance 47: 24.99 m  
Minimum Vertical Clearance Over Bridge 53: 99.90 m  
Minimum Vertical Underclearance Reference 54A: H Hwy beneath struct  
Minimum Vertical Underclearance 54B: 05.41 m  
Minimum Lateral Underclearance Reference R 55A: H Hwy beneath struct  
Minimum Lateral Underclearance R 55: 08.39 m  
Minimum Lateral Underclearance L 56: 00.00 m

### LOAD RATING AND POSTING

Inventory Rating Method 65: 1 LF Load Factor Operating Rating Method 63: 1 LF Load Factor  
Inventory Rating 66: MS24.7 Operating Rating 64: MS41.3  
Design Load 31: MS 22.5 or greater 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: 9 Above Desirable Crit  
Underclearance, Vertical and Horizontal 69: 7 Above 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: 16,660  
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.	1,539	0 %	0	100 %	1,539	0 %	0	0 %	0	0 %	0
1	107/2	Paint Stl Opn Girder	m.	535	90 %	481	5 %	27	3 %	16	2 %	11	0 %	0
1	215/2	R/Conc Abutment	m.	86	70 %	60	28 %	24	2 %	2	0 %	0	0 %	0
1	218/2	Undefined Wall Elem.	m.	44	40 %	18	50 %	22	9 %	4	1 %	0	0 %	0
1	302/2	Compressn Joint Seal	m.	86	0 %	0	75 %	65	25 %	22	0 %	0	0 %	0

## Structure Inventory and Appraisal Sheet

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	311/2	Moveable Bearing	ea.	15	0 %	0	0 %	0	100 %	15	0 %	0	0 %	0
1	313/2	Fixed Bearing	ea.	15	0 %	0	100 %	15	0 %	0	0 %	0	0 %	0
1	331/2	Conc Bridge Railing	m.	36	80 %	29	20 %	7	0 %	0	0 %	0	0 %	0
1	333/2	Other Bridge Railing	m.	102	98 %	100	2 %	2	0 %	0	0 %	0	0 %	0
1	360/2	Settlement SmFlag	ea.	1	100 %	1	0 %	0	0 %	0	0 %	0	0 %	0
1	383/2	Wear.Surf- AC+Membr.	sq.m.	1,385	80 %	1,108	20 %	277	0 %	0	0 %	0	0 %	0
1	388/2	Paint	sq.m.	2,474	92 %	2,276	5 %	124	2 %	49	1 %	25	0 %	0
1	389/2	Reinfor conc dk/slab	sq.m.	1,539	33 %	508	33 %	508	34 %	523	0 %	0	0 %	0

### BASIC DESIGN STRESSES

STRUCTURAL STEEL: A572  
A36  
A325

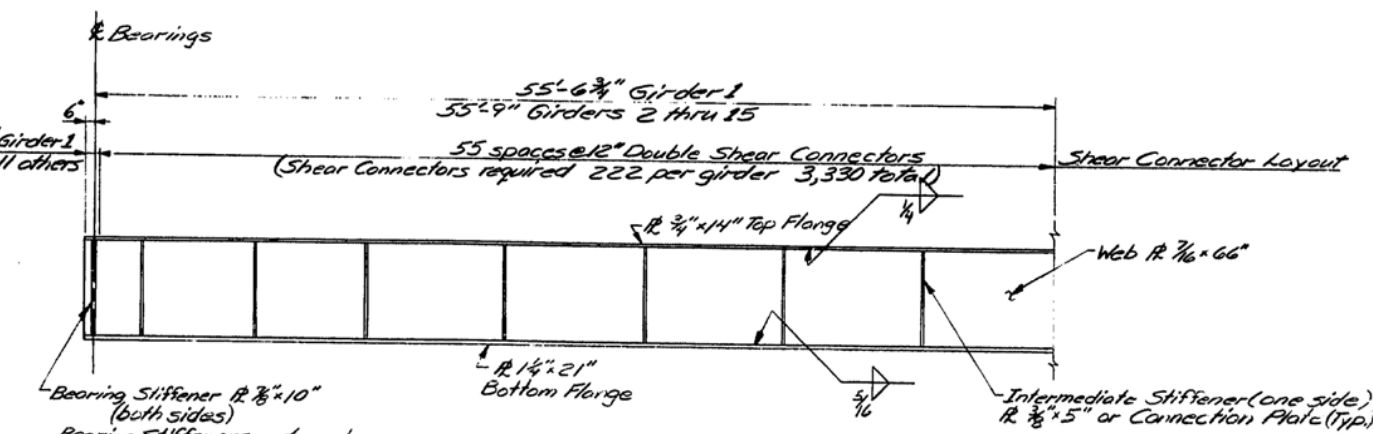
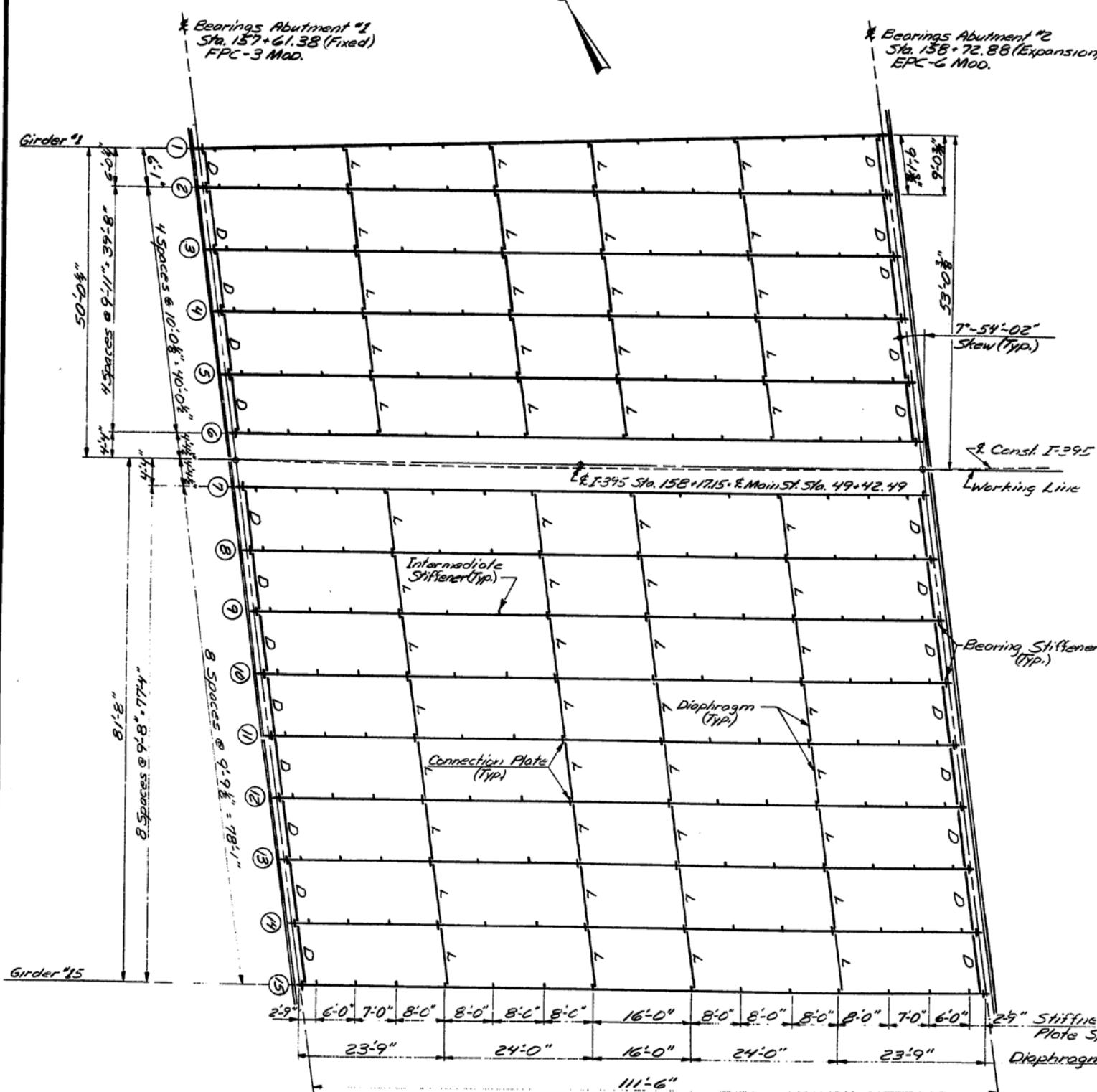
$F_y = 50,000 \text{ psi}$   
 $F_y = 36,000 \text{ psi}$   
 $F_v = 25,000 \text{ psi}$

### MATERIALS

STRUCTURAL STEEL: Flanges ASTM A572  
All other material (except as otherwise noted) ASTM A36  
High strength Bolts ASTM 325

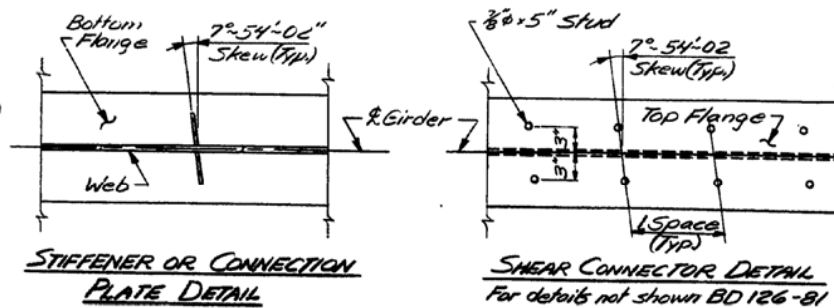
### STRUCTURAL STEEL NOTES

1. Camber ordinates, as shown, are computed to compensate for all dead load deflections and for the curvature of the finished grade profile.
2. No transverse butt weld splice will be allowed in the flange plates or web plates with in 10' from the point of maximum positive moment.
3. Sections of flange plates or web plates between transverse shop splices shall be not less than 20' in length unless otherwise shown on the plans.
4. Butt weld splices in flanges shall be not less than one foot from transverse welds in the web plates.
5. Bearing stiffeners shall be plumb after erection and dead loading of the structure. Intermediate web stiffeners may be either plumb or normal to the top flange.
6. Cross-frame or diaphragm connection plates may be either plumb or normal to the top flange.
7. The Expansion Pedestal Setting Chart indicates the required final position of the bearings. It is anticipated that the bearings at Abutment 2 will move 5" away from the fixed bearings due to the placement of the superstructure concrete. No separate payment will be made for resetting bearings to the final position if an adjustment is required.



GIRDER ELEVATION  
(Symmetrical - for details not shown see BD 113-81)

REFERENCE:  
BD 113-81 sheet #37  
BD 126-81 sheet #42



STIFFENER OR CONNECTION PLATE DETAIL

SHEAR CONNECTOR DETAIL  
For details not shown BD 126-81

97-251

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

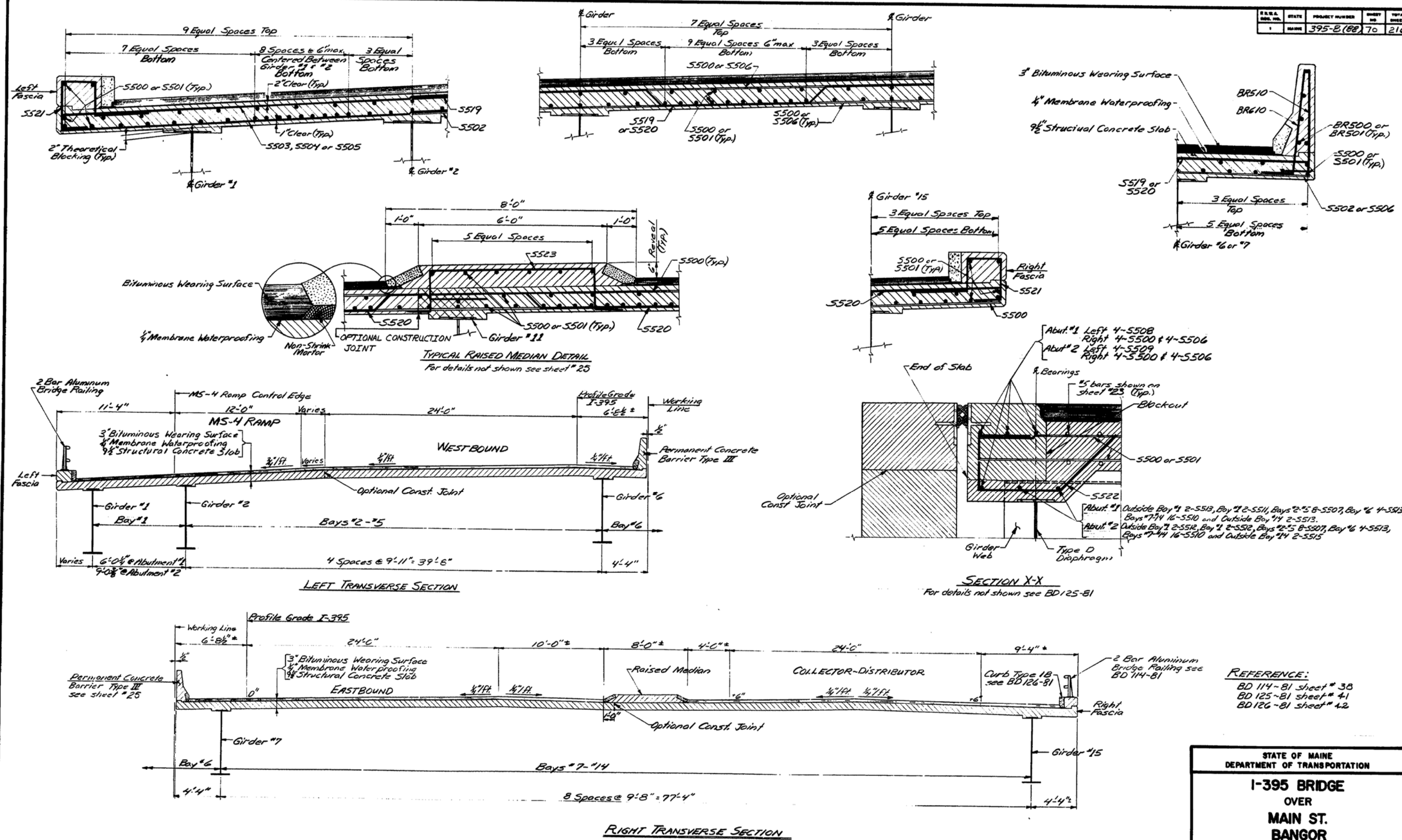
1-395 BRIDGE  
OVER  
MAIN ST.  
BANGOR  
PENOBSCOT COUNTY  
FRAMING PLAN  
(STEEL ALTERNATE)

SHEET 21 OF 43 AUGUSTA, MAINE Oct. 1984

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	9/84
CHECKED	10/84
REVISIONS	
FIELD CHANGES	



STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
MAINE	395-B(88) 70	216	



**REFERENCE:**  
 BD 114-81 sheet # 38  
 BD 125-81 sheet # 41  
 BD 126-81 sheet # 42

STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION

**I-395 BRIDGE  
 OVER  
 MAIN ST.  
 BANGOR  
 PENOBSCOT COUNTY  
 SUPERSTRUCTURE DETAILS  
 (STEEL ALTERNATE)**

**97-254**

SHEET 21 OF 43 AUGUSTA, MAINE Oct. 1984

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	10/1/84
CHECKED	10/1/84
REVISIONS	
FIELD CHANGES	

BRUNING 44-132-42710-1

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## **APPENDIX B**

### **LOAD RATING COMPUTATIONS**

**(See CD)**

- Bridge Stringer Dead Load Calculations
  - Left Bridge
- Bridge Live Load Distribution Factor Calculation
  - Left Bridge
- Mathcad Calculations:
  - Rigid Body Rotation Calculation Left Bridge Girder Mid-span
  - Rigid Body Rotation Calculation Left Bridge Girder At Support
- Merlin-Dash Output:
  - HL-93 Rating Interior Girder – Interior Left Bridge
  - HL-93 Rating Exterior Girder – Exterior Left Bridge
  - EV2 Rating Interior Girder – Interior Left Bridge (Updated 7/31/19)
  - EV3 Rating Interior Girder – Interior Left Bridge (Updated 7/31/19)
  - EV2 Rating Exterior Girder – Exterior Left Bridge (Updated 7/31/19)
  - EV3 Rating Exterior Girder – Exterior Left Bridge (Updated 7/31/19)
- Hand Calculations





The Manual for Bridge Evaluation - Second Edition 2011 &amp; 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 - 5799, 6 Girders Left Bridge**

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 =	$L_{span} =$	<u>111.50'</u>	
curb-to-curb width of bridge =	$b_{pvm} =$	<u>50.85'</u>	
clear width of sidewalk =	$b_{sw} =$	<u>0.00'</u>	(No sidewalk on this structure)
out-to-out width of bridge deck =	$b_{deck} =$	<u>53.98'</u>	
number of girders =	$N_g =$	<u>6</u>	
girder spacing =	$S_g =$	<u>9.917'</u>	
maximum top flange width =	$b_{f\_max} =$	<u>14.00"</u>	
maximum top flange thickness =	$t_{f\_max} =$	<u>0.75"</u>	
<del>thickness of concrete deck @ fascia =</del>	<del><math>t_{deck\_f} =</math></del>	<del><u>9.50"</u></del>	
thickness of concrete deck @ CL deck =	$t_{deck\_CL} =$	<u>9.50"</u>	(Constant Deck Thickness)
<del>cross slope =</del>	<del><math>\epsilon_s =</math></del>	<del><u>0.0%</u></del>	(Enter value if deck thickness varies)
curb reveal =	$h_{reveal} =$	<u>9.50"</u>	
width of overhang =	$b_{overhang} =$	<u>4.292'</u>	(assumes controlling case)

**Material Weights:**

Unit weight of concrete =	$W_c =$	<u>150 pcf</u>	with reinforcing.
Unit weight of asphalt =	$W_a =$	<u>140 pcf</u>	
Unit weight of granite =	$W_g =$	<u>170 pcf</u>	
Unit weight of utility fluid =	$W_{fluid} =$	<u>62.4 pcf</u>	
Unit weight of gas =	$W_{gas} =$	<u>8.0 pcf</u>	

LRFD

3.5.1 - Table 3.5.1-1

3.5.1 - Table 3.5.1-2

3.5.1 - Table 3.5.1-3

**Girder Haunch:**

Top flange within haunch? =		<u>Yes</u>	
thickness of haunch =	$t_{haunch} =$	<u>2.00"</u>	
width of haunch =	$b_{haunch} =$	<u>22.00"</u>	
weight of single girder haunch =	$W_{haunch} =$	<u>0.035 klf</u>	$b_{haunch} * t_{haunch} - b_{f\_max} * t_{f\_max} * w_c$

**Concrete Deck Loads:**

tributary width for interior girder =	$TW_{int} =$	9.917 ft	$= S_g$
tributary width for exterior girder =	$TW_{ext} =$	9.250 ft	$= S_g/2 + b_{overhang}$
deck weight for interior girder =	$W_{deck\_int} =$	<u>1.178 klf</u>	Average Thickness = 0.79'
deck weight for exterior girder =	$W_{deck\_ext} =$	<u>1.098 klf</u>	Average Thickness = 0.79'

**MERLIN Dead Loads - 5799, 6 Girders Left Bridge (CONT'D)**ReferencesWearing Surface:

type of wearing surface =		Asphalt	
thickness of wearing surface =	$t_{pmt}$ =	3.00"	(average)
thickness of waterproofing membrane =	$t_{wm}$ =	0.250"	
total weight of wearing surface =	$W_{ws}$ =	1.928 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 ft <sup>2</sup>	(concrete portion)
cross-sectional area of granite curb =	$A_g$ =	0.00 ft <sup>2</sup>	(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 =	$N_{curbs}$ =	1	
cross-sect area of curb and mortar bed =	$A_{curbs}$ =	1.57 ft <sup>2</sup>	(concrete portion)
cross-sectional area of granite curb =	$A_{curbs}$ =	0.48 ft <sup>2</sup>	(granite curb only, if used)
weight of curbs =	$W_{curbs}$ =	0.318 klf	

Concrete Soffit (exterior girder):

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



THE Louis Berger Group, INC.

482 Congress Street, Suite 401

BY IAS/KSW

DATE 04/25/13

SHEET 3 OF 5

CHKD BY TDM

DATE 06/06/13

PROJECT CKE410A4

SUBJECT #5799 - Interstate 395 Over Main Street

## MERLIN Dead Loads - 5799, 6 Girders Left Bridge (CONT'D)

[References](#)

### Railings:

Current Standard MaineDOT Bridge Rail?

No

Rail<sub>Left</sub> = 2-bar Alum. 0.025 klf Calcul

Rail<sub>Right</sub> = Other 0.400 klf Calcul

Barrier Mounted Rail Left = NONE 0.000 klf

Barrier Mounted Rail Right = NONE 0.000 klf

total weight of railing =  $W_{rails} = 0.425$  klf

### Snow / Pedestrian Fences:

left fence height =

Type II

Weight = 21 plf.

Verify Load

right fence height =

NONE

Weight = 0 plf.

Verify Load

total fence weight =  $W_{fences} = 0.021$  klf

### Diaphragms:

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

# of Attachment Plate 1 =

2

Attachment Plate 1 Height =

66.00"

Attachment Plate 1 Width =

7.00"

Attachment Plate 1 Thickness =

0.38"

Attachment Plate 1 Weight =

0.098 kip

# of Diagonals =

2

Diagonal Weight Per Foot =

9 plf

Length of Diagonal member =

10.30'

Diagonal Weight =

0.185 kip

# of Horizontal Members =

1

Horizontal Member Weight Per Foot =

11 plf

Length of Horizontal Member =

9.75'

Horizontal Member Weight =

0.107 kip

Interior Girder Diaphragm Weight =

0.39 kips

Exterior Girder Diaphragm Weight =

0.20 kips

1/2 Interior Girder Diaphragm Weight

**MERLIN Dead Loads - 5799, 6 Girders Left Bridge (CONT'D)**ReferencesBridge Mounted Utilities:

Are utilities present on this bridge?  
Are sewer main(s) present on this bridge?  
Are water main(s) present on this bridge?  
Are gas main(s) present on this bridge?  
Are duct bank(s) present on this bridge?

YesNoNoNoNo

Weight of sewer main line =  
Inside diameter of sewer main line =  
Insulation & Hardware Allowance =

0.0 lbs/ft0.00"0.0 lbs/ft

liquid weight =  $W_{\text{liquid}} =$  0.0 lbs/ft ( $W_{\text{fluid}} * A_i$ )  
Total line weight =  $W_{\text{sewer}} =$  0.000 klf

Weight of water main line =  
Inside diameter of water main line =  
Insulation & Hardware Allowance =

0.0 lbs/ft0.00"0.0 lbs/ft

liquid weight =  $W_{\text{liquid}} =$  0.0 lbs/ft ( $W_{\text{fluid}} * A_i$ )  
Total line weight =  $W_{\text{water}} =$  0.000 klf

Weight of gas line =  
Inside diameter of gas line =  
Insulation & Hardware Allowance =

0.0 lbs/ft0.00"0.0 lbs/ft

liquid weight =  $W_{\text{liquid}} =$  0.0 lbs/ft ( $W_{\text{gas}} * A_i$ )  
Total line weight =  $W_{\text{gas}} =$  0.000 klf

Weight of electrical duct bank =  
Hardware Allowance =  
Total duct bank weight =

0.0 lbs/ft0.0 lbs/ft $W_{\text{electric}} =$  0.000 klf

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



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SHEET 5 OF 5

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PROJECT CKE410A4

SUBJECT #5799 - Interstate 395 Over Main Street

## MERLIN Dead Loads - 5799, 6 Girders Left Bridge (CONT'D)

0.729

### MERLIN loads - Wet Concrete (DC1):

deck + haunch (interior girder) = 1.212 klf =  $W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = 1.133 klf =  $W_{deck\_ext} + W_{haunch} + W_{soffit}$

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

Diaphragm L (interior girder) = 0.391 kip per location

Diaphragm L (exterior girder) = 0.195 kip per location

### MERLIN loads - Composite (DC2):

wearing surface = 0.321 klf =  $W_{ws} \div N_g$

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

railing = 0.071 klf =  $W_{rails} \div N_g$

snow fence = 0.004 klf =  $W_{fence} \div N_g$

curb = 0.053 klf =  $W_{curbs} \div N_g$

References

**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:**

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	Strength	
	Moment	Shear
Interior Stringer	0.734	0.969
Exterior Stringer	0.807	0.872

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	11900	From SI&A
% <sub>trucks</sub>	10%	From SI&A
ADTT = % <sub>trucks</sub> * ADT	1190	

Condition Factor	$\phi_c =$	1
System Factor	$\phi_s =$	1
	$\phi_c * \phi_s =$	1

Merlin Dash 06012

Merlin Dash 09012  
input as ETA for Strength



## LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES

## LRFD AASHTO 4.6.2.2

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

## Variables

Skew Angle	$\theta$	7.90 degrees
Minimum Shoulder Width	$W_{\text{shoulder}}$	5.375 Ft.
Girder/Stringer Spacing	$S$	9.9166 Ft.
Girder/Stringer Span Length	$L$	111.5 Ft.
Deck Overhang	$d_{\text{ovhg}}$	4.291667 Ft.
Curb/Sidewalk/Rail	$d_{\text{curb/sw/rail}}$	1.292 Ft.
Roadway Overhang	$d_e$	3 Ft.

Corrugated Steel Plank ? (Yes/No)		No
Depth of steel grid or corrugated steel plank	$t_g$	0 in.
Depth of concrete slab	$t_s$	9.5 in.
Deck Concrete Strength	$f'_c$	3 ksi
Unit Weight of Concrete	$\gamma_{\text{conc}}$	0.145 kcf

MBE Table 6A.5.2.1-1

LRFD Table 3.5.1-1

Type of Beam		Welded Girder
No. of Beams	$N_b$	6
Number of Design Lanes	$N_L$	4
Depth of beam	$d$	68.00 in.
Area of Beam	$A$	65.63 in <sup>2</sup>
Moment of Inertia for Beam	$I$	47544.41 in <sup>4</sup>
Top Flange Thickness	$t_{\text{tf}}$	0.75 in
Overcamber Allowance		0.75 in

LRFD 3.6.1.1.1

See Moment of Inertia Calcs

See Moment of Inertia Calcs

See Moment of Inertia Calcs

From Plans

From NHBDM

Modulus of Elasticity for Beam	$E_B$	29000 ksi
Modulus of Elasticity for Deck	$E_D$	3156 ksi
Modular Ratio	$n$	9.0
CG dist. between deck and beam	$e_g$	47.11 in.
Longitudinal Stiffness Parameter	$K_g$	1738705

LRFD 5.4.2.4-1

LRFD 4.6.2.2.1-2

Hand Calculations

LRFD 4.6.2.2.1-1

Longitudinal Stiffness Constant for Moment	1.04	LRFD Table 4.6.2.2.2b-1
Longitudinal Stiffness Constant (Skew Correction for Moment)	1.11	LRFD Table 4.6.2.2.2e-1
Longitudinal Stiffness Constant (Skew Correction for Shear)	0.88	LRFD Table 4.6.2.2.3c-1
Skew Correction for Shear	1.02	LRFD Table 4.6.2.2.3c-1
Skew Correction for Moment	1.00	LRFD Table 4.6.2.2.2e-1



## Interior Beams

## REFERENCE

## Moment

## Corrugated Steel Plank Deck

LRFD Table 4.6.2.2.2c-1

Check Range of Applicability:

Spacing N/A  
Deck N/A $g_1 =$  N/A

One lane loaded

 $g_m =$  N/A

Two or more lanes loaded

Control  $g_c =$  N/A

## Reinforced Concrete Deck

LRFD Table 4.6.2.2.2b-1

Check Range of Applicability:

Spacing 9.9166 OK  
Slab 9.5 OK  
Length 111.5 OK  
No. Beams 6 OK  
 $K_g$  1738705 OK $g_1 =$  0.499

One lane loaded

 $g_m =$  0.734

Two or more lanes loaded

 $g_{1\_fat} =$  0.416

One lane loaded without multiple presence for fatigue analysis

Control  $g_c =$  0.734

Corrected for skew

## Shear

LRFD Table 4.6.2.2.3a-1

 $g_1 =$  0.757

One lane loaded

 $g_m =$  0.946

Two or more lanes loaded

 $g_{1\_fat} =$  0.646

One lane loaded without multiple presence for fatigue analysis

Control  $g_c =$  0.969

Corrected for skew





## Exterior Beams

## REFERENCE

## Moment

LRFD Table 4.6.2.2d-1

## Reinforced Concrete Deck

Check Range of Applicability:

Roadway Overhang 3.00 OK

	$g_1 =$	0.958	Lever Rule One lane loaded ( $m=1.2$ ) without accounting for lane line.
	$g_{1l} =$	0.550	Lever Rule One lane loaded ( $m=1.2$ ) accounting for lane line.
	$g_m =$	0.807	Two or more lanes loaded
	$g_{RB1} =$	0.546	Rigid Body Rotation C4.6.2.2d-1 (See MathCad Calculation)
	$g_{RB2} =$	0.729	Rigid Body Rotation C4.6.2.2d-2 (See MathCad Calculation)
	$g_{1\_fat} =$	0.458	One lane loaded without multiple presence for fatigue analysis
Control	$g_c =$	0.807	Corrected for skew

## Shear

LRFD Table 4.6.2.2b-1

	$g_1 =$	0.958	Lever Rule One lane loaded ( $m=1.2$ ) without accounting for lane line.
	$g_{1l} =$	0.550	Lever Rule One lane loaded ( $m=1.2$ ) accounting for lane line.
	$g_m =$	0.851	Two or more lanes loaded
	$g_{RB1} =$	0.540	Rigid Body Rotation C4.6.2.2d-1 (See MathCad Calculation)
	$g_{RB2} =$	0.723	Rigid Body Rotation C4.6.2.2d-2 (See MathCad Calculation)
	$g_{1\_fat} =$	0.469	One lane loaded without multiple presence for fatigue analysis
Control	$g_c =$	0.872	Corrected for skew



**THE Louis Berger Group, INC.**

482 Congress Street, Suite 401  
Portland, ME 04101

BY IAS/KSW DATE: 04/25/13 SHEET 1 OF 1  
CHKD BY TDM DATE: 06/06/13 PROJECT CKE410A4  
SUBJECT #5799 - Interstate 395 Over Main Street

**Moment of Inertia Calculation for Plate Girder, Section 111.5' Long**

Section	Depth (in)	Width (in)	A (in <sup>2</sup> )	y (in)	Ay (in <sup>3</sup> )	d	Ad <sup>2</sup>	I <sub>o</sub>	I
Plate (top)	0.75	14	10.50	67.63	710.0625	41.49	18070.55	0.49	18071.05
Web	66.00	0.44	28.88	34.25	988.97	8.11	1899.17	10481.63	12380.79
Plate (bottom)	1.25	21.00	26.25	0.63	16.41	-25.52	17089.15	3.42	17092.57
Total	68.00		65.63		1715.44				47544.41
				$\bar{y}$ (in)=	26.14				

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

BY: KSW

DATE: 6/3/13

PROJECT: Maine Load Rating Steel Bridges

CHK BY: TDM

DATE: 6/6/13

SUBJECT: #5799 I-395 over Main Street

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

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

**RIGID CROSS SECTION ANALYSIS - AT ABUT. NO. 2**

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

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.

$$B_{\text{shoulder}} := 5 \text{ ft} + 4.5 \cdot \text{in}$$

Width of Shoulder

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

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

$$N_L := 4 \quad N_{L,\text{Striped}} := 3$$

Number of Design/Striped Lanes

$$N_b := 6$$

Number of Beams

$$x_1 := 24.644 \text{ ft}$$

Distance from c.g. of beams to beam G1 (From Excel Spreadsheet).

$$x_2 := 14.727 \text{ ft}$$

Distance from c.g. of beams to beam G2 (From Excel Spreadsheet).

$$x_3 := 4.811 \text{ ft}$$

Distance from c.g. of beams to beam G3 (From Excel Spreadsheet).

$$x_4 := -5.106 \text{ ft}$$

Distance from c.g. of beams to beam G4 (From Excel Spreadsheet).

$$x_5 := -15.023 \text{ ft}$$

Distance from c.g. of beams to beam G5 (From Excel Spreadsheet).

$$x_6 := -24.054 \text{ ft}$$

Distance from c.g. of beams to beam G6 (From Excel Spreadsheet).

$$d_{e,\text{act}} := 3.0 \cdot \text{ft} \quad d_e := d_{e,\text{act}} - d_{\text{shoulder}} \quad d_e = -0.375 \text{ ft}$$

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

$$X_{\text{ext}} := x_1 \quad X_{\text{ext}} = 24.644 \text{ ft}$$

Horizontal Distance from the center of gravity of the pattern of girders to the exterior girder (From Excel Spreadsheet).

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

BY: KSW

DATE: 6/3/13

PROJECT:

Maine Load Rating Steel Bridges

CHK BY: TDM

DATE: 6/6/13

SUBJECT:

#5799 I-395 over Main Street

### RIGID CROSS SECTION ANALYSIS (cont'd)

$$e := \begin{pmatrix} X_{\text{ext}} + d_e - 5 \cdot \text{ft} \\ X_{\text{ext}} + d_e - 5 \cdot \text{ft} - B_{\text{lane}} \\ X_{\text{ext}} + d_e - 5 \cdot \text{ft} - 2B_{\text{lane}} \end{pmatrix} \quad e = \begin{pmatrix} 19.269 \\ 7.269 \\ -4.731 \end{pmatrix} \text{ ft}$$

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

$$m_p := \begin{pmatrix} 1.20 \\ 1.00 \\ 0.85 \end{pmatrix}$$

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

$$R_1 := \left( \frac{1}{N_b} + \frac{X_{\text{ext}} \cdot e_1}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right) \cdot m_{p1}$$

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

$$R_2 := \left[ \frac{2}{N_b} + \frac{X_{\text{ext}} \cdot (e_1 + e_2)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right] \cdot m_{p2}$$

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

$$R_3 := \left[ \frac{3}{N_b} + \frac{X_{\text{ext}} \cdot (e_1 + e_2 + e_3)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right] \cdot m_{p3}$$

Distribution Factor, 3 lanes loaded - Equation C4.6.2.2.2d-1

$$R = \begin{pmatrix} 0.54 \\ 0.723 \\ 0.697 \end{pmatrix}$$

Since there are 3 striped lanes on the bridge use a factor of 0.723 since it is the maximum of 1 2 or 3 lanes loaded.

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

BY: KSW

DATE: 6/3/13

PROJECT: Maine Load Rating Steel Bridges

CHK BY: TDM

DATE: 6/6/13

SUBJECT: #5799 I-395 over Main Street

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

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

**RIGID CROSS SECTION ANALYSIS - AT MIDSPAN**

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

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.

$B_{\text{shoulder}} := 5 \text{ft} + 4.5 \cdot \text{in}$

Width of Shoulder

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

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

$N_L := 4$        $N_{L,\text{Striped}} := 3$

Number of Design/Striped Lanes

$N_b := 6$

Number of Beams

$x_1 := 24.393 \text{ft}$

Distance from c.g. of beams to beam G1 (From Excel Spreadsheet).

$x_2 := 14.477 \text{ft}$

Distance from c.g. of beams to beam G2 (From Excel Spreadsheet).

$x_3 := 4.560 \text{ft}$

Distance from c.g. of beams to beam G3 (From Excel Spreadsheet).

$x_4 := -5.357 \text{ft}$

Distance from c.g. of beams to beam G4 (From Excel Spreadsheet).

$x_5 := -15.273 \text{ft}$

Distance from c.g. of beams to beam G5 (From Excel Spreadsheet).

$x_6 := -22.799 \text{ft}$

Distance from c.g. of beams to beam G6 (From Excel Spreadsheet).

$d_{e,\text{act}} := 3.0 \cdot \text{ft}$        $d_e := d_{e,\text{act}} - d_{\text{shoulder}}$        $d_e = -0.375 \text{ft}$

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

$X_{\text{ext}} := x_1$        $X_{\text{ext}} = 24.393 \text{ft}$

Horizontal Distance from the center of gravity of the pattern of girders to the exterior girder (From Excel Spreadsheet).

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

BY: KSW

DATE: 6/3/13

PROJECT:

Maine Load Rating Steel Bridges

CHK BY: TDM

DATE: 6/6/13

SUBJECT:

#5799 I-395 over Main Street

### RIGID CROSS SECTION ANALYSIS (cont'd)

$$e := \begin{pmatrix} X_{\text{ext}} + d_e - 5 \cdot \text{ft} \\ X_{\text{ext}} + d_e - 5 \cdot \text{ft} - B_{\text{lane}} \\ X_{\text{ext}} + d_e - 5 \cdot \text{ft} - 2B_{\text{lane}} \end{pmatrix} \quad e = \begin{pmatrix} 19.018 \\ 7.018 \\ -4.982 \end{pmatrix} \text{ ft}$$

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

$$m_p := \begin{pmatrix} 1.20 \\ 1.00 \\ 0.85 \end{pmatrix}$$

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

$$R_1 := \left( \frac{1}{N_b} + \frac{X_{\text{ext}} \cdot e_1}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right) \cdot m_{p1}$$

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

$$R_2 := \left[ \frac{2}{N_b} + \frac{X_{\text{ext}} \cdot (e_1 + e_2)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right] \cdot m_{p2}$$

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

$$R_3 := \left[ \frac{3}{N_b} + \frac{X_{\text{ext}} \cdot (e_1 + e_2 + e_3)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2} \right] \cdot m_{p3}$$

Distribution Factor, 3 lanes loaded - Equation C4.6.2.2.2d-1

$$R = \begin{pmatrix} 0.546 \\ 0.729 \\ 0.697 \end{pmatrix}$$

Since there are 3 striped lanes on the bridge use a factor of 0.728 since it is the maximum of 1 2 or 3 lanes loaded.

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TABLE 0.0.1.1 PROJECT DATA  
\*\*\*\*\*

DESCRIPTION DATE  
-----  
MaineDOT, Bridge No. 5799 I-395 Interior Girder Left Bridge 6/4/2013

CONTRACT NUMBER STR NO STR UNIT DES CHK SPECS. USED  
-----  
CKE410A4 5799 Interior IAS/KSW KSW/TDM

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 = 1  
0 - DEFAULT (2008)  
1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS  
\*\*\*\*\*

BEAM POSITION NUMBER OF GIRDERS	1=INT. 2=EXT.	WIDTH BETWEEN CURBS OR BARRIER (ft)	OVERHANG WIDTH (ft)	EDGE OF SLAB TO CURB (ft)	HAUNCH DEPTH (in)	WIDTH (in)	COMPOSITE PERCENTAGE AT NEG MOM REGION (%)	STEEL LOAD DETAIL FACTOR >= 1.0
6	1	50.85	4.29	1.29	1.25	22.00	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
111.50	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
9.92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS

SECTION NO.	ID.	STANDARD SECTN		PLATE GIRDER		ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in)		REINFORCED CONCRETE SECTION	
		NOMINAL DEPTH (in)	WEIGHT (lb/ft)	WEB DEPTH (in)	WEB THICK.	TOP PLATE WIDTH	BOT. PLATE THICK.	AREA (in**2)	Ix (in**4)
1	PG			66.0	0.4375	14.00	0.7500	21.00	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

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TABLE 0.0.5.1 DEFINITION OF MEMBERS

MEMBER NUMBER (IN ORDER)	MEMB END SECT ID		MEMBER DESCRIPTN		PARAMETERS FOR NONPRISMATIC MEMB		YIELD STRESS (KSI)		
	LEFT	RIGHT	LNTH (ft)	-->TYPE<-- 0=PRISMT	S(0)	S(1)	WEB	TOP	BOT
1	1	1	111.50				36.	50.	50.

