Town of Amherst, NH
BOARD OF SELECTMEN AGENDA
Barbara Landry Meeting Room
2 Main Street
THURSDAY, DECEMBER 1, 2022 11:30 AM

1. Call to Order

2. Staff Reports

2.1. Bid Award, Temporary Bridge Installation – Thornton Ferry Road I Over Beaver Brook

3. Action Items

4. Old/New Business

Adjournment

Next Meeting: December 12, 2022

You are invited to a Zoom webinar.
When: Dec 1, 2022 11:30 AM Eastern Time (US and Canada)
Topic: BOS Meeting

Please click the link below to join the webinar:
https://us02web.zoom.us/j/81639166653
Telephone: 1 312 626 6799
Webinar ID: 816 3916 6653

Town of Amherst, NH
BOARD OF SELECTMEN
STAFF REPORT

Title: Bid Award, Temporary Bridge Installation – Thornton Ferry Road I Over Beaver Brook

Department: Public Works

Meeting Date: December 1, 2022

Staff Contact: Eric Slosek

BACKGROUND INFORMATION:

Please find attached bid analysis letter from Hoyle Tanner. Their recommendation is to award the temporary bridge to Hansen Bridge, LLC. Specifically they are recommending to award the two-lane bridge option. We have also reviewed the bid documents internally and concur with Hoyle Tanner's recommendation. The total bid amount for the two-lane option from Hansen Bridge is $288,512.00. We received two other bids for two-lane bridges from R.M. Piper, Inc., and Cold River Bridges, LLC. They both proposed identical ADM bridges, manufactured in Pennsylvania. The only difference in the products they proposed was the type of bridge rail. Their prices were $555,135.00 and $613,535.00 respectively.

All three contractors also proposed single-lane bridges. They are all of the same type as the two-lane bridges, just smaller in dimensions. The prices for these options from Hansen, Piper, and Cold River, were $253,512.00, $364,555.00, and $497,285.00 respectively. Hoyle Tanner's own estimates were $331,845.00 for the two-lane option, and $261,690.00 for the single-lane option.

Hansen's bridge is built to the same specifications as the ADM product, however Hansen cuts out the middleman and fabricates the bridge themselves here in NH. This results in a significant cost savings. The Hoyle Tanner letter raises a few questions regarding the Hansen proposal. The questions were regarding what the geometric specifications of the bridge were (superstructure depth and width), and an explanation for why the traffic attenuation devices were bid so low (HTA estimated $20K, Hansen bid $2K). We were able to confirm this information this morning. The bridge will meet the same specifications as the ADM bridge, including depth and width. Hansen also confirmed that their price for the traffic attenuation devices is the $2000, and they are aware of the planned requirements.

An important consideration is also the time-frame for installation. Hansen's time-frame specifies 2-4 weeks for fabrication, and 4-6 hours for bridge and deck installation. Both Piper and Cold River specified 12-16 weeks for design, design review, material procurement, and fabrication.

A second important consideration is that all proposed two-lane bridges are suitable for the Brook Rd. bridge replacement.

Hansen Bridge specializes in ADM-style bridges and has installed more than 250 bridges, most of which were installed for NH municipalities. I received recommendations for Hansen Bridge from at least three communities including Derry, Newport, and I believe Rollinsford (I accidentally deleted the voicemail). Hoyle Tanner also has confirmed that Hansen Bridge is a reputable company and in their experience is often the low bidder.

BUDGET IMPACT:

(Include general ledger account numbers)

$288K. ($259K from bloc grant, $36K from Bridge CRF)

POLICY IMPLICATIONS:

DEPARTMENT HEAD RECOMMENDATION:

Recommend to award bid to Hansen Bridge, LLC, for a two-lane temporary bridge installed on Thornton Ferry Rd. 1, for the amount of $288,512.00, and to authorize the Town Administrator to sign any related contract documents.

SUGGESTED MOTION:

I move to award bid to Hansen Bridge, LLC, for a two-lane temporary bridge installed on Thornton Ferry Rd. 1, for the amount of $288,512.00, and to authorize the Town Administrator to sign any related contract documents.