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

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

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

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

AASHTO LOADING	TANDEM LIVE LOAD	AASHTO ROAD TYPE		SIDEWALK
HL - 93	1=YES : 0=NO	1, 2, 3 OR 4	ADTT ADTSL	LIVE LOAD---
				(k/ft)
HL-93	1	2	1190	1012
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 1190

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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							OPTIONAL		
LOADING TYPES							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	0.73	2 2 2 2 2		
1 0.00 0	0.00	0.00	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	SLAB DESIGN DEPTH		POUR DAY	MODULAR RATIO		SLAB LOAD DATA		
		INITIAL (in)	FINAL (in)		N1=3n	N2=n	INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1 Deck+haunch	0.0	9.5	0	27.0	9.0	1.21	0.00	111.50

AASHTO Art. 10.38.1.3 or LRFD Art. 6.10.1.1b

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:

$f_c'$  = 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		UNIFORM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	0 Wearing surf	0.321	0.00	111.50	0.00	0.00
2	1 Railing	0.071	0.00	111.50	0.00	0.00
3	1 Snow fence	0.004	0.00	111.50	0.00	0.00
4	1 Curb	0.053	0.00	111.50	0.00	0.00
5	2 Diaphragm L	0.000	0.00	0.00	0.39	23.75
6	2 Diaphragm L	0.000	0.00	0.00	0.39	47.75
7	2 Diaphragm L	0.000	0.00	0.00	0.39	63.75
8	2 Diaphragm L	0.000	0.00	0.00	0.39	87.75
9	2 Stiffeners	0.003	0.00	111.50	0.00	0.00

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

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

2 = Noncomposite Loads, (In LRFD, it is for DC1 load)  
where N = modulus ratio =  $E_s/E_c$

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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.	CONNECTION	CONNECTOR	AASHTO ART.	AASHTO ART.	AASHTO ART.	AASHTO ART.	REBAR BAR AREA	DIST.	COMP.	COMP.	
PER	DIA. IN	NEGAT.	6.10.10.4	6.10.10.2	6.10.10.2	6.10.10.2	YIELD PER FOOT	FROM	STRENG.	ALLOW	
TRAN.	M. REGION		-----	(kip / per	(kip / per	(kip / per	STRESS OF SLAB	TOP	AT 28	-ABLE	
SEC (in)	0=NO		(kip / per	connector)	connector)	connector)	Fy	-----	DAYS	-----	
	1=YES		connect.)	Truck	Lane	(ksi)	(in**2)	(in)	(ksi)	(ksi)	
-----											
2	0.875	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00	

2	0.875	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00
---	-------	---	------	-----	-----	-----	------	------	------	------

NOTE: Qr = nominal resistance of the shear connector

=  $(\phi)_{sc} \times Q_n$

... 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
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	OF LATERAL BRACING Lb (ft)
0.00	23.75	36.0	36.0	23.75
23.75	47.75	36.0	36.0	24.00

47.75	63.75	36.0	36.0	16.00
63.75	87.75	36.0	36.0	24.00
87.75	111.50	36.0	36.0	23.75

-----  
NOTE: [1] default  $F_y$  = 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 0.15.1.2 TRANSVERSE STIFFENER DATA  
\*\*\*\*\*

LOCATION		B PARAMETER	YIELD STRESS	STIFFENER SPACING	DIMENSIONS, (in)	
DI ST. FROM	DI ST. TO				STIFF. WIDTH	STIFF. THICK
(ft)	(ft)	AASHTO 6.10.8.1	$F_y$ (ksi)	(ft-in)		
0.00	2.75	1.0	36.0	2- 9	5.00	0.3750
2.75	8.75	2.4	36.0	6- 0	5.00	0.3750
8.75	15.75	2.4	36.0	7- 0	5.00	0.3750
15.75	23.75	1.0	36.0	8- 0	7.00	0.3750
23.75	39.75	2.4	36.0	8- 0	5.00	0.3750
39.75	47.75	1.0	36.0	8- 0	7.00	0.3750
47.75	63.75	1.0	36.0	16- 0	7.00	0.3750
63.75	79.75	2.4	36.0	8- 0	5.00	0.3750
79.75	87.75	1.0	36.0	8- 0	7.00	0.3750
87.75	97.75	2.4	36.0	8- 0	5.00	0.3750
95.75	102.75	2.4	36.0	7- 0	5.00	0.3750
102.75	108.75	2.4	36.0	6- 0	5.00	0.3750
108.75	111.50	1.0	36.0	2- 9	5.00	0.3750

-----  
NOTE: B parameter .... AASHTO LRFD Art. 6.10.8.1.4

B = 1.0 ..... for stiffener pairs (default)

= 1.8 ..... for single angles

= 2.4 ..... for single plates

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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	1.212		0.2233	1.4353

SUPERIMPOSED DEAD LOADS  
\*\*\*\*\*

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.32	0.00	111.50
1	DC2	0.071	0.000	111.500
2	DC2	0.004	0.000	111.500
3	DC2	0.053	0.000	111.500

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TABLE 1.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES

\*\*\*\*\*

CONCRETE		DECK		STEEL	SUPERSTRUCTURE TOTAL WEIGHT
UNIT WEIGHT (pcf)	VOLUME (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	(kip)
150.00	5134.6	190.2	770.2	149.4	919.59

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

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH

\*\*\*\*\*

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	9.92	119.00

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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	0.734	0.734	0.734	0.734	FOR STRENGTH POSITIVE MOMENT
1	0.969				FOR STRENGTH POSITIVE SHEAR
	0.416				FOR FATIGUE POSITIVE MOMENT
	0.631				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	WEB DEPTH	LOCATION OF N.A. FROM BOT OF STEEL	ELASTIC SECTION MODULUS	
			$I_x$ (in**4)	$D$ (in)	$Y(BS)$ (in)	STEEL BOT. (in**3)	TOP. (in**3)
1	0	0.00	47544.4	66.0	26.14	1818.8	1135.8
1	1	5.57	47544.4	66.0	26.14	1818.8	1135.8
1	2	11.15	47544.4	66.0	26.14	1818.8	1135.8
1	3	16.72	47544.4	66.0	26.14	1818.8	1135.8
1	4	22.30	47544.4	66.0	26.14	1818.8	1135.8
1	5	27.87	47544.4	66.0	26.14	1818.8	1135.8
1	6	33.45	47544.4	66.0	26.14	1818.8	1135.8
1	7	39.03	47544.4	66.0	26.14	1818.8	1135.8
1	8	44.60	47544.4	66.0	26.14	1818.8	1135.8
1	9	50.18	47544.4	66.0	26.14	1818.8	1135.8
1	10	55.75	47544.4	66.0	26.14	1818.8	1135.8
1	11	61.33	47544.4	66.0	26.14	1818.8	1135.8
1	12	66.90	47544.4	66.0	26.14	1818.8	1135.8
1	13	72.47	47544.4	66.0	26.14	1818.8	1135.8
1	14	78.05	47544.4	66.0	26.14	1818.8	1135.8
1	15	83.62	47544.4	66.0	26.14	1818.8	1135.8
1	16	89.20	47544.4	66.0	26.14	1818.8	1135.8
1	17	94.77	47544.4	66.0	26.14	1818.8	1135.8
1	18	100.35	47544.4	66.0	26.14	1818.8	1135.8
1	19	105.92	47544.4	66.0	26.14	1818.8	1135.8
1	20	111.50	47544.4	66.0	26.14	1818.8	1135.8

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

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00

\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA	Q/Ix	ELASTIC SECTION MODULUS, (in**3)		
			$I_x$ (in**4)	Q=1ST. MOMENT OF INERTIA (1/in)	STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	104937.4	0.011647	2351.4	4489.7	3144.4
1	1	5.57	104937.4	0.011647	2351.4	4489.7	3144.4
1	2	11.15	104937.4	0.011647	2351.4	4489.7	3144.4
1	3	16.72	104937.4	0.011647	2351.4	4489.7	3144.4
1	4	22.30	104937.4	0.011647	2351.4	4489.7	3144.4
1	5	27.87	104937.4	0.011647	2351.4	4489.7	3144.4
1	6	33.45	104937.4	0.011647	2351.4	4489.7	3144.4
1	7	39.03	104937.4	0.011647	2351.4	4489.7	3144.4
1	8	44.60	104937.4	0.011647	2351.4	4489.7	3144.4
1	9	50.18	104937.4	0.011647	2351.4	4489.7	3144.4
1	10	55.75	104937.4	0.011647	2351.4	4489.7	3144.4
1	11	61.33	104937.4	0.011647	2351.4	4489.7	3144.4
1	12	66.90	104937.4	0.011647	2351.4	4489.7	3144.4
1	13	72.47	104937.4	0.011647	2351.4	4489.7	3144.4
1	14	78.05	104937.4	0.011647	2351.4	4489.7	3144.4
1	15	83.62	104937.4	0.011647	2351.4	4489.7	3144.4
1	16	89.20	104937.4	0.011647	2351.4	4489.7	3144.4
1	17	94.77	104937.4	0.011647	2351.4	4489.7	3144.4
1	18	100.35	104937.4	0.011647	2351.4	4489.7	3144.4
1	19	105.92	104937.4	0.011647	2351.4	4489.7	3144.4
1	20	111.50	104937.4	0.011647	2351.4	4489.7	3144.4

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

SP	IN	D	FROM	MOMENT OF		Q/I x		ELASTIC SECTION MODULUS, (in**3)		
				INERTIA		Q=1ST. MOMENT		STEEL		CONCRETE (SLAB)
NO	NO	L	SUPT	I <sub>x</sub>		OF INERTIA				
			(ft)	(in**4)		(1/in)		BOT.	TOP.	TOP.
1	0	0.00		144379.7		0.014198		2524.5	13357.4	6938.3
1	1	5.57		144379.7		0.014198		2524.5	13357.4	6938.3
1	2	11.15		144379.7		0.014198		2524.5	13357.4	6938.3
1	3	16.72		144379.7		0.014198		2524.5	13357.4	6938.3
1	4	22.30		144379.7		0.014198		2524.5	13357.4	6938.3
1	5	27.87		144379.7		0.014198		2524.5	13357.4	6938.3
1	6	33.45		144379.7		0.014198		2524.5	13357.4	6938.3
1	7	39.03		144379.7		0.014198		2524.5	13357.4	6938.3
1	8	44.60		144379.7		0.014198		2524.5	13357.4	6938.3
1	9	50.18		144379.7		0.014198		2524.5	13357.4	6938.3
1	10	55.75		144379.7		0.014198		2524.5	13357.4	6938.3
1	11	61.33		144379.7		0.014198		2524.5	13357.4	6938.3
1	12	66.90		144379.7		0.014198		2524.5	13357.4	6938.3
1	13	72.47		144379.7		0.014198		2524.5	13357.4	6938.3
1	14	78.05		144379.7		0.014198		2524.5	13357.4	6938.3
1	15	83.62		144379.7		0.014198		2524.5	13357.4	6938.3
1	16	89.20		144379.7		0.014198		2524.5	13357.4	6938.3
1	17	94.77		144379.7		0.014198		2524.5	13357.4	6938.3
1	18	100.35		144379.7		0.014198		2524.5	13357.4	6938.3
1	19	105.92		144379.7		0.014198		2524.5	13357.4	6938.3
1	20	111.50		144379.7		0.014198		2524.5	13357.4	6938.3

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 positive & negative flexure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	DEAD LOAD		CONCENTRATED LOADS (k-ft)	UNI FORM LOADS (k-ft)	TOTAL (k-ft)	
	BEAM (k-ft)	SLAB (k-ft)			NONCOMPOSITE DEAD	LOAD
1 0 0.00	0.0	0.0	0.0	0.0	0.0	
1 1 5.57	65.9	357.9	4.4	0.9	429.0	
1 2 11.15	124.9	678.1	8.7	1.7	813.4	
1 3 16.72	177.0	960.6	13.1	2.4	1153.0	
1 4 22.30	222.1	1205.4	17.4	3.0	1448.0	
1 5 27.87	260.3	1412.6	20.2	3.5	1696.6	
1 6 33.45	291.5	1582.1	22.4	3.9	1899.9	
1 7 39.03	315.8	1714.0	24.5	4.2	2058.6	
1 8 44.60	333.1	1808.1	26.7	4.5	2172.5	
1 9 50.18	343.6	1864.7	28.0	4.6	2240.8	
1 10 55.75	347.0	1883.5	28.0	4.7	2263.1	
1 11 61.33	343.6	1864.7	28.0	4.6	2240.8	
1 12 66.90	333.1	1808.1	26.7	4.5	2172.5	
1 13 72.47	315.8	1714.0	24.5	4.2	2058.6	
1 14 78.05	291.5	1582.1	22.4	3.9	1899.9	
1 15 83.62	260.3	1412.6	20.2	3.5	1696.6	
1 16 89.20	222.1	1205.4	17.4	3.0	1448.0	
1 17 94.77	177.0	960.6	13.1	2.4	1153.0	
1 18 100.35	124.9	678.1	8.7	1.7	813.4	
1 19 105.92	65.9	357.9	4.4	0.9	429.0	
1 20 111.50	0.0	0.0	0.0	0.0	0.0	

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

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	UNI FORM SUPERIMPOSED DEAD LOAD (k-ft)		CONCENTRATED LOADS (k-ft)		OTHER UNI FORM LOADS	TOTAL (k-ft)	
	DW	DC2	DW	DC2		-- COMPOSITE DEAD LOAD	
1 0 0.00	0.0	0.0	0.0	0.0	0.0	0.0	
1 1 5.57	94.8	37.8	0.0	0.0	0.0	132.6	
1 2 11.15	179.6	71.6	0.0	0.0	0.0	251.2	
1 3 16.72	254.4	101.4	0.0	0.0	0.0	355.9	
1 4 22.30	319.3	127.3	0.0	0.0	0.0	446.6	
1 5 27.87	374.1	149.2	0.0	0.0	0.0	523.3	
1 6 33.45	419.0	167.1	0.0	0.0	0.0	586.1	
1 7 39.03	453.9	181.0	0.0	0.0	0.0	635.0	
1 8 44.60	478.9	191.0	0.0	0.0	0.0	669.8	
1 9 50.18	493.9	196.9	0.0	0.0	0.0	690.8	
1 10 55.75	498.8	198.9	0.0	0.0	0.0	697.8	
1 11 61.33	493.9	196.9	0.0	0.0	0.0	690.8	
1 12 66.90	478.9	191.0	0.0	0.0	0.0	669.8	
1 13 72.47	453.9	181.0	0.0	0.0	0.0	635.0	
1 14 78.05	419.0	167.1	0.0	0.0	0.0	586.1	
1 15 83.62	374.1	149.2	0.0	0.0	0.0	523.3	
1 16 89.20	319.3	127.3	0.0	0.0	0.0	446.6	
1 17 94.77	254.4	101.4	0.0	0.0	0.0	355.9	
1 18 100.35	179.6	71.6	0.0	0.0	0.0	251.2	
1 19 105.92	94.8	37.8	0.0	0.0	0.0	132.6	
1 20 111.50	0.0	0.0	0.0	0.0	0.0	0.0	

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

TABLE 1-2.8-3

\*\*\*\*\* (UNRESTORED) \*\*\*\*\*

SP NO	IN NO	D L FROM SUPT (ft)	SIDEWALK (MAXIMUM)		LL+I , (k-ft), LOAD TYPE= HL -93		MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
			POSITIVE (k-ft)	NEGATIVE (k-ft)						
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93		
1	1	5.57	0.0	0.0	478.2	HL-93	0.0	HL-93		
1	2	11.15	0.0	0.0	902.5	HL-93	0.0	HL-93		
1	3	16.72	0.0	0.0	1273.1	HL-93	0.0	HL-93		
1	4	22.30	0.0	0.0	1589.9	HL-93	0.0	HL-93		
1	5	27.87	0.0	0.0	1850.2	HL-93	0.0	HL-93		
1	6	33.45	0.0	0.0	2062.2	HL-93	0.0	HL-93		
1	7	39.03	0.0	0.0	2227.2	HL-93	0.0	HL-93		
1	8	44.60	0.0	0.0	2341.2	HL-93	0.0	HL-93		
1	9	50.18	0.0	0.0	2397.3	HL-93	0.0	HL-93		
1	10	55.75	0.0	0.0	2416.0	HL-93	0.0	HL-93		
1	11	61.33	0.0	0.0	2397.3	HL-93	0.0	HL-93		
1	12	66.90	0.0	0.0	2341.2	HL-93	0.0	HL-93		
1	13	72.47	0.0	0.0	2227.2	HL-93	0.0	HL-93		
1	14	78.05	0.0	0.0	2062.2	HL-93	0.0	HL-93		
1	15	83.62	0.0	0.0	1850.2	HL-93	0.0	HL-93		
1	16	89.20	0.0	0.0	1589.9	HL-93	0.0	HL-93		
1	17	94.77	0.0	0.0	1273.1	HL-93	0.0	HL-93		
1	18	100.35	0.0	0.0	902.5	HL-93	0.0	HL-93		
1	19	105.92	0.0	0.0	478.2	HL-93	0.0	HL-93		
1	20	111.50	0.0	0.0	0.0	HL-93	0.0	HL-93		

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

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	TRUCK ONLY		
	POS	NEG	RANGE
1 0	0.00	0.	0.
1 1	5.57	151.	151.
1 2	11.15	283.	283.
1 3	16.72	394.	394.
1 4	22.30	489.	489.
1 5	27.87	569.	569.
1 6	33.45	631.	631.
1 7	39.03	678.	678.
1 8	44.60	706.	706.
1 9	50.18	704.	704.
1 10	55.75	704.	704.
1 11	61.33	704.	704.
1 12	66.90	706.	706.
1 13	72.47	678.	678.
1 14	78.05	631.	631.
1 15	83.62	569.	569.
1 16	89.20	489.	489.
1 17	94.77	394.	394.
1 18	100.35	283.	283.
1 19	105.92	151.	151.
1 20	111.50	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)

\*\*\*\*\*

## 5799 Int Left.res

		DEAD LOAD		LL+I: N= 9.0		LOAD TYPE= HL - 93		TOTAL MAXIMUM	
SP NO	IN NO	D FROM L SUPT (ft)	NON COMP. N=Inf n. (k-ft)	COMP. N=27.0 (k-ft)	MAXI MUM POSI TIVE (k-ft)	GOVERN LOAD TYPE	MAXI MUM GOVERN NEGATIVE LOAD (k-ft)	POS I TIVE (k-ft)	NEGATI VE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	0.0	0.0
1	1	5.57	429.0	132.6	478.2	HL-93	0.0	1039.8	561.6
1	2	11.15	813.4	251.2	902.5	HL-93	0.0	1967.1	1064.6
1	3	16.72	1153.0	355.9	1273.1	HL-93	0.0	2782.0	1508.9
1	4	22.30	1448.0	446.6	1589.9	HL-93	0.0	3484.5	1894.5
1	5	27.87	1696.6	523.3	1850.2	HL-93	0.0	4070.1	2219.9
1	6	33.45	1899.9	586.1	2062.2	HL-93	0.0	4548.2	2486.0
1	7	39.03	2058.6	635.0	2227.2	HL-93	0.0	4920.7	2693.5
1	8	44.60	2172.5	669.8	2341.2	HL-93	0.0	5183.5	2842.3
1	9	50.18	2240.8	690.8	2397.3	HL-93	0.0	5328.8	2931.6
1	10	55.75	2263.1	697.8	2416.0	HL-93	0.0	5376.8	2960.9
1	11	61.33	2240.8	690.8	2397.3	HL-93	0.0	5328.8	2931.6
1	12	66.90	2172.5	669.8	2341.2	HL-93	0.0	5183.5	2842.3
1	13	72.47	2058.6	635.0	2227.2	HL-93	0.0	4920.7	2693.5
1	14	78.05	1899.9	586.1	2062.2	HL-93	0.0	4548.2	2486.0
1	15	83.62	1696.6	523.3	1850.2	HL-93	0.0	4070.1	2219.9
1	16	89.20	1448.0	446.6	1589.9	HL-93	0.0	3484.5	1894.5
1	17	94.77	1153.0	355.9	1273.1	HL-93	0.0	2782.0	1508.9
1	18	100.35	813.4	251.2	902.5	HL-93	0.0	1967.1	1064.6
1	19	105.92	429.0	132.6	478.2	HL-93	0.0	1039.8	561.6
1	20	111.50	0.0	0.0	0.0	HL-93	0.0	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	5.57	1039.8	1183.2	1562.5	1371.2	842.4	113.3
1	2	11.15	1967.1	2237.9	2955.1	2594.1	1596.9	212.2
1	3	16.72	2782.0	3164.0	4177.7	3668.4	2263.3	295.5
1	4	22.30	3484.5	3961.5	5230.4	4594.4	2841.8	366.8
1	5	27.87	4070.1	4625.2	6106.3	5366.2	3329.8	426.5
1	6	33.45	4548.2	5166.9	6821.2	5996.3	3729.0	473.6
1	7	39.03	4920.7	5588.9	7378.0	6487.1	4040.3	508.8
1	8	44.60	5183.5	5885.9	7769.7	6833.3	4263.5	529.6
1	9	50.18	5328.8	6048.0	7983.1	7024.2	4397.3	528.3
1	10	55.75	5376.8	6101.6	8053.7	7087.4	4441.3	527.9
1	11	61.33	5328.8	6048.0	7983.1	7024.2	4397.3	528.3
1	12	66.90	5183.5	5885.9	7769.7	6833.3	4263.5	529.6
1	13	72.47	4920.7	5588.9	7378.0	6487.1	4040.3	508.8
1	14	78.05	4548.2	5166.9	6821.2	5996.3	3729.0	473.6
1	15	83.62	4070.1	4625.2	6106.3	5366.2	3329.8	426.5
1	16	89.20	3484.5	3961.4	5230.4	4594.4	2841.8	366.8
1	17	94.77	2782.0	3164.0	4177.7	3668.4	2263.3	295.5
1	18	100.35	1967.1	2237.9	2955.1	2594.1	1596.9	212.2
1	19	105.92	1039.8	1183.2	1562.5	1371.2	842.4	113.3
1	20	111.50	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)

## 5799 Int Left.res

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SPAN NO.	IN NO	DIST LEFT (ft)	FROM SUPT	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL DEAD LOAD (kip)	NONCOMPOSITE LOAD
1	0	0.00		12.4	67.6	0.8	0.2	81.0	
1	1	5.57		11.2	60.8	0.8	0.2	72.9	
1	2	11.15		10.0	54.1	0.8	0.1	64.9	
1	3	16.72		8.7	47.3	0.8	0.1	56.9	
1	4	22.30		7.5	40.5	0.8	0.1	48.9	
1	5	27.87		6.2	33.8	0.4	0.1	40.5	
1	6	33.45		5.0	27.0	0.4	0.1	32.5	
1	7	39.03		3.7	20.3	0.4	0.1	24.4	
1	8	44.60		2.5	13.5	0.4	0.0	16.4	
1	9	50.18		1.2	6.8	0.0	0.0	8.0	
1	10	55.75		0.0	0.0	0.0	0.0	0.0	
1	11	61.33		-1.2	-6.8	0.0	0.0	-8.0	
1	12	66.90		-2.5	-13.5	-0.4	0.0	-16.4	
1	13	72.47		-3.7	-20.3	-0.4	-0.1	-24.4	
1	14	78.05		-5.0	-27.0	-0.4	-0.1	-32.5	
1	15	83.62		-6.2	-33.8	-0.4	-0.1	-40.5	
1	16	89.20		-7.5	-40.5	-0.8	-0.1	-48.9	
1	17	94.77		-8.7	-47.3	-0.8	-0.1	-56.9	
1	18	100.35		-10.0	-54.1	-0.8	-0.1	-64.9	
1	19	105.92		-11.2	-60.8	-0.8	-0.2	-72.9	
1	20	111.50		-12.4	-67.6	-0.8	-0.2	-81.0	

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

\*\*\*\*\*

SPAN NO.	IN NO	DIST LEFT (ft)	FROM SUPT	COMPOSITE (k)				TOTAL		TOTAL DEAD LOADS (k)
				UNIFORM LOADS		CONCENTRATED LOADS		COMPOSITE (k)	NONCOMPOSITE	
				DW	DC2	DW	DC2			
1	0	0.00		17.9	7.1	0.0	0.0	25.0	81.0	106.0
1	1	5.57		16.1	6.4	0.0	0.0	22.5	72.9	95.5
1	2	11.15		14.3	5.7	0.0	0.0	20.0	64.9	85.0
1	3	16.72		12.5	5.0	0.0	0.0	17.5	56.9	74.4
1	4	22.30		10.7	4.3	0.0	0.0	15.0	48.9	63.9
1	5	27.87		8.9	3.6	0.0	0.0	12.5	40.5	53.0
1	6	33.45		7.2	2.9	0.0	0.0	10.0	32.5	42.5
1	7	39.03		5.4	2.1	0.0	0.0	7.5	24.4	32.0
1	8	44.60		3.6	1.4	0.0	0.0	5.0	16.4	21.4
1	9	50.18		1.8	0.7	0.0	0.0	2.5	8.0	10.5
1	10	55.75		0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	11	61.33		-1.8	-0.7	0.0	0.0	-2.5	-8.0	-10.5
1	12	66.90		-3.6	-1.4	0.0	0.0	-5.0	-16.4	-21.4
1	13	72.47		-5.4	-2.1	0.0	0.0	-7.5	-24.4	-32.0
1	14	78.05		-7.2	-2.9	0.0	0.0	-10.0	-32.5	-42.5
1	15	83.62		-8.9	-3.6	0.0	0.0	-12.5	-40.5	-53.0
1	16	89.20		-10.7	-4.3	0.0	0.0	-15.0	-48.9	-63.9
1	17	94.77		-12.5	-5.0	0.0	0.0	-17.5	-56.9	-74.4
1	18	100.35		-14.3	-5.7	0.0	0.0	-20.0	-64.9	-85.0
1	19	105.92		-16.1	-6.4	0.0	0.0	-22.5	-72.9	-95.5
1	20	111.50		-17.9	-7.1	0.0	0.0	-25.0	-81.0	-106.0

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5799 Int Left.res  
TABLE 1.2.6.3=LIVE LOAD SHEAR FOR N = 9.0 (UNFACTORED)  
\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM)		LL+I , (kips),		LOAD TYPE= HL -93	
			POSITIVE (kips)	NEGATIVE (kips)	MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	119.6	HL-93	0.0	HL-93
1	1	5.57	0.0	0.0	111.6	HL-93	-2.6	HL-93
1	2	11.15	0.0	0.0	103.7	HL-93	-5.6	HL-93
1	3	16.72	0.0	0.0	96.1	HL-93	-9.3	HL-93
1	4	22.30	0.0	0.0	88.6	HL-93	-13.1	HL-93
1	5	27.87	0.0	0.0	81.3	HL-93	-17.8	HL-93
1	6	33.45	0.0	0.0	74.1	HL-93	-23.2	HL-93
1	7	39.03	0.0	0.0	67.1	HL-93	-28.9	HL-93
1	8	44.60	0.0	0.0	60.3	HL-93	-34.9	HL-93
1	9	50.18	0.0	0.0	53.7	HL-93	-41.0	HL-93
1	10	55.75	0.0	0.0	47.3	HL-93	-47.3	HL-93
1	11	61.33	0.0	0.0	41.0	HL-93	-53.7	HL-93
1	12	66.90	0.0	0.0	34.9	HL-93	-60.3	HL-93
1	13	72.47	0.0	0.0	28.9	HL-93	-67.1	HL-93
1	14	78.05	0.0	0.0	23.2	HL-93	-74.1	HL-93
1	15	83.62	0.0	0.0	17.8	HL-93	-81.3	HL-93
1	16	89.20	0.0	0.0	13.1	HL-93	-88.6	HL-93
1	17	94.77	0.0	0.0	9.3	HL-93	-96.1	HL-93
1	18	100.35	0.0	0.0	5.6	HL-93	-103.7	HL-93
1	19	105.92	0.0	0.0	2.6	HL-93	-111.6	HL-93
1	20	111.50	0.0	0.0	0.0	HL-93	-119.6	HL-93

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

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY		
			POS	NEG	RANGE
1	0	0.00	44.	0.	44.
1	1	5.57	41.	-1.	42.
1	2	11.15	38.	-2.	41.
1	3	16.72	36.	-4.	40.
1	4	22.30	33.	-5.	38.
1	5	27.87	31.	-6.	37.
1	6	33.45	28.	-8.	36.
1	7	39.03	25.	-10.	35.
1	8	44.60	23.	-12.	35.
1	9	50.18	20.	-15.	35.
1	10	55.75	18.	-18.	35.
1	11	61.33	15.	-20.	35.
1	12	66.90	12.	-23.	35.
1	13	72.47	10.	-25.	35.
1	14	78.05	8.	-28.	36.
1	15	83.62	6.	-31.	37.
1	16	89.20	5.	-33.	38.
1	17	94.77	4.	-36.	40.
1	18	100.35	2.	-38.	41.
1	19	105.92	1.	-41.	42.
1	20	111.50	0.	-44.	44.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE  
\*\*\*\*\*

TOP FLANGE	BOTTOM FLANGE
------------	---------------

SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	5799 Int Left.res Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	43.7	.14988E-01	0.33	0.2500*	10205E-01	0.22	0.3125*	
1	1	5.57	42.2	.14988E-01	0.32	0.2500*	10205E-01	0.22	0.3125*	
1	2	11.15	40.8	.14988E-01	0.31	0.2500*	10205E-01	0.21	0.3125*	
1	3	16.72	39.5	.14988E-01	0.30	0.2500*	10205E-01	0.20	0.3125*	
1	4	22.30	38.3	.14988E-01	0.29	0.2500*	10205E-01	0.20	0.3125*	
1	5	27.87	36.7	.14988E-01	0.28	0.2500*	10205E-01	0.19	0.3125*	
1	6	33.45	35.7	.14988E-01	0.27	0.2500*	10205E-01	0.18	0.3125*	
1	7	39.03	35.3	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	8	44.60	35.1	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	9	50.18	35.1	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	10	55.75	35.1	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	11	61.33	35.1	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	12	66.90	35.1	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	13	72.47	35.3	.14988E-01	0.26	0.2500*	10205E-01	0.18	0.3125*	
1	14	78.05	35.7	.14988E-01	0.27	0.2500*	10205E-01	0.18	0.3125*	
1	15	83.62	36.7	.14988E-01	0.28	0.2500*	10205E-01	0.19	0.3125*	
1	16	89.20	38.3	.14988E-01	0.29	0.2500*	10205E-01	0.20	0.3125*	
1	17	94.77	39.5	.14988E-01	0.30	0.2500*	10205E-01	0.20	0.3125*	
1	18	100.35	40.8	.14988E-01	0.31	0.2500*	10205E-01	0.21	0.3125*	
1	19	105.92	42.2	.14988E-01	0.32	0.2500*	10205E-01	0.22	0.3125*	
1	20	111.50	43.7	.14988E-01	0.33	0.2500*	10205E-01	0.22	0.3125*	

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

[2] For non-composite construction:

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

For composite construction:

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

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

\* -- minimum weld size governs

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

SP NO	IN NO	D L	FROM SUPT (ft)	TOTAL		TOTAL L+I , LOAD TYPE = HL - 93		MAX. SHEAR (kips)	
				DEAD LOADS (k)	POSITIVE GOVERN. (k)	NEGATIVE GOVERN. (k)	LOAD TYPE	POS.	NEG.
1	0	0.00		106.0	119.6	0.0	HL-93	225.6	106.0
1	1	5.57		95.5	111.6	-2.6	HL-93	207.1	92.9
1	2	11.15		85.0	103.7	-5.6	HL-93	188.7	79.3
1	3	16.72		74.4	96.1	-9.3	HL-93	170.5	65.2
1	4	22.30		63.9	88.6	-13.1	HL-93	152.5	50.8
1	5	27.87		53.0	81.3	-17.8	HL-93	134.3	35.2
1	6	33.45		42.5	74.1	-23.2	HL-93	116.6	19.3
1	7	39.03		32.0	67.1	-28.9	HL-93	99.1	3.0
1	8	44.60		21.4	60.3	-34.9	HL-93	81.8	-13.4
1	9	50.18		10.5	53.7	-41.0	HL-93	64.2	-30.5
1	10	55.75		0.0	47.3	-47.3	HL-93	47.3	-47.3
1	11	61.33		-10.5	41.0	-53.7	HL-93	30.5	-64.2
1	12	66.90		-21.4	34.9	-60.3	HL-93	13.4	-81.8

5799 Int Left.res									
1	13	72.47	-32.0	28.9	HL-93	-67.1	HL-93	-3.0	-99.1
1	14	78.05	-42.5	23.2	HL-93	-74.1	HL-93	-19.3	-116.6
1	15	83.62	-53.0	17.8	HL-93	-81.3	HL-93	-35.2	-134.3
1	16	89.20	-63.9	13.1	HL-93	-88.6	HL-93	-50.8	-152.5
1	17	94.77	-74.4	9.3	HL-93	-96.1	HL-93	-65.2	-170.5
1	18	100.35	-85.0	5.6	HL-93	-103.7	HL-93	-79.3	-188.7
1	19	105.92	-95.5	2.6	HL-93	-111.6	HL-93	-92.9	-207.1
1	20	111.50	-106.0	0.0	HL-93	-119.6	HL-93	-106.0	-225.6

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TABLE 1.2.6.5= SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

\*\*\*\*\*

SP IN D NO NO	FROM L SUPT	SERVICE I (ft)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	225.6	261.5	346.3	298.4	32.7
1	1	5.57	207.1	240.5	318.6	274.0	31.6
1	2	11.15	188.7	219.8	291.3	249.8	30.6
1	3	16.72	170.5	199.3	264.3	225.9	29.7
1	4	22.30	152.5	179.1	237.6	202.2	28.7
1	5	27.87	134.3	158.6	210.7	178.2	27.5
1	6	33.45	116.6	138.8	184.6	154.9	26.8
1	7	39.03	99.1	119.2	158.8	131.9	26.5
1	8	44.60	81.8	99.9	133.3	109.2	26.3
1	9	50.18	64.2	80.4	107.6	86.1	26.3
1	10	55.75	-47.3	-61.4	-82.7	-63.8	26.3
1	11	61.33	-64.2	-80.4	-107.6	-86.1	26.3
1	12	66.90	-81.8	-99.9	-133.3	-109.2	26.3
1	13	72.47	-99.1	-119.2	-158.8	-131.9	26.5
1	14	78.05	-116.6	-138.8	-184.6	-154.9	26.8
1	15	83.62	-134.3	-158.6	-210.7	-178.2	27.5
1	16	89.20	-152.5	-179.1	-237.6	-202.2	28.7
1	17	94.77	-170.5	-199.3	-264.3	-225.9	29.7
1	18	100.35	-188.7	-219.8	-291.3	-249.8	30.6
1	19	105.92	-207.1	-240.5	-318.6	-274.0	31.6
1	20	111.50	-225.6	-261.5	-346.3	-298.4	32.7

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TABLE 1.2.7.2= SUMMARY OF REACTIONS (UNFACTORED)

\*\*\*\*\*

SUPT NO.	TOTAL	LL+I , (K), LOAD TYPE = HL - 93				TOTAL DL+LL+I (L R F D)		
	DEAD LOADS (K)	MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	MINIMUM	MAXIMUM	
1	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47
2	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47

NOTE: [1] " - " Indicates Uplift



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

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

SP	IN	D FROM	NONCOMPOSITE DL		COMPOSITE DL		T O T A L		
			BEAM	SLAB	CONCENTRATED	UNI FORM	NONCOMPOSITE	COMPOSITE	= DL
NO	NO	L SUPT (ft)	(in)	(in)	(in)	(in)	(in)	(in)	(in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	5.57	-0.0897	-0.4867	0.0000	-0.0817	-0.5846	-0.0817	-0.6663
1	2	11.15	-0.1768	-0.9596	0.0000	-0.1611	-1.1526	-0.1611	-1.3137
1	3	16.72	-0.2591	-1.4063	0.0000	-0.2360	-1.6891	-0.2360	-1.9251
1	4	22.30	-0.3345	-1.8156	0.0000	-0.3047	-2.1808	-0.3047	-2.4855
1	5	27.87	-0.4013	-2.1781	0.0000	-0.3656	-2.6163	-0.3656	-2.9818
1	6	33.45	-0.4580	-2.4857	0.0000	-0.4172	-2.9858	-0.4172	-3.4030
1	7	39.03	-0.5033	-2.7317	0.0000	-0.4585	-3.2814	-0.4585	-3.7400
1	8	44.60	-0.5364	-2.9112	0.0000	-0.4886	-3.4971	-0.4886	-3.9857
1	9	50.18	-0.5565	-3.0203	0.0000	-0.5070	-3.6282	-0.5070	-4.1351
1	10	55.75	-0.5632	-3.0569	0.0000	-0.5131	-3.6722	-0.5131	-4.1853
1	11	61.33	-0.5565	-3.0203	0.0000	-0.5070	-3.6282	-0.5070	-4.1351
1	12	66.90	-0.5364	-2.9112	0.0000	-0.4886	-3.4971	-0.4886	-3.9857
1	13	72.47	-0.5033	-2.7317	0.0000	-0.4585	-3.2814	-0.4585	-3.7400
1	14	78.05	-0.4580	-2.4857	0.0000	-0.4172	-2.9858	-0.4172	-3.4030
1	15	83.62	-0.4013	-2.1781	0.0000	-0.3656	-2.6163	-0.3656	-2.9818
1	16	89.20	-0.3345	-1.8156	0.0000	-0.3047	-2.1808	-0.3047	-2.4855
1	17	94.77	-0.2591	-1.4063	0.0000	-0.2360	-1.6891	-0.2360	-1.9251
1	18	100.35	-0.1768	-0.9596	0.0000	-0.1611	-1.1526	-0.1611	-1.3137
1	19	105.92	-0.0897	-0.4867	0.0000	-0.0817	-0.5846	-0.0817	-0.6663
1	20	111.50	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	IN	D FROM	NONCOMPOSITE		DEAD	LOADS		COMPOSITE DL		T O T A L	
			STEEL	CAMBER	SLAB	CAMBER	DEFL.	CAMBER	DEFL.	CAMBER	SIZE
NO	NO	L SUPT (ft)	DEFL.	SIZE	DEFL.	SIZE	SIZE	SIZE	SIZE	SIZE	SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--	--
1	1	5.6	-0.090	0	1/8	-0.495	0	1/2	-0.082	0	11/16
1	2	11.1	-0.177	0	3/16	-0.976	1	0/16	-0.161	0	3/8
1	3	16.7	-0.259	0	5/16	-1.430	1	7/16	-0.236	0	15/16
1	4	22.3	-0.335	0	3/8	-1.846	1	7/8	-0.305	0	1/2
1	5	27.9	-0.401	0	7/16	-2.215	2	1/4	-0.366	0	0/16
1	6	33.5	-0.458	0	1/2	-2.528	2	9/16	-0.417	0	7/16
1	7	39.0	-0.503	0	9/16	-2.778	2	13/16	-0.459	0	3/4
1	8	44.6	-0.536	0	9/16	-2.961	3	0/16	-0.489	0	0/16
1	9	50.2	-0.556	0	9/16	-3.072	3	1/8	-0.507	0	3/16
1	10	55.8	-0.563	0	5/8	-3.109	3	1/8	-0.513	0	3/16
1	11	61.3	-0.556	0	9/16	-3.072	3	1/8	-0.507	0	3/16
1	12	66.9	-0.536	0	9/16	-2.961	3	0/16	-0.489	0	0/16
1	13	72.5	-0.503	0	9/16	-2.778	2	13/16	-0.459	0	3/4
1	14	78.0	-0.458	0	1/2	-2.528	2	9/16	-0.417	0	7/16
1	15	83.6	-0.401	0	7/16	-2.215	2	1/4	-0.366	0	0/16
1	16	89.2	-0.335	0	3/8	-1.846	1	7/8	-0.305	0	1/2

5799 Int Left.res  
 1 17 94.8 -0.259 0 5/16 -1.430 1 7/16 -0.236 0 1/ 4 -1.925 1 15/16  
 1 18 100.3 -0.177 0 3/16 -0.976 1 0/16 -0.161 0 3/16 -1.314 1 3/ 8  
 1 19 105.9 -0.090 0 1/ 8 -0.495 0 1/ 2 -0.082 0 1/ 8 -0.666 0 11/16  
 1 20 111.5 0.000 -- 0.000 -- 0.000 -- 0.000 --

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 NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2  
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TABLE 1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION  
 \*\*\*\*\*  
 (UNFACTORED)

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	55.75	4 0.433	-0.475 MAX 0.018 MIN -0.230 MAX LANE 0.018 MIN LANE	HL-93	1.67	0.00191
		5 0.542	-0.593 MAX 0.023 MIN -0.288 MAX LANE 0.023 MIN LANE	HL-93	1.67	0.00191
		4 0.433	-0.434 MAX FA 0.000 MIN FA			

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

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

$$DF = (NL/Ng) * (RF)_{lane} \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 aveereaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)  
 \*\*\*\*\*

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

## 5799 Int Left.res

1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	-0.70	0.44	-3.84	2.40	-4.53	2.83
1	2	11.15	-1.32	0.82	-7.27	4.54	-8.59	5.37
1	3	16.72	-1.87	1.17	-10.31	6.44	-12.18	7.61
1	4	22.30	-2.35	1.47	-12.95	8.09	-15.30	9.55
1	5	27.87	-2.75	1.72	-15.17	9.48	-17.92	11.19
1	6	33.45	-3.08	1.92	-16.99	10.61	-20.07	12.53
1	7	39.03	-3.34	2.08	-18.41	11.50	-21.75	13.58
1	8	44.60	-3.52	2.20	-19.43	12.14	-22.95	14.33
1	9	50.18	-3.63	2.27	-20.04	12.52	-23.67	14.78
1	10	55.75	-3.67	2.29	-20.24	12.64	-23.91	14.93
1	11	61.33	-3.63	2.27	-20.04	12.52	-23.67	14.78
1	12	66.90	-3.52	2.20	-19.43	12.14	-22.95	14.33
1	13	72.47	-3.34	2.08	-18.41	11.50	-21.75	13.58
1	14	78.05	-3.08	1.92	-16.99	10.61	-20.07	12.53
1	15	83.62	-2.75	1.72	-15.17	9.48	-17.92	11.19
1	16	89.20	-2.35	1.47	-12.95	8.09	-15.30	9.55
1	17	94.77	-1.87	1.17	-10.31	6.44	-12.18	7.61
1	18	100.35	-1.32	0.82	-7.27	4.54	-8.59	5.37
1	19	105.92	-0.70	0.44	-3.84	2.40	-4.53	2.83
1	20	111.50	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD, (ksi)			TOTAL DEAD LOAD, (ksi)		
			CONCRETE	STEEL BEAM		CONCRETE	STEEL BEAM	
				TOP	BOT		TOP	BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	-0.02	-0.35	0.68	-0.02	-4.89	3.51
1	2	11.15	-0.04	-0.67	1.28	-0.04	-9.26	6.65
1	3	16.72	-0.05	-0.95	1.82	-0.05	-13.13	9.42
1	4	22.30	-0.06	-1.19	2.28	-0.06	-16.49	11.83
1	5	27.87	-0.07	-1.40	2.67	-0.07	-19.32	13.86
1	6	33.45	-0.08	-1.57	2.99	-0.08	-21.64	15.53
1	7	39.03	-0.09	-1.70	3.24	-0.09	-23.45	16.82
1	8	44.60	-0.09	-1.79	3.42	-0.09	-24.74	17.75
1	9	50.18	-0.10	-1.85	3.53	-0.10	-25.52	18.31
1	10	55.75	-0.10	-1.86	3.56	-0.10	-25.78	18.49
1	11	61.33	-0.10	-1.85	3.53	-0.10	-25.52	18.31
1	12	66.90	-0.09	-1.79	3.42	-0.09	-24.74	17.75
1	13	72.47	-0.09	-1.70	3.24	-0.09	-23.45	16.82
1	14	78.05	-0.08	-1.57	2.99	-0.08	-21.64	15.53
1	15	83.62	-0.07	-1.40	2.67	-0.07	-19.32	13.86
1	16	89.20	-0.06	-1.19	2.28	-0.06	-16.49	11.83
1	17	94.77	-0.05	-0.95	1.82	-0.05	-13.13	9.42
1	18	100.35	-0.04	-0.67	1.28	-0.04	-9.27	6.65
1	19	105.92	-0.02	-0.35	0.68	-0.02	-4.89	3.51
1	20	111.50	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D FROM L SUPT	MAXIMUM POSITIVE, (MPa)		MAXIMUM NEGATIVE, (ksi)	
			CONCR.	STEEL	CONCR.	STEEL

	(ft)	TOP	TOP	BOT	5799 Int Left.res TOP TOP BOT
1 0	0.00	0.00	0.00	0.00	0.00
1 1	5.57	-0.09	-0.43	2.27	0.00
1 2	11.15	-0.17	-0.81	4.29	0.00
1 3	16.72	-0.24	-1.14	6.05	0.00
1 4	22.30	-0.31	-1.43	7.56	0.00
1 5	27.87	-0.36	-1.66	8.79	0.00
1 6	33.45	-0.40	-1.85	9.80	0.00
1 7	39.03	-0.43	-2.00	10.59	0.00
1 8	44.60	-0.45	-2.10	11.13	0.00
1 9	50.18	-0.46	-2.15	11.40	0.00
1 10	55.75	-0.46	-2.17	11.48	0.00
1 11	61.33	-0.46	-2.15	11.40	0.00
1 12	66.90	-0.45	-2.10	11.13	0.00
1 13	72.47	-0.43	-2.00	10.59	0.00
1 14	78.05	-0.40	-1.85	9.80	0.00
1 15	83.62	-0.36	-1.66	8.79	0.00
1 16	89.20	-0.31	-1.43	7.56	0.00
1 17	94.77	-0.24	-1.14	6.05	0.00
1 18	100.35	-0.17	-0.81	4.29	0.00
1 19	105.92	-0.09	-0.43	2.27	0.00
1 20	111.50	0.00	0.00	0.00	0.00

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

SP IN D FROM NO NO L SUPT	TOP OF TOP FLANGE (ft)	TRUCK	ONLY	GOVERNING STRESS LOAD RANGE TP TYPE	BOT OF BOT FLANGE (ft)	TRUCK	ONLY	GOVERNING STRESS LOAD RANGE TP TYPE
1 0	0.00	0.0	0.0	0.0 T HL	0.0	0.0	0.0	0.0 T HL
1 1	5.57	0.1	0.0	0.1 C HL	0.7	0.0	0.0	0.7 T HL
1 2	11.15	0.3	0.0	0.3 C HL	1.3	0.0	0.0	1.3 T HL
1 3	16.72	0.4	0.0	0.4 C HL	1.9	0.0	0.0	1.9 T HL
1 4	22.30	0.4	0.0	0.4 C HL	2.3	0.0	0.0	2.3 T HL
1 5	27.87	0.5	0.0	0.5 C HL	2.7	0.0	0.0	2.7 T HL
1 6	33.45	0.6	0.0	0.6 C HL	3.0	0.0	0.0	3.0 T HL
1 7	39.03	0.6	0.0	0.6 C HL	3.2	0.0	0.0	3.2 T HL
1 8	44.60	0.6	0.0	0.6 C HL	3.4	0.0	0.0	3.4 T HL
1 9	50.18	0.6	0.0	0.6 C HL	3.3	0.0	0.0	3.3 T HL
1 10	55.75	0.6	0.0	0.6 C HL	3.3	0.0	0.0	3.3 T HL
1 11	61.33	0.6	0.0	0.6 C HL	3.3	0.0	0.0	3.3 T HL
1 12	66.90	0.6	0.0	0.6 C HL	3.4	0.0	0.0	3.4 T HL
1 13	72.47	0.6	0.0	0.6 C HL	3.2	0.0	0.0	3.2 T HL
1 14	78.05	0.6	0.0	0.6 C HL	3.0	0.0	0.0	3.0 T HL
1 15	83.62	0.5	0.0	0.5 C HL	2.7	0.0	0.0	2.7 T HL
1 16	89.20	0.4	0.0	0.4 C HL	2.3	0.0	0.0	2.3 T HL
1 17	94.77	0.4	0.0	0.4 C HL	1.9	0.0	0.0	1.9 T HL
1 18	100.35	0.3	0.0	0.3 C HL	1.3	0.0	0.0	1.3 T HL
1 19	105.92	0.1	0.0	0.1 C HL	0.7	0.0	0.0	0.7 T HL
1 20	111.50	0.0	0.0	0.0 T HL	0.0	0.0	0.0	0.0 T HL

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

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

YIELD TOTAL POSITIVE, (ksi) TOTAL NEGATIVE, (ksi)

5799 Int Left.res										
			STRESS							
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.11	-5.32	5.78	-0.02	-4.89	3.51
1	2	11.15	50.	50.	-0.21	-10.08	10.94	-0.04	-9.26	6.65
1	3	16.72	50.	50.	-0.29	-14.28	15.47	-0.05	-13.13	9.42
1	4	22.30	50.	50.	-0.37	-17.92	19.39	-0.06	-16.49	11.83
1	5	27.87	50.	50.	-0.43	-20.99	22.66	-0.07	-19.32	13.86
1	6	33.45	50.	50.	-0.48	-23.49	25.33	-0.08	-21.64	15.53
1	7	39.03	50.	50.	-0.52	-25.45	27.41	-0.09	-23.45	16.82
1	8	44.60	50.	50.	-0.54	-26.85	28.88	-0.09	-24.74	17.75
1	9	50.18	50.	50.	-0.56	-27.67	29.70	-0.10	-25.52	18.31
1	10	55.75	50.	50.	-0.56	-27.95	29.98	-0.10	-25.78	18.49
1	11	61.33	50.	50.	-0.56	-27.67	29.70	-0.10	-25.52	18.31
1	12	66.90	50.	50.	-0.54	-26.85	28.88	-0.09	-24.74	17.75
1	13	72.47	50.	50.	-0.52	-25.45	27.41	-0.09	-23.45	16.82
1	14	78.05	50.	50.	-0.48	-23.49	25.33	-0.08	-21.64	15.53
1	15	83.62	50.	50.	-0.43	-20.99	22.66	-0.07	-19.32	13.86
1	16	89.20	50.	50.	-0.37	-17.92	19.39	-0.06	-16.49	11.83
1	17	94.77	50.	50.	-0.29	-14.28	15.47	-0.05	-13.13	9.42
1	18	100.35	50.	50.	-0.21	-10.08	10.94	-0.04	-9.27	6.65
1	19	105.92	50.	50.	-0.11	-5.32	5.78	-0.02	-4.89	3.51
1	20	111.50	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00

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

\*\*\*\*\*

			YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.14	-5.45	6.46	-0.02	-4.89	3.51
1	2	11.15	50.	50.	-0.26	-10.32	12.23	-0.04	-9.26	6.65
1	3	16.72	50.	50.	-0.37	-14.62	17.29	-0.05	-13.13	9.42
1	4	22.30	50.	50.	-0.46	-18.35	21.66	-0.06	-16.49	11.83
1	5	27.87	50.	50.	-0.54	-21.48	25.30	-0.07	-19.32	13.86
1	6	33.45	50.	50.	-0.60	-24.05	28.27	-0.08	-21.64	15.53
1	7	39.03	50.	50.	-0.65	-26.05	30.58	-0.09	-23.45	16.82
1	8	44.60	50.	50.	-0.68	-27.48	32.22	-0.09	-24.74	17.75
1	9	50.18	50.	50.	-0.70	-28.32	33.12	-0.10	-25.52	18.31
1	10	55.75	50.	50.	-0.70	-28.60	33.42	-0.10	-25.78	18.49
1	11	61.33	50.	50.	-0.70	-28.32	33.12	-0.10	-25.52	18.31
1	12	66.90	50.	50.	-0.68	-27.48	32.22	-0.09	-24.74	17.75
1	13	72.47	50.	50.	-0.65	-26.05	30.58	-0.09	-23.45	16.82
1	14	78.05	50.	50.	-0.60	-24.05	28.27	-0.08	-21.64	15.53
1	15	83.62	50.	50.	-0.54	-21.48	25.30	-0.07	-19.32	13.86
1	16	89.20	50.	50.	-0.46	-18.35	21.66	-0.06	-16.49	11.83
1	17	94.77	50.	50.	-0.37	-14.62	17.29	-0.05	-13.13	9.42
1	18	100.35	50.	50.	-0.26	-10.32	12.23	-0.04	-9.27	6.65
1	19	105.92	50.	50.	-0.14	-5.45	6.46	-0.02	-4.89	3.51
1	20	111.50	50.	50.	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

\*\*\*\*\*

			YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	

NO	NO	L	SUPT (ft)	Fy (ksi)		TOP	5799 Int Left.res		TOP	TOP	BOT
				TOP	BOT		TOP	BOT			
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.19	-6.92	8.48	-0.03	-6.17	4.50	
1	2	11.15	50.	50.	-0.35	-13.12	16.05	-0.05	-11.70	8.54	
1	3	16.72	50.	50.	-0.50	-18.59	22.69	-0.07	-16.59	12.10	
1	4	22.30	50.	50.	-0.62	-23.33	28.42	-0.09	-20.83	15.20	
1	5	27.87	50.	50.	-0.73	-27.31	33.20	-0.11	-24.40	17.81	
1	6	33.45	50.	50.	-0.81	-30.57	37.10	-0.12	-27.33	19.94	
1	7	39.03	50.	50.	-0.88	-33.11	40.13	-0.13	-29.61	21.61	
1	8	44.60	50.	50.	-0.92	-34.93	42.28	-0.14	-31.25	22.80	
1	9	50.18	50.	50.	-0.95	-36.00	43.46	-0.14	-32.23	23.52	
1	10	55.75	50.	50.	-0.95	-36.35	43.85	-0.14	-32.55	23.75	
1	11	61.33	50.	50.	-0.95	-36.00	43.46	-0.14	-32.23	23.52	
1	12	66.90	50.	50.	-0.92	-34.93	42.28	-0.14	-31.25	22.80	
1	13	72.47	50.	50.	-0.88	-33.11	40.13	-0.13	-29.61	21.61	
1	14	78.05	50.	50.	-0.81	-30.57	37.10	-0.12	-27.33	19.94	
1	15	83.62	50.	50.	-0.73	-27.31	33.20	-0.11	-24.40	17.81	
1	16	89.20	50.	50.	-0.62	-23.33	28.42	-0.09	-20.83	15.20	
1	17	94.77	50.	50.	-0.50	-18.59	22.69	-0.07	-16.59	12.10	
1	18	100.35	50.	50.	-0.35	-13.12	16.05	-0.05	-11.70	8.54	
1	19	105.92	50.	50.	-0.19	-6.92	8.48	-0.03	-6.17	4.50	
1	20	111.50	50.	50.	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	IN	D	FROM	YIELD STRESS	TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
					CONCR.	STEEL BEAM		CONCR.	STEEL BEAM	
NO	NO	L	SUPT	Fy (ksi)	TOP	TOP	BOT	TOP	TOP	BOT
			(ft)	TOP BOT						
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.17	-5.09	7.01	-0.01	-4.34	3.04
1	2	11.15	50.	50.	-0.33	-9.64	13.26	-0.03	-8.22	5.75
1	3	16.72	50.	50.	-0.46	-13.65	18.75	-0.04	-11.65	8.16
1	4	22.30	50.	50.	-0.58	-17.13	23.47	-0.05	-14.63	10.24
1	5	27.87	50.	50.	-0.68	-20.05	27.39	-0.05	-17.14	12.00
1	6	33.45	50.	50.	-0.75	-22.44	30.59	-0.06	-19.20	13.44
1	7	39.03	50.	50.	-0.81	-24.30	33.09	-0.06	-20.80	14.56
1	8	44.60	50.	50.	-0.86	-25.63	34.84	-0.07	-21.95	15.37
1	9	50.18	50.	50.	-0.88	-26.41	35.79	-0.07	-22.64	15.85
1	10	55.75	50.	50.	-0.88	-26.66	36.10	-0.07	-22.86	16.01
1	11	61.33	50.	50.	-0.88	-26.41	35.79	-0.07	-22.64	15.85
1	12	66.90	50.	50.	-0.86	-25.63	34.84	-0.07	-21.95	15.37
1	13	72.47	50.	50.	-0.81	-24.30	33.09	-0.06	-20.80	14.56
1	14	78.05	50.	50.	-0.75	-22.44	30.59	-0.06	-19.20	13.44
1	15	83.62	50.	50.	-0.68	-20.05	27.39	-0.05	-17.14	12.00
1	16	89.20	50.	50.	-0.58	-17.13	23.47	-0.05	-14.63	10.24
1	17	94.77	50.	50.	-0.46	-13.65	18.75	-0.04	-11.65	8.16
1	18	100.35	50.	50.	-0.33	-9.64	13.26	-0.03	-8.22	5.75
1	19	105.92	50.	50.	-0.17	-5.09	7.01	-0.01	-4.34	3.04
1	20	111.50	50.	50.	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	IN	D	FROM	YIELD STRESS	TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
					CONCR.	STEEL BEAM		CONCR.	STEEL BEAM	
NO	NO	L	SUPT	Fy (ksi)	TOP	TOP	BOT	TOP	TOP	BOT
			(ft)	TOP BOT						

## 5799 Int Left.res

1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.15	-6.75	7.57	-0.03	-6.17	4.50
1	2	11.15	50.	50.	-0.28	-12.80	14.33	-0.05	-11.70	8.54
1	3	16.72	50.	50.	-0.40	-18.13	20.27	-0.07	-16.59	12.10
1	4	22.30	50.	50.	-0.50	-22.76	25.40	-0.09	-20.83	15.20
1	5	27.87	50.	50.	-0.59	-26.65	29.68	-0.11	-24.40	17.81
1	6	33.45	50.	50.	-0.65	-29.83	33.18	-0.12	-27.33	19.94
1	7	39.03	50.	50.	-0.71	-32.31	35.90	-0.13	-29.61	21.61
1	8	44.60	50.	50.	-0.74	-34.09	37.82	-0.14	-31.25	22.80
1	9	50.18	50.	50.	-0.76	-35.14	38.90	-0.14	-32.23	23.52
1	10	55.75	50.	50.	-0.77	-35.48	39.25	-0.14	-32.55	23.75
1	11	61.33	50.	50.	-0.76	-35.14	38.90	-0.14	-32.23	23.52
1	12	66.90	50.	50.	-0.74	-34.09	37.82	-0.14	-31.25	22.80
1	13	72.47	50.	50.	-0.71	-32.31	35.90	-0.13	-29.61	21.61
1	14	78.05	50.	50.	-0.65	-29.83	33.18	-0.12	-27.33	19.94
1	15	83.62	50.	50.	-0.59	-26.65	29.68	-0.11	-24.40	17.81
1	16	89.20	50.	50.	-0.50	-22.76	25.40	-0.09	-20.83	15.20
1	17	94.77	50.	50.	-0.40	-18.13	20.27	-0.07	-16.59	12.10
1	18	100.35	50.	50.	-0.28	-12.80	14.33	-0.05	-11.70	8.54
1	19	105.92	50.	50.	-0.15	-6.75	7.57	-0.03	-6.17	4.50
1	20	111.50	50.	50.	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 L	FROM SUPT (ft)	YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
				Fy TOP	(ksi) BOT	CONCR. TOP	STEEL BEAM		CONCR. TOP	STEEL BEAM	
							TOP	BOT		TOP	BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.14	-4.92	6.10	-0.01	-4.34	3.04	3.04
1	2	11.15	50.	50.	-0.26	-9.31	11.55	-0.03	-8.22	5.75	5.75
1	3	16.72	50.	50.	-0.37	-13.19	16.33	-0.04	-11.65	8.16	8.16
1	4	22.30	50.	50.	-0.46	-16.56	20.44	-0.05	-14.63	10.24	10.24
1	5	27.87	50.	50.	-0.53	-19.39	23.87	-0.05	-17.14	12.00	12.00
1	6	33.45	50.	50.	-0.59	-21.70	26.67	-0.06	-19.20	13.44	13.44
1	7	39.03	50.	50.	-0.64	-23.50	28.85	-0.06	-20.80	14.56	14.56
1	8	44.60	50.	50.	-0.68	-24.79	30.39	-0.07	-21.95	15.37	15.37
1	9	50.18	50.	50.	-0.69	-25.55	31.23	-0.07	-22.64	15.85	15.85
1	10	55.75	50.	50.	-0.70	-25.79	31.51	-0.07	-22.86	16.01	16.01
1	11	61.33	50.	50.	-0.69	-25.55	31.23	-0.07	-22.64	15.85	15.85
1	12	66.90	50.	50.	-0.68	-24.79	30.39	-0.07	-21.95	15.37	15.37
1	13	72.47	50.	50.	-0.64	-23.50	28.85	-0.06	-20.80	14.56	14.56
1	14	78.05	50.	50.	-0.59	-21.70	26.67	-0.06	-19.20	13.44	13.44
1	15	83.62	50.	50.	-0.53	-19.39	23.87	-0.05	-17.14	12.00	12.00
1	16	89.20	50.	50.	-0.46	-16.56	20.44	-0.05	-14.63	10.24	10.24
1	17	94.77	50.	50.	-0.37	-13.19	16.33	-0.04	-11.65	8.16	8.16
1	18	100.35	50.	50.	-0.26	-9.31	11.55	-0.03	-8.22	5.75	5.75
1	19	105.92	50.	50.	-0.14	-4.92	6.10	-0.01	-4.34	3.04	3.04
1	20	111.50	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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

MEMBER		STEEL TYPE	TOP FLANGE		WEB		BOT FLANGE		COVER PLATE -- (in)			
NO	LENGTH (ft)		WIDTH (in)	THICK (in)	DEPTH (in)	THICK	WIDTH (in)	THICK	TOP		BOT	
									WIDTH	THICK	WIDTH	THICK
1	111.50	PG	14.0	0.7500	66.0	0.4375	21.0	1.2500				

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) = 111.5

MEMBER NO	SLAB THICK. (ft)	GIRDER DEPTH (in)	[2]=OVERALL D.		[2]/(12*[1])		LIMITATION 1/25 1/30	STATUS
			[3]=	GIRDER D. (in)	[3]/(12*[1])	<=>		
1	111.50	9.50	68.00	[2]= 78.00	0.05830	>	0.04000	OK
				[3]= 68.00	0.05082	>	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	depth	Web thick	D/tw	[2]	2Dcp/tw [3] [4]		2Dc/tw [5] [6]		Web Cat
1	0	0.00	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	1	5.57	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	2	11.15	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	3	16.72	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	4	22.30	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	5	27.87	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	6	33.45	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	7	39.03	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	8	44.60	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	9	50.18	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	10	55.75	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	11	61.33	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	12	66.90	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	13	72.47	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	14	78.05	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	15	83.62	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	16	89.20	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	17	94.77	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	18	100.35	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	19	105.92	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2
1	20	111.50	0	1	66.00	0.438	150.9	150.0	150.0	0.00	90.55			2



## 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:  $3.76 \cdot \sqrt{E/Fyc}$  (Eq. 6.10.6.2.2-1)  
 [5] Use Article D6.3.1 to calculate Dc and 2\*Dc/tw  
 [6] 2Dc/tw limit:  $5.7 \cdot \sqrt{E/Fyc}$  (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	D FROM NO	Lo Co ng mp	Web depth	thick	2Dc/tw [2]	Web Category [3]
-----						
1	0	0.00	0	1	66.00 0.438 187.93 137.27	3
1	1	5.57	0	1	66.00 0.438 187.93 137.27	3
1	2	11.15	0	1	66.00 0.438 187.93 137.27	3
1	3	16.72	0	1	66.00 0.438 187.93 137.27	3
1	4	22.30	0	1	66.00 0.438 187.93 137.27	3
1	5	27.87	0	1	66.00 0.438 187.93 137.27	3
1	6	33.45	0	1	66.00 0.438 187.93 137.27	3
1	7	39.03	0	1	66.00 0.438 187.93 137.27	3
1	8	44.60	0	1	66.00 0.438 187.93 137.27	3
1	9	50.18	0	1	66.00 0.438 187.93 137.27	3
1	10	55.75	0	1	66.00 0.438 187.93 137.27	3
1	11	61.33	0	1	66.00 0.438 187.93 137.27	3
1	12	66.90	0	1	66.00 0.438 187.93 137.27	3
1	13	72.47	0	1	66.00 0.438 187.93 137.27	3
1	14	78.05	0	1	66.00 0.438 187.93 137.27	3
1	15	83.62	0	1	66.00 0.438 187.93 137.27	3
1	16	89.20	0	1	66.00 0.438 187.93 137.27	3
1	17	94.77	0	1	66.00 0.438 187.93 137.27	3
1	18	100.35	0	1	66.00 0.438 187.93 137.27	3
1	19	105.92	0	1	66.00 0.438 187.93 137.27	3
1	20	111.50	0	1	66.00 0.438 187.93 137.27	3

## NOTE:

- [1] 0 - No long. stiffeners  
 1 - long. stiffeners  
 [2] For non-composite sections, calculate Dc and 2\*Dc/tw  
 [3] 2Dc/tw limit:  $5.7 \cdot \sqrt{E/Fyc}$  (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	bf/2tf [1]	bf (in)	[2]	tf (in)	[3]	Iyc/Iyt	FLAG
-----									
1	0	0.00	9.33 12.	14.0	11.0	0.750	0.481		
			8.40 12.	21.0	11.0	1.250	0.481	0.18	0

5799 Int Left.res									
1	1	5.57	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	2	11.15	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	3	16.72	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	4	22.30	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	5	27.87	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	6	33.45	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	7	39.03	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	8	44.60	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	9	50.18	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	10	55.75	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	11	61.33	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	12	66.90	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	13	72.47	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	14	78.05	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	15	83.62	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	16	89.20	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	17	94.77	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	18	100.35	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	19	105.92	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	20	111.50	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0

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

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

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

LATERAL BRACING NO	DIST. LEFT ----(FT)----	FROM SUPT TO	f <sub>o</sub> (ksi)	f <sub>2</sub> (ksi)	f <sub>mid</sub> (ksi)	f <sub>1</sub> (ksi)	C <sub>b</sub>
1	0.0	23.7	0.000	23.972	13.590	3.209	1.615
			0.000	0.000	0.000	0.000	1.000
2	23.7	47.7	23.972	35.041	31.147	27.253	1.115
			0.000	0.000	0.000	0.000	1.000
3	47.7	63.7	35.041	35.041	35.866	36.691	1.000
			0.000	0.000	0.000	0.000	1.000
4	63.7	87.7	23.972	35.041	31.147	27.253	1.115
			0.000	0.000	0.000	0.000	1.000
5	87.7	111.5	0.000	23.972	13.590	3.209	1.615
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case  
 f<sub>o</sub>, f<sub>2</sub>, f<sub>mid</sub>, f<sub>1</sub>, and C<sub>b</sub> 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

5799 Int Left.res  
and larger lateral moment of inertia of the flange  
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES

\*\*\*\*\*

SP	IN	D	FROM	Co	FLB			LTB			GOV
NO	NO	L	SUPT	mp	LMDAf	LMDApf	LMDArf	Lb	Lp	Lr	CAT
			(ft)					(ft)	(ft)	(ft)	
1	0	0.00	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	1	5.57	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	2	11.15	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	3	16.72	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	4	22.30	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	5	27.87	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	6	33.45	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	7	39.03	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	8	44.60	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	9	50.18	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	10	55.75	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	11	61.33	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	12	66.90	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	13	72.47	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	14	78.05	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	15	83.62	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	16	89.20	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	17	94.77	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	18	100.35	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	19	105.92	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	20	111.50	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2

NOTE: LMDAf = slenderness ratio for the comp. flange

=  $bfc / 2tfc$  (Eq. 6.10.8.2.2-3)

LMDApf = limiting slenderness ratio for a compact flange

=  $0.38 \cdot \sqrt{E/Fyc}$  (Eq. 6.10.8.2.2-4)

LMDArf = limiting slenderness ratio for a non-compact flange

=  $0.56 \cdot \sqrt{E/Fyr}$  (Eq. 6.10.8.2.2-5)

Lb = unbraced length

Lp = limiting unbraced length to achieve the nominal flexural of  $RbRhFyc$  under uniform bending

=  $1.0rt \cdot \sqrt{E/Fyc}$  (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

=  $\pi \cdot rt \cdot \sqrt{E/Fyr}$  (Eq. 6.10.8.2.3-5)

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

Flange Category

0 = compact section

2 = non-compact section

3 = slender section

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE

\*\*\*\*\*

SP	IN	D	FROM	Co	Rh	Rb	Cb	FLB	LTB	GOV
NO	NO	L	SUPT	mp				Fnc	Fnc	Fnc
			(ft)					(Ksi)	(Ksi)	(Ksi)
1	0	0.00	1	0.971	1.000	1.61		48.22	48.57	48.22
1	1	5.57	1	0.982	1.000	1.61		48.74	49.11	48.74

5799 Int Left.res								
1	2	11.15	1	0.982	1.000	1.61	48.74	49.11
1	3	16.72	1	0.982	1.000	1.61	48.74	49.11
1	4	22.30	1	0.982	1.000	1.61	48.74	49.11
1	5	27.87	1	0.982	1.000	1.11	48.74	39.28
1	6	33.45	1	0.982	1.000	1.11	48.74	39.28
1	7	39.03	1	0.982	1.000	1.11	48.74	39.28
1	8	44.60	1	0.982	1.000	1.11	48.74	39.28
1	9	50.18	1	0.982	1.000	1.00	48.74	41.57
1	10	55.75	1	0.982	1.000	1.00	48.74	41.57
1	11	61.33	1	0.982	1.000	1.00	48.74	41.57
1	12	66.90	1	0.982	1.000	1.11	48.74	39.28
1	13	72.47	1	0.982	1.000	1.11	48.74	39.28
1	14	78.05	1	0.982	1.000	1.11	48.74	39.28
1	15	83.62	1	0.982	1.000	1.11	48.74	39.28
1	16	89.20	1	0.982	1.000	1.61	48.74	49.11
1	17	94.77	1	0.982	1.000	1.61	48.74	49.11
1	18	100.35	1	0.982	1.000	1.61	48.74	49.11
1	19	105.92	1	0.982	1.000	1.61	48.74	49.11
1	20	111.50	1	0.971	1.000	1.61	48.22	48.57

Note: In the positive moment region, the result is for DL case  
In the negative moment region, the 1st line is for DL case  
and the 2nd line is for LL case

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TABLE 1.2.22.7C=INFORMATION FOR DUCTILITY CHECK  
\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0	0.00	1	9.48	32.76
1	1	5.57	1	9.48	32.76
1	2	11.15	1	9.48	32.76
1	3	16.72	1	9.48	32.76
1	4	22.30	1	9.48	32.76
1	5	27.87	1	9.48	32.76
1	6	33.45	1	9.48	32.76
1	7	39.03	1	9.48	32.76
1	8	44.60	1	9.48	32.76
1	9	50.18	1	9.48	32.76
1	10	55.75	1	9.48	32.76
1	11	61.33	1	9.48	32.76
1	12	66.90	1	9.48	32.76
1	13	72.47	1	9.48	32.76
1	14	78.05	1	9.48	32.76
1	15	83.62	1	9.48	32.76
1	16	89.20	1	9.48	32.76
1	17	94.77	1	9.48	32.76
1	18	100.35	1	9.48	32.76
1	19	105.92	1	9.48	32.76
1	20	111.50	1	9.48	32.76

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION  
\*\*\*\*\*

				S T R E N G T H C A T E G O R Y,				Category	
SP NO	IN NO	D FROM L SUPT (ft)	Section Region	Noncomposite Web [1]	Flange [2]	Composite Web [3]	Flange [4]	Non- Comp.	Comp.
1	0	0.00	1	2	2	2		2	2
1	1	5.57	1	2	2	2		2	2

						5799	Int	Left.res		
1	2	11.15	1	2	2	2	2		2	2
1	3	16.72	1	2	2	2	2		2	2
1	4	22.30	1	2	2	2	2		2	2
1	5	27.87	1	2	2	2	2		2	2
1	6	33.45	1	2	2	2	2		2	2
1	7	39.03	1	2	2	2	2		2	2
1	8	44.60	1	2	2	2	2		2	2
1	9	50.18	1	2	2	2	2		2	2
1	10	55.75	1	2	2	2	2		2	2
1	11	61.33	1	2	2	2	2		2	2
1	12	66.90	1	2	2	2	2		2	2
1	13	72.47	1	2	2	2	2		2	2
1	14	78.05	1	2	2	2	2		2	2
1	15	83.62	1	2	2	2	2		2	2
1	16	89.20	1	2	2	2	2		2	2
1	17	94.77	1	2	2	2	2		2	2
1	18	100.35	1	2	2	2	2		2	2
1	19	105.92	1	2	2	2	2		2	2
1	20	111.50	1	2	2	2	2		2	2

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

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- NOTE: [1] For non-composite 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  $F_y$  of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCTIBILITY CHECK (STRENGTH IV)

\*\*\*\*\*

SP	IN	D	FROM (ft)	f <sub>l</sub> (ksi)	0.6F <sub>yt</sub> (ksi)	f <sub>bu</sub> (ksi)	[1] (ksi)	f <sub>bu</sub> +f <sub>l</sub> (ksi)	[2] (ksi)	f <sub>bu</sub> +1/3f <sub>l</sub> (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	30.0	0.0	26.6	0.0	48.6	0.0	48.2	0	
			0.0	-	0.0	-	0.0	48.6	-	-	0	
1	1	5.57	0.0	30.0	6.8	26.6	6.8	49.1	6.8	48.7	0	
			0.0	-	4.2	-	4.2	49.1	-	-	0	
1	2	11.15	0.0	30.0	12.9	26.6	12.9	49.1	12.9	48.7	0	
			0.0	-	8.0	-	8.0	49.1	-	-	0	
1	3	16.72	0.0	30.0	18.3	26.6	18.3	49.1	18.3	48.7	0	
			0.0	-	11.4	-	11.4	49.1	-	-	0	
1	4	22.30	0.0	30.0	22.9	26.6	22.9	49.1	22.9	48.7	0	
			0.0	-	14.3	-	14.3	49.1	-	-	0	
1	5	27.87	0.0	30.0	26.9	26.6	26.9	49.1	26.9	39.3	1	
			0.0	-	16.8	-	16.8	49.1	-	-	0	
1	6	33.45	0.0	30.0	30.1	26.6	30.1	49.1	30.1	39.3	1	

5799 Int Left.res										
1	7	39.03	0.0	-	18.8	-	18.8	49.1	-	0
			0.0	30.0	32.6	26.6	32.6	49.1	32.6	1
			0.0	-	20.4	-	20.4	49.1	-	0
1	8	44.60	0.0	30.0	34.4	26.6	34.4	49.1	34.4	1
			0.0	-	21.5	-	21.5	49.1	-	0
1	9	50.18	0.0	30.0	35.5	26.6	35.5	49.1	35.5	1
			0.0	-	22.2	-	22.2	49.1	-	0
1	10	55.75	0.0	30.0	35.9	26.6	35.9	49.1	35.9	1
			0.0	-	22.4	-	22.4	49.1	-	0
1	11	61.33	0.0	30.0	35.5	26.6	35.5	49.1	35.5	1
			0.0	-	22.2	-	22.2	49.1	-	0
1	12	66.90	0.0	30.0	34.4	26.6	34.4	49.1	34.4	1
			0.0	-	21.5	-	21.5	49.1	-	0
1	13	72.47	0.0	30.0	32.6	26.6	32.6	49.1	32.6	1
			0.0	-	20.4	-	20.4	49.1	-	0
1	14	78.05	0.0	30.0	30.1	26.6	30.1	49.1	30.1	1
			0.0	-	18.8	-	18.8	49.1	-	0
1	15	83.62	0.0	30.0	26.9	26.6	26.9	49.1	26.9	1
			0.0	-	16.8	-	16.8	49.1	-	0
1	16	89.20	0.0	30.0	22.9	26.6	22.9	49.1	22.9	1
			0.0	-	14.3	-	14.3	49.1	-	0
1	17	94.77	0.0	30.0	18.3	26.6	18.3	49.1	18.3	1
			0.0	-	11.4	-	11.4	49.1	-	0
1	18	100.35	0.0	30.0	12.9	26.6	12.9	49.1	12.9	1
			0.0	-	8.0	-	8.0	49.1	-	0
1	19	105.92	0.0	30.0	6.8	26.6	6.8	49.1	6.8	1
			0.0	-	4.2	-	4.2	49.1	-	0
1	20	111.50	0.0	30.0	0.0	26.6	0.0	48.6	0.0	1
			0.0	-	0.0	-	0.0	48.6	-	0

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

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG

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

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

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

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

SP	IN	D FROM	fl / 0.6Fyt	fbu/ [1]	fbu+fl / [2]	fbu+1/3fl / [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	5.57	0.000	0.256	0.138	0.140	0.256	2
					0.086		0.086	3
1	2	11.15	0.000	0.485	0.262	0.264	0.485	2
					0.164		0.164	3
1	3	16.72	0.000	0.687	0.372	0.375	0.687	2
					0.232		0.232	3
1	4	22.30	0.000	0.863	0.467	0.471	0.863	2
					0.292		0.292	3
1	5	27.87	0.000	1.011	0.547	0.684	1.011	2
					0.342		0.342	3
1	6	33.45	0.000	1.132	0.613	0.766	1.132	2
					0.383		0.383	3
1	7	39.03	0.000	1.226	0.664	0.830	1.226	2
					0.415		0.415	3
1	8	44.60	0.000	1.294	0.701	0.876	1.294	2
					0.438		0.438	3

				5799 Int Left.res					
1	9	50.18	0.000	1.335	0.723	0.854	1.335	2	
					0.452		0.452	3	
1	10	55.75	0.000	1.348	0.730	0.863	1.348	2	
					0.456		0.456	3	
1	11	61.33	0.000	1.335	0.723	0.854	1.335	2	
					0.452		0.452	3	
1	12	66.90	0.000	1.294	0.701	0.876	1.294	2	
					0.438		0.438	3	
1	13	72.47	0.000	1.226	0.664	0.830	1.226	2	
					0.415		0.415	3	
1	14	78.05	0.000	1.132	0.613	0.766	1.132	2	
					0.383		0.383	3	
1	15	83.62	0.000	1.011	0.547	0.684	1.011	2	
					0.342		0.342	3	
1	16	89.20	0.000	0.863	0.467	0.471	0.863	2	
					0.292		0.292	3	
1	17	94.77	0.000	0.687	0.372	0.375	0.687	2	
					0.232		0.232	3	
1	18	100.35	0.000	0.485	0.262	0.264	0.485	2	
					0.164		0.164	3	
1	19	105.92	0.000	0.256	0.138	0.140	0.256	2	
					0.086		0.086	3	
1	20	111.50	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 IN D FROM ID	Mu+1/3fl	Sxt	[1]	fbu	[2]	fl	[3]	fbu+	[4]	FLAG
NO NO L SUPT	(ft)	(k-ft)	(k-ft)	(ksi)	(ksi)	(ksi)	(ksi)	1/3fl	(ksi)	MCTD
1 0	0.00	2	-	-	0.0	48.1	-	-	-	-0-0
			-	-	0.0	-	0.0	30.0	0.0	48.1 --00
1 1	5.57	2	-	-	6.9	48.1	-	-	-	-0-0
			-	-	8.5	-	0.0	30.0	8.5	48.1 --00
1 2	11.15	2	-	-	13.1	48.1	-	-	-	-0-0
			-	-	16.0	-	0.0	30.0	16.0	48.1 --00
1 3	16.72	2	-	-	18.6	48.1	-	-	-	-0-0
			-	-	22.7	-	0.0	30.0	22.7	48.1 --00
1 4	22.30	2	-	-	23.3	48.1	-	-	-	-0-0
			-	-	28.4	-	0.0	30.0	28.4	48.1 --00
1 5	27.87	2	-	-	27.3	48.1	-	-	-	-0-0
			-	-	33.2	-	0.0	30.0	33.2	48.1 --00
1 6	33.45	2	-	-	30.6	48.1	-	-	-	-0-0
			-	-	37.1	-	0.0	30.0	37.1	48.1 --00
1 7	39.03	2	-	-	33.1	48.1	-	-	-	-0-0
			-	-	40.1	-	0.0	30.0	40.1	48.1 --00
1 8	44.60	2	-	-	34.9	48.1	-	-	-	-0-0
			-	-	42.3	-	0.0	30.0	42.3	48.1 --00
1 9	50.18	2	-	-	36.0	48.1	-	-	-	-0-0
			-	-	43.5	-	0.0	30.0	43.5	48.1 --00
1 10	55.75	2	-	-	36.4	48.1	-	-	-	-0-0
			-	-	43.8	-	0.0	30.0	43.8	48.1 --00
1 11	61.33	2	-	-	36.0	48.1	-	-	-	-0-0
			-	-	43.5	-	0.0	30.0	43.5	48.1 --00
1 12	66.90	2	-	-	34.9	48.1	-	-	-	-0-0
			-	-	42.3	-	0.0	30.0	42.3	48.1 --00
1 13	72.47	2	-	-	33.1	48.1	-	-	-	-0-0
			-	-	40.1	-	0.0	30.0	40.1	48.1 --00
1 14	78.05	2	-	-	30.6	48.1	-	-	-	-0-0

5799 Int Left.res											
1 15	83.62	2	-	-	37.1	-	0.0	30.0	37.1	48.1	--00
			-	-	27.3	48.1	-	-	-	-0-0	
			-	-	33.2	-	0.0	30.0	33.2	48.1	--00
1 16	89.20	2	-	-	23.3	48.1	-	-	-	-0-0	
			-	-	28.4	-	0.0	30.0	28.4	48.1	--00
			-	-	18.6	48.1	-	-	-	-0-0	
1 17	94.77	2	-	-	22.7	-	0.0	30.0	22.7	48.1	--00
			-	-	13.1	48.1	-	-	-	-0-0	
			-	-	16.0	-	0.0	30.0	16.0	48.1	--00
1 18	100.35	2	-	-	6.9	48.1	-	-	-	-0-0	
			-	-	8.5	-	0.0	30.0	8.5	48.1	--00
			-	-	0.0	48.1	-	-	-	-0-0	
1 20	111.50	2	-	-	0.0	-	0.0	30.0	0.0	48.1	--00
			-	-							
			-	-							

NOTE: [1] =  $(\Phi) f * M_n$  (Eq. 6.10.7.1.1-1)  
[2] =  $(\Phi) f * F_{nc}$  for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)  
=  $(\Phi) f * R_h * F_{yt}$  for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)  
[3] =  $0.6 * F_{yt}$  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)  
=  $0.6 * F_{yc}$  for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)  
[4] =  $(\Phi) f * F_{nt}$  for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)  
=  $(\Phi) f * F_{nc}$  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  $f_{bu}$ ,  $M_u$  and  $f_l$  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  $f_{bu}$  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+1/3fI	Sxt/	f <sub>bu</sub> /	f <sub>l</sub> /	f <sub>bu</sub> +1/3f <sub>l</sub> /	MAX.	
						[1]	[2]	[3]	[4]	RAT.	GOVN.
1	0	0.00	2				0.000			0.000	2
1	1	5.57	2				0.144	0.000	0.000	0.000	4
										0.144	2
1	2	11.15	2				0.273	0.000	0.176	0.176	4
										0.273	2
1	3	16.72	2				0.387	0.000	0.334	0.334	4
										0.387	2
1	4	22.30	2				0.485	0.000	0.472	0.472	4
										0.485	2
1	5	27.87	2				0.568	0.000	0.591	0.591	4
										0.568	2
1	6	33.45	2				0.636	0.000	0.691	0.691	4
										0.636	2
1	7	39.03	2				0.689	0.000	0.772	0.772	4
										0.689	2
1	8	44.60	2				0.727	0.000	0.835	0.835	4
										0.727	2
1	9	50.18	2				0.749	0.000	0.880	0.880	4
										0.749	2
								0.000	0.904	0.904	4



5799 Int Left.res									
1	10	55.75	2	0.757		0.000	0.912	0.757	2
1	11	61.33	2	0.749		0.000	0.912	0.749	4
1	12	66.90	2	0.727		0.000	0.904	0.727	2
1	13	72.47	2	0.689		0.000	0.880	0.689	2
1	14	78.05	2	0.636		0.000	0.835	0.636	2
1	15	83.62	2	0.568		0.000	0.772	0.568	2
1	16	89.20	2	0.485		0.000	0.691	0.485	2
1	17	94.77	2	0.387		0.000	0.591	0.387	2
1	18	100.35	2	0.273		0.000	0.472	0.273	2
1	19	105.92	2	0.144		0.000	0.334	0.144	2
1	20	111.50	2	0.000		0.000	0.176	0.000	2
						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+1/3fl Sxt) / [1]
- 2 = fbu / [2]
- 3 = fl / [3]
- 4 = (fbu + 1/3fl) / [4]

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY

*****											
SP NO	IN NO	D L	FROM SUPT (ft)	Fy (ksi)	k [1]	Art. 6.10.9.2		D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq. 6.10.9.2-- Vn = C x Vp
-----											
1	0	0.00	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	1	5.57	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	2	11.15	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	3	16.72	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	4	22.30	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	5	27.87	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	6	33.45	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	7	39.03	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	8	44.60	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	9	50.18	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	10	55.75	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	11	61.33	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	12	66.90	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	13	72.47	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	14	78.05	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	15	83.62	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	16	89.20	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	17	94.77	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	18	100.35	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	19	105.92	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	20	111.50	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	

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NOTE: [1]  $k = \text{buckling coefficient} = 5 + 5 / ((d_0/D)^{**2})$   
= 5 for unstiffened beams and girders  
= 5 for stiffened girders when  $(d_0/D) > 3$ , or  
 $(d_0/D) > (260/(D/tw))^{**2}$

[2] =  $1.12 * \sqrt{E_k / F_yw}$

[3] =  $1.40 * \sqrt{E_k / F_yw}$

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

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

For  $[2] \leq D/tw \leq [3]$ ,  $C = 1.12 * \sqrt{E_k / F_yw} / (D/tw)$   
..... AASHTO LRFD Eq. 6.10.9.3.2-6

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

\*\* AASHTO LRFD Art. 6.10.9.1 --- For interior web panels considered  
stiffened  
if  $(d_0/D) < 3$  without long. stiffener, or  
if  $(d_0/D) > 3$  with both transv. and long. stiffeners.  
For handling requirement,  $(d_0/D) > (260/(D/tw))^{**2}$

$V_u = C V_p$

where D = clear, unsupported distance between flange components

$d_0$  = distance between transverse stiffeners

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

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY

\*\*\*\*\*

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

SP	IN	D FROM	WEB STABILITY	LFD/LRFD	UNSTIFFENED	OVERALL
NO	NO	L SUPT	UNSTIFFENED, [1]	MAX. SHEAR <=>	SHEAR CAPA- CATEGORY	WEB
(ft)	----	D/tw	-----	(kip)	CITY, [2]	
1	0	0.00	2	346.3	> 167.5	2
1	1	5.57	2	318.6	> 167.5	2
1	2	11.15	2	291.3	> 167.5	2
1	3	16.72	2	264.3	> 167.5	2
1	4	22.30	2	237.6	> 167.5	2
1	5	27.87	2	210.7	> 167.5	2
1	6	33.45	2	184.6	> 167.5	2
1	7	39.03	2	158.8	< 167.5	2
1	8	44.60	2	133.3	< 167.5	2
1	9	50.18	2	107.6	< 167.5	2
1	10	55.75	2	82.7	< 167.5	2
1	11	61.33	2	107.6	< 167.5	2
1	12	66.90	2	133.3	< 167.5	2
1	13	72.47	2	158.8	< 167.5	2
1	14	78.05	2	184.6	> 167.5	2
1	15	83.62	2	210.7	> 167.5	2
1	16	89.20	2	237.6	> 167.5	2
1	17	94.77	2	264.3	> 167.5	2
1	18	100.35	2	291.3	> 167.5	2

				5799 Int Left.res		
1	19	105.92	2	318.6	>	167.5
1	20	111.50	2	346.3	>	167.5
						2
						2

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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.