TOWN ADMINISTRATOR RECOMMENDATION:

ATTACHMENTS:

1. Thornton Ferry Bid Analysis
2. Bid Tab
3. Hansen Documents
4. RM Piper Documents
5. Cold River Documents

November 29, 2022

Mr. Eric Slosek
Public Works Director
Town of Amherst
22 Dodge Road
Amherst, NH 03031

Re: Thornton Ferry Road I Bridge over Beaver Brook
NHDOT Bridge No. 145/106
Temporary Bridge Bid Review and Recommendation
Hoyle Tanner Project No. 21.919111

Dear Eric:

The purpose of this letter is to provide the Town of Amherst (Town) with a review of the bids and our recommendations for the subject project. Based on the findings discussed below, we recommend that the Town award the project to Hansen Bridge, LLC (Hansen) for Alternate A – Two-Lane Temporary Bridge.

EVALUATION METHODOLOGY

Per the Project Specifications, all bids were required to include unit pricing along with a technical proposal for either, or both of, the two Alternates included in the Contract Documents (Alternate A – Two-Lane Temporary Bridge and Alternate B – One Lane Temporary Bridge). The following methodology as defined in the Project Specifications is used to determine the scoring for the project.

- 70% of the total score shall be based on the technical proposals. All bids will be ranked on their technical proposals with preference given to schedule and ease of re-use of the temporary bridge at other sites.
- 30% of the total score shall be based on total project costs ranking.

BID OPENING:

Bids for the subject project were opened publicly on November 22, 2022 at 2:00 PM local time. A total of six bids and technical proposals were received from three contractors. Each contractor submitted a bid and technical proposal for each of the two alternates.

- For Alternate A, the Engineer's estimate was $331,845.00. Of the three bids opened, one was less than the engineer's estimate and two were higher. The bids ranged from $288,512.00 to $613,535.00

- For Alternate B, the Engineer's estimate was $261,690.00. Of the three bids opened, one was less than the engineer's estimate and two were higher. The bids ranged from $253,512.00 to $497,285.00.

TOTAL PROJECT COST SCORING

Hansen was the low bidder for both alternates. Per the Project Specifications, 30% of the total score is based on cost. Assigning the full 30% to the low bid yields the following scoring.

Table 1. Total Project Cost Scoring

	Alternate A		Alternate B	
	Bid	Scoring	Bid	Scoring
Hansen Bridge LLC	$288,512.00	30.0%	$253,512.00	30.0%
R.M. Piper, Inc.	$555,135.00	15.6%	$364,555.00	20.9%
Cold River Bridges. LLC	$613,535.00	14.1%	$497,285.00	15.3%

TECHNICAL PROPOSAL SCORING

Per the Project Specifications, 70% of the total score is based on the technical proposal which should include the following:

- The geometric characteristics (span, roadway width, total bridge width, and superstructure depth) and description of the materials of the bridge.
- A certification that the proposed temporary bridge meets the load capacity requirements and bridge rail test level as indicated in the Special Provisions for Section 501.
- The anticipated assembly, installation, disassembly, and transportation procedure.
- The anticipated project schedule, including approximate lead time for temporary bridge fabrication if applicable.

Hansen Bridge LLC

Hansen submitted technical proposals for both bridge alternates.

In a review of Hansen's proposals, we made the following observations:

- The bridge can be installed with an excavator eliminating the need for large equipment such as a crane.
- The structural steel will be galvanized which has a lower long term maintenance cost as compared to painted systems.
- Hansen does not explicitly state the anticipated total bridge width or superstructure depth. We recommend confirming the geometric criteria with Hansen prior to awarding the contract.
- A detailed description of the assembly and installation procedure is not provided, however, a statement as to the maximum pick weight and that all connections will be bolted is made in the proposal.
- A bridge fabrication schedule of 2-4 weeks is provided for both alternates.

The proposal for Alternate B – One-Lane Temporary bridge should receive a lower technical proposal score based on the limited potential for future re-use. Although Thornton Ferry Road I has a relatively low traffic count and site conditions that allow for a "stop, stop" traffic control condition, it is unknown if the future location of the temporary bridge will share these same characteristics.

The complete technical proposals are included in Appendix B.