$D/tw \leq 150$  (Eq. 6.10.2.1.1-1)

w / long. stiff.

$D/tw \leq 300$  (Eq. 6.10.2.1.2-1)

$2Dcp/tw \leq 3.76 \sqrt{E/Fyc}$  ..... 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)

$2Dc/tw \leq 5.7 \sqrt{E/Fyc}$  ..... AASHTO Eq.6.10.6.2.3-1

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

$V_n = CV_p$  ..... AASHTO Eq.6.10.9.2-1

$V_n = V_p R_{c1}$  ..... AASHTO Eq.6.10.9.3.2-2

$R_{c1} = C + [0.87(1-C)] / \sqrt{1 + (d_0/D)^2}$  for  $2Dtw / (bfctfc + bfttft) \leq 2.5$

$V_n = V_p R_{c2}$  ..... AASHTO Eq.6.10.9.3.2-8

$R_{c2} = C + [0.87(1-C)] / [\sqrt{1 + (d_0/D)^2} + (d_0/D)]$  for  $2Dtw / (bfctfc + bfttft) > 2.5$

$V_p = 0.58 F_y D^* tw$  ..... 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	IN	D	FROM	YIELD	LFD/LRFD	UNSTIFFENED	REQUIREMENT OF	MAX. ALLOWABLE
NO	NO	L	SUPT	STRESS	MAXIMUM	SHEAR	TRANS. STIFFENERS	TRANS. STIFFENERS
			(ft)	$F_y$	SHEAR	CAPACITY	1=YES, 0=NO	SPACING
				(ksi)	(k)	(k)		(ft-in)
1	0	0.00		36.	346.25	167.53	1	8 - 3
1	1	5.57		36.	318.64	167.53	1	14 - 4
1	2	11.15		36.	291.32	167.53	1	16 - 6
1	3	16.72		36.	264.31	167.53	1	16 - 6
1	4	22.30		36.	237.60	167.53	1	16 - 6
1	5	27.87		36.	210.70	167.53	1	16 - 6
1	6	33.45		36.	184.60	167.53	1	16 - 6
1	7	39.03		36.	158.79	167.53	0	
1	8	44.60		36.	133.29	167.53	0	
1	9	50.18		36.	107.61	167.53	0	
1	10	55.75		36.	82.71	167.53	0	

5799 Int Left.res

\*\*\*\*\*

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5799 Int Left.res

1	9	50.18	151.	1.0000	603.	16-	0	0.506	107.61	<	304.86
1	10	55.75	151.	1.0000	603.	16-	0	0.506	82.71	<	304.86
1	11	61.33	151.	1.0000	603.	16-	0	0.506	107.61	<	304.86
1	12	66.90	151.	1.0000	603.	8-	0	0.700	133.29	<	422.27
1	13	72.47	151.	1.0000	603.	8-	0	0.700	158.79	<	422.27
1	14	78.05	151.	1.0000	603.	8-	0	0.700	184.60	<	422.27
1	15	83.62	151.	1.0000	603.	8-	0	0.700	210.70	<	422.27
1	16	89.20	151.	1.0000	603.	8-	0	0.700	237.60	<	422.27
1	17	94.77	151.	1.0000	603.	8-	0	0.700	264.31	<	422.27
1	18	100.35	151.	1.0000	603.	7-	0	0.745	291.32	<	449.38
1	19	105.92	151.	1.0000	603.	6-	0	0.799	318.64	<	481.49
1	20	111.50	151.	1.0000	603.	2-	9	1.000	346.25	<	602.91

-----  
NOTE: Assumed that all the supports have stiffeners

STATUS: (1) --- blank: shear capacity is ok  
(2) --- <= : insufficient shear capacity  
(3) ---  $V_n = CC * V_p$ ; CC see AASHTO LRFD 6.10.9.3.2-2 OR -8  
or Table .22.15

CC includes post-buckling strength due to tension field action

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TABLE 1.2.22.20=SECTION PROPERTY CHECK OF TRANSVERSE STIFFENERS

TABLE 1-2. 22. 20

\*\*\*\*\*

			MIN. [1]		WIDTH (in)		MAX. [2]		I, 6. 10. 11. 1. 3, (in**4)					FL	
SP	IN	D FROM	AASHTO	Eq. 6. 10. 11. 1. 2-1, 2	It	[3]	[4]	[5]	[6]			AG			
NO	NO	L SUPT	MIN. <=>	ACTUAL <=>	MAX.	It1	It2	It3	It4						
-----															
1	0	0.0	4.2 <	5.0 <	6.0	36.	22.	21.	0.			0.			
1	1	5.6	4.2 <	5.0 <	6.0	16.	3.	21.	11.			0.			
1	2	11.1	4.2 <	5.0 <	6.0	16.	3.	21.	11.			0.			
1	3	16.7	4.2 <	7.0 >	6.0	94.	3.	32.	14.			0. <=			
1	4	22.3	4.2 <	7.0 >	6.0	94.	3.	32.	11.			0. <=			
1	5	27.9	4.2 <	5.0 <	6.0	16.	3.	21.	6.			0.			
1	6	33.5	4.2 <	5.0 <	6.0	16.	3.	21.	4.			0.			
1	7	39.03													
1	8	44.60													
1	9	50.18													
1	10	55.75													
1	11	61.33													
1	12	66.90													
1	13	72.47													
1	14	78.0	4.2 <	5.0 <	6.0	16.	3.	21.	4.			0.			
1	15	83.6	4.2 <	7.0 >	6.0	94.	3.	32.	8.			0. <=			
1	16	89.2	4.2 <	5.0 <	6.0	16.	3.	21.	8.			0.			
1	17	94.8	4.2 <	5.0 <	6.0	16.	3.	21.	10.			0.			
1	18	100.3	4.2 <	5.0 <	6.0	16.	3.	21.	11.			0.			
1	19	105.9	4.2 <	5.0 <	6.0	16.	3.	21.	11.			0.			
1	20	111.5	4.2 <	5.0 <	6.0	36.	22.	21.	11.			0.			

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\*\* NOTE [1 or 2]: Minimum width of transverse stiffeners  
AASHTO LRFD Eq. 6.10.11.1.2-1 or -2

[3]: required moment of inertia for trans. stiff.  
AASHTO LRFD Eq. 6.10.11.1.3-3

[4]: required moment of inertia for trans. stiff.  
AASHTO LRFD Eq. 6.10.11.1.3-4

[5]: required moment of inertia for trans. stiff.  
AASHTO LRFD Eq. 6.10.11.1.3-7

[6]: required moment of inertia for trans. stiff.

FLAG : (1) --- BLANK : acceptable trans. stiff.  
(2) --- <= : trans. stiff. violates AASHTO requirement(s)

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TABLE 1.2.22.20A=REQ. SECTION PROPERTIES FOR TRANS. STIFFENERS.  
\*\*\*\*\*

SP IN D FROM	WEB	Do	ASSUMED	J	REQ'D I
NO NO L SUPT	DEPTH THICK.	-(ft-in)-	B Y	AASHTO	-(in**4)-
(ft)	(in)	ASSUMED		6.10.8.1.3-1,2	
		SPACING			
1 0 0.00	66.0 0.4375	2- 9	1.0 1.0	8.0	22.1
1 1 5.57	66.0 0.4375	6- 0	2.4 1.0	0.5	3.0
1 2 11.15	66.0 0.4375	7- 0	2.4 1.0	0.5	3.5
1 3 16.72	66.0 0.4375	8- 0	1.0 1.0	0.5	4.0
1 4 22.30	66.0 0.4375	8- 0	1.0 1.0	0.5	4.0
1 5 27.87	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 6 33.45	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 7 39.03	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 8 44.60	66.0 0.4375	8- 0	1.0 1.0	0.5	4.0
1 9 50.18	66.0 0.4375	16- 0	1.0 1.0	0.5	8.0
1 10 55.75	66.0 0.4375	16- 0	1.0 1.0	0.5	8.0
1 11 61.33	66.0 0.4375	16- 0	1.0 1.0	0.5	8.0
1 12 66.90	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 13 72.47	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 14 78.05	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 15 83.62	66.0 0.4375	8- 0	1.0 1.0	0.5	4.0
1 16 89.20	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 17 94.77	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 18 100.35	66.0 0.4375	7- 0	2.4 1.0	0.5	3.5
1 19 105.92	66.0 0.4375	6- 0	2.4 1.0	0.5	3.0
1 20 111.50	66.0 0.4375	2- 9	1.0 1.0	8.0	22.1

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK  
\*\*\*\*\*

SP IN D FROM	TOP FLANGE	BOT. FLANGE	COMP. FLG	FLAG
NO NO L SUPT	ff	ff+fl/2	fc	fcrw
(ft)	(ksi)	(ksi)	(ksi)	(ksi)
				TBC
1 0 0.00	0.0	-	45.7	0.0
1 1 5.57	5.4	-	45.7	6.5
1 2 11.15	10.3	-	45.7	12.2
1 3 16.72	14.6	-	45.7	17.3
1 4 22.30	18.3	-	45.7	21.7
1 5 27.87	21.5	-	45.7	25.3
1 6 33.45	24.0	-	45.7	28.3
1 7 39.03	26.0	-	45.7	30.6
1 8 44.60	27.5	-	45.7	32.2
1 9 50.18	28.3	-	45.7	33.1
1 10 55.75	28.6	-	45.7	33.4
1 11 61.33	28.3	-	45.7	33.1
1 12 66.90	27.5	-	45.7	32.2
1 13 72.47	26.0	-	45.7	30.6
1 14 78.05	24.0	-	45.7	28.3
1 15 83.62	21.5	-	45.7	25.3
1 16 89.20	18.3	-	45.7	21.7
1 17 94.77	14.6	-	45.7	17.3
1 18 100.35	10.3	-	45.7	12.2
1 19 105.92	5.4	-	45.7	6.5
1 20 111.50	0.0	-	45.7	0.0

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NOTE: [1] =  $0.95R_h F_y f$  for composite sections (Eq. 6.10.4.2.2-1)  
=  $0.80R_h F_y f$  for non-composite sections (Eq. 6.10.4.2.2-3)  
[2] =  $0.95R_h F_y f$  for composite sections (Eq. 6.10.4.2.2-2)  
=  $0.80R_h F_y f$  for non-composite sections (Eq. 6.10.4.2.2-3)  
[3] =  $0.9E_k / (D/tw)^2$  (Eq. 6.10.1.9.1-1)  
but not to exceed the smaller of  $R_h F_{yc}$  and  $F_{yw}/0.7$   
k = bending-buckling coefficient  
=  $9 / (D_c/D)^2$  (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 IN D FROM NO NO L SUPT (ft)	TOP OF TOP FLANGE			BOTTOM OF BOTTOM FLANGE		
	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1 0 0.00	TR	0.0	A B B^C^C D E E^A	TR	0.0	A B B^C^C D E E^A
1 1 5.57	TR	-0.2	A B B^C^C D E E^A	TR	1.1	A B B^C^C D E E^A
1 2 11.15	TR	-0.4	A B B^C^C D E E^A	TR	2.0	A B B^C^C D E E^A
1 3 16.72	TR	-0.5	A B B^C^C D E E^A	TR	2.8	A B B^C^C D E
1 4 22.30	TR	-0.7	A B B^C^C D E E^A	TR	3.5	A B B^C^C D E
1 5 27.87	TR	-0.8	A B B^C^C D E E^A	TR	4.1	A B B^C^C D E
1 6 33.45	TR	-0.9	A B B^C^C D E E^A	TR	4.5	A B B^C^C D
1 7 39.03	TR	-0.9	A B B^C^C D E E^A	TR	4.8	A B B^C^C D
1 8 44.60	TR	-1.0	A B B^C^C D E E^A	TR	5.0	A B B^C^C D
1 9 50.18	TR	-0.9	A B B^C^C D E E^A	TR	5.0	A B B^C^C D
1 10 55.75	TR	-0.9	A B B^C^C D E E^A	TR	5.0	A B B^C^C D
1 11 61.33	TR	-0.9	A B B^C^C D E E^A	TR	5.0	A B B^C^C D
1 12 66.90	TR	-1.0	A B B^C^C D E E^A	TR	5.0	A B B^C^C D
1 13 72.47	TR	-0.9	A B B^C^C D E E^A	TR	4.8	A B B^C^C D
1 14 78.05	TR	-0.9	A B B^C^C D E E^A	TR	4.5	A B B^C^C D
1 15 83.62	TR	-0.8	A B B^C^C D E E^A	TR	4.1	A B B^C^C D E
1 16 89.20	TR	-0.7	A B B^C^C D E E^A	TR	3.5	A B B^C^C D E
1 17 94.77	TR	-0.5	A B B^C^C D E E^A	TR	2.8	A B B^C^C D E
1 18 100.35	TR	-0.4	A B B^C^C D E E^A	TR	2.0	A B B^C^C D E E^A
1 19 105.92	TR	-0.2	A B B^C^C D E E^A	TR	1.1	A B B^C^C D E E^A
1 20 111.50	TR	0.0	A B B^C^C D E E^A	TR	0.0	A B B^C^C D E E^A

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 IN D FROM NO NO L SUPT (ft)	TOP OF TOP FLANGE			BOTTOM OF BOTTOM FLANGE		
	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	GOVERN. LOADING	STRESS RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1 0 0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1 1 5.57	TR	-0.1	A B B^C^C D E E^	TR	0.5	A B B^C^C D E E^
1 2 11.15	TR	-0.2	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1 3 16.72	TR	-0.3	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1 4 22.30	TR	-0.3	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1 5 27.87	TR	-0.4	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1 6 33.45	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1 7 39.03	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E E^
1 8 44.60	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E E^
1 9 50.18	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E E^
1 10 55.75	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E E^
1 11 61.33	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E E^
1 12 66.90	TR	-0.5	A B B^C^C D E E^	TR	2.5	A B B^C^C D E E^
1 13 72.47	TR	-0.5	A B B^C^C D E E^	TR	2.4	A B B^C^C D E E^
1 14 78.05	TR	-0.4	A B B^C^C D E E^	TR	2.3	A B B^C^C D E E^
1 15 83.62	TR	-0.4	A B B^C^C D E E^	TR	2.0	A B B^C^C D E E^
1 16 89.20	TR	-0.3	A B B^C^C D E E^	TR	1.7	A B B^C^C D E E^
1 17 94.77	TR	-0.3	A B B^C^C D E E^	TR	1.4	A B B^C^C D E E^
1 18 100.35	TR	-0.2	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1 19 105.92	TR	-0.1	A B B^C^C D E E^	TR	0.5	A B B^C^C D E E^
1 20 111.50	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; LRF D Fatigue II Limit State with 0.75 load factor.  
Design for Finite Life w/ ADTT Single Lane= 1012 & No. of Cycles= 27703500  
\* 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 IN D FROM AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN. MAX. ALLOW.  
6.10.10.2-1 PER TRANS. SHEAR CONN.



NO		L SUPT		Vf	Q/I	Vsr	5799 Int Left.res		SECTION	---PITCH---	
		(ft)		[1]	[2]	[3]	[4], Zr <---Input		[5], input	(in), [6]	
1	0	0.00	65.5	.14198E-01	0.9	TR	4.21	2	9.06		
1	1	5.57	63.2	.14198E-01	0.9	TR	4.21	2	9.38		
1	2	11.15	61.1	.14198E-01	0.9	TR	4.21	2	9.70		
1	3	16.72	59.3	.14198E-01	0.8	TR	4.21	2	10.00		
1	4	22.30	57.4	.14198E-01	0.8	TR	4.21	2	10.33		
1	5	27.87	55.1	.14198E-01	0.8	TR	4.21	2	10.77		
1	6	33.45	53.5	.14198E-01	0.8	TR	4.21	2	11.08		
1	7	39.03	53.0	.14198E-01	0.8	TR	4.21	2	11.19		
1	8	44.60	52.7	.14198E-01	0.7	TR	4.21	2	11.26		
1	9	50.18	52.7	.14198E-01	0.7	TR	4.21	2	11.26		
1	10	55.75	52.7	.14198E-01	0.7	TR	4.21	2	11.26		
1	11	61.33	52.7	.14198E-01	0.7	TR	4.21	2	11.26		
1	12	66.90	52.7	.14198E-01	0.7	TR	4.21	2	11.26		
1	13	72.47	53.0	.14198E-01	0.8	TR	4.21	2	11.19		
1	14	78.05	53.5	.14198E-01	0.8	TR	4.21	2	11.08		
1	15	83.62	55.1	.14198E-01	0.8	TR	4.21	2	10.77		
1	16	89.20	57.4	.14198E-01	0.8	TR	4.21	2	10.33		
1	17	94.77	59.3	.14198E-01	0.8	TR	4.21	2	10.00		
1	18	100.35	61.1	.14198E-01	0.9	TR	4.21	2	9.70		
1	19	105.92	63.2	.14198E-01	0.9	TR	4.21	2	9.38		
1	20	111.50	65.5	.14198E-01	0.9	TR	4.21	2	9.06		

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

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

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

[3] Vsr = range of horizontal shear, in Kips per inches, at the junction of the slab and girder at the point in the span under consideration.

=  $\sqrt{Vfat^2 + Ffat^2}$  (Eq. 6.10.10.1.2-2)

Vfat =  $Vf \cdot Q/I$  (Eq. 6.10.10.1.2-3)

Ffat = radial fatigue shear range per unit length

=  $Ffat^2 = Frc/w$  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= 1012

\*\*\* 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	IN	D FROM	AASHTO E6. 10. 10. 1. 2			AASHTO LRFD Eqn.		SHEAR CONN.	MAX. ALLOW.
NO	NO	L SUPT	Vf	Q/I	Vsr	6. 10. 10. 2-2	4], Zr <--Input	PER TRANS. SECTION	SHEAR CONN.
		(ft)	[1]	[2]	[3]			[5], input	PI TCH----
									(in), [6]
1	0	0.00	32.7	.14198E-01	0.5	TR	2.03	2	8.71
1	1	5.57	31.6	.14198E-01	0.4	TR	2.03	2	9.02
1	2	11.15	30.6	.14198E-01	0.4	TR	2.03	2	9.33
1	3	16.72	29.7	.14198E-01	0.4	TR	2.03	2	9.62
1	4	22.30	28.7	.14198E-01	0.4	TR	2.03	2	9.94
1	5	27.87	27.5	.14198E-01	0.4	TR	2.03	2	10.36
1	6	33.45	26.8	.14198E-01	0.4	TR	2.03	2	10.66
1	7	39.03	26.5	.14198E-01	0.4	TR	2.03	2	10.77
1	8	44.60	26.3	.14198E-01	0.4	TR	2.03	2	10.83
1	9	50.18	26.3	.14198E-01	0.4	TR	2.03	2	10.83
1	10	55.75	26.3	.14198E-01	0.4	TR	2.03	2	10.83
1	11	61.33	26.3	.14198E-01	0.4	TR	2.03	2	10.83
1	12	66.90	26.3	.14198E-01	0.4	TR	2.03	2	10.83
1	13	72.47	26.5	.14198E-01	0.4	TR	2.03	2	10.77
1	14	78.05	26.8	.14198E-01	0.4	TR	2.03	2	10.66
1	15	83.62	27.5	.14198E-01	0.4	TR	2.03	2	10.36
1	16	89.20	28.7	.14198E-01	0.4	TR	2.03	2	9.94
1	17	94.77	29.7	.14198E-01	0.4	TR	2.03	2	9.62
1	18	100.35	30.6	.14198E-01	0.4	TR	2.03	2	9.33
1	19	105.92	31.6	.14198E-01	0.4	TR	2.03	2	9.02
1	20	111.50	32.7	.14198E-01	0.5	TR	2.03	2	8.71

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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.

$$= \sqrt{Vfat^2 + Ffat^2} \quad (\text{Eq. 6. 10. 10. 1. 2-2})$$

$$Vfat = Vf \cdot Q/I \quad (\text{Eq. 6. 10. 10. 1. 2-3})$$

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= 1012 & No. of Cycles= 27703500  
\*\*\* If ADTT Single Lane greater than or equal to 960, refer  
to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT REGION			NO OF SHEAR CONNECTOR N(PER ZONE) <=> FATIG. CRI.	SEE NOTE		STATUS
				1=POS. 0=NEG.	(k) LRFD	(k) 6.10.10.4.2		N1	N2	
1	10	55.75	1	2877.0	2882.9		150	>	110	BLANK=OK CHECK=**
1	20	111.50	1	2877.0	2882.9		150	>	110	

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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 :

$$N1 (N2) = P / Q_r \quad \text{--- (Eq. 6.10.10.4.1-2)}$$

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

Q<sub>r</sub> = factored shear resistance of one shear connector  
= (Phi)<sub>sc</sub> Q<sub>n</sub> --- (Eq. 6.10.10.4.1-1)

Q<sub>n</sub> = nominal shear resistance

(Phi)<sub>sc</sub> = 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:

$$P1p = 0.85 f'_c b_s t_s \quad \text{--- (Eq. 6.10.10.4.2-2)}$$

or

$$P2p, 1n = F_y w D t_w + F_y t b_{ft} t_{ft} + F_y c b_{fc} t_{fc} \quad \text{(Eq. 6.10.10.4.2-3, 7)}$$

Where

f'<sub>c</sub> = specified 28-day compressive strength of the concrete

b<sub>s</sub> = effective width of the slab

b<sub>fc</sub> = width of compression flange

b<sub>ft</sub> = width of tension flange

t<sub>s</sub> = slab thickness

F<sub>yw</sub> = specified minimum yield strength of the web

F<sub>yt</sub> = specified minimum yield strength of the tension flange

F<sub>yc</sub> = specified minimum yield strength of the compression flange

D = web depth

t<sub>ft</sub> = thickness of tension flange

t<sub>fc</sub> = 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:

$$P2n = 0.45 f'c b_s t_s \quad \text{--- (Eq. 6.10.10.4.2-8)}$$

Where

$b_s$  = effective width of the concrete deck  
 $t_s$  = 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=RECOMMENDED SHEAR CONNECTOR REQUIRED PITCH  
 \*\*\*\*\*

SPAN NO.	CURRENT FROM (ft)	SPAN TO (ft)	REQUIRED PITCH (in)
1	0.000	55.750	9.000
1	55.750	111.500	9.000

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

SPAN NO.	CURRENT FROM (ft)	SPAN TO (ft)	T C R
1	0.0	111.5	C

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

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY  
 \*\*\*\*\*

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
NG	1.2.22.10=CONSTRUCTIBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.18=STATUS OF TRANSVERSE STIFFENERS

OK 1.2.22.19=SHEAR CAPACITY CHECK  
 NG 1.2.22.20=SECTION PROPERTY CHECK OF TRANSVERSE STIFFENERS  
 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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 LRF -- 2010  
 RATING

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesignPAGE 85\5799\Rating\MD\5

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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
					D1; k-ft ksi	D2	MAX. MOMENT					
							POS. MAX. STRESS	NEG. STRESS				
1	0	0.00	48.06	TOP	0.00	0.00	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	0.00	0.00	0.00	0.00	9.99	HL-93	9.99	2
1	1	5.57	48.06	TOP	-4.94	-0.43	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	3.60	2.27	0.00	0.00	9.99	HL-93	9.99	2
1	2	11.15	48.06	TOP	-9.36	-0.81	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	3	16.72	48.06	TOP	-13.27	-1.14	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	4	22.30	48.06	TOP	-16.66	-1.43	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	5	27.87	48.06	TOP	-19.52	-1.66	0.00	0.00	9.99	HL-93	8.13	2
			48.06	BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	6	33.45	48.06	TOP	-21.86	-1.85	0.00	0.00	8.29	HL-93	6.39	2
			48.06	BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	7	39.03	48.06	TOP	-23.69	-2.00	0.00	0.00	6.83	HL-93	5.27	2
			48.06	BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	8	44.60	48.06	TOP	-25.00	-2.10	0.00	0.00	5.92	HL-93	4.57	2
			48.06	BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	9	50.18	48.05	TOP	-25.78	-2.15	0.00	0.00	5.44	HL-93	4.20	2
			48.06	BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	10	55.75	48.05	TOP	-26.04	-2.17	0.00	0.00	5.29	HL-93	4.08	2
			48.06	BOT	19.00	11.48	0.00	1.57	HL-93	1.21	2	
1	11	61.33	48.05	TOP	-25.78	-2.15	0.00	0.00	5.44	HL-93	4.20	2
			48.06	BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	12	66.90	48.06	TOP	-25.00	-2.10	0.00	0.00	5.92	HL-93	4.57	2
			48.06	BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	13	72.47	48.06	TOP	-23.69	-2.00	0.00	0.00	6.83	HL-93	5.27	2
			48.06	BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	14	78.05	48.06	TOP	-21.86	-1.85	0.00	0.00	8.29	HL-93	6.39	2
			48.06	BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	15	83.62	48.06	TOP	-19.52	-1.66	0.00	0.00	9.99	HL-93	8.13	2
			48.06	BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	16	89.20	48.06	TOP	-16.66	-1.43	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	17	94.77	48.06	TOP	-13.27	-1.14	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	18	100.35	48.06	TOP	-9.36	-0.81	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	19	105.92	48.06	TOP	-4.94	-0.43	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	3.60	2.27	0.00	0.00	9.99	HL-93	9.99	2
1	20	111.50	48.06	TOP	0.00	0.00	0.00	0.00	9.99	HL-93	9.99	2
			48.06	BOT	0.00	0.00	0.00	0.00	9.99	HL-93	9.99	2

Please read NOTES on the following page

1

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MERLIN V 10.4

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  

$$RF = (Mu - ENS1 * (GDC * D1 * S2 / SO + GDW * D2 * S2 / S1)) / (ENS1 * GLLST1 * (L + I))$$
 WHERE SO, S1 & S2 ARE SECTION MODULUS OF D1, D2 & L  
 STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)

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

TABLE 1.2.32.2=RATING; SERVICEABILITY STRENGTH  
 \*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS---		UNFACTORED L+I STRESS				OPERAT. RATING FACTOR [3]	INVENT.	
				STEEL TOP; ksi; BOT	STEEL TOP; ksi; BOT	MAX. POS. ---STEEL---	MAX. NEG. ---STEEL---	MAX. POS. TOP; ksi; BOT	MAX. NEG. TOP; ksi; BOT		LIVE LOAD TYPE	RATING FACTOR
1	0	0.00	47.5	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	5.57	47.5	-4.89	3.51	-0.43	2.27	0.00	0.00	9.99	HL-93	7.68
1	2	11.15	47.5	-9.26	6.65	-0.81	4.29	0.00	0.00	9.52	HL-93	7.32
1	3	16.72	47.5	-13.13	9.42	-1.14	6.05	0.00	0.00	6.29	HL-93	4.84
1	4	22.30	47.5	-16.49	11.83	-1.43	7.56	0.00	0.00	4.72	HL-93	3.63
1	5	27.87	47.5	-19.32	13.86	-1.66	8.79	0.00	0.00	3.82	HL-93	2.94
1	6	33.45	47.5	-21.64	15.53	-1.85	9.80	0.00	0.00	3.26	HL-93	2.51
1	7	39.03	47.5	-23.45	16.82	-2.00	10.59	0.00	0.00	2.90	HL-93	2.23
1	8	44.60	47.5	-24.74	17.75	-2.10	11.13	0.00	0.00	2.67	HL-93	2.06
1	9	50.18	47.5	-25.52	18.31	-2.15	11.40	0.00	0.00	2.56	HL-93	1.97
1	10	55.75	47.5	-25.78	18.49	-2.17	11.48	0.00	0.00	2.53	HL-93	1.94
1	11	61.33	47.5	-25.52	18.31	-2.15	11.40	0.00	0.00	2.56	HL-93	1.97
1	12	66.90	47.5	-24.74	17.75	-2.10	11.13	0.00	0.00	2.67	HL-93	2.06
1	13	72.47	47.5	-23.45	16.82	-2.00	10.59	0.00	0.00	2.90	HL-93	2.23
1	14	78.05	47.5	-21.64	15.53	-1.85	9.80	0.00	0.00	3.26	HL-93	2.51
1	15	83.62	47.5	-19.32	13.86	-1.66	8.79	0.00	0.00	3.82	HL-93	2.94
1	16	89.20	47.5	-16.49	11.83	-1.43	7.56	0.00	0.00	4.72	HL-93	3.63
1	17	94.77	47.5	-13.13	9.42	-1.14	6.05	0.00	0.00	6.29	HL-93	4.84
1	18	100.35	47.5	-9.27	6.65	-0.81	4.29	0.00	0.00	9.52	HL-93	7.32
1	19	105.92	47.5	-4.89	3.51	-0.43	2.27	0.00	0.00	9.99	HL-93	7.68
1	20	111.50	47.5	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 I 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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COMPOSITE  
LRF -- 2010  
RATING

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

NO.	SPAN		CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
	LENGTH (ft)						
1	111.50		55.75	STRENGTH L.S.	1.57	1.21	HL-93
1	111.50		55.75	SERVICEABILITY	2.53	1.94	HL-93

NOTE: Control Rating Factor for Current Bridge:  
STRENGTH L.S.

Operating= 1.568

Inventory= 1.210

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

NO.	SPAN		CRITICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
	LENGTH (ft)					
1	111.50		55.75	STRENGTH L.S.	2.04	2382.50
1	111.50		55.75	SERVICEABILITY	3.27	

NOTE: control Rating Factor for Current Bridge:  
STRENGTH L.S. with Live Load Factor =1.660 & ADTT = 1190  
Legal Load Rating= 2.036

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

SP IN D FROM NO NO L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
		D1; kips	D2	MAX. POS.	SHEAR NEG.			
1 0 0.00	602.9	81.0	25.0	119.6	0.0	2.89	HL-93	2.23
1 1 5.57	481.5	72.9	22.5	111.6	-2.6	2.38	HL-93	1.83
1 2 11.15	449.4	64.9	20.0	103.7	-5.6	2.42	HL-93	1.87
1 3 16.72	422.3	56.9	17.5	96.1	-9.3	2.51	HL-93	1.94
1 4 22.30	422.3	48.9	15.0	88.6	-13.1	2.84	HL-93	2.19
1 5 27.87	422.3	40.5	12.5	81.3	-17.8	3.22	HL-93	2.49

5799 Int Left.res										
1	6	33.45	422.3	32.5	10.0	74.1	-23.2	3.67	HL-93	2.83
1	7	39.03	422.3	24.4	7.5	67.1	-28.9	4.20	HL-93	3.24
1	8	44.60	422.3	16.4	5.0	60.3	-34.9	4.84	HL-93	3.74
1	9	50.18	304.9	8.0	2.5	53.7	-41.0	4.02	HL-93	3.10
1	10	55.75	304.9	0.0	0.0	47.3	-47.3	4.78	HL-93	3.69
1	11	61.33	304.9	-8.0	-2.5	41.0	-53.7	4.02	HL-93	3.10
1	12	66.90	422.3	-16.4	-5.0	34.9	-60.3	4.84	HL-93	3.74
1	13	72.47	422.3	-24.4	-7.5	28.9	-67.1	4.20	HL-93	3.24
1	14	78.05	422.3	-32.5	-10.0	23.2	-74.1	3.67	HL-93	2.83
1	15	83.62	422.3	-40.5	-12.5	17.8	-81.3	3.22	HL-93	2.49
1	16	89.20	422.3	-48.9	-15.0	13.1	-88.6	2.84	HL-93	2.19
1	17	94.77	422.3	-56.9	-17.5	9.3	-96.1	2.51	HL-93	1.94
1	18	100.35	449.4	-64.9	-20.0	5.6	-103.7	2.42	HL-93	1.87
1	19	105.92	481.5	-72.9	-22.5	2.6	-111.6	2.38	HL-93	1.83
1	20	111.50	602.9	-81.0	-25.0	0.0	-119.6	2.89	HL-93	2.23

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TABLE 1.2.33. 3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
NO.	LENGTH (ft)	D FROM L	SUPT				
			(ft)				
1	111.50	5.57		STRENGTH L. S.	2.38	1.83	HL-93

NOTE: Control Rating Factor for Current Bridge:

STRENGTH L. S.

Operating= 2.377

Inventory= 1.834

TABLE 1.2.33. 3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
NO.	LENGTH (ft)	D FROM L	SUPT			
			(ft)			
1	111.50	5.57		STRENGTH L. S.	2.96	121.09

NOTE: control Rating Factor for Current Bridge:

STRENGTH L. S. with Live Load Factor =1.660 & ADTT = 1190

Legal Load Rating= 2.958

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TABLE 1.2.34. 1=RATING; LIVE LOAD DEFLECTION

\*\*\*\*\*

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE		LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
		AND----- DIST. FACTOR FOR LL DEFL.	----- LL DEFL.			
1	55.75	4	0.433	-0.47	HL-93	3.52
		5	0.542	-0.59		2.82

NOTE: [1] "-" indicates downward deflection



5799 Int Left.res

[2]  $RF * (\text{live load defl.}) = \text{span length}/800$ , therefore

$RF = L/(800 * LL \text{ Defl.})$ , if = 99, No rating

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TABLE 0.0.1.1 PROJECT DATA  
\*\*\*\*\*

DESCRIPTION DATE  
-----  
MaineDOT, Bridge No. 5799 I-395 Left Bridge Exterior Girder 6/4/2013

CONTRACT NUMBER STR NO STR UNIT DES CHK SPECS. USED  
-----  
CKE410A4 5799 Exterior IAS/KSW KSW/TDM

TABLE 0.0.1.2 GENERAL PROGRAM OPTIONS  
\*\*\*\*\*

OUTPUT LEVEL (0, 1)	SPAN INTERVAL (MAX=20)	CONSTRUCTION 1= COMPOSITE 2= NONCOMP.	ANALYSIS CODE CODE YEAR UNIT DESIGN ID 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 = 1  
0 - DEFAULT (2008)  
1 - "PRIOR TO 2007" WIDTH IS USED

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TABLE 0.0.3.1 STRUCTURAL DETAILS  
\*\*\*\*\*

BEAM POSITION NUMBER OF GIRDERS	1=INT. 2=EXT.	WIDTH BETWEEN CURBS OR BARRIER (ft)	OVERHANG WIDTH (ft)	EDGE OF SLAB TO CURB (ft)	HAUNCH DEPTH (in)	WIDTH (in)	COMPOSITE PERCENTAGE AT NEG MOM REGION (%)	STEEL LOAD DETAIL FACTOR >= 1.0
6	2	50.85	4.29	1.29	1.25	22.00	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
111.50	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
9.92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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TABLE 0.0.4.1 DEFINITION OF SECTIONS

SECTION NO.	ID.	STANDARD SECTN		PLATE GIRDER		ROLLED SECTIONS WITH COVER PLATES OR PLATE GIRDERS ... (in)		REINFORCED CONCRETE SECTION	
		NOMINAL DEPTH (in)	WEIGHT (lb/ft)	WEB DEPTH (in)	WEB THICK.	TOP PLATE WIDTH	BOT. PLATE THICK.	AREA (in**2)	I x (in**4)
1	PG			66.0	0.4375	14.00	0.7500	21.00	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

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TABLE 0.0.5.1 DEFINITION OF MEMBERS

MEMBER NUMBER (IN ORDER)	MEMB END SECT ID		MEMBER DESCRIPTN		PARAMETERS FOR NONPRISMATIC MEMB		YIELD STRESS (KSI)		
	LEFT	RIGHT	LNTH (ft)	-->TYPE<-- 0=PRISMT	S(0)	S(1)	WEB	TOP	BOT
1	1	1	111.50				36.	50.	50.

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

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

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

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

AASHTO LOADING	TANDEM LIVE LOAD	AASHTO ROAD TYPE		SIDEWALK
HL - 93	1=YES : 0=NO	1, 2, 3 OR 4	ADTT ADTSL	LIVE LOAD---
				(k/ft)
HL-93	1	2	1190	1012
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 1190

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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							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	0.81	2 2 2 2 2		
1 0.00 0	0.00	0.00	0.00	0.00	0 0 0 0 0	0.87	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	SLAB DESIGN DEPTH		POUR DAY	MODULAR RATIO		SLAB LOAD DATA		
		INITIAL (in)	FINAL (in)		N1=3n	N2=n	INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)
1	1 Deck+huanch	0.0	9.5	0	27.0	9.0	1.13	0.00	111.50

AASHTO Art. 10.38.1.3 or LRFD Art. 6.10.1.1b

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:

$f_c'$  = 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		UNIFORM LOAD DATA			CONCENTRATED LOAD DATA	
LOAD NO.	DESCRIPTION	INTENSITY (k/ft)	POSITION FROM (ft)	TO (ft)	INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	0 Wearing surf	0.321	0.00	111.50	0.00	0.00
2	1 Railing	0.071	0.00	111.50	0.00	0.00
3	1 Snow fence	0.004	0.00	111.50	0.00	0.00
4	1 Curb	0.053	0.00	111.50	0.00	0.00
5	2 Diaphragm L	0.000	0.00	0.00	0.19	23.75
6	2 Diaphragm L	0.000	0.00	0.00	0.19	47.75
7	2 Diaphragm L	0.000	0.00	0.00	0.19	63.75
8	2 Diaphragm L	0.000	0.00	0.00	0.19	87.75
9	2 Stiffeners	0.003	0.00	111.50	0.00	0.00

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

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

2 = Noncomposite Loads, (In LRFD, it is for DC1 load)  
where N = modulus ratio =  $E_s/E_c$

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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	AASHTO ART.	AASHTO ART.	AASHTO ART.	AASHTO ART.	REBAR BAR AREA	DI ST.	COMP.	COMP.	
PER	IN	NEGAT.	6. 10. 10. 4	6. 10. 10. 2	6. 10. 10. 2	6. 10. 10. 2	YIELD PER FOOT	FROM	STRENG.	ALLOW	
TRAN.	M.	REGION	-----	(kip / per	(kip / per	(kip / per	STRESS OF SLAB	TOP	AT 28	-ABLE	
SEC	(in)	0=NO	(kip / per	connector)	connector)	connector)	Fy	-----	DAYS	-----	
		1=YES	connect.)	Truck	Lane	Lane	(ksi)	(in**2)	(in)	(ksi)	
2	0.875	0	0.00	0.0	0.0	0.0	0.00	0.00	3.00	0.00	

NOTE: Qr = nominal resistance of the shear connector  
=  $(\phi)_{sc} \times Q_n$   
... 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
DISTANCE FROM (ft)	DISTANCE TO (ft)	Fy (ksi)	Fy (WEB) (ksi)	OF LATERAL BRACING Lb (ft)
0.00	23.75	36.0	36.0	23.75
23.75	47.75	36.0	36.0	24.00

47.75	63.75	36.0	36.0	16.00
63.75	87.75	36.0	36.0	24.00
87.75	111.50	36.0	36.0	23.75

-----  
NOTE: [1] default  $F_y$  = 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 0.15.1.2 TRANSVERSE STIFFENER DATA  
\*\*\*\*\*

LOCATION		B PARAMETER	YIELD STRESS	STIFFENER SPACING	DIMENSIONS, (in)	
DI ST. FROM	DI ST. TO				STIFF. WIDTH	STIFF. THICK
(ft)	(ft)	AASHTO 6.10.8.1	$F_y$ (ksi)	(ft-in)		
0.00	2.75	2.4	36.0	2- 9	5.00	0.3750
2.75	8.75	2.4	36.0	6- 0	5.00	0.3750
8.75	15.75	2.4	36.0	7- 0	5.00	0.3750
15.75	23.75	2.4	36.0	8- 0	7.00	0.3750
23.75	39.75	2.4	36.0	8- 0	5.00	0.3750
39.75	47.75	2.4	36.0	8- 0	7.00	0.3750
47.75	63.75	2.4	36.0	16- 0	7.00	0.3750
63.75	79.75	2.4	36.0	8- 0	5.00	0.3750
79.75	87.75	2.4	36.0	8- 0	7.00	0.3750
87.75	97.75	2.4	36.0	8- 0	5.00	0.3750
95.75	102.75	2.4	36.0	7- 0	5.00	0.3750
102.75	108.75	2.4	36.0	6- 0	5.00	0.3750
108.75	111.50	2.4	36.0	2- 9	5.00	0.3750

-----  
NOTE: B parameter .... AASHTO LRFD Art. 6.10.8.1.4

B = 1.0 ..... for stiffener pairs (default)

= 1.8 ..... for single angles

= 2.4 ..... for single plates

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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	1.133		0.2233	1.3563

SUPERIMPOSED DEAD LOADS  
\*\*\*\*\*

LOAD		INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	DW	0.32	0.00	111.50
1	DC2	0.071	0.000	111.500
2	DC2	0.004	0.000	111.500
3	DC2	0.053	0.000	111.500

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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	5134.6	190.2	770.2	149.4	919.59

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

[2] Superimposed dead load not included

TABLE 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH

\*\*\*\*\*

SPAN NO.	SPACING (ft)	EFF. WIDTH (in)
1	9.92	111.01

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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	0.807	0.807	0.958	0.958	FOR STRENGTH POSITIVE MOMENT
1	0.872				FOR STRENGTH POSITIVE SHEAR
	0.798				FOR FATIGUE POSITIVE MOMENT
	0.798				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	WEB DEPTH	LOCATION OF N.A. FROM BOT OF STEEL	ELASTIC SECTION MODULUS	
			$I_x$ (in**4)	$D$ (in)	$Y(BS)$ (in)	STEEL BOT. (in**3)	TOP. (in**3)
1	0	0.00	47544.4	66.0	26.14	1818.8	1135.8
1	1	5.57	47544.4	66.0	26.14	1818.8	1135.8
1	2	11.15	47544.4	66.0	26.14	1818.8	1135.8
1	3	16.72	47544.4	66.0	26.14	1818.8	1135.8
1	4	22.30	47544.4	66.0	26.14	1818.8	1135.8
1	5	27.87	47544.4	66.0	26.14	1818.8	1135.8
1	6	33.45	47544.4	66.0	26.14	1818.8	1135.8
1	7	39.03	47544.4	66.0	26.14	1818.8	1135.8
1	8	44.60	47544.4	66.0	26.14	1818.8	1135.8
1	9	50.18	47544.4	66.0	26.14	1818.8	1135.8
1	10	55.75	47544.4	66.0	26.14	1818.8	1135.8
1	11	61.33	47544.4	66.0	26.14	1818.8	1135.8
1	12	66.90	47544.4	66.0	26.14	1818.8	1135.8
1	13	72.47	47544.4	66.0	26.14	1818.8	1135.8
1	14	78.05	47544.4	66.0	26.14	1818.8	1135.8
1	15	83.62	47544.4	66.0	26.14	1818.8	1135.8
1	16	89.20	47544.4	66.0	26.14	1818.8	1135.8
1	17	94.77	47544.4	66.0	26.14	1818.8	1135.8
1	18	100.35	47544.4	66.0	26.14	1818.8	1135.8
1	19	105.92	47544.4	66.0	26.14	1818.8	1135.8
1	20	111.50	47544.4	66.0	26.14	1818.8	1135.8

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

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TABLE 1.2.4.2=COMPOSITE SECTION PROPERTIES FOR N = 27.00

\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	MOMENT OF INERTIA	Q/Ix	ELASTIC SECTION MODULUS, (in**3)		
			$I_x$ (in**4)	Q=1ST. MOMENT OF INERTIA (1/in)	STEEL BOT.	STEEL TOP.	CONCRETE(SLAB) TOP.
1	0	0.00	102549.1	0.011431	2338.0	4248.5	3004.0
1	1	5.57	102549.1	0.011431	2338.0	4248.5	3004.0
1	2	11.15	102549.1	0.011431	2338.0	4248.5	3004.0
1	3	16.72	102549.1	0.011431	2338.0	4248.5	3004.0
1	4	22.30	102549.1	0.011431	2338.0	4248.5	3004.0
1	5	27.87	102549.1	0.011431	2338.0	4248.5	3004.0
1	6	33.45	102549.1	0.011431	2338.0	4248.5	3004.0
1	7	39.03	102549.1	0.011431	2338.0	4248.5	3004.0
1	8	44.60	102549.1	0.011431	2338.0	4248.5	3004.0
1	9	50.18	102549.1	0.011431	2338.0	4248.5	3004.0
1	10	55.75	102549.1	0.011431	2338.0	4248.5	3004.0
1	11	61.33	102549.1	0.011431	2338.0	4248.5	3004.0
1	12	66.90	102549.1	0.011431	2338.0	4248.5	3004.0
1	13	72.47	102549.1	0.011431	2338.0	4248.5	3004.0
1	14	78.05	102549.1	0.011431	2338.0	4248.5	3004.0
1	15	83.62	102549.1	0.011431	2338.0	4248.5	3004.0
1	16	89.20	102549.1	0.011431	2338.0	4248.5	3004.0
1	17	94.77	102549.1	0.011431	2338.0	4248.5	3004.0
1	18	100.35	102549.1	0.011431	2338.0	4248.5	3004.0
1	19	105.92	102549.1	0.011431	2338.0	4248.5	3004.0
1	20	111.50	102549.1	0.011431	2338.0	4248.5	3004.0

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

SP	IN	D	FROM	MOMENT OF		Q/I x		ELASTIC SECTION MODULUS, (in**3)		
				INERTIA		Q=1ST. MOMENT		STEEL		CONCRETE (SLAB)
NO	NO	L	SUPT	I x		OF INERTIA				
			(ft)	(in**4)		(1/in)		BOT.	TOP.	TOP.
1	0	0.00		142039.5		0.014099		2515.9	12305.2	6593.3
1	1	5.57		142039.5		0.014099		2515.9	12305.2	6593.3
1	2	11.15		142039.5		0.014099		2515.9	12305.2	6593.3
1	3	16.72		142039.5		0.014099		2515.9	12305.2	6593.3
1	4	22.30		142039.5		0.014099		2515.9	12305.2	6593.3
1	5	27.87		142039.5		0.014099		2515.9	12305.2	6593.3
1	6	33.45		142039.5		0.014099		2515.9	12305.2	6593.3
1	7	39.03		142039.5		0.014099		2515.9	12305.2	6593.3
1	8	44.60		142039.5		0.014099		2515.9	12305.2	6593.3
1	9	50.18		142039.5		0.014099		2515.9	12305.2	6593.3
1	10	55.75		142039.5		0.014099		2515.9	12305.2	6593.3
1	11	61.33		142039.5		0.014099		2515.9	12305.2	6593.3
1	12	66.90		142039.5		0.014099		2515.9	12305.2	6593.3
1	13	72.47		142039.5		0.014099		2515.9	12305.2	6593.3
1	14	78.05		142039.5		0.014099		2515.9	12305.2	6593.3
1	15	83.62		142039.5		0.014099		2515.9	12305.2	6593.3
1	16	89.20		142039.5		0.014099		2515.9	12305.2	6593.3
1	17	94.77		142039.5		0.014099		2515.9	12305.2	6593.3
1	18	100.35		142039.5		0.014099		2515.9	12305.2	6593.3
1	19	105.92		142039.5		0.014099		2515.9	12305.2	6593.3
1	20	111.50		142039.5		0.014099		2515.9	12305.2	6593.3

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 positive & negative flexure.

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TABLE 1.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	DEAD LOAD		CONCENTRATED LOADS (k-ft)	UNI FORM LOADS (k-ft)	TOTAL (k-ft)	
	BEAM (k-ft)	SLAB (k-ft)			NONCOMPOSITE DEAD	LOAD
1 0 0.00	0.0	0.0	0.0	0.0	0.0	
1 1 5.57	65.9	334.5	2.2	0.9	403.5	
1 2 11.15	124.9	633.9	4.3	1.7	764.8	
1 3 16.72	177.0	898.0	6.5	2.4	1083.8	
1 4 22.30	222.1	1126.9	8.7	3.0	1360.6	
1 5 27.87	260.3	1320.5	10.1	3.5	1594.4	
1 6 33.45	291.5	1479.0	11.2	3.9	1785.6	
1 7 39.03	315.8	1602.3	12.2	4.2	1934.5	
1 8 44.60	333.1	1690.3	13.3	4.5	2041.2	
1 9 50.18	343.6	1743.1	13.9	4.6	2105.2	
1 10 55.75	347.0	1760.7	13.9	4.7	2126.3	
1 11 61.33	343.6	1743.1	13.9	4.6	2105.2	
1 12 66.90	333.1	1690.3	13.3	4.5	2041.2	
1 13 72.47	315.8	1602.3	12.2	4.2	1934.5	
1 14 78.05	291.5	1479.0	11.2	3.9	1785.6	
1 15 83.62	260.3	1320.5	10.1	3.5	1594.4	
1 16 89.20	222.1	1126.9	8.7	3.0	1360.6	
1 17 94.77	177.0	898.0	6.5	2.4	1083.8	
1 18 100.35	124.9	633.9	4.3	1.7	764.8	
1 19 105.92	65.9	334.5	2.2	0.9	403.5	
1 20 111.50	0.0	0.0	0.0	0.0	0.0	

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

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUPT (ft)	UNI FORM SUPERIMPOSED DEAD LOAD (k-ft)		CONCENTRATED LOADS (k-ft)		OTHER UNI FORM LOADS	TOTAL (k-ft)	
				DW	DC2	DW	DC2		-- COMPOSITE DEAD LOAD	
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
1	1	5.57	94.8	37.8	0.0	0.0	0.0	0.0	132.6	
1	2	11.15	179.6	71.6	0.0	0.0	0.0	0.0	251.2	
1	3	16.72	254.4	101.4	0.0	0.0	0.0	0.0	355.9	
1	4	22.30	319.3	127.3	0.0	0.0	0.0	0.0	446.6	
1	5	27.87	374.1	149.2	0.0	0.0	0.0	0.0	523.3	
1	6	33.45	419.0	167.1	0.0	0.0	0.0	0.0	586.1	
1	7	39.03	453.9	181.0	0.0	0.0	0.0	0.0	635.0	
1	8	44.60	478.9	191.0	0.0	0.0	0.0	0.0	669.8	
1	9	50.18	493.9	196.9	0.0	0.0	0.0	0.0	690.8	
1	10	55.75	498.8	198.9	0.0	0.0	0.0	0.0	697.8	
1	11	61.33	493.9	196.9	0.0	0.0	0.0	0.0	690.8	
1	12	66.90	478.9	191.0	0.0	0.0	0.0	0.0	669.8	
1	13	72.47	453.9	181.0	0.0	0.0	0.0	0.0	635.0	
1	14	78.05	419.0	167.1	0.0	0.0	0.0	0.0	586.1	
1	15	83.62	374.1	149.2	0.0	0.0	0.0	0.0	523.3	
1	16	89.20	319.3	127.3	0.0	0.0	0.0	0.0	446.6	
1	17	94.77	254.4	101.4	0.0	0.0	0.0	0.0	355.9	
1	18	100.35	179.6	71.6	0.0	0.0	0.0	0.0	251.2	
1	19	105.92	94.8	37.8	0.0	0.0	0.0	0.0	132.6	
1	20	111.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

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

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	SIDEWALK (MAXIMUM)		LL+I , (k-ft), LOAD TYPE= HL -93		MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
	POSITIVE (k-ft)	NEGATIVE (k-ft)						
1 0	0.00	0.0	0.0	0.0	0.0	HL-93	0.0	HL-93
1 1	5.57	0.0	0.0	525.7	525.7	HL-93	0.0	HL-93
1 2	11.15	0.0	0.0	992.3	992.3	HL-93	0.0	HL-93
1 3	16.72	0.0	0.0	1399.8	1399.8	HL-93	0.0	HL-93
1 4	22.30	0.0	0.0	1748.1	1748.1	HL-93	0.0	HL-93
1 5	27.87	0.0	0.0	2034.3	2034.3	HL-93	0.0	HL-93
1 6	33.45	0.0	0.0	2267.3	2267.3	HL-93	0.0	HL-93
1 7	39.03	0.0	0.0	2448.7	2448.7	HL-93	0.0	HL-93
1 8	44.60	0.0	0.0	2574.0	2574.0	HL-93	0.0	HL-93
1 9	50.18	0.0	0.0	2635.7	2635.7	HL-93	0.0	HL-93
1 10	55.75	0.0	0.0	2656.2	2656.2	HL-93	0.0	HL-93
1 11	61.33	0.0	0.0	2635.7	2635.7	HL-93	0.0	HL-93
1 12	66.90	0.0	0.0	2574.0	2574.0	HL-93	0.0	HL-93
1 13	72.47	0.0	0.0	2448.7	2448.7	HL-93	0.0	HL-93
1 14	78.05	0.0	0.0	2267.3	2267.3	HL-93	0.0	HL-93
1 15	83.62	0.0	0.0	2034.3	2034.3	HL-93	0.0	HL-93
1 16	89.20	0.0	0.0	1748.1	1748.1	HL-93	0.0	HL-93
1 17	94.77	0.0	0.0	1399.8	1399.8	HL-93	0.0	HL-93
1 18	100.35	0.0	0.0	992.3	992.3	HL-93	0.0	HL-93
1 19	105.92	0.0	0.0	525.7	525.7	HL-93	0.0	HL-93
1 20	111.50	0.0	0.0	0.0	0.0	HL-93	0.0	HL-93

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

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	TRUCK ONLY		
	POS	NEG	RANGE
1 0	0.00	0.	0.
1 1	5.57	290.	290.
1 2	11.15	543.	543.
1 3	16.72	756.	756.
1 4	22.30	938.	938.
1 5	27.87	1091.	1091.
1 6	33.45	1211.	1211.
1 7	39.03	1301.	1301.
1 8	44.60	1354.	1354.
1 9	50.18	1351.	1351.
1 10	55.75	1350.	1350.
1 11	61.33	1351.	1351.
1 12	66.90	1354.	1354.
1 13	72.47	1301.	1301.
1 14	78.05	1211.	1211.
1 15	83.62	1091.	1091.
1 16	89.20	938.	938.
1 17	94.77	756.	756.
1 18	100.35	543.	543.
1 19	105.92	290.	290.
1 20	111.50	0.	0.

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TABLE 1.2.5.4=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (UNFACTORED)

\*\*\*\*\*

## 5799 Ext Left.res

		DEAD LOAD		LL+I: N= 9.0		LOAD TYPE= HL - 93		TOTAL MAXIMUM	
SP NO	IN NO	D FROM L SUPT (ft)	NON COMP. N=Inf n. (k-ft)	COMP. N=27.0 (k-ft)	MAXI MUM POSI TIVE (k-ft)	GOVERN LOAD TYPE	MAXI MUM GOVERN NEGATIVE LOAD (k-ft)	POS I TIVE (k-ft)	NEGATI VE (k-ft)
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	0.0	0.0
1	1	5.57	403.5	132.6	525.7	HL-93	0.0	1061.8	536.1
1	2	11.15	764.8	251.2	992.3	HL-93	0.0	2008.3	1016.0
1	3	16.72	1083.8	355.9	1399.8	HL-93	0.0	2839.5	1439.7
1	4	22.30	1360.6	446.6	1748.1	HL-93	0.0	3555.3	1807.2
1	5	27.87	1594.4	523.3	2034.3	HL-93	0.0	4151.9	2117.7
1	6	33.45	1785.6	586.1	2267.3	HL-93	0.0	4639.0	2371.7
1	7	39.03	1934.5	635.0	2448.7	HL-93	0.0	5018.2	2569.5
1	8	44.60	2041.2	669.8	2574.0	HL-93	0.0	5285.1	2711.1
1	9	50.18	2105.2	690.8	2635.7	HL-93	0.0	5431.7	2796.0
1	10	55.75	2126.3	697.8	2656.2	HL-93	0.0	5480.3	2824.1
1	11	61.33	2105.2	690.8	2635.7	HL-93	0.0	5431.7	2796.0
1	12	66.90	2041.2	669.8	2574.0	HL-93	0.0	5285.1	2711.1
1	13	72.47	1934.5	635.0	2448.7	HL-93	0.0	5018.2	2569.5
1	14	78.05	1785.6	586.1	2267.3	HL-93	0.0	4639.0	2371.7
1	15	83.62	1594.4	523.3	2034.3	HL-93	0.0	4151.9	2117.7
1	16	89.20	1360.6	446.6	1748.1	HL-93	0.0	3555.3	1807.2
1	17	94.77	1083.8	355.9	1399.8	HL-93	0.0	2839.5	1439.7
1	18	100.35	764.8	251.2	992.3	HL-93	0.0	2008.3	1016.0
1	19	105.92	403.5	132.6	525.7	HL-93	0.0	1061.8	536.1
1	20	111.50	0.0	0.0	0.0	HL-93	0.0	0.0	0.0

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TABLE 1.2.5.5=MOMENT SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

\*\*\*\*\*

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0	0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1	5.57	1061.8	1219.5	1613.8	1403.6	804.2	217.4
1	2	11.15	2008.3	2306.0	3051.4	2654.5	1524.0	407.1
1	3	16.72	2839.5	3259.4	4312.8	3752.9	2159.6	566.9
1	4	22.30	3555.3	4079.7	5398.0	4698.7	2710.8	703.7
1	5	27.87	4151.9	4762.2	6300.6	5486.9	3176.5	818.2
1	6	33.45	4639.0	5319.2	7037.2	6130.2	3557.5	908.4
1	7	39.03	5018.2	5752.8	7610.6	6631.1	3854.2	975.9
1	8	44.60	5285.1	6057.3	8013.1	6983.5	4066.6	1015.8
1	9	50.18	5431.7	6222.4	8230.9	7176.6	4194.0	1013.4
1	10	55.75	5480.3	6277.2	8303.3	7240.8	4236.2	1012.6
1	11	61.33	5431.7	6222.4	8230.9	7176.6	4194.0	1013.4
1	12	66.90	5285.1	6057.3	8013.1	6983.5	4066.6	1015.8
1	13	72.47	5018.2	5752.8	7610.6	6631.1	3854.2	975.9
1	14	78.05	4639.0	5319.2	7037.2	6130.2	3557.5	908.4
1	15	83.62	4151.9	4762.2	6300.6	5486.9	3176.5	818.2
1	16	89.20	3555.3	4079.7	5398.0	4698.7	2710.8	703.7
1	17	94.77	2839.5	3259.4	4312.8	3752.9	2159.6	566.9
1	18	100.35	2008.3	2306.0	3051.4	2654.5	1524.0	407.1
1	19	105.92	1061.8	1219.5	1613.8	1403.6	804.2	217.4
1	20	111.50	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)

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\*\*\*\*\*

SPAN NO.	IN NO	DIST FROM LEFT SUPT (ft)	STEEL BEAM (kip)	CONC. SLAB (kip)	CONCENT. LOAD (kip)	OTHER UNIFORM LOAD (kip)	TOTAL DEAD LOAD (kip)	NONCOMPOSITE LOAD
1	0	0.00	12.4	63.2	0.4	0.2	76.2	
1	1	5.57	11.2	56.8	0.4	0.2	68.6	
1	2	11.15	10.0	50.5	0.4	0.1	61.0	
1	3	16.72	8.7	44.2	0.4	0.1	53.4	
1	4	22.30	7.5	37.9	0.4	0.1	45.9	
1	5	27.87	6.2	31.6	0.2	0.1	38.1	
1	6	33.45	5.0	25.3	0.2	0.1	30.5	
1	7	39.03	3.7	18.9	0.2	0.1	22.9	
1	8	44.60	2.5	12.6	0.2	0.0	15.4	
1	9	50.18	1.2	6.3	0.0	0.0	7.6	
1	10	55.75	0.0	0.0	0.0	0.0	0.0	
1	11	61.33	-1.2	-6.3	0.0	0.0	-7.6	
1	12	66.90	-2.5	-12.6	-0.2	0.0	-15.4	
1	13	72.47	-3.7	-18.9	-0.2	-0.1	-22.9	
1	14	78.05	-5.0	-25.3	-0.2	-0.1	-30.5	
1	15	83.62	-6.2	-31.6	-0.2	-0.1	-38.1	
1	16	89.20	-7.5	-37.9	-0.4	-0.1	-45.9	
1	17	94.77	-8.7	-44.2	-0.4	-0.1	-53.4	
1	18	100.35	-10.0	-50.5	-0.4	-0.1	-61.0	
1	19	105.92	-11.2	-56.8	-0.4	-0.2	-68.6	
1	20	111.50	-12.4	-63.2	-0.4	-0.2	-76.2	

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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)				TOTAL		TOTAL
			UNIFORM LOADS		CONCENTRATED LOADS		COMPOSITE	NONCOMPOSITE	DEAD LOADS
			DW	DC2	DW	DC2	(k)		(k)
1	0	0.00	17.9	7.1	0.0	0.0	25.0	76.2	101.2
1	1	5.57	16.1	6.4	0.0	0.0	22.5	68.6	91.1
1	2	11.15	14.3	5.7	0.0	0.0	20.0	61.0	81.0
1	3	16.72	12.5	5.0	0.0	0.0	17.5	53.4	71.0
1	4	22.30	10.7	4.3	0.0	0.0	15.0	45.9	60.9
1	5	27.87	8.9	3.6	0.0	0.0	12.5	38.1	50.6
1	6	33.45	7.2	2.9	0.0	0.0	10.0	30.5	40.5
1	7	39.03	5.4	2.1	0.0	0.0	7.5	22.9	30.4
1	8	44.60	3.6	1.4	0.0	0.0	5.0	15.4	20.4
1	9	50.18	1.8	0.7	0.0	0.0	2.5	7.6	10.1
1	10	55.75	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	11	61.33	-1.8	-0.7	0.0	0.0	-2.5	-7.6	-10.1
1	12	66.90	-3.6	-1.4	0.0	0.0	-5.0	-15.4	-20.4
1	13	72.47	-5.4	-2.1	0.0	0.0	-7.5	-22.9	-30.4
1	14	78.05	-7.2	-2.9	0.0	0.0	-10.0	-30.5	-40.5
1	15	83.62	-8.9	-3.6	0.0	0.0	-12.5	-38.1	-50.6
1	16	89.20	-10.7	-4.3	0.0	0.0	-15.0	-45.9	-60.9
1	17	94.77	-12.5	-5.0	0.0	0.0	-17.5	-53.4	-71.0
1	18	100.35	-14.3	-5.7	0.0	0.0	-20.0	-61.0	-81.0
1	19	105.92	-16.1	-6.4	0.0	0.0	-22.5	-68.6	-91.1
1	20	111.50	-17.9	-7.1	0.0	0.0	-25.0	-76.2	-101.2

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

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM)		LL+I, (kips), LOAD TYPE= HL -93			
			POSITIVE (kips)	NEGATIVE (kips)	MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	107.6	HL-93	0.0	HL-93
1	1	5.57	0.0	0.0	100.4	HL-93	-2.3	HL-93
1	2	11.15	0.0	0.0	93.4	HL-93	-5.1	HL-93
1	3	16.72	0.0	0.0	86.5	HL-93	-8.4	HL-93
1	4	22.30	0.0	0.0	79.7	HL-93	-11.8	HL-93
1	5	27.87	0.0	0.0	73.1	HL-93	-16.0	HL-93
1	6	33.45	0.0	0.0	66.7	HL-93	-20.9	HL-93
1	7	39.03	0.0	0.0	60.4	HL-93	-26.0	HL-93
1	8	44.60	0.0	0.0	54.3	HL-93	-31.4	HL-93
1	9	50.18	0.0	0.0	48.3	HL-93	-36.9	HL-93
1	10	55.75	0.0	0.0	42.5	HL-93	-42.5	HL-93
1	11	61.33	0.0	0.0	36.9	HL-93	-48.3	HL-93
1	12	66.90	0.0	0.0	31.4	HL-93	-54.3	HL-93
1	13	72.47	0.0	0.0	26.0	HL-93	-60.4	HL-93
1	14	78.05	0.0	0.0	20.9	HL-93	-66.7	HL-93
1	15	83.62	0.0	0.0	16.0	HL-93	-73.1	HL-93
1	16	89.20	0.0	0.0	11.8	HL-93	-79.7	HL-93
1	17	94.77	0.0	0.0	8.4	HL-93	-86.5	HL-93
1	18	100.35	0.0	0.0	5.1	HL-93	-93.4	HL-93
1	19	105.92	0.0	0.0	2.3	HL-93	-100.4	HL-93
1	20	111.50	0.0	0.0	0.0	HL-93	-107.6	HL-93

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

SP NO	IN NO	D FROM L SUPT (ft)	TRUCK ONLY		
			POS	NEG	RANGE
1	0	0.00	55.	0.	55.
1	1	5.57	52.	-1.	53.
1	2	11.15	49.	-3.	52.
1	3	16.72	45.	-5.	50.
1	4	22.30	42.	-6.	48.
1	5	27.87	39.	-8.	47.
1	6	33.45	35.	-10.	45.
1	7	39.03	32.	-13.	45.
1	8	44.60	29.	-16.	44.
1	9	50.18	26.	-19.	44.
1	10	55.75	22.	-22.	44.
1	11	61.33	19.	-26.	44.
1	12	66.90	16.	-29.	44.
1	13	72.47	13.	-32.	45.
1	14	78.05	10.	-35.	45.
1	15	83.62	8.	-39.	47.
1	16	89.20	6.	-42.	48.
1	17	94.77	5.	-45.	50.
1	18	100.35	3.	-49.	52.
1	19	105.92	1.	-52.	53.
1	20	111.50	0.	-55.	55.

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TABLE 1.2.6.3B=MINIMUM WELD SIZE  
\*\*\*\*\*

TOP FLANGE	BOTTOM FLANGE
------------	---------------

5799 Ext Left.res										
SP NO	IN NO	D L	FROM SUPT (ft)	Vr [1] (Kip)	Q/I [2] (1/in)	SHEAR FLOW [3]	WELD SIZE (in)	Q/I (1/in)	SHEAR FLOW	WELD SIZE (in)
1	0	0.00	55.3	.14957E-01	0.41	0.2500*	10237E-01	0.28	0.3125*	
1	1	5.57	53.4	.14957E-01	0.40	0.2500*	10237E-01	0.27	0.3125*	
1	2	11.15	51.6	.14957E-01	0.39	0.2500*	10237E-01	0.26	0.3125*	
1	3	16.72	50.1	.14957E-01	0.37	0.2500*	10237E-01	0.26	0.3125*	
1	4	22.30	48.5	.14957E-01	0.36	0.2500*	10237E-01	0.25	0.3125*	
1	5	27.87	46.5	.14957E-01	0.35	0.2500*	10237E-01	0.24	0.3125*	
1	6	33.45	45.2	.14957E-01	0.34	0.2500*	10237E-01	0.23	0.3125*	
1	7	39.03	44.7	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	8	44.60	44.5	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	9	50.18	44.5	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	10	55.75	44.5	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	11	61.33	44.5	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	12	66.90	44.5	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	13	72.47	44.7	.14957E-01	0.33	0.2500*	10237E-01	0.23	0.3125*	
1	14	78.05	45.2	.14957E-01	0.34	0.2500*	10237E-01	0.23	0.3125*	
1	15	83.62	46.5	.14957E-01	0.35	0.2500*	10237E-01	0.24	0.3125*	
1	16	89.20	48.5	.14957E-01	0.36	0.2500*	10237E-01	0.25	0.3125*	
1	17	94.77	50.1	.14957E-01	0.37	0.2500*	10237E-01	0.26	0.3125*	
1	18	100.35	51.6	.14957E-01	0.39	0.2500*	10237E-01	0.26	0.3125*	
1	19	105.92	53.4	.14957E-01	0.40	0.2500*	10237E-01	0.27	0.3125*	
1	20	111.50	55.3	.14957E-01	0.41	0.2500*	10237E-01	0.28	0.3125*	

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

[2] For non-composite construction:

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

For composite construction:

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

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

\* -- minimum weld size governs

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

SP NO	IN NO	D L	FROM SUPT (ft)	TOTAL DEAD LOADS (k)	TOTAL L+I , POSITIVE GOVERN. (k)	LOAD TYPE	NEGATIVE GOVERN. (k)	LOAD TYPE	MAX. SHEAR (kips)
									POS. NEG.
1	0	0.00	101.2	107.6	HL-93	0.0	HL-93	208.8	101.2
1	1	5.57	91.1	100.4	HL-93	-2.3	HL-93	191.5	88.8
1	2	11.15	81.0	93.4	HL-93	-5.1	HL-93	174.4	76.0
1	3	16.72	71.0	86.5	HL-93	-8.4	HL-93	157.4	62.6
1	4	22.30	60.9	79.7	HL-93	-11.8	HL-93	140.6	49.1
1	5	27.87	50.6	73.1	HL-93	-16.0	HL-93	123.7	34.6
1	6	33.45	40.5	66.7	HL-93	-20.9	HL-93	107.2	19.7
1	7	39.03	30.4	60.4	HL-93	-26.0	HL-93	90.9	4.4
1	8	44.60	20.4	54.3	HL-93	-31.4	HL-93	74.7	-11.0
1	9	50.18	10.1	48.3	HL-93	-36.9	HL-93	58.4	-26.8
1	10	55.75	0.0	42.5	HL-93	-42.5	HL-93	42.5	-42.5
1	11	61.33	-10.1	36.9	HL-93	-48.3	HL-93	26.8	-58.4
1	12	66.90	-20.4	31.4	HL-93	-54.3	HL-93	11.0	-74.7

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1	13	72.47	-30.4	26.0	HL-93	-60.4	HL-93	-4.4	-90.9
1	14	78.05	-40.5	20.9	HL-93	-66.7	HL-93	-19.7	-107.2
1	15	83.62	-50.6	16.0	HL-93	-73.1	HL-93	-34.6	-123.7
1	16	89.20	-60.9	11.8	HL-93	-79.7	HL-93	-49.1	-140.6
1	17	94.77	-71.0	8.4	HL-93	-86.5	HL-93	-62.6	-157.4
1	18	100.35	-81.0	5.1	HL-93	-93.4	HL-93	-76.0	-174.4
1	19	105.92	-91.1	2.3	HL-93	-100.4	HL-93	-88.8	-191.5
1	20	111.50	-101.2	0.0	HL-93	-107.6	HL-93	-101.2	-208.8

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TABLE 1.2.6.5= SHEAR SUMMARY FOR COMPOSITE CONSTRUCTION (LRFD)

\*\*\*\*\*

SP IN D	FROM	SERVICE I	SERVICE II	STRENGTH I	STRENGTH II	STRENGTH IV	FATIGUE
NO	NO	L SUPT					RANGE
		(ft)	(ki ps)	(ki ps)	(ki ps)	(ki ps)	(ki ps)
1	0	0.00	208.8	241.1	319.3	276.3	41.5
1	1	5.57	191.5	221.7	293.6	253.5	40.0
1	2	11.15	174.4	202.4	268.3	230.9	38.7
1	3	16.72	157.4	183.4	243.1	208.5	37.6
1	4	22.30	140.6	164.5	218.3	186.4	36.4
1	5	27.87	123.7	145.7	193.5	164.2	34.9
1	6	33.45	107.2	127.2	169.2	142.5	33.9
1	7	39.03	90.9	109.0	145.1	121.0	33.6
1	8	44.60	74.7	91.0	121.4	99.7	33.4
1	9	50.18	58.4	72.9	97.6	78.3	33.4
1	10	55.75	-42.5	-55.3	-74.4	-57.4	33.4
1	11	61.33	-58.4	-72.9	-97.6	-78.3	33.4
1	12	66.90	-74.7	-91.0	-121.4	-99.7	33.4
1	13	72.47	-90.9	-109.0	-145.1	-121.0	33.6
1	14	78.05	-107.2	-127.2	-169.2	-142.5	33.9
1	15	83.62	-123.7	-145.7	-193.5	-164.2	34.9
1	16	89.20	-140.6	-164.5	-218.3	-186.4	36.4
1	17	94.77	-157.4	-183.4	-243.1	-208.5	37.6
1	18	100.35	-174.4	-202.4	-268.3	-230.9	38.7
1	19	105.92	-191.5	-221.7	-293.6	-253.5	40.0
1	20	111.50	-208.8	-241.1	-319.3	-276.3	41.5

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TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

\*\*\*\*\*

SUPT	TOTAL	LL+I , (K)	LOAD TYPE =	HL - 93	TOTAL DL+LL+I (L R F D)
NO.	DEAD LOADS	MINIMUM	GOVERN.	MAXIMUM	MINIMUM
	(K)		LOAD TYPE	LOAD TYPE	MAXIMUM
1	101.20	0.00	HL-93	107.62	HL-93
				ST1	86.61
				ST2	86.61
				ST4	124.96
				SE1	101.20
				SE2	101.20
					319.31
					276.26
					151.80
					208.82
					241.11
2	101.20	0.00	HL-93	107.62	HL-93
				ST1	86.61
				ST2	86.61
				ST4	124.96
				SE1	101.20
				SE2	101.20
					319.31
					276.26
					151.80
					208.82
					241.11

NOTE: [1] " - " Indicates Uplift

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

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

*****									
SP NO	IN NO	D FROM L SUPT (ft)	NONCOMPOSITE DL		COMPOSITE DL		T O T A L		
			BEAM (in)	SLAB (in)	CONCENTRATED (in)	UNI FORM (in)	NONCOMPOSITE (in)	COMPOSITE (in)	= DL (in)
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	5.57	-0.0897	-0.4550	0.0000	-0.0836	-0.5494	-0.0836	-0.6330
1	2	11.15	-0.1768	-0.8971	0.0000	-0.1648	-1.0831	-0.1648	-1.2480
1	3	16.72	-0.2591	-1.3146	0.0000	-0.2415	-1.5873	-0.2415	-1.8288
1	4	22.30	-0.3345	-1.6972	0.0000	-0.3118	-2.0493	-0.3118	-2.3612
1	5	27.87	-0.4013	-2.0361	0.0000	-0.3741	-2.4585	-0.3741	-2.8326
1	6	33.45	-0.4580	-2.3236	0.0000	-0.4269	-2.8057	-0.4269	-3.2326
1	7	39.03	-0.5033	-2.5537	0.0000	-0.4692	-3.0835	-0.4692	-3.5527
1	8	44.60	-0.5364	-2.7214	0.0000	-0.5000	-3.2861	-0.5000	-3.7861
1	9	50.18	-0.5565	-2.8235	0.0000	-0.5188	-3.4093	-0.5188	-3.9281
1	10	55.75	-0.5632	-2.8577	0.0000	-0.5250	-3.4507	-0.5250	-3.9757
1	11	61.33	-0.5565	-2.8235	0.0000	-0.5188	-3.4093	-0.5188	-3.9281
1	12	66.90	-0.5364	-2.7214	0.0000	-0.5000	-3.2861	-0.5000	-3.7861
1	13	72.47	-0.5033	-2.5537	0.0000	-0.4692	-3.0835	-0.4692	-3.5527
1	14	78.05	-0.4580	-2.3236	0.0000	-0.4269	-2.8057	-0.4269	-3.2326
1	15	83.62	-0.4013	-2.0361	0.0000	-0.3741	-2.4585	-0.3741	-2.8326
1	16	89.20	-0.3345	-1.6972	0.0000	-0.3118	-2.0493	-0.3118	-2.3612
1	17	94.77	-0.2591	-1.3146	0.0000	-0.2415	-1.5873	-0.2415	-1.8288
1	18	100.35	-0.1768	-0.8971	0.0000	-0.1648	-1.0831	-0.1648	-1.2480
1	19	105.92	-0.0897	-0.4550	0.0000	-0.0836	-0.5494	-0.0836	-0.6330
1	20	111.50	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	IN	D FROM	NONCOMPOSITE		DEAD	LOADS		COMPOSITE DL		T O T A L	
			STEEL	CAMBER	SLAB	CAMBER	DEFL.	CAMBER	DEFL.	CAMBER	
NO	NO	L SUPT (ft)	DEFL.	SIZE	DEFL.	SIZE	SIZE	SIZE	SIZE	SIZE	
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--	
1	1	5.6	-0.090	0	1/8	-0.460	0	1/2	-0.084	0	11/16
1	2	11.1	-0.177	0	3/16	-0.906	0	15/16	-0.165	0	1/4
1	3	16.7	-0.259	0	5/16	-1.328	1	3/8	-0.242	0	7/8
1	4	22.3	-0.335	0	3/8	-1.715	1	3/4	-0.312	0	3/8
1	5	27.9	-0.401	0	7/16	-2.057	2	1/16	-0.374	0	7/8
1	6	33.5	-0.458	0	1/2	-2.348	2	3/8	-0.427	0	1/4
1	7	39.0	-0.503	0	9/16	-2.580	2	5/8	-0.469	0	9/16
1	8	44.6	-0.536	0	9/16	-2.750	2	3/4	-0.500	0	13/16
1	9	50.2	-0.556	0	9/16	-2.853	2	7/8	-0.519	0	15/16
1	10	55.8	-0.563	0	5/8	-2.887	2	15/16	-0.525	0	0/16
1	11	61.3	-0.556	0	9/16	-2.853	2	7/8	-0.519	0	15/16
1	12	66.9	-0.536	0	9/16	-2.750	2	3/4	-0.500	0	13/16
1	13	72.5	-0.503	0	9/16	-2.580	2	5/8	-0.469	0	9/16
1	14	78.0	-0.458	0	1/2	-2.348	2	3/8	-0.427	0	1/4
1	15	83.6	-0.401	0	7/16	-2.057	2	1/16	-0.374	0	7/8
1	16	89.2	-0.335	0	3/8	-1.715	1	3/4	-0.312	0	3/8

5799 Ext Left.res  
 1 17 94.8 -0.259 0 5/16 -1.328 1 3/8 -0.242 0 1/4 -1.829 1 7/8  
 1 18 100.3 -0.177 0 3/16 -0.906 0 15/16 -0.165 0 3/16 -1.248 1 1/4  
 1 19 105.9 -0.090 0 1/8 -0.460 0 1/2 -0.084 0 1/8 -0.633 0 11/16  
 1 20 111.5 0.000 -- 0.000 -- 0.000 -- 0.000 --

-----  
 NOTE: for camber, please refer to AASHTO Art.10.14 or LRFD Art. 6.7.2  
 1

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

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	55.75	4 0.433	-0.482 MAX 0.019 MIN -0.234 MAX LANE 0.019 MIN LANE	HL-93	1.67	0.00194
		5 0.542	-0.603 MAX 0.023 MIN -0.293 MAX LANE 0.023 MIN LANE	HL-93	1.67	0.00194
		4 0.433	-0.442 MAX FA 0.000 MIN FA			

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

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

$$DF = (NL/Ng) * (RF)_{lane} \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 averaging.