R.M. Piper, Inc., and Cold River Bridges LLC.

R.M. Piper, Inc. (Piper) and Cold River Bridges LLC (CRB) submitted technical proposals for both bridge alternates. Their proposals were prepared by ADM Welding & Fabrication, LLC, who is a supplier of modular bridge systems. In a review of the proposals, we made the following observations:

- The proposed bridges are prefabricated into panels and bolted in the field. The bridge can be "skidded" into place eliminating the need for large equipment such as a crane.
- The deck will be removable nail laminated timber panels with sacrificial longitudinal running boards.
- The structural steel will be painted with a system that does not meet the NHDOT Standard Specifications for Road and Bridge Construction 2016. Galvanized steel, or a paint system meeting the NHDOT Specifications can be selected at an additional cost and lead time.
- These proposals include all the required information as listed in the project specifications including fabrication times of 3 weeks for Alternate B and 4 weeks for Alternate A.

These proposals are nearly identical except for the bridge rail. Piper is proposing a "beam and post" type bridge rail comprised of wide flange beam posts at 3'-1.5" spacing with galvanized standard W-beam rail. Piper's proposal states that this rail system will meet TL-1 loading equivalency by calculation. Alternatively, CRB is proposing to install braced portable concrete barrier per the NHDOT's standard details. This system is often used for phased construction of bridges and has a higher rating of TL-3 as proven by crash testing. This barrier system will also tie into the proposed portable concrete barrier used for approach guardrail which will provide a "smoother" transition between the approach and bridge rail if a vehicle impacts the rail system. The disadvantage to providing the concrete barrier for bridge rail is that snow and ice may form a berm at the base of the barrier since the snow cannot be pushed off the bridge between the bridge rails.

As stated above for Hansen, Alternate B – One-Lane Temporary bridge should receive a lower technical proposal score based on the limited potential for future re-use.

The complete technical proposals are included in Appendix B.

Summary

Table 2. Technical Proposal Scoring

	Alternate A Scoring	Alternate B Scoring
Hansen Bridge LLC	60%	30%
R.M. Piper, Inc.	65%	33%
Cold River Bridges. LLC	70%	35%

RECOMMENDATIONS

Table 3. Combined Scoring

	Alternate A Scoring	Alternate B Scoring
Hansen Bridge LLC	90%	60%
R.M. Piper, Inc.	80.6%	53.9%
Cold River Bridges. LLC	84.1%	50.3%

Hansen's bids have the highest combined score for technical proposals and cost for each alternate. As such, a further investigation of Hansen's bid prices is warranted to determine if the bids are balanced. Appendix A attached to this letter provides a detailed analysis of bid items that deviate from the Engineer's estimated price by more than 1% of the total bid price of the project. Based on the results of this analysis, we believe that Hansen's bids are balanced for both alternates.

The total cost difference between Alternate A – Two-Lane Bridge and Alternate B – One-Lane Bridge for Hansen's bids is $35,000; however, Alternate A provides a safer two-lane option, wider travelway for snow plowing, and greater flexibility for future re-use of the bridge. As such, we recommend that the Town award the project to Hansen Bridge, LLC (Hansen) for Alternate A – Two-Lane Temporary Bridge pending the confirmation of bridge geometry as discussed above.

Should you require additional information, please contact me at (603) 460-5211 or via email at jripley@hoyletanner.com.

Sincerely,
HOYLE, TANNER & ASSOCIATES, INC.

Joseph C Ripley, PE
Project Manager

Enclosures

APPENDIX A
BID ANALYSIS FOR HANSEN BRIDGE

HOYLE, TANNER & ASSOCIATES, INC.
Project No. 21.919111

BID ANALYSIS
Construction of the Temporary Bridge Installation - Thornton Ferry Road I Over Beaver Brook
Town of Amherst, NH
Alternate A - Two-Lane Temporary Bridge