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TABLE 1.2.9.1=NONCOMPOSITE DEAD LOAD STRESSES FOR N=INFINITY (UNFACTORED)  
 \*\*\*\*\*

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

## 5799 Ext Left.res

1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	-0.70	0.44	-3.57	2.23	-4.26	2.66
1	2	11.15	-1.32	0.82	-6.76	4.22	-8.08	5.05
1	3	16.72	-1.87	1.17	-9.58	5.98	-11.45	7.15
1	4	22.30	-2.35	1.47	-12.03	7.51	-14.38	8.98
1	5	27.87	-2.75	1.72	-14.10	8.80	-16.84	10.52
1	6	33.45	-3.08	1.92	-15.79	9.86	-18.87	11.78
1	7	39.03	-3.34	2.08	-17.10	10.68	-20.44	12.76
1	8	44.60	-3.52	2.20	-18.05	11.27	-21.57	13.47
1	9	50.18	-3.63	2.27	-18.61	11.62	-22.24	13.89
1	10	55.75	-3.67	2.29	-18.80	11.74	-22.47	14.03
1	11	61.33	-3.63	2.27	-18.61	11.62	-22.24	13.89
1	12	66.90	-3.52	2.20	-18.05	11.27	-21.57	13.47
1	13	72.47	-3.34	2.08	-17.10	10.68	-20.44	12.76
1	14	78.05	-3.08	1.92	-15.79	9.86	-18.87	11.78
1	15	83.62	-2.75	1.72	-14.10	8.80	-16.84	10.52
1	16	89.20	-2.35	1.47	-12.03	7.51	-14.38	8.98
1	17	94.77	-1.87	1.17	-9.58	5.98	-11.45	7.15
1	18	100.35	-1.32	0.82	-6.76	4.22	-8.08	5.05
1	19	105.92	-0.70	0.44	-3.57	2.23	-4.26	2.66
1	20	111.50	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D FROM L SUPT (ft)	SUPERIMPOSED DEAD LOAD, (ksi)			TOTAL DEAD LOAD, (ksi)		
			CONCRETE	STEEL BEAM		CONCRETE	STEEL BEAM	
				TOP	BOT		TOP	BOT
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	-0.02	-0.37	0.68	-0.02	-4.64	3.34
1	2	11.15	-0.04	-0.71	1.29	-0.04	-8.79	6.34
1	3	16.72	-0.05	-1.01	1.83	-0.05	-12.46	8.98
1	4	22.30	-0.07	-1.26	2.29	-0.07	-15.64	11.27
1	5	27.87	-0.08	-1.48	2.69	-0.08	-18.32	13.21
1	6	33.45	-0.09	-1.66	3.01	-0.09	-20.52	14.79
1	7	39.03	-0.09	-1.79	3.26	-0.09	-22.23	16.02
1	8	44.60	-0.10	-1.89	3.44	-0.10	-23.46	16.91
1	9	50.18	-0.10	-1.95	3.55	-0.10	-24.19	17.44
1	10	55.75	-0.10	-1.97	3.58	-0.10	-24.44	17.61
1	11	61.33	-0.10	-1.95	3.55	-0.10	-24.19	17.44
1	12	66.90	-0.10	-1.89	3.44	-0.10	-23.46	16.91
1	13	72.47	-0.09	-1.79	3.26	-0.09	-22.23	16.02
1	14	78.05	-0.09	-1.66	3.01	-0.09	-20.52	14.79
1	15	83.62	-0.08	-1.48	2.69	-0.08	-18.32	13.21
1	16	89.20	-0.07	-1.26	2.29	-0.07	-15.64	11.27
1	17	94.77	-0.05	-1.01	1.83	-0.05	-12.46	8.98
1	18	100.35	-0.04	-0.71	1.29	-0.04	-8.79	6.34
1	19	105.92	-0.02	-0.37	0.68	-0.02	-4.64	3.34
1	20	111.50	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D FROM L SUPT	MAXIMUM POSITIVE, (MPa)		MAXIMUM NEGATIVE, (ksi)	
			CONCR.	STEEL	CONCR.	STEEL

	(ft)	TOP	TOP	BOT	5799 Ext Left.res TOP TOP BOT
1 0	0.00	0.00	0.00	0.00	0.00
1 1	5.57	-0.11	-0.51	2.51	0.00
1 2	11.15	-0.20	-0.97	4.73	0.00
1 3	16.72	-0.28	-1.37	6.68	0.00
1 4	22.30	-0.35	-1.70	8.34	0.00
1 5	27.87	-0.41	-1.98	9.70	0.00
1 6	33.45	-0.46	-2.21	10.81	0.00
1 7	39.03	-0.50	-2.39	11.68	0.00
1 8	44.60	-0.52	-2.51	12.28	0.00
1 9	50.18	-0.53	-2.57	12.57	0.00
1 10	55.75	-0.54	-2.59	12.67	0.00
1 11	61.33	-0.53	-2.57	12.57	0.00
1 12	66.90	-0.52	-2.51	12.28	0.00
1 13	72.47	-0.50	-2.39	11.68	0.00
1 14	78.05	-0.46	-2.21	10.81	0.00
1 15	83.62	-0.41	-1.98	9.70	0.00
1 16	89.20	-0.35	-1.70	8.34	0.00
1 17	94.77	-0.28	-1.37	6.68	0.00
1 18	100.35	-0.20	-0.97	4.73	0.00
1 19	105.92	-0.11	-0.51	2.51	0.00
1 20	111.50	0.00	0.00	0.00	0.00

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

SP IN D FROM NO NO L SUPT (ft)	TOP OF TOP FLANGE ----- TRUCK ONLY	GOVERNING STRESS LOAD RANGE TP TYPE	BOT OF BOT FLANGE ----- TRUCK ONLY	GOVERNING STRESS LOAD RANGE TP TYPE
1 0	0.0	0.0	0.0	0.0 T HL
1 1	5.57	0.3	0.0	0.3 C HL
1 2	11.15	0.5	0.0	0.5 C HL
1 3	16.72	0.7	0.0	0.7 C HL
1 4	22.30	0.9	0.0	0.9 C HL
1 5	27.87	1.1	0.0	1.1 C HL
1 6	33.45	1.2	0.0	1.2 C HL
1 7	39.03	1.3	0.0	1.3 C HL
1 8	44.60	1.3	0.0	1.3 C HL
1 9	50.18	1.3	0.0	1.3 C HL
1 10	55.75	1.3	0.0	1.3 C HL
1 11	61.33	1.3	0.0	1.3 C HL
1 12	66.90	1.3	0.0	1.3 C HL
1 13	72.47	1.3	0.0	1.3 C HL
1 14	78.05	1.2	0.0	1.2 C HL
1 15	83.62	1.1	0.0	1.1 C HL
1 16	89.20	0.9	0.0	0.9 C HL
1 17	94.77	0.7	0.0	0.7 C HL
1 18	100.35	0.5	0.0	0.5 C HL
1 19	105.92	0.3	0.0	0.3 C HL
1 20	111.50	0.0	0.0	0.0 T HL

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

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

YIELD TOTAL POSITIVE, (ksi) TOTAL NEGATIVE, (ksi)

5799 Ext Left.res										
			STRESS							
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.13	-5.15	5.85	-0.02	-4.64	3.34
1	2	11.15	50.	50.	-0.24	-9.76	11.07	-0.04	-8.79	6.34
1	3	16.72	50.	50.	-0.34	-13.82	15.65	-0.05	-12.46	8.98
1	4	22.30	50.	50.	-0.42	-17.34	19.61	-0.07	-15.64	11.27
1	5	27.87	50.	50.	-0.49	-20.31	22.91	-0.08	-18.32	13.21
1	6	33.45	50.	50.	-0.55	-22.73	25.60	-0.09	-20.52	14.79
1	7	39.03	50.	50.	-0.59	-24.62	27.70	-0.09	-22.23	16.02
1	8	44.60	50.	50.	-0.62	-25.97	29.18	-0.10	-23.46	16.91
1	9	50.18	50.	50.	-0.64	-26.76	30.01	-0.10	-24.19	17.44
1	10	55.75	50.	50.	-0.64	-27.03	30.28	-0.10	-24.44	17.61
1	11	61.33	50.	50.	-0.64	-26.76	30.01	-0.10	-24.19	17.44
1	12	66.90	50.	50.	-0.62	-25.97	29.18	-0.10	-23.46	16.91
1	13	72.47	50.	50.	-0.59	-24.62	27.70	-0.09	-22.23	16.02
1	14	78.05	50.	50.	-0.55	-22.73	25.60	-0.09	-20.52	14.79
1	15	83.62	50.	50.	-0.49	-20.31	22.91	-0.08	-18.32	13.21
1	16	89.20	50.	50.	-0.42	-17.34	19.61	-0.07	-15.64	11.27
1	17	94.77	50.	50.	-0.34	-13.82	15.65	-0.05	-12.46	8.98
1	18	100.35	50.	50.	-0.24	-9.76	11.07	-0.04	-8.79	6.34
1	19	105.92	50.	50.	-0.13	-5.15	5.85	-0.02	-4.64	3.34
1	20	111.50	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00

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

			YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.16	-5.30	6.60	-0.02	-4.64	3.34
1	2	11.15	50.	50.	-0.30	-10.05	12.49	-0.04	-8.79	6.34
1	3	16.72	50.	50.	-0.42	-14.23	17.66	-0.05	-12.46	8.98
1	4	22.30	50.	50.	-0.53	-17.85	22.11	-0.07	-15.64	11.27
1	5	27.87	50.	50.	-0.61	-20.90	25.82	-0.08	-18.32	13.21
1	6	33.45	50.	50.	-0.68	-23.39	28.85	-0.09	-20.52	14.79
1	7	39.03	50.	50.	-0.74	-25.34	31.21	-0.09	-22.23	16.02
1	8	44.60	50.	50.	-0.78	-26.72	32.87	-0.10	-23.46	16.91
1	9	50.18	50.	50.	-0.80	-27.53	33.78	-0.10	-24.19	17.44
1	10	55.75	50.	50.	-0.80	-27.80	34.08	-0.10	-24.44	17.61
1	11	61.33	50.	50.	-0.80	-27.53	33.78	-0.10	-24.19	17.44
1	12	66.90	50.	50.	-0.78	-26.72	32.87	-0.10	-23.46	16.91
1	13	72.47	50.	50.	-0.74	-25.34	31.21	-0.09	-22.23	16.02
1	14	78.05	50.	50.	-0.68	-23.39	28.85	-0.09	-20.52	14.79
1	15	83.62	50.	50.	-0.61	-20.90	25.82	-0.08	-18.32	13.21
1	16	89.20	50.	50.	-0.53	-17.85	22.11	-0.07	-15.64	11.27
1	17	94.77	50.	50.	-0.42	-14.23	17.66	-0.05	-12.46	8.98
1	18	100.35	50.	50.	-0.30	-10.05	12.49	-0.04	-8.79	6.34
1	19	105.92	50.	50.	-0.16	-5.30	6.60	-0.02	-4.64	3.34
1	20	111.50	50.	50.	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  
\*\*\*\*\*

			YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
SP NO	IN NO	D FROM L SUPT (ft)	Fy (ksi)	CONCR. TOP	STEEL TOP	BEAM BOT	CONCR. TOP	STEEL TOP	BEAM BOT	



NO	NO	L	SUPT	Fy	(ksi)	TOP	5799 Ext Left.res		TOP	TOP	BOT
							TOP	BOT			
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.21	-6.76	8.69	-0.03	-5.86	4.30	
1	2	11.15	50.	50.	-0.40	-12.81	16.43	-0.05	-11.11	8.15	
1	3	16.72	50.	50.	-0.57	-18.14	23.23	-0.08	-15.75	11.55	
1	4	22.30	50.	50.	-0.71	-22.75	29.09	-0.09	-19.77	14.50	
1	5	27.87	50.	50.	-0.83	-26.64	33.97	-0.11	-23.17	16.99	
1	6	33.45	50.	50.	-0.93	-29.82	37.95	-0.12	-25.95	19.02	
1	7	39.03	50.	50.	-1.00	-32.29	41.05	-0.13	-28.11	20.61	
1	8	44.60	50.	50.	-1.05	-34.05	43.23	-0.14	-29.66	21.75	
1	9	50.18	50.	50.	-1.08	-35.09	44.43	-0.15	-30.59	22.43	
1	10	55.75	50.	50.	-1.09	-35.43	44.82	-0.15	-30.90	22.65	
1	11	61.33	50.	50.	-1.08	-35.09	44.43	-0.15	-30.59	22.43	
1	12	66.90	50.	50.	-1.05	-34.05	43.23	-0.14	-29.66	21.75	
1	13	72.47	50.	50.	-1.00	-32.29	41.05	-0.13	-28.11	20.61	
1	14	78.05	50.	50.	-0.93	-29.82	37.95	-0.12	-25.95	19.02	
1	15	83.62	50.	50.	-0.83	-26.64	33.97	-0.11	-23.17	16.99	
1	16	89.20	50.	50.	-0.71	-22.75	29.09	-0.09	-19.77	14.50	
1	17	94.77	50.	50.	-0.57	-18.14	23.23	-0.08	-15.75	11.55	
1	18	100.35	50.	50.	-0.40	-12.81	16.43	-0.05	-11.11	8.15	
1	19	105.92	50.	50.	-0.21	-6.76	8.69	-0.03	-5.86	4.30	
1	20	111.50	50.	50.	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	IN	D	FROM	YIELD	STRESS	TOTAL POSITIVE , (ksi)		TOTAL NEGATIVE , (ksi)		BOT
						CONCR.	STEEL BEAM	CONCR.	STEEL BEAM	
NO	NO	L	SUPT	Fy	(ksi)	TOP	TOP BOT	TOP	TOP BOT	
		(ft)		TOP	BOT					
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.20	-5.00	7.28	-0.01	-4.11	2.89
1	2	11.15	50.	50.	-0.38	-9.48	13.75	-0.03	-7.78	5.47
1	3	16.72	50.	50.	-0.53	-13.42	19.44	-0.04	-11.03	7.75
1	4	22.30	50.	50.	-0.67	-16.83	24.32	-0.05	-13.85	9.73
1	5	27.87	50.	50.	-0.78	-19.70	28.38	-0.06	-16.23	11.40
1	6	33.45	50.	50.	-0.86	-22.04	31.70	-0.06	-18.17	12.77
1	7	39.03	50.	50.	-0.93	-23.87	34.28	-0.07	-19.69	13.84
1	8	44.60	50.	50.	-0.98	-25.17	36.09	-0.07	-20.77	14.60
1	9	50.18	50.	50.	-1.01	-25.92	37.06	-0.07	-21.43	15.06
1	10	55.75	50.	50.	-1.01	-26.17	37.38	-0.07	-21.64	15.21
1	11	61.33	50.	50.	-1.01	-25.92	37.06	-0.07	-21.43	15.06
1	12	66.90	50.	50.	-0.98	-25.17	36.09	-0.07	-20.77	14.60
1	13	72.47	50.	50.	-0.93	-23.87	34.28	-0.07	-19.69	13.84
1	14	78.05	50.	50.	-0.86	-22.04	31.70	-0.06	-18.17	12.77
1	15	83.62	50.	50.	-0.78	-19.70	28.38	-0.06	-16.23	11.40
1	16	89.20	50.	50.	-0.67	-16.83	24.32	-0.05	-13.85	9.73
1	17	94.77	50.	50.	-0.53	-13.42	19.44	-0.04	-11.03	7.75
1	18	100.35	50.	50.	-0.38	-9.48	13.75	-0.03	-7.78	5.47
1	19	105.92	50.	50.	-0.20	-5.00	7.28	-0.01	-4.11	2.89
1	20	111.50	50.	50.	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	IN	D	FROM	YIELD	STRESS	TOTAL POSITIVE , (ksi)		TOTAL NEGATIVE , (ksi)		BOT
						CONCR.	STEEL BEAM	CONCR.	STEEL BEAM	
NO	NO	L	SUPT	Fy	(ksi)	TOP	TOP BOT	TOP	TOP BOT	
		(ft)		TOP	BOT					

## 5799 Ext Left.res

1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.17	-6.56	7.69	-0.03	-5.86	4.30
1	2	11.15	50.	50.	-0.32	-12.42	14.54	-0.05	-11.11	8.15
1	3	16.72	50.	50.	-0.46	-17.59	20.56	-0.08	-15.75	11.55
1	4	22.30	50.	50.	-0.57	-22.07	25.75	-0.09	-19.77	14.50
1	5	27.87	50.	50.	-0.67	-25.85	30.09	-0.11	-23.17	16.99
1	6	33.45	50.	50.	-0.74	-28.93	33.62	-0.12	-25.95	19.02
1	7	39.03	50.	50.	-0.80	-31.33	36.38	-0.13	-28.11	20.61
1	8	44.60	50.	50.	-0.84	-33.05	38.32	-0.14	-29.66	21.75
1	9	50.18	50.	50.	-0.87	-34.06	39.40	-0.15	-30.59	22.43
1	10	55.75	50.	50.	-0.87	-34.39	39.76	-0.15	-30.90	22.65
1	11	61.33	50.	50.	-0.87	-34.06	39.40	-0.15	-30.59	22.43
1	12	66.90	50.	50.	-0.84	-33.05	38.32	-0.14	-29.66	21.75
1	13	72.47	50.	50.	-0.80	-31.33	36.38	-0.13	-28.11	20.61
1	14	78.05	50.	50.	-0.74	-28.93	33.62	-0.12	-25.95	19.02
1	15	83.62	50.	50.	-0.67	-25.85	30.09	-0.11	-23.17	16.99
1	16	89.20	50.	50.	-0.57	-22.07	25.75	-0.09	-19.77	14.50
1	17	94.77	50.	50.	-0.46	-17.59	20.56	-0.08	-15.75	11.55
1	18	100.35	50.	50.	-0.32	-12.42	14.54	-0.05	-11.11	8.15
1	19	105.92	50.	50.	-0.17	-6.56	7.69	-0.03	-5.86	4.30
1	20	111.50	50.	50.	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 NO	FROM L SUPT (ft)	YIELD STRESS		TOTAL POSITIVE , (ksi)			TOTAL NEGATIVE , (ksi)		
				Fy TOP	(ksi) BOT	CONCR. TOP	STEEL BEAM		CONCR. TOP	STEEL BEAM	
							TOP	BOT		TOP	BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	5.57	50.	50.	-0.16	-4.80	6.27	-0.01	-4.11	2.89	
1	2	11.15	50.	50.	-0.30	-9.09	11.86	-0.03	-7.78	5.47	
1	3	16.72	50.	50.	-0.42	-12.87	16.77	-0.04	-11.03	7.75	
1	4	22.30	50.	50.	-0.52	-16.15	20.99	-0.05	-13.85	9.73	
1	5	27.87	50.	50.	-0.61	-18.90	24.50	-0.06	-16.23	11.40	
1	6	33.45	50.	50.	-0.68	-21.16	27.37	-0.06	-18.17	12.77	
1	7	39.03	50.	50.	-0.74	-22.91	29.61	-0.07	-19.69	13.84	
1	8	44.60	50.	50.	-0.77	-24.16	31.17	-0.07	-20.77	14.60	
1	9	50.18	50.	50.	-0.79	-24.90	32.03	-0.07	-21.43	15.06	
1	10	55.75	50.	50.	-0.80	-25.14	32.31	-0.07	-21.64	15.21	
1	11	61.33	50.	50.	-0.79	-24.90	32.03	-0.07	-21.43	15.06	
1	12	66.90	50.	50.	-0.77	-24.16	31.17	-0.07	-20.77	14.60	
1	13	72.47	50.	50.	-0.74	-22.91	29.61	-0.07	-19.69	13.84	
1	14	78.05	50.	50.	-0.68	-21.16	27.37	-0.06	-18.17	12.77	
1	15	83.62	50.	50.	-0.61	-18.90	24.50	-0.06	-16.23	11.40	
1	16	89.20	50.	50.	-0.52	-16.15	20.99	-0.05	-13.85	9.73	
1	17	94.77	50.	50.	-0.42	-12.87	16.77	-0.04	-11.03	7.75	
1	18	100.35	50.	50.	-0.30	-9.09	11.86	-0.03	-7.78	5.47	
1	19	105.92	50.	50.	-0.16	-4.80	6.27	-0.01	-4.11	2.89	
1	20	111.50	50.	50.	0.00	0.00	0.00	0.00	0.00	0.00	

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

MEMBER		STEEL TYPE	TOP FLANGE		WEB		BOT FLANGE		COVER PLATE -- (in)			
NO	LENGTH (ft)		WIDTH (in)	THICK (in)	DEPTH (in)	THICK (in)	WIDTH (in)	THICK (in)	TOP		BOT	
									WIDTH	THICK	WIDTH	THICK
1	111.50	PG	14.0	0.7500	66.0	0.4375	21.0	1.2500				

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) = 111.5

MEMBER NO	SLAB THICK. (ft)	GIRDER DEPTH (in)	[2]=OVERALL D.		[2]/(12*[1])		LIMITATION 1/25 1/30	STATUS
			[3]= GIRDER D.	(in)	[3]/(12*[1])	<=>		
1	111.50	9.50	68.00	[2]= 78.00	0.05830	>	0.04000	OK
				[3]= 68.00	0.05082	>	0.03300	OK

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

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

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

[3] -- same as NOTE [2]

\*\* -- cover plates not taken into account

\*\* -- these criteria are related to the structural stability during construction and the limitation for the live load deflection.

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TABLE 1.2.22.5 =DEPTH/THICKNESS RATIOS (N = n)

\*\*\*\*\*

SP IN D FROM	Lo Co	Web	D/tw		2Dcp/tw		2Dc/tw		Web
NO NO L SUPT	mp	depth	thick	[2]	[3]	[4]	[5]	[6]	Cat
	[1]								
1 0	0.00	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 1	5.57	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 2	11.15	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 3	16.72	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 4	22.30	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 5	27.87	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 6	33.45	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 7	39.03	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 8	44.60	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 9	50.18	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 10	55.75	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 11	61.33	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 12	66.90	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 13	72.47	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 14	78.05	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 15	83.62	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 16	89.20	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 17	94.77	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 18	100.35	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 19	105.92	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2
1 20	111.50	0	1 66.00	0.438	150.9	150.0	0.00	90.55	2

## 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:  $3.76 \cdot \sqrt{E/Fyc}$  (Eq. 6.10.6.2.2-1)  
 [5] Use Article D6.3.1 to calculate Dc and 2\*Dc/tw  
 [6] 2Dc/tw limit:  $5.7 \cdot \sqrt{E/Fyc}$  (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	D	FROM	Lo	Co	Web	2Dc/tw		Web
NO	NO	L	SUPT	ng	mp	depth	thick	[2]	Category
				[1]				[3]	
1	0	0.00	0	1	66.00	0.438	187.93	137.27	3
1	1	5.57	0	1	66.00	0.438	187.93	137.27	3
1	2	11.15	0	1	66.00	0.438	187.93	137.27	3
1	3	16.72	0	1	66.00	0.438	187.93	137.27	3
1	4	22.30	0	1	66.00	0.438	187.93	137.27	3
1	5	27.87	0	1	66.00	0.438	187.93	137.27	3
1	6	33.45	0	1	66.00	0.438	187.93	137.27	3
1	7	39.03	0	1	66.00	0.438	187.93	137.27	3
1	8	44.60	0	1	66.00	0.438	187.93	137.27	3
1	9	50.18	0	1	66.00	0.438	187.93	137.27	3
1	10	55.75	0	1	66.00	0.438	187.93	137.27	3
1	11	61.33	0	1	66.00	0.438	187.93	137.27	3
1	12	66.90	0	1	66.00	0.438	187.93	137.27	3
1	13	72.47	0	1	66.00	0.438	187.93	137.27	3
1	14	78.05	0	1	66.00	0.438	187.93	137.27	3
1	15	83.62	0	1	66.00	0.438	187.93	137.27	3
1	16	89.20	0	1	66.00	0.438	187.93	137.27	3
1	17	94.77	0	1	66.00	0.438	187.93	137.27	3
1	18	100.35	0	1	66.00	0.438	187.93	137.27	3
1	19	105.92	0	1	66.00	0.438	187.93	137.27	3
1	20	111.50	0	1	66.00	0.438	187.93	137.27	3

## NOTE:

- [1] 0 - No long. stiffeners  
 1 - long. stiffeners  
 [2] For non-composite sections, calculate Dc and 2\*Dc/tw  
 [3] 2Dc/tw limit:  $5.7 \cdot \sqrt{E/Fyc}$  (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	bf	2tf	[1]	bf	[2]	tf	[3]	Iyc/Iyt	FLAG
NO	NO	L	SUPT	(ft)			(in)	(in)	(in)	(in)		
1	0	0.00	9.33	12.	14.0	11.0	0.750	0.481				
			8.40	12.	21.0	11.0	1.250	0.481	0.18	0		

5799 Ext Left.res									
1	1	5.57	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	2	11.15	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	3	16.72	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	4	22.30	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	5	27.87	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	6	33.45	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	7	39.03	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	8	44.60	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	9	50.18	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	10	55.75	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	11	61.33	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	12	66.90	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	13	72.47	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	14	78.05	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	15	83.62	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	16	89.20	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	17	94.77	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	18	100.35	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	19	105.92	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	0
1	20	111.50	9.33	12.14.0	11.0	0.750	0.481		
			8.40	12.21.0	11.0	1.250	0.481	0.18	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 SUPT TO	f <sub>0</sub> (ksi)	f <sub>2</sub> (ksi)	f <sub>mid</sub> (ksi)	f <sub>1</sub> (ksi)	C <sub>b</sub>
1	0.0	23.7	0.000	22.527	12.778	3.030	1.614
			0.000	0.000	0.000	0.000	1.000
2	23.7	47.7	22.527	32.922	29.272	25.621	1.115
			0.000	0.000	0.000	0.000	1.000
3	47.7	63.7	32.922	32.922	33.698	34.474	1.000
			0.000	0.000	0.000	0.000	1.000
4	63.7	87.7	22.527	32.922	29.272	25.621	1.115
			0.000	0.000	0.000	0.000	1.000
5	87.7	111.5	0.000	22.527	12.778	3.030	1.614
			0.000	0.000	0.000	0.000	1.000

Note: The 1st line is for DL case and the 2nd line is for LL case  
 f<sub>0</sub>, f<sub>2</sub>, f<sub>mid</sub>, f<sub>1</sub>, and C<sub>b</sub> 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

5799 Ext Left.res  
and larger lateral moment of inertia of the flange  
is smaller than 50%, then Cb=1.0.

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES

\*\*\*\*\*

SP	IN	D	FROM	Co	FLB			LTB			GOV
NO	NO	L	SUPT	mp	LMDAf	LMDApf	LMDArf	Lb	Lp	Lr	CAT
(ft)								(ft)	(ft)	(ft)	
1	0	0.00	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	1	5.57	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	2	11.15	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	3	16.72	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	4	22.30	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	5	27.87	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	6	33.45	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	7	39.03	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	8	44.60	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	9	50.18	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	10	55.75	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	11	61.33	1	9.33	9.15	16.12	2	16.00	6.47	24.30	2
1	12	66.90	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	13	72.47	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	14	78.05	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	15	83.62	1	9.33	9.15	16.12	2	24.00	6.47	24.30	2
1	16	89.20	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	17	94.77	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	18	100.35	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	19	105.92	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2
1	20	111.50	1	9.33	9.15	16.12	2	23.75	6.47	24.30	2

NOTE: LMDAf = slenderness ratio for the comp. flange

=  $bfc / 2tfc$  (Eq. 6.10.8.2.2-3)

LMDApf = limiting slenderness ratio for a compact flange

=  $0.38 \cdot \sqrt{E/Fyc}$  (Eq. 6.10.8.2.2-4)

LMDArf = limiting slenderness ratio for a non-compact flange

=  $0.56 \cdot \sqrt{E/Fyr}$  (Eq. 6.10.8.2.2-5)

Lb = unbraced length

Lp = limiting unbraced length to achieve the nominal flexural

of  $RbRhFyc$  under uniform bending

=  $1.0rt \cdot \sqrt{E/Fyc}$  (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

=  $\pi \cdot rt \cdot \sqrt{E/Fyr}$  (Eq. 6.10.8.2.3-5)

In negative moment region, the first line is for non-composite sections

and the second line is for composite sections

Flange Category

0 = compact section

2 = non-compact section

3 = slender section

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE

\*\*\*\*\*

SP	IN	D	FROM	Co	Rh	Rb	Cb	FLB	LTB	GOV
NO	NO	L	SUPT	mp				Fnc	Fnc	Fnc
(ft)								(Ksi)	(Ksi)	(Ksi)
1	0	0.00	1	0.971	1.000	1.61		48.22	48.57	48.22
1	1	5.57	1	0.982	1.000	1.61		48.72	49.09	48.72

5799 Ext Left.res									
1	2	11.15	1	0.982	1.000	1.61	48.72	49.09	48.72
1	3	16.72	1	0.982	1.000	1.61	48.72	49.09	48.72
1	4	22.30	1	0.982	1.000	1.61	48.73	49.09	48.73
1	5	27.87	1	0.982	1.000	1.11	48.73	39.27	39.27
1	6	33.45	1	0.982	1.000	1.11	48.73	39.27	39.27
1	7	39.03	1	0.982	1.000	1.11	48.73	39.27	39.27
1	8	44.60	1	0.982	1.000	1.11	48.73	39.27	39.27
1	9	50.18	1	0.982	1.000	1.00	48.73	41.56	41.56
1	10	55.75	1	0.982	1.000	1.00	48.73	41.56	41.56
1	11	61.33	1	0.982	1.000	1.00	48.73	41.56	41.56
1	12	66.90	1	0.982	1.000	1.11	48.73	39.27	39.27
1	13	72.47	1	0.982	1.000	1.11	48.73	39.27	39.27
1	14	78.05	1	0.982	1.000	1.11	48.73	39.27	39.27
1	15	83.62	1	0.982	1.000	1.11	48.73	39.27	39.27
1	16	89.20	1	0.982	1.000	1.61	48.73	49.09	48.73
1	17	94.77	1	0.982	1.000	1.61	48.72	49.09	48.72
1	18	100.35	1	0.982	1.000	1.61	48.72	49.09	48.72
1	19	105.92	1	0.982	1.000	1.61	48.72	49.09	48.72
1	20	111.50	1	0.971	1.000	1.61	48.22	48.57	48.22

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=INFORMATION FOR DUCTILITY CHECK

\*\*\*\*\*

SP NO	IN NO	FROM L SUPT (ft)	Co mp	Dp (in)	0.42Dt (in)
1	0	0.00	1	10.13	32.76
1	1	5.57	1	10.13	32.76
1	2	11.15	1	10.13	32.76
1	3	16.72	1	10.13	32.76
1	4	22.30	1	10.13	32.76
1	5	27.87	1	10.13	32.76
1	6	33.45	1	10.13	32.76
1	7	39.03	1	10.13	32.76
1	8	44.60	1	10.13	32.76
1	9	50.18	1	10.13	32.76
1	10	55.75	1	10.13	32.76
1	11	61.33	1	10.13	32.76
1	12	66.90	1	10.13	32.76
1	13	72.47	1	10.13	32.76
1	14	78.05	1	10.13	32.76
1	15	83.62	1	10.13	32.76
1	16	89.20	1	10.13	32.76
1	17	94.77	1	10.13	32.76
1	18	100.35	1	10.13	32.76
1	19	105.92	1	10.13	32.76
1	20	111.50	1	10.13	32.76

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TABLE 1.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION

\*\*\*\*\*

				S T R E N G T H C A T E G O R Y,				Category	
SP NO	IN NO	FROM L SUPT (ft)	Section Region	Noncomposite		Composite		Non-Comp.	Comp.
				Web [1]	Flange [2]	Web [3]	Flange [4]		
1	0	0.00	1	2	2	2		2	2
1	1	5.57	1	2	2	2		2	2

						5799	Ext	Left.res		
1	2	11.15	1	2	2	2	2		2	2
1	3	16.72	1	2	2	2	2		2	2
1	4	22.30	1	2	2	2	2		2	2
1	5	27.87	1	2	2	2	2		2	2
1	6	33.45	1	2	2	2	2		2	2
1	7	39.03	1	2	2	2	2		2	2
1	8	44.60	1	2	2	2	2		2	2
1	9	50.18	1	2	2	2	2		2	2
1	10	55.75	1	2	2	2	2		2	2
1	11	61.33	1	2	2	2	2		2	2
1	12	66.90	1	2	2	2	2		2	2
1	13	72.47	1	2	2	2	2		2	2
1	14	78.05	1	2	2	2	2		2	2
1	15	83.62	1	2	2	2	2		2	2
1	16	89.20	1	2	2	2	2		2	2
1	17	94.77	1	2	2	2	2		2	2
1	18	100.35	1	2	2	2	2		2	2
1	19	105.92	1	2	2	2	2		2	2
1	20	111.50	1	2	2	2	2		2	2

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

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- NOTE: [1] For non-composite 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  $F_y$  of the flanges > 70 ksi (483 MPa)

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TABLE 1.2.22.10=CONSTRUCTIBILITY CHECK (STRENGTH IV)

\*\*\*\*\*

SP	IN	D	FROM	f <sub>l</sub>	0.6F <sub>yt</sub>	f <sub>bu</sub>	[1]	f <sub>bu</sub> +f <sub>l</sub>	[2]	f <sub>bu</sub> +1/3f <sub>l</sub>	[3]	FLAG
			(ft)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	
1	0		0.00	0.0	30.0	0.0	26.6	0.0	48.6	0.0	48.2	0
				0.0	-	0.0	-	0.0	48.6	-	-	0
1	1		5.57	0.0	30.0	6.4	26.6	6.4	49.1	6.4	48.7	0
				0.0	-	4.0	-	4.0	49.1	-	-	0
1	2		11.15	0.0	30.0	12.1	26.6	12.1	49.1	12.1	48.7	0
				0.0	-	7.6	-	7.6	49.1	-	-	0
1	3		16.72	0.0	30.0	17.2	26.6	17.2	49.1	17.2	48.7	0
				0.0	-	10.7	-	10.7	49.1	-	-	0
1	4		22.30	0.0	30.0	21.6	26.6	21.6	49.1	21.6	48.7	0
				0.0	-	13.5	-	13.5	49.1	-	-	0
1	5		27.87	0.0	30.0	25.3	26.6	25.3	49.1	25.3	39.3	0
				0.0	-	15.8	-	15.8	49.1	-	-	0
1	6		33.45	0.0	30.0	28.3	26.6	28.3	49.1	28.3	39.3	1



5799 Ext Left.res										
1	7	39.03	0.0	-	17.7	-	17.7	49.1	-	0
			0.0	30.0	30.7	26.6	30.7	49.1	30.7	39.3
			0.0	-	19.1	-	19.1	49.1	-	0
1	8	44.60	0.0	30.0	32.3	26.6	32.3	49.1	32.3	39.3
			0.0	-	20.2	-	20.2	49.1	-	0
1	9	50.18	0.0	30.0	33.4	26.6	33.4	49.1	33.4	41.6
			0.0	-	20.8	-	20.8	49.1	-	0
1	10	55.75	0.0	30.0	33.7	26.6	33.7	49.1	33.7	41.6
			0.0	-	21.0	-	21.0	49.1	-	0
1	11	61.33	0.0	30.0	33.4	26.6	33.4	49.1	33.4	41.6
			0.0	-	20.8	-	20.8	49.1	-	0
1	12	66.90	0.0	30.0	32.3	26.6	32.3	49.1	32.3	39.3
			0.0	-	20.2	-	20.2	49.1	-	0
1	13	72.47	0.0	30.0	30.7	26.6	30.7	49.1	30.7	39.3
			0.0	-	19.1	-	19.1	49.1	-	0
1	14	78.05	0.0	30.0	28.3	26.6	28.3	49.1	28.3	39.3
			0.0	-	17.7	-	17.7	49.1	-	0
1	15	83.62	0.0	30.0	25.3	26.6	25.3	49.1	25.3	39.3
			0.0	-	15.8	-	15.8	49.1	-	0
1	16	89.20	0.0	30.0	21.6	26.6	21.6	49.1	21.6	48.7
			0.0	-	13.5	-	13.5	49.1	-	0
1	17	94.77	0.0	30.0	17.2	26.6	17.2	49.1	17.2	48.7
			0.0	-	10.7	-	10.7	49.1	-	0
1	18	100.35	0.0	30.0	12.1	26.6	12.1	49.1	12.1	48.7
			0.0	-	7.6	-	7.6	49.1	-	0
1	19	105.92	0.0	30.0	6.4	26.6	6.4	49.1	6.4	48.7
			0.0	-	4.0	-	4.0	49.1	-	0
1	20	111.50	0.0	30.0	0.0	26.6	0.0	48.6	0.0	48.2
			0.0	-	0.0	-	0.0	48.6	-	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	0.000	3
1	1	5.57	0.000	0.240	0.130	0.131	0.240	2
					0.081	0.081	0.081	3
1	2	11.15	0.000	0.456	0.247	0.249	0.456	2
					0.154	0.154	0.154	3
1	3	16.72	0.000	0.646	0.350	0.353	0.646	2
					0.218	0.218	0.218	3
1	4	22.30	0.000	0.811	0.439	0.443	0.811	2
					0.274	0.274	0.274	3
1	5	27.87	0.000	0.950	0.515	0.643	0.950	2
					0.321	0.321	0.321	3
1	6	33.45	0.000	1.064	0.576	0.721	1.064	2
					0.360	0.360	0.360	3
1	7	39.03	0.000	1.152	0.624	0.781	1.152	2
					0.390	0.390	0.390	3
1	8	44.60	0.000	1.216	0.659	0.824	1.216	2
					0.411	0.411	0.411	3

				5799 Ext Left.res				
1	9	50.18	0.000	1.254	0.680	0.803	1.254	2
					0.424		0.424	3
1	10	55.75	0.000	1.267	0.686	0.811	1.267	2
					0.429		0.429	3
1	11	61.33	0.000	1.254	0.680	0.803	1.254	2
					0.424		0.424	3
1	12	66.90	0.000	1.216	0.659	0.824	1.216	2
					0.411		0.411	3
1	13	72.47	0.000	1.152	0.624	0.781	1.152	2
					0.390		0.390	3
1	14	78.05	0.000	1.064	0.576	0.721	1.064	2
					0.360		0.360	3
1	15	83.62	0.000	0.950	0.515	0.643	0.950	2
					0.321		0.321	3
1	16	89.20	0.000	0.811	0.439	0.443	0.811	2
					0.274		0.274	3
1	17	94.77	0.000	0.646	0.350	0.353	0.646	2
					0.218		0.218	3
1	18	100.35	0.000	0.456	0.247	0.249	0.456	2
					0.154		0.154	3
1	19	105.92	0.000	0.240	0.130	0.131	0.240	2
					0.081		0.081	3
1	20	111.50	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 IN D	FROM	ID	Mu+1/3fl	Sxt	[1]	fbu	[2]	fl	[3]	fbu+	[4]	FLAG
NO	NO	L	SUPT	(ft)	(k-ft)	(k-ft)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	MCTD
				(ft)						1/3fl		
1	0	0.00	2	-	-	0.0	48.1	-	-	-	-	-0-0
				-	-	0.0	-	0.0	30.0	0.0	48.1	--00
1	1	5.57	2	-	-	6.8	48.1	-	-	-	-	-0-0
				-	-	8.7	-	0.0	30.0	8.7	48.1	--00
1	2	11.15	2	-	-	12.8	48.1	-	-	-	-	-0-0
				-	-	16.4	-	0.0	30.0	16.4	48.1	--00
1	3	16.72	2	-	-	18.1	48.1	-	-	-	-	-0-0
				-	-	23.2	-	0.0	30.0	23.2	48.1	--00
1	4	22.30	2	-	-	22.8	48.1	-	-	-	-	-0-0
				-	-	29.1	-	0.0	30.0	29.1	48.1	--00
1	5	27.87	2	-	-	26.6	48.1	-	-	-	-	-0-0
				-	-	34.0	-	0.0	30.0	34.0	48.1	--00
1	6	33.45	2	-	-	29.8	48.1	-	-	-	-	-0-0
				-	-	37.9	-	0.0	30.0	37.9	48.1	--00
1	7	39.03	2	-	-	32.3	48.1	-	-	-	-	-0-0
				-	-	41.0	-	0.0	30.0	41.0	48.1	--00
1	8	44.60	2	-	-	34.1	48.1	-	-	-	-	-0-0
				-	-	43.2	-	0.0	30.0	43.2	48.1	--00
1	9	50.18	2	-	-	35.1	48.1	-	-	-	-	-0-0
				-	-	44.4	-	0.0	30.0	44.4	48.1	--00
1	10	55.75	2	-	-	35.4	48.1	-	-	-	-	-0-0
				-	-	44.8	-	0.0	30.0	44.8	48.1	--00
1	11	61.33	2	-	-	35.1	48.1	-	-	-	-	-0-0
				-	-	44.4	-	0.0	30.0	44.4	48.1	--00
1	12	66.90	2	-	-	34.1	48.1	-	-	-	-	-0-0
				-	-	43.2	-	0.0	30.0	43.2	48.1	--00
1	13	72.47	2	-	-	32.3	48.1	-	-	-	-	-0-0
				-	-	41.0	-	0.0	30.0	41.0	48.1	--00
1	14	78.05	2	-	-	29.8	48.1	-	-	-	-	-0-0

										5799 Ext Left.res
1	15	83.62	2	-	-	37.9	-	0.0	30.0	37.9 48.1 --00
				-	-	26.6	48.1	-	-	-0-0
				-	-	34.0	-	0.0	30.0	34.0 48.1 --00
1	16	89.20	2	-	-	22.8	48.1	-	-	-0-0
				-	-	29.1	-	0.0	30.0	29.1 48.1 --00
1	17	94.77	2	-	-	18.1	48.1	-	-	-0-0
				-	-	23.2	-	0.0	30.0	23.2 48.1 --00
1	18	100.35	2	-	-	12.8	48.1	-	-	-0-0
				-	-	16.4	-	0.0	30.0	16.4 48.1 --00
1	19	105.92	2	-	-	6.8	48.1	-	-	-0-0
				-	-	8.7	-	0.0	30.0	8.7 48.1 --00
1	20	111.50	2	-	-	0.0	48.1	-	-	-0-0
				-	-	0.0	-	0.0	30.0	0.0 48.1 --00

NOTE: [1] =  $(\Phi) f * M_n$  (Eq. 6.10.7.1.1-1)  
 [2] =  $(\Phi) f * F_{nc}$  for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)  
 =  $(\Phi) f * R_h * F_{yt}$  for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)  
 [3] =  $0.6 * F_{yt}$  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)  
 =  $0.6 * F_{yc}$  for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)  
 [4] =  $(\Phi) f * F_{nt}$  for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)  
 =  $(\Phi) f * F_{nc}$  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  $f_{bu}$ ,  $M_u$  and  $f_l$  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  $f_{bu}$  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+1/3f <sub>l</sub> Sxt/	f <sub>bu</sub> /	f <sub>l</sub> /	f <sub>bu</sub> +1/3f <sub>l</sub> /	MAX.
					[1]	[2]	[3]	[4]	RAT. GOVN.
1	0	0.00	2			0.000		0.000	0.000 2
1	1	5.57	2			0.141	0.000	0.000	0.141 4
1	2	11.15	2			0.266	0.000	0.181	0.181 2
1	3	16.72	2			0.377	0.000	0.342	0.342 4
1	4	22.30	2			0.473	0.000	0.377	0.377 2
1	5	27.87	2			0.554	0.000	0.483	0.483 4
1	6	33.45	2			0.620	0.000	0.473	0.473 2
1	7	39.03	2			0.672	0.000	0.605	0.605 4
1	8	44.60	2			0.709	0.000	0.707	0.707 2
1	9	50.18	2			0.730	0.000	0.790	0.790 4
							0.000	0.672	0.672 2
							0.000	0.854	0.854 4
							0.000	0.709	0.709 2
							0.000	0.899	0.899 4
							0.000	0.730	0.730 2
							0.000	0.924	0.924 4

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1	10	55.75	2	0.737		0.000	0.933	0.737	2
1	11	61.33	2	0.730		0.000	0.924	0.730	4
1	12	66.90	2	0.709		0.000	0.899	0.709	2
1	13	72.47	2	0.672		0.000	0.854	0.672	4
1	14	78.05	2	0.620		0.000	0.790	0.620	2
1	15	83.62	2	0.554		0.000	0.707	0.554	4
1	16	89.20	2	0.473		0.000	0.605	0.473	2
1	17	94.77	2	0.377		0.000	0.483	0.377	4
1	18	100.35	2	0.266		0.000	0.342	0.266	2
1	19	105.92	2	0.141		0.000	0.181	0.141	4
1	20	111.50	2	0.000		0.000	0.000	0.000	2
						0.000	0.000	0.000	4

NOTE: [1] =  $(\Phi) f \cdot M_n$  (Eq. 6.10.7.1.1-1)  
 [2] =  $(\Phi) f \cdot F_{nc}$  for comp. flange of composite sections in positive flexure (Eq. 6.10.7.2.1-1)  
 =  $(\Phi) f \cdot R_h \cdot F_{yt}$  for tension flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.8.1.3-1)  
 [3] =  $0.6 \cdot F_{yt}$  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)  
 =  $0.6 \cdot F_{yc}$  for comp. flange of composite sections in negative flexure and non-composite sections (Eq. 6.10.1.6-1)  
 [4] =  $(\Phi) f \cdot F_{nt}$  for non-compact tension flange of composite sections in positive flexure (Eq. 6.10.8.1.2-1)  
 =  $(\Phi) f \cdot F_{nc}$  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 =  $(\mu + 1/3 f_l S_{xt}) / [1]$
- 2 =  $f_{bu} / [2]$
- 3 =  $f_l / [3]$
- 4 =  $(f_{bu} + 1/3 f_l) / [4]$

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TABLE 1.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY

*****											
SP NO	IN NO	D L	FROM SUPT (ft)	Fy (ksi)	k [1]	Art. 6.10.9.2		D/tw	C [4]	Vp (ksi)	SHEAR CAPACITY -Eq. 6.10.9.2-- Vn = C x Vp
-----											
1	0	0.00	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	1	5.57	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	2	11.15	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	3	16.72	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	4	22.30	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	5	27.87	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	6	33.45	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	7	39.03	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	8	44.60	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	9	50.18	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	10	55.75	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	11	61.33	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	12	66.90	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	13	72.47	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	14	78.05	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	15	83.62	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	16	89.20	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	17	94.77	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	18	100.35	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	19	105.92	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	
1	20	111.50	36.0	5.0	71.1	88.9	150.9	0.278	602.9	167.5	

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NOTE: [1]  $k = \text{buckling coefficient} = 5 + 5 / ((d_0/D)^{**2})$   
= 5 for unstiffened beams and girders  
= 5 for stiffened girders when  $(d_0/D) > 3$ , or  
 $(d_0/D) > (260/(D/tw))^{**2}$

[2] =  $1.12 * \text{sqrt}(E_k / F_yw)$

[3] =  $1.40 * \text{sqrt}(E_k / F_yw)$

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

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

For  $[2] \leq D/tw \leq [3]$ ,  $C = 1.12 * \text{sqrt}(E_k / F_yw) / (D/tw)$   
..... AASHTO LRFD Eq. 6.10.9.3.2-6

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

\*\* AASHTO LRFD Art. 6.10.9.1 --- For interior web panels considered  
stiffened  
if  $(d_0/D) < 3$  without long. stiffener, or  
if  $(d_0/D) > 3$  with both transv. and long. stiffeners.  
For handling requirement,  $(d_0/D) > (260/(D/tw))^{**2}$

$V_u = C V_p$

where D = clear, unsupported distance between flange components

$d_0$  = distance between transverse stiffeners

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

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TABLE 1.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY

\*\*\*\*\*

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

SP	IN	D	FROM	WEB	STABILITY	LFD/LRFD	UNSTIFFENED	OVERALL
NO	NO	L	SUPT	UNSTIFFENED, [1]	MAX. SHEAR <=>	SHEAR CAPA-	CATEGORY	WEB
(ft)	----	D/tw	-----	(kip)		CITY, [2]		
1	0	0.00		2	319.3	>	167.5	2
1	1	5.57		2	293.6	>	167.5	2
1	2	11.15		2	268.3	>	167.5	2
1	3	16.72		2	243.1	>	167.5	2
1	4	22.30		2	218.3	>	167.5	2
1	5	27.87		2	193.5	>	167.5	2
1	6	33.45		2	169.2	>	167.5	2
1	7	39.03		2	145.1	<	167.5	2
1	8	44.60		2	121.4	<	167.5	2
1	9	50.18		2	97.6	<	167.5	2
1	10	55.75		2	74.4	<	167.5	2
1	11	61.33		2	97.6	<	167.5	2
1	12	66.90		2	121.4	<	167.5	2
1	13	72.47		2	145.1	<	167.5	2
1	14	78.05		2	169.2	>	167.5	2
1	15	83.62		2	193.5	>	167.5	2
1	16	89.20		2	218.3	>	167.5	2
1	17	94.77		2	243.1	>	167.5	2
1	18	100.35		2	268.3	>	167.5	2

				5799 Ext Left.res		
1	19	105.92	2	293.6	>	167.5
1	20	111.50	2	319.3	>	167.5

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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.

$D/tw \leq 150$  (Eq. 6.10.2.1.1-1)

w / long. stiff.

$D/tw \leq 300$  (Eq. 6.10.2.1.2-1)

$2Dcp/tw \leq 3.76 \sqrt{E/Fyc}$  ..... 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)

$2Dc/tw \leq 5.7 \sqrt{E/Fyc}$  ..... AASHTO Eq.6.10.6.2.3-1

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

$V_n = CV_p$  ..... AASHTO Eq.6.10.9.2-1

$V_n = V_p Rc_1$  ..... AASHTO Eq.6.10.9.3.2-2

$Rc_1 = C + [0.87(1-C)] / \sqrt{1 + (d_0/D)^2}$  for  $2Dtw / (bfctfc + bfttft) \leq 2.5$

$V_n = V_p Rc_2$  ..... AASHTO Eq.6.10.9.3.2-8

$Rc_2 = C + [0.87(1-C)] / [\sqrt{1 + (d_0/D)^2} + (d_0/D)]$  for  $2Dtw / (bfctfc + bfttft) > 2.5$

$V_p = 0.58 F_y D^* tw$  ..... 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	IN	D	FROM	YIELD	LFD/LRFD	UNSTIFFENED	REQUIREMENT OF	MAX. ALLOWABLE
NO	NO	L	SUPT	STRESS	MAXIMUM	SHEAR	TRANS. STIFFENERS	TRANS. STIFFENERS
			(ft)	$F_y$	SHEAR	CAPACITY	1=YES, 0=NO	SPACING
				(ksi)	(k)	(k)		(ft-in)
1	0	0.00		36.	319.31	167.53	1	8 - 3
1	1	5.57		36.	293.65	167.53	1	16 - 6
1	2	11.15		36.	268.25	167.53	1	16 - 6
1	3	16.72		36.	243.13	167.53	1	16 - 6
1	4	22.30		36.	218.29	167.53	1	16 - 6
1	5	27.87		36.	193.47	167.53	1	16 - 6
1	6	33.45		36.	169.16	167.53	1	16 - 6
1	7	39.03		36.	145.13	167.53	0	
1	8	44.60		36.	121.37	167.53	0	
1	9	50.18		36.	97.64	167.53	0	
1	10	55.75		36.	74.43	167.53	0	

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1	11	61.33	36.	97.64	167.53	0		
1	12	66.90	36.	121.37	167.53	0		
1	13	72.47	36.	145.13	167.53	0		
1	14	78.05	36.	169.16	167.53	1	16 -	6
1	15	83.62	36.	193.47	167.53	1	16 -	6
1	16	89.20	36.	218.29	167.53	1	16 -	6
1	17	94.77	36.	243.13	167.53	1	16 -	6
1	18	100.35	36.	268.25	167.53	1	16 -	6
1	19	105.92	36.	293.65	167.53	1	16 -	6
1	20	111.50	36.	319.31	167.53	1	8 -	3

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TABLE 1.2.22.18=STATUS OF TRANSVERSE STIFFENERS

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	YIELD STRESS		SPACING, (ft-in)		STATUS	
	BEAM (ksi)	TRANS. STIFF.	FROM INPUT	MAXIMUM ALLOWABLE	SEE NOTE	
1 0 0.00	36.	36.	2- 9 <	8- 3		
1 1 5.57	36.	36.	6- 0 <	16- 6		
1 2 11.15	36.	36.	7- 0 <	16- 6		
1 3 16.72	36.	36.	8- 0 <	16- 6		
1 4 22.30	36.	36.	8- 0 <	16- 6		
1 5 27.87	36.	36.	8- 0 <	16- 6		
1 6 33.45	36.	36.	8- 0 <	16- 6		
1 7 39.03	36.	36.	8- 0		**	
1 8 44.60	36.	36.	8- 0		**	
1 9 50.18	36.	36.	16- 0		**	
1 10 55.75	36.	36.	16- 0		**	
1 11 61.33	36.	36.	16- 0		**	
1 12 66.90	36.	36.	8- 0		**	
1 13 72.47	36.	36.	8- 0		**	
1 14 78.05	36.	36.	8- 0 <	16- 6		
1 15 83.62	36.	36.	8- 0 <	16- 6		
1 16 89.20	36.	36.	8- 0 <	16- 6		
1 17 94.77	36.	36.	8- 0 <	16- 6		
1 18 100.35	36.	36.	7- 0 <	16- 6		
1 19 105.92	36.	36.	6- 0 <	16- 6		
1 20 111.50	36.	36.	2- 9 <	8- 3		

\*\* NOTE [1] : STATUS

BLANK = Acceptable trans. stiff. spacing  
<= = Trans. stiff. spacing violated  
\*\* = Trans. stiff. is redundant

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TABLE 1.2.22.19=SHEAR CAPACITY CHECK

\*\*\*\*\*

SP IN D FROM NO NO L SUPT (ft)	WEB		RB	YIELD TRANS.		CC	LRFD		SHEAR CAPACITY (Phi) Vn (k)	STATUS
	----- D/T	----- D/T		SHEAR Vp (k)	STIFF. SPACING (Ft-in)		MAXIMUM SHEAR (k)	==>		
1 0 0.00	151.	1.0000		603.	2- 9	1.000	319.31	<	602.91	
1 1 5.57	151.	1.0000		603.	6- 0	0.799	293.65	<	481.49	
1 2 11.15	151.	1.0000		603.	7- 0	0.745	268.25	<	449.38	
1 3 16.72	151.	1.0000		603.	8- 0	0.700	243.13	<	422.27	
1 4 22.30	151.	1.0000		603.	8- 0	0.700	218.29	<	422.27	
1 5 27.87	151.	1.0000		603.	8- 0	0.700	193.47	<	422.27	
1 6 33.45	151.	1.0000		603.	8- 0	0.700	169.16	<	422.27	
1 7 39.03	151.	1.0000		603.	8- 0	0.700	145.13	<	422.27	
1 8 44.60	151.	1.0000		603.	8- 0	0.700	121.37	<	422.27	

5799 Ext Left.res

1	9	50.18	151.	1.0000	603.	16-	0	0.506	97.64	<	304.86
1	10	55.75	151.	1.0000	603.	16-	0	0.506	74.43	<	304.86
1	11	61.33	151.	1.0000	603.	16-	0	0.506	97.64	<	304.86
1	12	66.90	151.	1.0000	603.	8-	0	0.700	121.37	<	422.27
1	13	72.47	151.	1.0000	603.	8-	0	0.700	145.13	<	422.27
1	14	78.05	151.	1.0000	603.	8-	0	0.700	169.16	<	422.27
1	15	83.62	151.	1.0000	603.	8-	0	0.700	193.47	<	422.27
1	16	89.20	151.	1.0000	603.	8-	0	0.700	218.29	<	422.27
1	17	94.77	151.	1.0000	603.	8-	0	0.700	243.13	<	422.27
1	18	100.35	151.	1.0000	603.	7-	0	0.745	268.25	<	449.38
1	19	105.92	151.	1.0000	603.	6-	0	0.799	293.65	<	481.49
1	20	111.50	151.	1.0000	603.	2-	9	1.000	319.31	<	602.91

NOTE: Assumed that all the supports have stiffeners

STATUS: (1) --- blank: shear capacity is ok  
 (2) --- <= : insufficient shear capacity  
 (3) ---  $V_n = CC * V_p$ ; CC see AASHTO LRFD 6.10.9.3.2-2 OR -8  
 or Table 22.15

CC includes post-buckling strength due to tension field action

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TABLE 1.2.22.20=SECTION PROPERTY CHECK OF TRANSVERSE STIFFENERS  
 \*\*\*\*\*

SP IN D FROM			MIN. [1] WIDTH (in) MAX. [2]			I, 6.10.11.1.3, (in**4)					FL AG
NO	NO	L SUPT (ft)	AASHTO MIN. <=>	Eq. 6.10.11.1.2-1,2 ACTUAL <=>	MAX. MAX.	It	[3] It1	[4] It2	[5] It3	[6] It4	
1	0	0.0	4.2 <	5.0 <	6.0	16.	22.	21.	0.	0.	<=
1	1	5.6	4.2 <	5.0 <	6.0	16.	3.	21.	10.	0.	
1	2	11.1	4.2 <	5.0 <	6.0	16.	3.	21.	9.	0.	
1	3	16.7	4.2 <	7.0 >	6.0	43.	3.	32.	11.	0.	<=
1	4	22.3	4.2 <	7.0 >	6.0	43.	3.	32.	9.	0.	<=
1	5	27.9	4.2 <	5.0 <	6.0	16.	3.	21.	5.	0.	
1	6	33.5	4.2 <	5.0 <	6.0	16.	3.	21.	3.	0.	
1	7	39.03									
1	8	44.60									
1	9	50.18									
1	10	55.75									
1	11	61.33									
1	12	66.90									
1	13	72.47									
1	14	78.0	4.2 <	5.0 <	6.0	16.	3.	21.	3.	0.	
1	15	83.6	4.2 <	7.0 >	6.0	43.	3.	32.	6.	0.	<=
1	16	89.2	4.2 <	5.0 <	6.0	16.	3.	21.	6.	0.	
1	17	94.8	4.2 <	5.0 <	6.0	16.	3.	21.	8.	0.	
1	18	100.3	4.2 <	5.0 <	6.0	16.	3.	21.	9.	0.	
1	19	105.9	4.2 <	5.0 <	6.0	16.	3.	21.	10.	0.	
1	20	111.5	4.2 <	5.0 <	6.0	16.	22.	21.	10.	0.	<=

Please read NOTE on the following page

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\*\* NOTE [1 or 2]: Minimum width of transverse stiffeners  
 AASHTO LRFD Eq. 6.10.11.1.2-1 or -2

[3]: required moment of inertia for trans. stiff.  
 AASHTO LRFD Eq. 6.10.11.1.3-3

[4]: required moment of inertia for trans. stiff.  
 AASHTO LRFD Eq. 6.10.11.1.3-4

[5]: required moment of inertia for trans. stiff.  
 AASHTO LRFD Eq. 6.10.11.1.3-7

[6]: required moment of inertia for trans. stiff.



FLAG : (1) --- BLANK : acceptable trans. stiff.  
(2) --- <= : trans. stiff. violates AASHTO requirement(s)

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TABLE 1.2.22.20A=REQ. SECTION PROPERTIES FOR TRANS. STIFFENERS.  
\*\*\*\*\*

SP IN D FROM	WEB	Do	ASSUMED	J	REQ'D I
NO NO L SUPT	DEPTH THICK.	-(ft-in)-	B Y	AASHTO	-(in**4)-
(ft)	(in)	ASSUMED		6.10.8.1.3-1,2	
		SPACING			
1 0 0.00	66.0 0.4375	2- 9	2.4 1.0	8.0	22.1
1 1 5.57	66.0 0.4375	6- 0	2.4 1.0	0.5	3.0
1 2 11.15	66.0 0.4375	7- 0	2.4 1.0	0.5	3.5
1 3 16.72	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 4 22.30	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 5 27.87	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 6 33.45	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 7 39.03	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 8 44.60	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 9 50.18	66.0 0.4375	16- 0	2.4 1.0	0.5	8.0
1 10 55.75	66.0 0.4375	16- 0	2.4 1.0	0.5	8.0
1 11 61.33	66.0 0.4375	16- 0	2.4 1.0	0.5	8.0
1 12 66.90	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 13 72.47	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 14 78.05	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 15 83.62	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 16 89.20	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 17 94.77	66.0 0.4375	8- 0	2.4 1.0	0.5	4.0
1 18 100.35	66.0 0.4375	7- 0	2.4 1.0	0.5	3.5
1 19 105.92	66.0 0.4375	6- 0	2.4 1.0	0.5	3.0
1 20 111.50	66.0 0.4375	2- 9	2.4 1.0	8.0	22.1

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TABLE 1.2.22.21=SERVICE LIMIT STATE CHECK  
\*\*\*\*\*

SP IN D FROM	TOP FLANGE	BOT. FLANGE	COMP. FLG	FLAG
NO NO L SUPT	ff ff+fl/2	[1] ff+fl/2 [2]	fc fcrw	[3] TBC
(ft)	(ksi) (ksi)	(ksi) (ksi)	(ksi) (ksi)	
1 0 0.00	0.0 -	45.7 0.0	45.7 0.0	48.6 000
1 1 5.57	5.3 -	45.7 6.6	45.7 5.3	49.1 000
1 2 11.15	10.0 -	45.7 12.5	45.7 10.0	49.1 000
1 3 16.72	14.2 -	45.7 17.7	45.7 14.2	49.1 000
1 4 22.30	17.9 -	45.7 22.1	45.7 17.9	49.1 000
1 5 27.87	20.9 -	45.7 25.8	45.7 20.9	49.1 000
1 6 33.45	23.4 -	45.7 28.8	45.7 23.4	49.1 000
1 7 39.03	25.3 -	45.7 31.2	45.7 25.3	49.1 000
1 8 44.60	26.7 -	45.7 32.9	45.7 26.7	49.1 000
1 9 50.18	27.5 -	45.7 33.8	45.7 27.5	49.1 000
1 10 55.75	27.8 -	45.7 34.1	45.7 27.8	49.1 000
1 11 61.33	27.5 -	45.7 33.8	45.7 27.5	49.1 000
1 12 66.90	26.7 -	45.7 32.9	45.7 26.7	49.1 000
1 13 72.47	25.3 -	45.7 31.2	45.7 25.3	49.1 000
1 14 78.05	23.4 -	45.7 28.8	45.7 23.4	49.1 000
1 15 83.62	20.9 -	45.7 25.8	45.7 20.9	49.1 000
1 16 89.20	17.9 -	45.7 22.1	45.7 17.9	49.1 000
1 17 94.77	14.2 -	45.7 17.7	45.7 14.2	49.1 000
1 18 100.35	10.0 -	45.7 12.5	45.7 10.0	49.1 000
1 19 105.92	5.3 -	45.7 6.6	45.7 5.3	49.1 000
1 20 111.50	0.0 -	45.7 0.0	45.7 0.0	48.6 000

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NOTE: [1] =  $0.95R_h F_y f$  for composite sections (Eq. 6.10.4.2.2-1)  
=  $0.80R_h F_y f$  for non-composite sections (Eq. 6.10.4.2.2-3)  
[2] =  $0.95R_h F_y f$  for composite sections (Eq. 6.10.4.2.2-2)  
=  $0.80R_h F_y f$  for non-composite sections (Eq. 6.10.4.2.2-3)  
[3] =  $0.9E_k / (D/tw)^2$  (Eq. 6.10.1.9.1-1)  
but not to exceed the smaller of  $R_h F_y c$  and  $F_y w/0.7$   
k = bending-buckling coefficient  
=  $9 / (D_c/D)^2$  (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	IN	D	FROM	TOP OF TOP FLANGE			BOTTOM OF BOTTOM FLANGE		
				GOVERN.	STRESS	ACCEPTABLE	GOVERN.	STRESS	ACCEPTABLE
NO	NO	L	SUPT	LOADING	RANGE	STRESS	LOADING	RANGE	STRESS
(ft)					(ksi)	CATEGORY		(ksi)	CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^	
1	1	5.57	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^	
1	2	11.15	TR	-0.8	A B B^C^C D E E^	TR	3.9	A B B^C^C D E	
1	3	16.72	TR	-1.1	A B B^C^C D E E^	TR	5.4	A B B^C^C D	
1	4	22.30	TR	-1.4	A B B^C^C D E E^	TR	6.7	A B B^C^C D	
1	5	27.87	TR	-1.6	A B B^C^C D E E^	TR	7.8	A B B^C^C	
1	6	33.45	TR	-1.8	A B B^C^C D E E^	TR	8.7	A B B^C^C	
1	7	39.03	TR	-1.9	A B B^C^C D E E^	TR	9.3	A B B^C^C	
1	8	44.60	TR	-2.0	A B B^C^C D E E^	TR	9.7	A B B^C^C	
1	9	50.18	TR	-2.0	A B B^C^C D E E^	TR	9.7	A B B^C^C	
1	10	55.75	TR	-2.0	A B B^C^C D E E^	TR	9.7	A B B^C^C	
1	11	61.33	TR	-2.0	A B B^C^C D E E^	TR	9.7	A B B^C^C	
1	12	66.90	TR	-2.0	A B B^C^C D E E^	TR	9.7	A B B^C^C	
1	13	72.47	TR	-1.9	A B B^C^C D E E^	TR	9.3	A B B^C^C	
1	14	78.05	TR	-1.8	A B B^C^C D E E^	TR	8.7	A B B^C^C	
1	15	83.62	TR	-1.6	A B B^C^C D E E^	TR	7.8	A B B^C^C	
1	16	89.20	TR	-1.4	A B B^C^C D E E^	TR	6.7	A B B^C^C D	
1	17	94.77	TR	-1.1	A B B^C^C D E E^	TR	5.4	A B B^C^C D	
1	18	100.35	TR	-0.8	A B B^C^C D E E^	TR	3.9	A B B^C^C D E	
1	19	105.92	TR	-0.4	A B B^C^C D E E^	TR	2.1	A B B^C^C D E E^	
1	20	111.50	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

			TOP OF TOP FLANGE			BOTTOM OF BOTTOM FLANGE		
SP	IN	D FROM	GOVERN.	STRESS	ACCEPTABLE	GOVERN.	STRESS	ACCEPTABLE
NO	NO	L SUPT	LOADING	RANGE	STRESS	LOADING	RANGE	STRESS
		(ft)		(ksi)	CATEGORY		(ksi)	CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	5.57	TR	-0.2	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	2	11.15	TR	-0.4	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	3	16.72	TR	-0.6	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	4	22.30	TR	-0.7	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	5	27.87	TR	-0.8	A B B^C^C D E E^	TR	3.9	A B B^C^C D
1	6	33.45	TR	-0.9	A B B^C^C D E E^	TR	4.3	A B B^C^C
1	7	39.03	TR	-1.0	A B B^C^C D E E^	TR	4.7	A B B^C^C
1	8	44.60	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C
1	9	50.18	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C
1	10	55.75	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C
1	11	61.33	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C
1	12	66.90	TR	-1.0	A B B^C^C D E E^	TR	4.8	A B B^C^C
1	13	72.47	TR	-1.0	A B B^C^C D E E^	TR	4.7	A B B^C^C
1	14	78.05	TR	-0.9	A B B^C^C D E E^	TR	4.3	A B B^C^C
1	15	83.62	TR	-0.8	A B B^C^C D E E^	TR	3.9	A B B^C^C D
1	16	89.20	TR	-0.7	A B B^C^C D E E^	TR	3.4	A B B^C^C D E
1	17	94.77	TR	-0.6	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	18	100.35	TR	-0.4	A B B^C^C D E E^	TR	1.9	A B B^C^C D E E^
1	19	105.92	TR	-0.2	A B B^C^C D E E^	TR	1.0	A B B^C^C D E E^
1	20	111.50	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; LRF Fatigue II Limit State with 0.75 load factor.  
 Design for Finite Life w/ ADTT Single Lane= 1012 & No. of Cycles= 27703500  
 \* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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

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

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

\*\*\*\*\*

AASHTO E6.10.10.1.2 AASHTO LRFD Eqn. SHEAR CONN. MAX. ALLOW.  
 SP IN D FROM ----- 6.10.10.2-1 PER TRANS. SHEAR CONN.

NO		L SUPT		Vf	Q/I	Vsr	5799 Ext Left.res		SECTION	---PITCH---	
		(ft)		[1]	[2]	[3]	[4], Zr	<---Input	[5], input	(in), [6]	
1	0	0.00	82.9	.14099E-01	1.2	TR	4.21	2	7.20		
1	1	5.57	80.1	.14099E-01	1.1	TR	4.21	2	7.46		
1	2	11.15	77.4	.14099E-01	1.1	TR	4.21	2	7.72		
1	3	16.72	75.1	.14099E-01	1.1	TR	4.21	2	7.95		
1	4	22.30	72.7	.14099E-01	1.0	TR	4.21	2	8.21		
1	5	27.87	69.8	.14099E-01	1.0	TR	4.21	2	8.56		
1	6	33.45	67.8	.14099E-01	1.0	TR	4.21	2	8.82		
1	7	39.03	67.1	.14099E-01	0.9	TR	4.21	2	8.90		
1	8	44.60	66.7	.14099E-01	0.9	TR	4.21	2	8.95		
1	9	50.18	66.7	.14099E-01	0.9	TR	4.21	2	8.95		
1	10	55.75	66.7	.14099E-01	0.9	TR	4.21	2	8.95		
1	11	61.33	66.7	.14099E-01	0.9	TR	4.21	2	8.95		
1	12	66.90	66.7	.14099E-01	0.9	TR	4.21	2	8.95		
1	13	72.47	67.1	.14099E-01	0.9	TR	4.21	2	8.90		
1	14	78.05	67.8	.14099E-01	1.0	TR	4.21	2	8.82		
1	15	83.62	69.8	.14099E-01	1.0	TR	4.21	2	8.56		
1	16	89.20	72.7	.14099E-01	1.0	TR	4.21	2	8.21		
1	17	94.77	75.1	.14099E-01	1.1	TR	4.21	2	7.95		
1	18	100.35	77.4	.14099E-01	1.1	TR	4.21	2	7.72		
1	19	105.92	80.1	.14099E-01	1.1	TR	4.21	2	7.46		
1	20	111.50	82.9	.14099E-01	1.2	TR	4.21	2	7.20		

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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 =  $Vf \cdot Q/I$  (Eq. 6.10.10.1.2-3)

Ffat = radial fatigue shear range per unit length

=  $Ffat^2 = Frc/w$  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= 1012  
\*\*\* 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	IN	D FROM	AASHTO E6. 10. 10. 1. 2			AASHTO LRFD Eqn.		SHEAR CONN.	MAX. ALLOW.
NO	NO	L SUPT	Vf	Q/I	Vsr	6. 10. 10. 2-2	4], Zr <--Input	PER TRANS. SECTION	SHEAR CONN.
		(ft)	[1]	[2]	[3]			[5], input	PI TCH--
									(in), [6]
1	0	0.00	41.5	.14099E-01	0.6	TR	2.03	2	6.93
1	1	5.57	40.0	.14099E-01	0.6	TR	2.03	2	7.18
1	2	11.15	38.7	.14099E-01	0.5	TR	2.03	2	7.42
1	3	16.72	37.6	.14099E-01	0.5	TR	2.03	2	7.65
1	4	22.30	36.4	.14099E-01	0.5	TR	2.03	2	7.90
1	5	27.87	34.9	.14099E-01	0.5	TR	2.03	2	8.24
1	6	33.45	33.9	.14099E-01	0.5	TR	2.03	2	8.48
1	7	39.03	33.6	.14099E-01	0.5	TR	2.03	2	8.56
1	8	44.60	33.4	.14099E-01	0.5	TR	2.03	2	8.61
1	9	50.18	33.4	.14099E-01	0.5	TR	2.03	2	8.61
1	10	55.75	33.4	.14099E-01	0.5	TR	2.03	2	8.61
1	11	61.33	33.4	.14099E-01	0.5	TR	2.03	2	8.61
1	12	66.90	33.4	.14099E-01	0.5	TR	2.03	2	8.61
1	13	72.47	33.6	.14099E-01	0.5	TR	2.03	2	8.56
1	14	78.05	33.9	.14099E-01	0.5	TR	2.03	2	8.48
1	15	83.62	34.9	.14099E-01	0.5	TR	2.03	2	8.24
1	16	89.20	36.4	.14099E-01	0.5	TR	2.03	2	7.90
1	17	94.77	37.6	.14099E-01	0.5	TR	2.03	2	7.65
1	18	100.35	38.7	.14099E-01	0.5	TR	2.03	2	7.42
1	19	105.92	40.0	.14099E-01	0.6	TR	2.03	2	7.18
1	20	111.50	41.5	.14099E-01	0.6	TR	2.03	2	6.93

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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 =  $Vf \cdot Q/I$  (Eq. 6. 10. 10. 1. 2-3)

Ffat = radial fatigue shear range per unit length

=  $Ffat^2 = Frc/w$  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= 1012 & No. of Cycles= 27703500  
\*\*\* If ADTT Single Lane greater than or equal to 960, refer  
to Fatigue I Table.

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TABLE 1.2.22.24A=SHEAR CONNECTOR (STRENGTH LIMIT STATE)

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT REGION			NO OF SHEAR CONNECTOR N(PER ZONE) <=> FATIG. CRI.	SEE NOTE		STATUS
				1=POS. 0=NEG.	(k) LRFD	(k) 6.10.10.4.2		N1	N2	
1	10	55.75	1	2877.0	2689.1		192	> 103		BLANK=OK CHECK=**
1	20	111.50	1	2877.0	2689.1		192	> 103		

-----  
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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 :

$$N1 (N2) = P / Q_r \quad \text{--- (Eq. 6.10.10.4.1-2)}$$

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

Q<sub>r</sub> = factored shear resistance of one shear connector  
= (Phi)<sub>sc</sub> Q<sub>n</sub> --- (Eq. 6.10.10.4.1-1)

Q<sub>n</sub> = nominal shear resistance

(Phi)<sub>sc</sub> = 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:

$$P1p = 0.85 f'_c b_s t_s \quad \text{--- (Eq. 6.10.10.4.2-2)}$$

or

$$P2p, 1n = F_y w D t_w + F_y t b_{ft} t_{ft} + F_y c b_{fc} t_{fc} \quad \text{(Eq. 6.10.10.4.2-3, 7)}$$

Where

f'<sub>c</sub> = specified 28-day compressive strength of the concrete

b<sub>s</sub> = effective width of the slab

b<sub>fc</sub> = width of compression flange

b<sub>ft</sub> = width of tension flange

t<sub>s</sub> = slab thickness

F<sub>yw</sub> = specified minimum yield strength of the web

F<sub>yt</sub> = specified minimum yield strength of the tension flange

F<sub>yc</sub> = specified minimum yield strength of the compression flange

D = web depth

t<sub>ft</sub> = thickness of tension flange

t<sub>fc</sub> = 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:

$$P2n = 0.45 f'c bs ts - - - - - \text{ (Eq. 6.10.10.4.2-8)}$$

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=RECOMMENDED SHEAR CONNECTOR REQUIRED PITCH  
\*\*\*\*\*

SPAN NO.	CURRENT FROM (ft)	SPAN TO (ft)	REQUIRED PITCH (in)
1	0.000	55.750	7.000
1	55.750	111.500	7.000

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

SPAN NO.	CURRENT FROM (ft)	SPAN TO (ft)	T C R
1	0.0	111.5	C

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

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TABLE 1.2.30.1=CODE CHECK STATUS SUMMARY  
\*\*\*\*\*

STATUS	TABLE NO.
OK	1.2.8.2=MAXIMUM LIVE LOAD DEFLECTION FOR COMPOSITE CONSTRUCTION
OK	1.2.22.4=DEPTH RATIOS
NG	1.2.22.10=CONSTRUCTIBILITY CHECK
OK	1.2.22.14=STRENGTH LIMIT STATE CHECK
OK	1.2.22.17=TRANSVERSE STIFFENER SPACING
OK	1.2.22.18=STATUS OF TRANSVERSE STIFFENERS

OK 1.2.22.19=SHEAR CAPACITY CHECK  
 NG 1.2.22.20=SECTION PROPERTY CHECK OF TRANSVERSE STIFFENERS  
 OK 1.2.22.21=SERVICE LIMIT STATE CHECK

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)  
 1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)  
 1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)  
 1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

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TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
					D1; k-ft ksi	D2	MAX. MOMENT					
							POS. MAX. STRESS	NEG. STRESS				
1	0	0.00		48.06 TOP	0.00	0.00	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	0.00	0.00	0.00	9.99	HL-93	9.99	2	
1	1	5.57		48.06 TOP	-4.69	-0.51	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	3.44	2.51	0.00	9.99	HL-93	9.97	2	
1	2	11.15		48.06 TOP	-8.89	-0.97	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2	
1	3	16.72		48.06 TOP	-12.60	-1.37	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2	
1	4	22.30		48.06 TOP	-15.82	-1.70	0.00	9.99	HL-93	9.98	2	
				48.06 BOT	11.60	8.34	0.00	2.98	HL-93	2.40	2	
1	5	27.87		48.06 TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2	
				48.06 BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2	
1	6	33.45		48.06 TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2	
				48.06 BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2	
1	7	39.03		48.06 TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2	
				48.06 BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2	
1	8	44.60		48.06 TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2	
				48.06 BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2	
1	9	50.18		48.06 TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2	
				48.06 BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2	
1	10	55.75		48.06 TOP	-24.72	-2.59	0.00	4.91	HL-93	3.79	2	
				48.06 BOT	18.12	12.67	0.00	1.49	HL-93	1.15	2	
1	11	61.33		48.06 TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2	
				48.06 BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2	
1	12	66.90		48.06 TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2	
				48.06 BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2	
1	13	72.47		48.06 TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2	
				48.06 BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2	
1	14	78.05		48.06 TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2	
				48.06 BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2	
1	15	83.62		48.06 TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2	
				48.06 BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2	
1	16	89.20		48.06 TOP	-15.82	-1.70	0.00	9.99	HL-93	9.48	2	
				48.06 BOT	11.60	8.34	0.00	2.98	HL-93	2.30	2	
1	17	94.77		48.06 TOP	-12.60	-1.37	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2	
1	18	100.35		48.06 TOP	-8.89	-0.97	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2	
1	19	105.92		48.06 TOP	-4.69	-0.51	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	3.44	2.51	0.00	9.99	HL-93	9.97	2	
1	20	111.50		48.06 TOP	0.00	0.00	0.00	9.99	HL-93	9.99	2	
				48.06 BOT	0.00	0.00	0.00	9.99	HL-93	9.99	2	

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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  

$$RF = (Mu - ENS1 * (GDC * D1 * S2 / SO + GDW * D2 * S2 / S1)) / (ENS1 * GLLST1 * (L + I))$$
 WHERE SO, 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---		UNFACTORED L+I STRESS				OPERAT. RATING FACTOR [3]	INVENT.	
				STEEL TOP; ksi; BOT	STEEL TOP; ksi; BOT	MAX. POS. ---STEEL---	MAX. NEG. ---STEEL---	MAX. POS. TOP; ksi; BOT	MAX. NEG. TOP; ksi; BOT		LIVE LOAD TYPE	RATING FACTOR
1	0	0.00	47.5	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68
1	1	5.57	47.5	-4.64	3.34	-0.51	2.51	0.00	0.00	9.99	HL-93	7.68
1	2	11.15	47.5	-8.79	6.34	-0.97	4.73	0.00	0.00	8.70	HL-93	6.69
1	3	16.72	47.5	-12.46	8.98	-1.37	6.68	0.00	0.00	5.77	HL-93	4.44
1	4	22.30	47.5	-15.64	11.27	-1.70	8.34	0.00	0.00	4.35	HL-93	3.34
1	5	27.87	47.5	-18.32	13.21	-1.98	9.70	0.00	0.00	3.53	HL-93	2.72
1	6	33.45	47.5	-20.52	14.79	-2.21	10.81	0.00	0.00	3.02	HL-93	2.33
1	7	39.03	47.5	-22.23	16.02	-2.39	11.68	0.00	0.00	2.70	HL-93	2.07
1	8	44.60	47.5	-23.46	16.91	-2.51	12.28	0.00	0.00	2.49	HL-93	1.92
1	9	50.18	47.5	-24.19	17.44	-2.57	12.57	0.00	0.00	2.39	HL-93	1.84
1	10	55.75	47.5	-24.44	17.61	-2.59	12.67	0.00	0.00	2.36	HL-93	1.81
1	11	61.33	47.5	-24.19	17.44	-2.57	12.57	0.00	0.00	2.39	HL-93	1.84
1	12	66.90	47.5	-23.46	16.91	-2.51	12.28	0.00	0.00	2.49	HL-93	1.92
1	13	72.47	47.5	-22.23	16.02	-2.39	11.68	0.00	0.00	2.70	HL-93	2.07
1	14	78.05	47.5	-20.52	14.79	-2.21	10.81	0.00	0.00	3.02	HL-93	2.33
1	15	83.62	47.5	-18.32	13.21	-1.98	9.70	0.00	0.00	3.53	HL-93	2.72
1	16	89.20	47.5	-15.64	11.27	-1.70	8.34	0.00	0.00	4.35	HL-93	3.34
1	17	94.77	47.5	-12.46	8.98	-1.37	6.68	0.00	0.00	5.77	HL-93	4.44
1	18	100.35	47.5	-8.79	6.34	-0.97	4.73	0.00	0.00	8.70	HL-93	6.69
1	19	105.92	47.5	-4.64	3.34	-0.51	2.51	0.00	0.00	9.99	HL-93	7.68
1	20	111.50	47.5	0.00	0.00	0.00	0.00	0.00	0.00	9.99	HL-93	7.68

-----  
 Please read NOTES on the following page

1

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MERLIN V 10.4  
COMPOSITE  
LRF -- 2010  
RATING

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\Design PAGE 88\5799\Rating\MD\5

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

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

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

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

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

1

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COMPOSITE  
LRF -- 2010  
RATING

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesignPAGE 89\5799\Rating\MD\5

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

NO.	SPAN LENGTH (ft)	CRI TICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	111.50	55.75	STRENGTH L. S.	1.49	1.15	HL-93
1	111.50	55.75	SERVICEABILITY	2.36	1.81	HL-93

NOTE: Control Rating Factor for Current Bridge:  
STRENGTH L. S.

Operating= 1.486

Inventory= 1.146

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

NO.	SPAN LENGTH (ft)	CRI TICAL LOCATION D FROM L SUPT (ft)	GOVERNING RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	111.50	55.75	STRENGTH L. S.	1.93	2619.45
1	111.50	55.75	SERVICEABILITY	3.05	

NOTE: control Rating Factor for Current Bridge:  
STRENGTH L. S. with Live Load Factor =1.660 & ADTT = 1190  
Legal Load Rating= 1.929

1

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MERLIN V 10.4  
COMPOSITE  
LRF -- 2010  
RATING

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesignPAGE 90\5799\Rating\MD\5

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

SP IN D FROM NO NO L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
		D1; kips	D2	MAX. POS.	SHEAR NEG.			
1 0 0.00	602.9	76.2	25.0	107.6	0.0	3.25	HL-93	2.51
1 1 5.57	481.5	68.6	22.5	100.4	-2.3	2.68	HL-93	2.07
1 2 11.15	449.4	61.0	20.0	93.4	-5.1	2.73	HL-93	2.11
1 3 16.72	422.3	53.4	17.5	86.5	-8.4	2.83	HL-93	2.18
1 4 22.30	422.3	45.9	15.0	79.7	-11.8	3.19	HL-93	2.46
1 5 27.87	422.3	38.1	12.5	73.1	-16.0	3.61	HL-93	2.79

5799 Ext Left.res										
1	6	33.45	422.3	30.5	10.0	66.7	-20.9	4.11	HL-93	3.17
1	7	39.03	422.3	22.9	7.5	60.4	-26.0	4.69	HL-93	3.62
1	8	44.60	422.3	15.4	5.0	54.3	-31.4	5.40	HL-93	4.17
1	9	50.18	304.9	7.6	2.5	48.3	-36.9	4.47	HL-93	3.45
1	10	55.75	304.9	0.0	0.0	42.5	-42.5	5.31	HL-93	4.10
1	11	61.33	304.9	-7.6	-2.5	36.9	-48.3	4.47	HL-93	3.45
1	12	66.90	422.3	-15.4	-5.0	31.4	-54.3	5.40	HL-93	4.17
1	13	72.47	422.3	-22.9	-7.5	26.0	-60.4	4.69	HL-93	3.62
1	14	78.05	422.3	-30.5	-10.0	20.9	-66.7	4.11	HL-93	3.17
1	15	83.62	422.3	-38.1	-12.5	16.0	-73.1	3.61	HL-93	2.79
1	16	89.20	422.3	-45.9	-15.0	11.8	-79.7	3.19	HL-93	2.46
1	17	94.77	422.3	-53.4	-17.5	8.4	-86.5	2.83	HL-93	2.18
1	18	100.35	449.4	-61.0	-20.0	5.1	-93.4	2.73	HL-93	2.11
1	19	105.92	481.5	-68.6	-22.5	2.3	-100.4	2.68	HL-93	2.07
1	20	111.50	602.9	-76.2	-25.0	0.0	-107.6	3.25	HL-93	2.51

1

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COMPOSITE  
LRF -- 2010  
RATING

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TABLE 1.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK

SPAN		CRITICAL LOCATION		GOVERNING	OPERATING		GOVERNING
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION	RATING	INVENTORY RATING	LIVE LOAD TYPE
1	111.50	105.92		STRENGTH L. S.	2.68	2.07	HL-93

NOTE: Control Rating Factor for Current Bridge:

STRENGTH L. S.

Operating= 2.682

Inventory= 2.069

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

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD	MAXIMUM
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION	RATING	LL SHEAR (kips)
1	111.50	5.57		STRENGTH L. S.	3.34	108.97

NOTE: control Rating Factor for Current Bridge:

STRENGTH L. S. with Live Load Factor =1.660 & ADTT = 1190

Legal Load Rating= 3.336

1

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COMPOSITE  
LRF -- 2010  
RATING

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesignPAGE 92\5799\Rating\MD\5

TABLE 1.2.34.1=RATING; LIVE LOAD DEFLECTION

SPAN		NUMBER OF LANE		LL + I.	GOVERN.	RATING FACTOR FOR LL DEFL.
NO.	D FROM L	AND	DEFLECTION	LOAD		
	SUPT	DIST.	(inch)	TYPE		
	(ft)	FOR LL DEFL.				RF = L/(800 * LL DEFL.)
1	55.75	4	0.433	-0.48	HL-93	3.47
		5	0.542	-0.60		2.77

NOTE: [1] "-" indicates downward deflection

5799 Ext Left.res

[2]  $RF * (\text{live load defl.}) = \text{span length}/800$ , therefore

$RF = L/(800 * LL \text{ Defl.})$ , if = 99, No rating

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 6\5799\Rating\Rati  
5799 Int Left\_19 ev2

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]

this note not valid  
because no  
transitions?

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 1.1.3.1A=BRIDGE SPACING AND EFFECTIVE WIDTH

\*\*\*\*\*

## SPAN SPACING EFF. WIDTH

NO. (ft) (in)

-----

1 9.92 119.00

-----

## SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2

\*\*\*\*\*

NO. OF AXLES = 2

AXLE SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(k) (ft) (k) (ft) (k) (ft) (k) (ft)

-----

24.00 15.00 33.50 0.00

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 35\5799\Rating\Rati

TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

\*\*\*\*\*

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I , (K), LOAD TYPE = HL - 93				TOTAL DL+LL+I (L R F D)		
		MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	MINIMUM	MAXIMUM	
1	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47
2	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47

NOTE: [1] " - " Indicates Uplift

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

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

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	55.75	4	0.667	-0.730 MAX HL-93 0.028 MIN -0.354 MAX LANE 0.028 MIN LANE	1.67	0.00191
		5	0.833	-0.913 MAX HL-93 0.036 MIN -0.443 MAX LANE 0.036 MIN LANE	1.67	0.00191
		4	0.667	-0.668 MAX FA 0.000 MIN FA		
		4	0.667	-0.591 MAX EV2 0.028 MIN EV2		

NOTE: [1] " - " indicates downward deflection

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

$$DF = (NL/Ng) \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 averaging.