ITEM NO.	DESCRIPTION	QUANTITY		ENGINEER'S ESTIMATE UNIT PRICE	ENGINEER'S ESTIMATE ITEM AMOUNT	LOW BID Hansen Bridge, LLC UNIT PRICE	LOW BID Hansen Bridge, LLC ITEM AMOUNT	LOW BID ITEM AMOUNT DIFFERENCE	DIFFERENCE AS % OF THE TOTAL BID	REMARKS
201.1	CLEARING AND GRUBBING (F)	0.15	A	$10,000.00	$ 1,500.00	$10,000.00	$ 1,500.00	$0.00	0.0%	
201.881	INVASIVE SPECIES CONTROL TYPE I	20	SY	$30.00	$ 600.00	$100.00	$ 2,000.00	+ $1,400.00	0.5%	
201.882	INVASIVE SPECIES CONTROL TYPE II	20	SY	$30.00	$ 600.00	$100.00	$ 2,000.00	+ $1,400.00	0.5%	
202.7	REMOVAL OF GUARDRAIL	245	LF	$5.00	$ 1,225.00	$20.00	$ 4,900.00	+ $3,675.00	1.3%	This is a low risk item. The limits of guardrail are well defined.
203.1	COMMON EXCAVATION	150	CY	$30.00	$ 4,500.00	$30.00	$ 4,500.00	$0.00	0.0%	
203.601	EMBANKMENT-IN-PLACE	15	CY	$30.00	$ 450.00	$20.00	$ 300.00	-$150.00	-0.1%	
214	FINE GRADING	1	U	$3,000.00	$ 3,000.00	$2,000.00	$ 2,000.00	-$1,000.00	-0.3%	
304.301	CRUSHED GRAVEL	610	CY	$45.00	$ 27,450.00	$40.00	$ 24,400.00	-$3,050.00	-1.1%	This is a low risk item. Although the total cost difference is over 1% of the total bid, the unit cost is reasonable.
403.11033	HBP- 3/4" WINTER BINDER, MACHINE METHOD	110	TON	$180.00	$ 19,800.00	$200.00	$ 22,000.00	+ $2,200.00	0.8%	
403.11043	HBP-1/2" SURFACE MIX, MACHINE METHOD	70	TON	$200.00	$ 14,000.00	$200.00	$ 14,000.00	$0.00	0.0%	
410.22	ASPHALT EMULSION FOR TACK COAT	25	GAL	$10.00	$ 250.00	$50.00	$ 1,250.00	+ $1,000.00	0.3%	
417	COLD PLANING BITUMINOUS SURFACES	25	SY	$40.00	$ 1,000.00	$40.00	$ 1,000.00	$0.00	0.0%	
501.191	TEMPORARY BRIDGE (TWO-LANE) LEFT IN PLACE	1	U	$160,000.00	$ 160,000.00	$131,437.00	$ 131,437.00	-$28,563.00	-9.9%	Hansen specializes in bridges of this type. The materials included in his technical proposal match those required in the specifications.
592.9	MODULAR CONCRETE BLOCK RETAINING WALL	279	LF	$75.00	$ 20,925.00	$60.00	$ 16,740.00	-$4,185.00	-1.5%	This is a low risk item. The limits of the wall are well defined. The cost difference is only slightly more than 1% of the total bid
606.4171	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL - LEFT IN PLACE	300	LF	$50.00	$ 15,000.00	$80.00	$ 24,000.00	+ $9,000.00	3.1%	This is a low risk item. The pay limits are well defined and will not be exceeded in the field.
606.9522	TEMP. IMPACT ATTENUATION DEVICE (NON-REDIRECTIVE) TEST LEVEL 2	4	U	$5,000.00	$ 20,000.00	$500.00	$ 2,000.00	-$18,000.00	-6.2%	This cost appears to be low based on Hoyle Tanner's experience. These attenuators need to be treated to prevent freezing. We recommend that further discussion with Hansen about this item take place prior to award of the contract to ensure that Hansen is aware of the specification requirements..
615.0301	TRAFFIC SIGN TYPE C	18	SF	$100.00	$ 1,800.00	$10.00	$ 180.00	-$1,620.00	-0.6%	
615.0601	TRAFFIC SIGN TYPE CC	5	SF	$40.00	$ 200.00	$10.00	$ 50.00	-$150.00	-0.1%	
628.2	SAWED BITUMINOUS PAVEMENT	95	LF	$6.00	$ 570.00	$3.00	$ 285.00	-$285.00	-0.1%	
645.44	TEMPORARY SLOPE MATTING TYPE D (WILDLIFE FRIENDLY)	210	SY	$5.00	$ 1,050.00	$20.00	$ 4,200.00	+ $3,150.00	1.1%	This unit price is higher than expected. Careful coordination between the resident engineer and Hansen should take place to ensure that additional quantities beyond what is required are not installed.
645.512	COMPOST SOCK FOR PERIMETER BERM	735	LF	$5.00	$ 3,675.00	$10.00	$ 7,350.00	+ $3,675.00	1.3%	This unit price is higher than expected. Careful coordination between the resident engineer and Hansen should take place to ensure that additional quantities beyond what is required are not installed.
646.51	TURF ESTABLISHMENT WITH MULCH, TACKIFIERS AND LOAM	210	SY	$5.00	$ 1,050.00	$2.00	$ 420.00	-$630.00	-0.2%	
692	MOBILIZATION	1	U	$30,200.00	$ 30,200.00	$20,000.00	$ 20,000.00	-$10,200.00	-3.5%	This is a low risk item.
697.11	INVASIVE SPECIES CONTROL AND MANAGEMENT PLAN	1	U	$3,000.00	$ 3,000.00	$2,000.00	$ 2,000.00	-$1,000.00	-0.3%	
				TOTAL	**$ 331,845.00**		**$ 288,512.00**			

Remarks: 1. The relatively small difference in cost between the estimated and bid prices appear to present no advantage to the contractor nor disadvantage to the Owner.

APPENDIX B
TECHNICAL PROPOSALS

HANSEN BRIDGE

ALTERNATE A

HANSEN BRIDGE

1325 County Road, New London, NH 03257
603-491- 5295
hansenbridge@tds.net

22' x 50' GALVANIZED STEEL STRINGER TEMPORARY BRIDGE ESTIMATE

November 20, 2022

CLIENT:
Town Of Amherst NH
Attn: Town Administrator

RE: Temporary Bridge Construction- Thornton Ferry Road Bridge

Description: Construct precast modular abutments for a 22'-0" x 50'-0" galvanized steel stringer bridge with removable laminated timber deck panels and TL-1 crash rated W Beam bridge rail. Installation per HTA plans and specifications.

Payment Schedule : #1 $144,256.00 deposit prior to construction.
#2 $144,256.00 due after bridge is complete.

BRIDGE ITEMS

Description	Estimated Qty	Unit		
Modular Precast Abutments	1	UNIT		
Laminated Pressure Treated Timber Deck Panels	1	UNIT		
Galvanized Steel Stringer Bridge w/ TL-1 W Beam Rail	1	UNIT		
Erosion Control	1	UNIT		
Paving	1	UNIT		
			TOTAL	$288,512.00

- ☐ Price is based on Hansen Bridge to provide job supervision.
- ☐ Please allow **2-4** weeks for bridge fabrication and galvanizing.
- ☐ NHPE bridge design load rating HL-93 and TL-1 rail calculations by GM2 Engineers
- ☐ All steel beams, clips and diaphragm's to be hot dipped galvanized and delivered in kit form. A325 galvanized bolted connections only.
- ☐ Please allow 4-6 hrs for galvanized bridge superstructure and deck installation. 20 Ton excavator required.
- ☐ **Heaviest pick 5,000 lbs, Total number of trailer loads = 1.**

Acceptance of above contract by owner, _____ Date_____ Signature_____

Thank you,
Hansen Bridge LLC
Michael Hansen

HANSEN BRIDGE

1325 County Road, New London, NH 03257
603-491- 5295
hansenbridge@tds.net

12' x 50' GALVANIZED STEEL STRINGER TEMPORARY BRIDGE ESTIMATE

November 20, 2022

CLIENT:
Town Of Amherst NH
Attn: Town Administrator

RE: Temporary Bridge Construction- Thornton Ferry Road Bridge

Description: Construct precast modular abutments for a 12'-0" x 50'-0" galvanized steel stringer bridge with removable laminated timber deck panels and TL-1 crash rated W Beam bridge rail. Installation per HTA plans and specifications.