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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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
					MOMENT		MAX. MOMENT					
					D1; k-ft	D2 ksi	POS. MAX. STRESS	NEG. MAX. STRESS				
1	0	0.00	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2	
			48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2	
1	1	5.57	48.06	TOP	-4.94	-0.43	0.00	72.23	HL-93	55.72	2	
			48.06	BOT	3.60	2.27	0.00	14.20	HL-93	10.95	2	
1	2	11.15	48.06	TOP	-9.36	-0.81	0.00	33.22	HL-93	25.63	2	
			48.06	BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	3	16.72	48.06	TOP	-13.27	-1.14	0.00	20.38	HL-93	15.73	2	
			48.06	BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	4	22.30	48.06	TOP	-16.66	-1.43	0.00	14.12	HL-93	10.90	2	
			48.06	BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	5	27.87	48.06	TOP	-19.52	-1.66	0.00	10.54	HL-93	8.13	2	
			48.06	BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	6	33.45	48.06	TOP	-21.86	-1.85	0.00	8.29	HL-93	6.39	2	
			48.06	BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	7	39.03	48.06	TOP	-23.69	-2.00	0.00	6.83	HL-93	5.27	2	
			48.06	BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	8	44.60	48.06	TOP	-25.00	-2.10	0.00	5.92	HL-93	4.57	2	
			48.06	BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	9	50.18	48.05	TOP	-25.78	-2.15	0.00	5.44	HL-93	4.20	2	
			48.06	BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	10	55.75	48.05	TOP	-26.04	-2.17	0.00	5.29	HL-93	4.08	2	
			48.06	BOT	19.00	11.48	0.00	1.57	HL-93	1.21	2	
1	11	61.33	48.05	TOP	-25.78	-2.15	0.00	5.44	HL-93	4.20	2	
			48.06	BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	12	66.90	48.06	TOP	-25.00	-2.10	0.00	5.92	HL-93	4.57	2	
			48.06	BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	13	72.47	48.06	TOP	-23.69	-2.00	0.00	6.83	HL-93	5.27	2	
			48.06	BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	14	78.05	48.06	TOP	-21.86	-1.85	0.00	8.29	HL-93	6.39	2	
			48.06	BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	15	83.62	48.06	TOP	-19.52	-1.66	0.00	10.54	HL-93	8.13	2	
			48.06	BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	16	89.20	48.06	TOP	-16.66	-1.43	0.00	14.12	HL-93	10.90	2	
			48.06	BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	17	94.77	48.06	TOP	-13.27	-1.14	0.00	20.38	HL-93	15.73	2	
			48.06	BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	18	100.35	48.06	TOP	-9.36	-0.81	0.00	33.22	HL-93	25.63	2	
			48.06	BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	19	105.92	48.06	TOP	-4.94	-0.43	0.00	72.23	HL-93	55.72	2	
			48.06	BOT	3.60	2.27	0.00	14.20	HL-93	10.95	2	
1	20	111.50	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2	
			48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2	

Please read NOTES on the following page

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 89\5799\Rating\Rati

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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	55.75		STRENGTH L.S.	2.82	1815.55
1	111.50	55.75		SERVICEABILITY	3.38	

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 & ADTT = 1190

Legal Load Rating= 2.817

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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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
					SHEAR D1; kips	D2	MAX. POS.	SHEAR NEG.			
1	0		0.00	602.9	81.0	25.0	119.6	0.0	2.89	HL-93	2.23
1	1		5.57	481.5	72.9	22.5	111.6	-2.6	2.38	HL-93	1.83
1	2		11.15	449.4	64.9	20.0	103.7	-5.6	2.42	HL-93	1.87
1	3		16.72	422.3	56.9	17.5	96.1	-9.3	2.51	HL-93	1.94
1	4		22.30	422.3	48.9	15.0	88.6	-13.1	2.84	HL-93	2.19
1	5		27.87	422.3	40.5	12.5	81.3	-17.8	3.22	HL-93	2.49
1	6		33.45	422.3	32.5	10.0	74.1	-23.2	3.67	HL-93	2.83
1	7		39.03	422.3	24.4	7.5	67.1	-28.9	4.20	HL-93	3.24
1	8		44.60	422.3	16.4	5.0	60.3	-34.9	4.84	HL-93	3.74
1	9		50.18	304.9	8.0	2.5	53.7	-41.0	4.02	HL-93	3.10
1	10		55.75	304.9	0.0	0.0	47.3	-47.3	4.78	HL-93	3.69
1	11		61.33	304.9	-8.0	-2.5	41.0	-53.7	4.02	HL-93	3.10
1	12		66.90	422.3	-16.4	-5.0	34.9	-60.3	4.84	HL-93	3.74
1	13		72.47	422.3	-24.4	-7.5	28.9	-67.1	4.20	HL-93	3.24
1	14		78.05	422.3	-32.5	-10.0	23.2	-74.1	3.67	HL-93	2.83
1	15		83.62	422.3	-40.5	-12.5	17.8	-81.3	3.22	HL-93	2.49
1	16		89.20	422.3	-48.9	-15.0	13.1	-88.6	2.84	HL-93	2.19
1	17		94.77	422.3	-56.9	-17.5	9.3	-96.1	2.51	HL-93	1.94
1	18		100.35	449.4	-64.9	-20.0	5.6	-103.7	2.42	HL-93	1.87
1	19		105.92	481.5	-72.9	-22.5	2.6	-111.6	2.38	HL-93	1.83
1	20		111.50	602.9	-81.0	-25.0	0.0	-119.6	2.89	HL-93	2.23

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	5.57		STRENGTH L.S.	4.14	86.57

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 4.137

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 6\5799\Rating\Rati  
5799 Int Left\_19 ev3

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 SPACING EFF. WIDTH

NO. (ft) (in)

-----

1 9.92 119.00

-----

## SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3

\*\*\*\*\*

NO. OF AXLES = 3

AXLE SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT 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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT 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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(k) (ft) (k) (ft) (k) (ft) (k) (ft)

-----

24.00 15.00 31.00 4.00 31.00 0.00

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 35\5799\Rating\Rati

TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

\*\*\*\*\*

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I , (K), LOAD TYPE = HL - 93				TOTAL DL+LL+I (L R F D)		
		MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	MINIMUM	MAXIMUM	
1	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47
2	106.00	0.00	HL-93	119.59	HL-93	ST1	90.93	346.25
						ST2	90.93	298.42
						ST4	132.16	159.00
						SE1	106.00	225.59
						SE2	106.00	261.47

NOTE: [1] " - " Indicates Uplift

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



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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 ROTATION AASHTO 2.5.2.6.2 [5] Rad.
1	55.75	4 0.667	-0.730 MAX 0.028 MIN -0.354 MAX LANE 0.028 MIN LANE	HL-93	1.67 0.00191
		5 0.833	-0.913 MAX 0.036 MIN -0.443 MAX LANE 0.036 MIN LANE	HL-93	1.67 0.00191
		4 0.667	-0.668 MAX 0.000 MIN	FA	
		4 0.667	-0.883 MAX 0.028 MIN	EV3	

NOTE: [1] " - " indicates downward deflection

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

$$DF = (NL/Ng) \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 averaging.

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 88\5799\Rating\Rati

TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUCT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. C RATING FACTOR	A T
					MOMENT		MAX. MOMENT					
					D1; k-ft	D2	POS. MAX. STRESS	NEG. MAX. STRESS				
1	0		0.00	48.06 TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2	
				48.06 BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2	
1	1		5.57	48.06 TOP	-4.94	-0.43	0.00	72.23	HL-93	55.72	2	
				48.06 BOT	3.60	2.27	0.00	14.20	HL-93	10.95	2	
1	2		11.15	48.06 TOP	-9.36	-0.81	0.00	33.22	HL-93	25.63	2	
				48.06 BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	3		16.72	48.06 TOP	-13.27	-1.14	0.00	20.38	HL-93	15.73	2	
				48.06 BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	4		22.30	48.06 TOP	-16.66	-1.43	0.00	14.12	HL-93	10.90	2	
				48.06 BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	5		27.87	48.06 TOP	-19.52	-1.66	0.00	10.54	HL-93	8.13	2	
				48.06 BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	6		33.45	48.06 TOP	-21.86	-1.85	0.00	8.29	HL-93	6.39	2	
				48.06 BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	7		39.03	48.06 TOP	-23.69	-2.00	0.00	6.83	HL-93	5.27	2	
				48.06 BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	8		44.60	48.06 TOP	-25.00	-2.10	0.00	5.92	HL-93	4.57	2	
				48.06 BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	9		50.18	48.05 TOP	-25.78	-2.15	0.00	5.44	HL-93	4.20	2	
				48.06 BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	10		55.75	48.05 TOP	-26.04	-2.17	0.00	5.29	HL-93	4.08	2	
				48.06 BOT	19.00	11.48	0.00	1.57	HL-93	1.21	2	
1	11		61.33	48.05 TOP	-25.78	-2.15	0.00	5.44	HL-93	4.20	2	
				48.06 BOT	18.81	11.40	0.00	1.60	HL-93	1.23	2	
1	12		66.90	48.06 TOP	-25.00	-2.10	0.00	5.92	HL-93	4.57	2	
				48.06 BOT	18.24	11.13	0.00	1.68	HL-93	1.30	2	
1	13		72.47	48.06 TOP	-23.69	-2.00	0.00	6.83	HL-93	5.27	2	
				48.06 BOT	17.29	10.59	0.00	1.85	HL-93	1.43	2	
1	14		78.05	48.06 TOP	-21.86	-1.85	0.00	8.29	HL-93	6.39	2	
				48.06 BOT	15.95	9.80	0.00	2.12	HL-93	1.64	2	
1	15		83.62	48.06 TOP	-19.52	-1.66	0.00	10.54	HL-93	8.13	2	
				48.06 BOT	14.25	8.79	0.00	2.55	HL-93	1.97	2	
1	16		89.20	48.06 TOP	-16.66	-1.43	0.00	14.12	HL-93	10.90	2	
				48.06 BOT	12.16	7.56	0.00	3.22	HL-93	2.48	2	
1	17		94.77	48.06 TOP	-13.27	-1.14	0.00	20.38	HL-93	15.73	2	
				48.06 BOT	9.68	6.05	0.00	4.40	HL-93	3.40	2	
1	18		100.35	48.06 TOP	-9.36	-0.81	0.00	33.22	HL-93	25.63	2	
				48.06 BOT	6.83	4.29	0.00	6.82	HL-93	5.26	2	
1	19		105.92	48.06 TOP	-4.94	-0.43	0.00	72.23	HL-93	55.72	2	
				48.06 BOT	3.60	2.27	0.00	14.20	HL-93	10.95	2	
1	20		111.50	48.06 TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2	
				48.06 BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2	

Please read NOTES on the following page

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 89\5799\Rating\Rati

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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	55.75		STRENGTH L.S.	1.86	2750.15
1	111.50	55.75		SERVICEABILITY	2.23	

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 1.860

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 93\5799\Rating\Rati

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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
					SHEAR D1; kips	D2	MAX. POS.	SHEAR NEG.			
1	0		0.00	602.9	81.0	25.0	119.6	0.0	2.89	HL-93	2.23
1	1		5.57	481.5	72.9	22.5	111.6	-2.6	2.38	HL-93	1.83
1	2		11.15	449.4	64.9	20.0	103.7	-5.6	2.42	HL-93	1.87
1	3		16.72	422.3	56.9	17.5	96.1	-9.3	2.51	HL-93	1.94
1	4		22.30	422.3	48.9	15.0	88.6	-13.1	2.84	HL-93	2.19
1	5		27.87	422.3	40.5	12.5	81.3	-17.8	3.22	HL-93	2.49
1	6		33.45	422.3	32.5	10.0	74.1	-23.2	3.67	HL-93	2.83
1	7		39.03	422.3	24.4	7.5	67.1	-28.9	4.20	HL-93	3.24
1	8		44.60	422.3	16.4	5.0	60.3	-34.9	4.84	HL-93	3.74
1	9		50.18	304.9	8.0	2.5	53.7	-41.0	4.02	HL-93	3.10
1	10		55.75	304.9	0.0	0.0	47.3	-47.3	4.78	HL-93	3.69
1	11		61.33	304.9	-8.0	-2.5	41.0	-53.7	4.02	HL-93	3.10
1	12		66.90	422.3	-16.4	-5.0	34.9	-60.3	4.84	HL-93	3.74
1	13		72.47	422.3	-24.4	-7.5	28.9	-67.1	4.20	HL-93	3.24
1	14		78.05	422.3	-32.5	-10.0	23.2	-74.1	3.67	HL-93	2.83
1	15		83.62	422.3	-40.5	-12.5	17.8	-81.3	3.22	HL-93	2.49
1	16		89.20	422.3	-48.9	-15.0	13.1	-88.6	2.84	HL-93	2.19
1	17		94.77	422.3	-56.9	-17.5	9.3	-96.1	2.51	HL-93	1.94
1	18		100.35	449.4	-64.9	-20.0	5.6	-103.7	2.42	HL-93	1.87
1	19		105.92	481.5	-72.9	-22.5	2.6	-111.6	2.38	HL-93	1.83
1	20		111.50	602.9	-81.0	-25.0	0.0	-119.6	2.89	HL-93	2.23

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
			(ft)			
1	111.50	5.57		STRENGTH L.S.	2.78	128.85

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 2.779

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 6\5799\Rating\Rati  
5799 Ext Left\_19 ev2

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 SPACING EFF. WIDTH

NO. (ft) (in)

-----

1 9.92 111.01

-----

## SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV2

\*\*\*\*\*

NO. OF AXLES = 2

AXLE SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(k) (ft) (k) (ft) (k) (ft) (k) (ft)

-----

24.00 15.00 33.50 0.00



FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 35\5799\Rating\Rati

TABLE 1.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

\*\*\*\*\*

SUPT	TOTAL	LL+I , (K), LOAD TYPE = HL - 93				TOTAL DL+LL+I (L R F D)		
	NO. DEAD LOADS (K)	MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	MINIMUM	MAXIMUM	
1	101.20	0.00	HL-93	107.62	HL-93	ST1	86.61	319.31
						ST2	86.61	276.26
						ST4	124.96	151.80
						SE1	101.20	208.82
						SE2	101.20	241.11
2	101.20	0.00	HL-93	107.62	HL-93	ST1	86.61	319.31
						ST2	86.61	276.26
						ST4	124.96	151.80
						SE1	101.20	208.82
						SE2	101.20	241.11

NOTE: [1] " - " Indicates Uplift

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

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 38\5799\Rating\Rati

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

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	55.75	4	0.667	-0.742 MAX HL-93 0.029 MIN -0.360 MAX LANE 0.029 MIN LANE	1.67	0.00194
		5	0.833	-0.928 MAX HL-93 0.036 MIN -0.450 MAX LANE 0.036 MIN LANE	1.67	0.00194
		4	0.667	-0.679 MAX FA 0.000 MIN FA		
		4	0.667	-0.601 MAX EV2 0.029 MIN EV2		

NOTE: [1] " - " indicates downward deflection

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

$$DF = (NL/Ng) \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 averaging.

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 88\5799\Rating\Rati

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		UNFACTORED L+I MAX. MOMENT		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR	C A T
					D1; k-ft	D2	POS. MAX. STRESS	NEG. MAX. STRESS				
1	0		0.00	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2
				48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2
1	1		5.57	48.06	TOP	-4.69	-0.51	0.00	60.97	HL-93	47.03	2
				48.06	BOT	3.44	2.51	0.00	12.93	HL-93	9.97	2
1	2		11.15	48.06	TOP	-8.89	-0.97	0.00	28.28	HL-93	21.82	2
				48.06	BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2
1	3		16.72	48.06	TOP	-12.60	-1.37	0.00	17.53	HL-93	13.53	2
				48.06	BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2
1	4		22.30	48.06	TOP	-15.82	-1.70	0.00	12.29	HL-93	9.48	2
				48.06	BOT	11.60	8.34	0.00	2.98	HL-93	2.30	2
1	5		27.87	48.06	TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2
				48.06	BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2
1	6		33.45	48.06	TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2
				48.06	BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2
1	7		39.03	48.06	TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2
				48.06	BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2
1	8		44.60	48.06	TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2
				48.06	BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2
1	9		50.18	48.06	TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2
				48.06	BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2
1	10		55.75	48.06	TOP	-24.72	-2.59	0.00	4.91	HL-93	3.79	2
				48.06	BOT	18.12	12.67	0.00	1.49	HL-93	1.15	2
1	11		61.33	48.06	TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2
				48.06	BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2
1	12		66.90	48.06	TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2
				48.06	BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2
1	13		72.47	48.06	TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2
				48.06	BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2
1	14		78.05	48.06	TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2
				48.06	BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2
1	15		83.62	48.06	TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2
				48.06	BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2
1	16		89.20	48.06	TOP	-15.82	-1.70	0.00	12.29	HL-93	9.48	2
				48.06	BOT	11.60	8.34	0.00	2.98	HL-93	2.30	2
1	17		94.77	48.06	TOP	-12.60	-1.37	0.00	17.53	HL-93	13.53	2
				48.06	BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2
1	18		100.35	48.06	TOP	-8.89	-0.97	0.00	28.28	HL-93	21.82	2
				48.06	BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2
1	19		105.92	48.06	TOP	-4.69	-0.51	0.00	60.97	HL-93	47.03	2
				48.06	BOT	3.44	2.51	0.00	12.93	HL-93	9.97	2
1	20		111.50	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2
				48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2

Please read NOTES on the following page

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 89\5799\Rating\Rati

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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV2

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	55.75		STRENGTH L.S.	2.67	1996.12
1	111.50	55.75		SERVICEABILITY	3.16	

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 & ADTT = 1190

Legal Load Rating= 2.669

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 93\5799\Rating\Rati

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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
					SHEAR D1; kips	D2	MAX. POS.	SHEAR NEG.			
1	0		0.00	602.9	76.2	25.0	107.6	0.0	3.25	HL-93	2.51
1	1		5.57	481.5	68.6	22.5	100.4	-2.3	2.68	HL-93	2.07
1	2		11.15	449.4	61.0	20.0	93.4	-5.1	2.73	HL-93	2.11
1	3		16.72	422.3	53.4	17.5	86.5	-8.4	2.83	HL-93	2.18
1	4		22.30	422.3	45.9	15.0	79.7	-11.8	3.19	HL-93	2.46
1	5		27.87	422.3	38.1	12.5	73.1	-16.0	3.61	HL-93	2.79
1	6		33.45	422.3	30.5	10.0	66.7	-20.9	4.11	HL-93	3.17
1	7		39.03	422.3	22.9	7.5	60.4	-26.0	4.69	HL-93	3.62
1	8		44.60	422.3	15.4	5.0	54.3	-31.4	5.40	HL-93	4.17
1	9		50.18	304.9	7.6	2.5	48.3	-36.9	4.47	HL-93	3.45
1	10		55.75	304.9	0.0	0.0	42.5	-42.5	5.31	HL-93	4.10
1	11		61.33	304.9	-7.6	-2.5	36.9	-48.3	4.47	HL-93	3.45
1	12		66.90	422.3	-15.4	-5.0	31.4	-54.3	5.40	HL-93	4.17
1	13		72.47	422.3	-22.9	-7.5	26.0	-60.4	4.69	HL-93	3.62
1	14		78.05	422.3	-30.5	-10.0	20.9	-66.7	4.11	HL-93	3.17
1	15		83.62	422.3	-38.1	-12.5	16.0	-73.1	3.61	HL-93	2.79
1	16		89.20	422.3	-45.9	-15.0	11.8	-79.7	3.19	HL-93	2.46
1	17		94.77	422.3	-53.4	-17.5	8.4	-86.5	2.83	HL-93	2.18
1	18		100.35	449.4	-61.0	-20.0	5.1	-93.4	2.73	HL-93	2.11
1	19		105.92	481.5	-68.6	-22.5	2.3	-100.4	2.68	HL-93	2.07
1	20		111.50	602.9	-76.2	-25.0	0.0	-107.6	3.25	HL-93	2.51

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV2

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	5.57		STRENGTH L.S.	4.67	77.91

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 4.667

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 6\5799\Rating\Rati  
5799 Ext Left\_19 ev3

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 SPACING EFF. WIDTH

NO. (ft) (in)

-----

1 9.92 111.01

-----

## SPECIFIED VEHICLE DATA FROM TRUCK FILE --- EV3

\*\*\*\*\*

NO. OF AXLES = 3

AXLE SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT 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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT 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 SPACING AXLE SPACING AXLE SPACING AXLE SPACING

-----

WHT DIST WHT DIST WHT DIST WHT DIST

(k) (ft) (k) (ft) (k) (ft) (k) (ft)

-----

24.00 15.00 31.00 4.00 31.00 0.00

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 35\5799\Rating\Rati

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

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I , (K), LOAD TYPE = HL - 93				TOTAL DL+LL+I (L R F D)		
		MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	MINIMUM	MAXIMUM	
1	101.20	0.00	HL-93	107.62	HL-93	ST1	86.61	319.31
						ST2	86.61	276.26
						ST4	124.96	151.80
						SE1	101.20	208.82
						SE2	101.20	241.11
2	101.20	0.00	HL-93	107.62	HL-93	ST1	86.61	319.31
						ST2	86.61	276.26
						ST4	124.96	151.80
						SE1	101.20	208.82
						SE2	101.20	241.11

NOTE: [1] " - " Indicates Uplift

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

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 38\5799\Rating\Rati

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	55.75	4 0.667	-0.742 MAX 0.029 MIN -0.360 MAX LANE 0.029 MIN LANE	HL-93	1.67	0.00194
		5 0.833	-0.928 MAX 0.036 MIN -0.450 MAX LANE 0.036 MIN LANE	HL-93	1.67	0.00194
		4 0.667	-0.679 MAX 0.000 MIN	FA		
		4 0.667	-0.898 MAX 0.029 MIN	EV3		

NOTE: [1] " - " indicates downward deflection

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

$$DF = (NL/Ng) \dots \text{AASHTO LRFD Art. 2.5.2.6}$$

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 averaging.

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 88\5799\Rating\Rati

TABLE 1.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT

\*\*\*\*\*

SP NO	IN NO	D L	FROM SUPT	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT		UNFACTORED L+I MAX. MOMENT		OPERAT. RATING FACTOR	LIVE LOAD TYPE	INVENT. C RATING A FACTOR T	
					D1; k-ft	D2 ksi	POS. MAX. STRESS	NEG. MAX. STRESS				
1	0		0.00	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2
				48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2
1	1		5.57	48.06	TOP	-4.69	-0.51	0.00	60.97	HL-93	47.03	2
				48.06	BOT	3.44	2.51	0.00	12.93	HL-93	9.97	2
1	2		11.15	48.06	TOP	-8.89	-0.97	0.00	28.28	HL-93	21.82	2
				48.06	BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2
1	3		16.72	48.06	TOP	-12.60	-1.37	0.00	17.53	HL-93	13.53	2
				48.06	BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2
1	4		22.30	48.06	TOP	-15.82	-1.70	0.00	12.29	HL-93	9.48	2
				48.06	BOT	11.60	8.34	0.00	2.98	HL-93	2.30	2
1	5		27.87	48.06	TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2
				48.06	BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2
1	6		33.45	48.06	TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2
				48.06	BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2
1	7		39.03	48.06	TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2
				48.06	BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2
1	8		44.60	48.06	TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2
				48.06	BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2
1	9		50.18	48.06	TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2
				48.06	BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2
1	10		55.75	48.06	TOP	-24.72	-2.59	0.00	4.91	HL-93	3.79	2
				48.06	BOT	18.12	12.67	0.00	1.49	HL-93	1.15	2
1	11		61.33	48.06	TOP	-24.47	-2.57	0.00	5.04	HL-93	3.88	2
				48.06	BOT	17.94	12.57	0.00	1.51	HL-93	1.17	2
1	12		66.90	48.06	TOP	-23.73	-2.51	0.00	5.43	HL-93	4.19	2
				48.06	BOT	17.40	12.28	0.00	1.59	HL-93	1.22	2
1	13		72.47	48.06	TOP	-22.49	-2.39	0.00	6.19	HL-93	4.77	2
				48.06	BOT	16.49	11.68	0.00	1.74	HL-93	1.34	2
1	14		78.05	48.06	TOP	-20.76	-2.21	0.00	7.41	HL-93	5.72	2
				48.06	BOT	15.22	10.81	0.00	1.99	HL-93	1.53	2
1	15		83.62	48.06	TOP	-18.53	-1.98	0.00	9.30	HL-93	7.17	2
				48.06	BOT	13.59	9.70	0.00	2.37	HL-93	1.83	2
1	16		89.20	48.06	TOP	-15.82	-1.70	0.00	12.29	HL-93	9.48	2
				48.06	BOT	11.60	8.34	0.00	2.98	HL-93	2.30	2
1	17		94.77	48.06	TOP	-12.60	-1.37	0.00	17.53	HL-93	13.53	2
				48.06	BOT	9.24	6.68	0.00	4.05	HL-93	3.13	2
1	18		100.35	48.06	TOP	-8.89	-0.97	0.00	28.28	HL-93	21.82	2
				48.06	BOT	6.52	4.73	0.00	6.25	HL-93	4.82	2
1	19		105.92	48.06	TOP	-4.69	-0.51	0.00	60.97	HL-93	47.03	2
				48.06	BOT	3.44	2.51	0.00	12.93	HL-93	9.97	2
1	20		111.50	48.06	TOP	0.00	0.00	0.00	99.99	HL-93	99.99	2
				48.06	BOT	0.00	0.00	0.00	99.99	HL-93	99.99	2

Please read NOTES on the following page

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 89\5799\Rating\Rati

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 1.2.32.3D=BRIDGE MOMENT RATING INFORMATION FOR MAXIMUM TRUCK EV3

\*\*\*\*\*

SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
			(ft)			
1	111.50	55.75		STRENGTH L.S.	1.76	3023.67
1	111.50	55.75		SERVICEABILITY	2.08	

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 1.762

FILE NAME = I:\Projects\CKE410A4\_MaineDOT Bridge Load Ratings\DesigPAGE 93\5799\Rating\Rati

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		UNFACTORED L+I		OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
					SHEAR D1; kips	D2	MAX. POS.	SHEAR NEG.			
1	0		0.00	602.9	76.2	25.0	107.6	0.0	3.25	HL-93	2.51
1	1		5.57	481.5	68.6	22.5	100.4	-2.3	2.68	HL-93	2.07
1	2		11.15	449.4	61.0	20.0	93.4	-5.1	2.73	HL-93	2.11
1	3		16.72	422.3	53.4	17.5	86.5	-8.4	2.83	HL-93	2.18
1	4		22.30	422.3	45.9	15.0	79.7	-11.8	3.19	HL-93	2.46
1	5		27.87	422.3	38.1	12.5	73.1	-16.0	3.61	HL-93	2.79
1	6		33.45	422.3	30.5	10.0	66.7	-20.9	4.11	HL-93	3.17
1	7		39.03	422.3	22.9	7.5	60.4	-26.0	4.69	HL-93	3.62
1	8		44.60	422.3	15.4	5.0	54.3	-31.4	5.40	HL-93	4.17
1	9		50.18	304.9	7.6	2.5	48.3	-36.9	4.47	HL-93	3.45
1	10		55.75	304.9	0.0	0.0	42.5	-42.5	5.31	HL-93	4.10
1	11		61.33	304.9	-7.6	-2.5	36.9	-48.3	4.47	HL-93	3.45
1	12		66.90	422.3	-15.4	-5.0	31.4	-54.3	5.40	HL-93	4.17
1	13		72.47	422.3	-22.9	-7.5	26.0	-60.4	4.69	HL-93	3.62
1	14		78.05	422.3	-30.5	-10.0	20.9	-66.7	4.11	HL-93	3.17
1	15		83.62	422.3	-38.1	-12.5	16.0	-73.1	3.61	HL-93	2.79
1	16		89.20	422.3	-45.9	-15.0	11.8	-79.7	3.19	HL-93	2.46
1	17		94.77	422.3	-53.4	-17.5	8.4	-86.5	2.83	HL-93	2.18
1	18		100.35	449.4	-61.0	-20.0	5.1	-93.4	2.73	HL-93	2.11
1	19		105.92	481.5	-68.6	-22.5	2.3	-100.4	2.68	HL-93	2.07
1	20		111.50	602.9	-76.2	-25.0	0.0	-107.6	3.25	HL-93	2.51

TABLE 1.2.33.3D=BRIDGE SHEAR RATING INFORMATION FOR MAXIMUM TRUCK EV3

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SPAN		CRITICAL LOCATION		GOVERNING	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
NO.	LENGTH (ft)	D FROM L	SUPT	RATING CRITERION		
		(ft)				
1	111.50	105.92		STRENGTH L.S.	3.14	115.95

NOTE: control Rating Factor for Current Bridge:

STRENGTH L.S. with Live Load Factor =1.307 &amp; ADTT = 1190

Legal Load Rating= 3.135



For MathCAD input of Rigid Body Rotation at the two points of interest:

**(1) Determine CG of girders at mid-span**

	Spacing	Dist from G6	$x_n$
G6	0	0	24.393
G5	9.917	9.917	14.477
G4	9.917	19.833	4.560
G3	9.917	29.750	-5.357
G2	9.917	39.667	-15.273
G1	7.526	47.193	-22.799

146.359

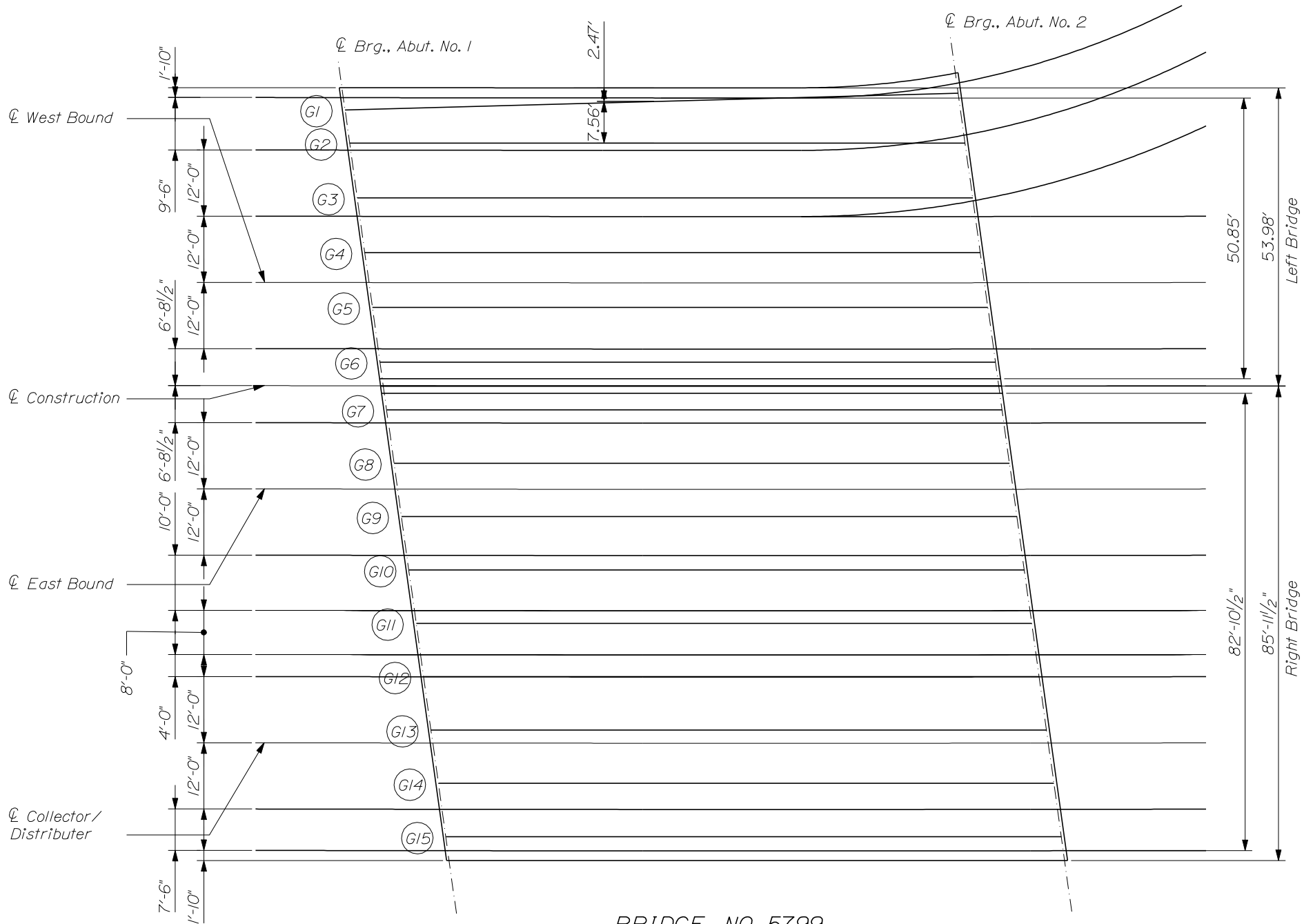
y cg = 24.393

**(2) Determine CG of girders at Abutment No. 2**

	Spacing	Dist from G6	$x_n$
G6	0	0	24.644
G5	9.917	9.917	14.727
G4	9.917	19.833	4.811
G3	9.917	29.750	-5.106
G2	9.917	39.667	-15.023
G1	9.031	48.698	-24.054

147.865

y cg = 24.644

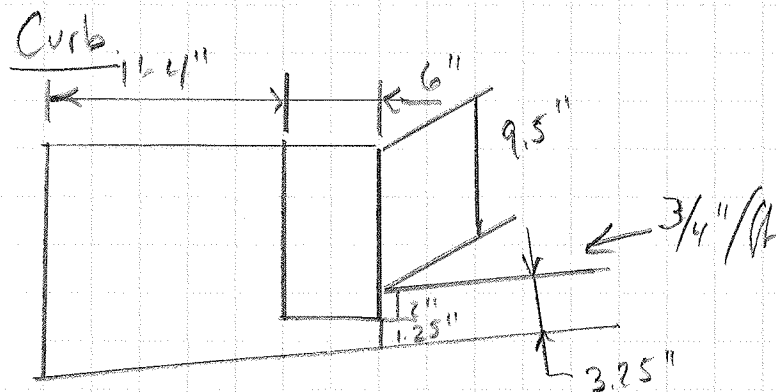


BRIDGE NO. 5799

This bridge is actually 2 bridges in 1. The "Left" Bridge for the 1984 Construction plan Stationing has 6 girders spaced at 9'-11" and 1 with a variable spacing. The "Right" bridge has 9 girders spaced @ 9'-8".

Since the girder sections are the same for both bridges, the interior girder on the Left bridge controls over the right bridge since it has a larger spacing and tributary width. Except for the left overhang on the left bridge all of the overhangs are 1'-4" or 4'-3 1/2". Based on the painted lane lines, the left exterior girder on the left bridge does not see any live load except for abutment within the flare. Therefore, the inside exterior girder on the left bridge is the controlling exterior girder since it has the largest tributary width and carries the largest live load based on the striping. See Attached CAD sketch for more detail.

Therefore Analyze the Left Bridge and ignore the right bridge since it does not control. Girders G3 and G6 are rated.



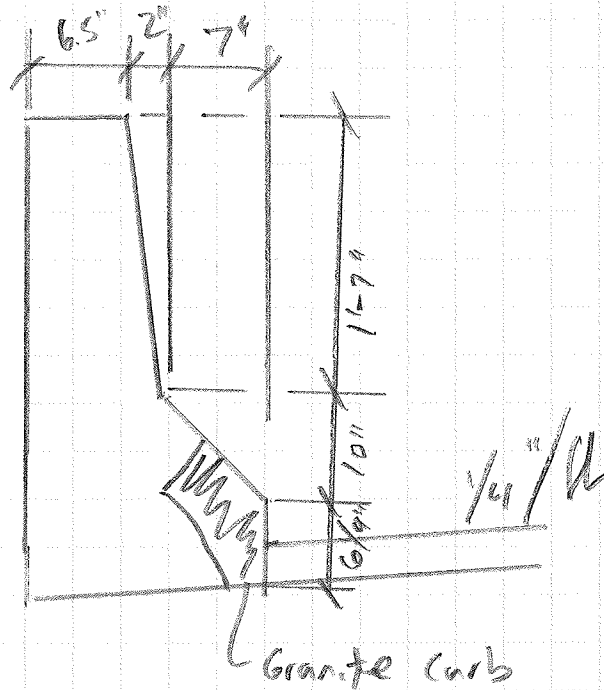
A curb

$$= \frac{1.25'' \times 6'' + 16'' \times 12.75''}{144} + \frac{(22'')^2 \times 0.0625 \times 1.5}{144}$$

$$= 1.574 \text{ ft}^2$$

$$A_{\text{girder}} = 0.5' \times \frac{(9.5'' + 2'')}{12} = 0.479 \text{ ft}^2$$

Type III Corner



$A_{curb} = 0.373 \text{ ft}^2$  from  
 Cadd sketch

$$A_{rail} = \frac{6.5" \times 19" + \frac{1}{2} \times 2" \times 15" + 8.5" \times 10" + \frac{1}{2} \times 10" \times 7" + 6.75" \times 15\frac{1}{2}"}{144 \text{ in}^2/\text{ft}^2}$$

$$= \frac{123.5 \text{ in}^2 + 15 \text{ in}^2 + 85 \text{ in}^2 + 35 \text{ in}^2 + 96.875 \text{ in}^2}{144 \text{ in}^2/\text{ft}^2} = \frac{350.375 \text{ in}^2}{144 \text{ in}^2/\text{ft}^2}$$

$$= 2.50 \text{ ft}^2$$

$$A_{rail \text{ concrete}} = 2.565 \text{ ft}^2 - 0.373 \text{ ft}^2 = 2.13 \text{ ft}^2$$

$$W_{rail} = 2.13 \text{ ft}^2 \times 150 \text{ lb/ft}^3 + 0.373 \text{ ft}^2 \times 170 \text{ lb/ft}^3$$

$$= 319 \text{ lb/ft} + 63.41 \text{ lb/ft} = 382 \text{ lb/ft}$$

See 400 lb/ft

### Diaphragm Load

Beam spacing = 9'-11"

L WT 5x11  $L = 9'-9"$   $W = 9.75(11) = .107k$   
 diagonals  $9'-11" - 2" = 9'-9"$

$$66" - 7\frac{1}{2}" - 11" - 6 = 99.2(1/4) = 40"$$

$$L = \sqrt{\left(\frac{40"}{12}\right)^2 + (9.75)^2} = 10.30'$$

2 WT 4x9  $W = 2 \times 9 \times 10.30 = .185k$

2 plates  $7" \times 3/8" \times 66" = 2 \left(7\frac{3}{8}" \times 66\right) (.490/ft) = .098k$   
.390k

### Blocking

Per plan sheet 21 of 43 note 1. The girders are cambered for all dead load. Therefore, blocking per Haunch detail on sheet 28 of 43  $h = 2"$

min input =  $2" - 0.75" = 1.25"$   
 $\uparrow$  over camber tolerance

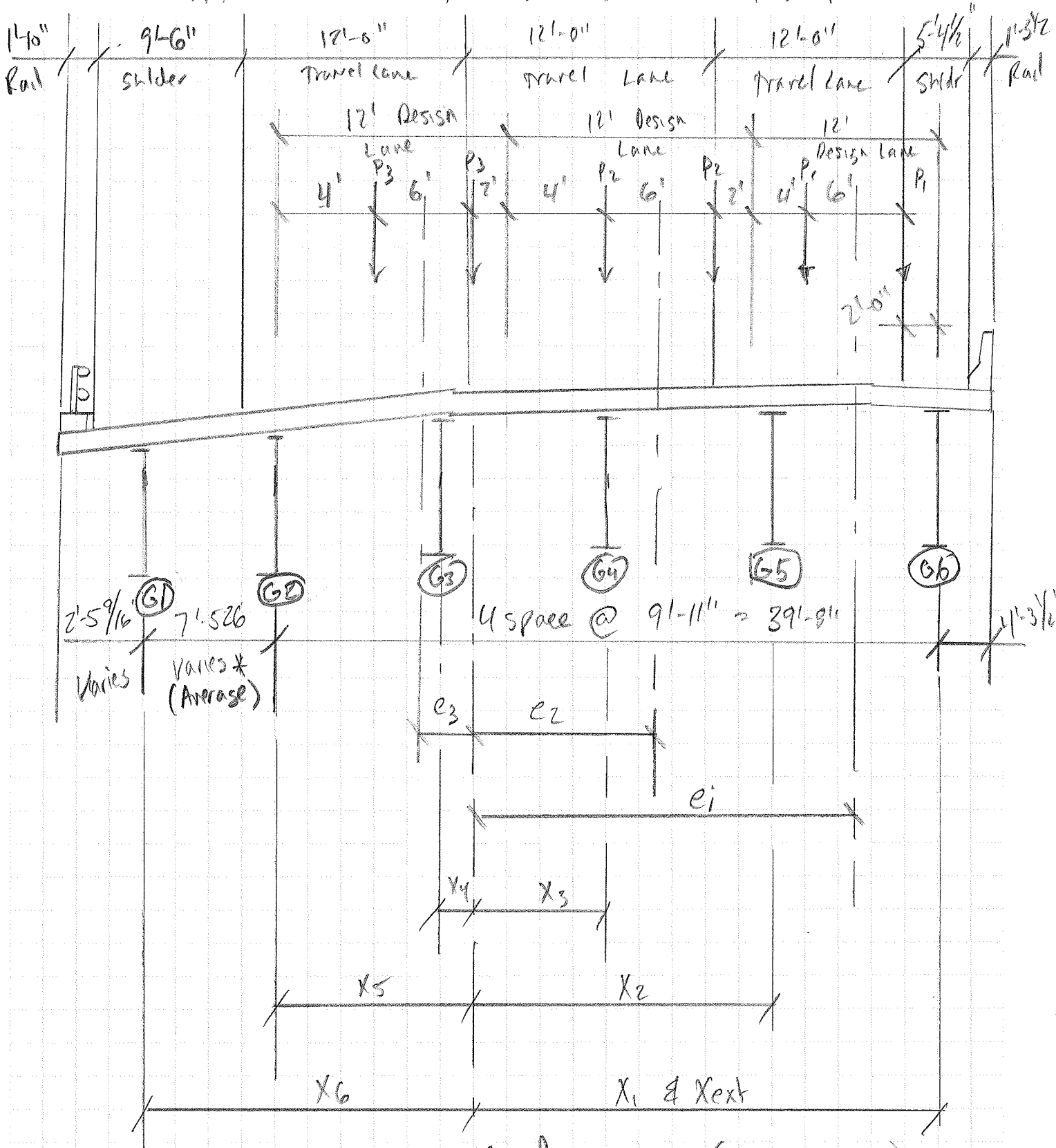
$$Eg = \left(\frac{9.5"}{2}\right) + 2" - 0.75" - 0.75" + (68" - 26.14") = 47.11"$$

$\uparrow$  over camber tol.  $\uparrow$   $\uparrow$  b6

### Intermediate Stiffeners

$$PL \ 3/8 \times 5" \times 66" \Rightarrow \frac{3/8 \times 5" \times 66"}{125} \times .490 = .035k/\text{plate}$$

$$\frac{.035k/\text{plate} \times 10}{111.5} = .0031k/ft$$



\* Midspan values shown

C.G. of  
 LEFT Bridge  
 End (Abut No. 2.)

(Lane spacings  
 per plans)

Similar

## **APPENDIX C**

### **SECTION LOSSES SKETCHES**

**(See CD)**

**(Not APPLICABLE)**