Payment Schedule : #1 $126,756.00 deposit prior to construction.
#2 $126,756.00 due after bridge is complete.

BRIDGE ITEMS

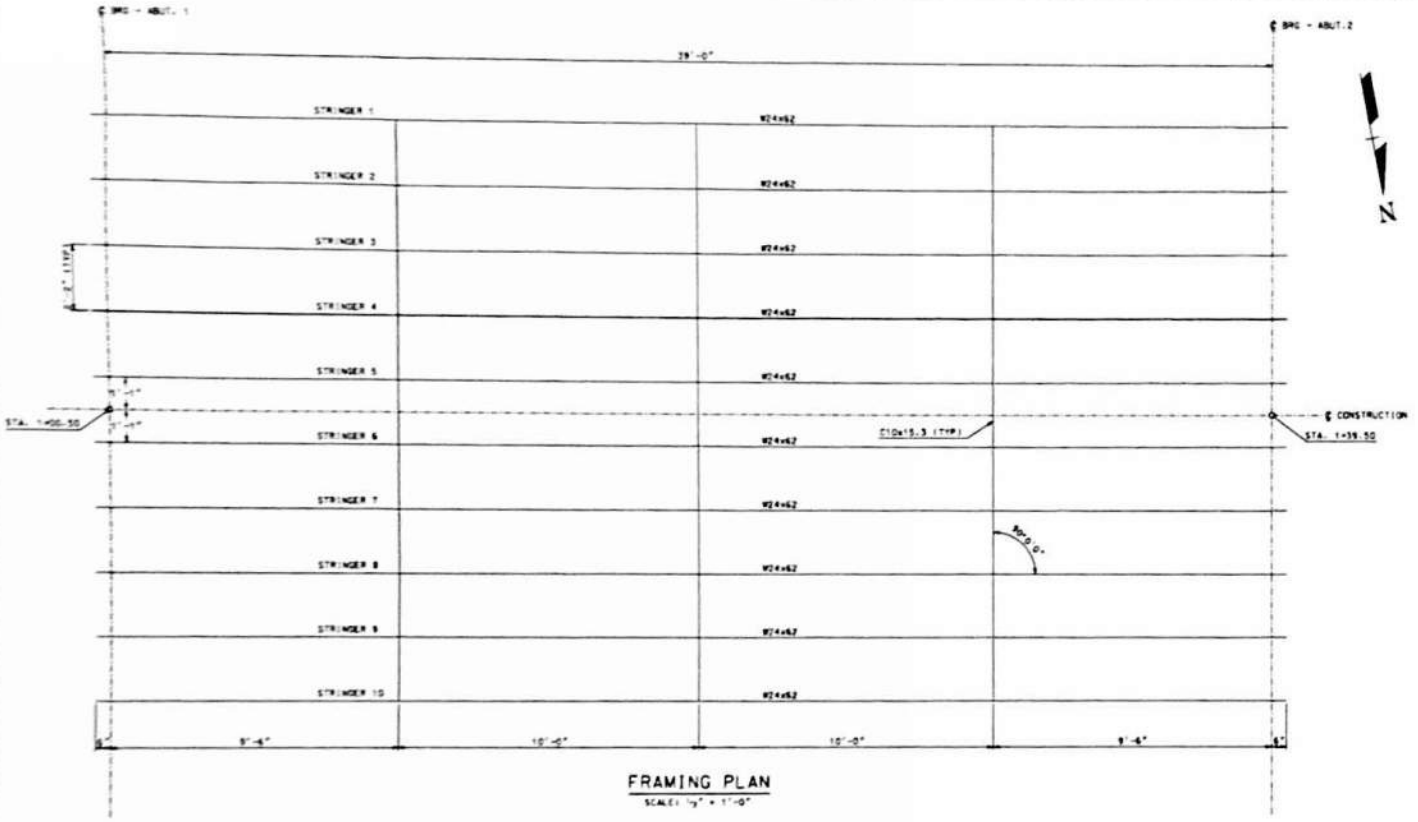
Description	Estimated Qty	Unit		
Modular Precast Abutments	1	UNIT		
Laminated Pressure Treated Timber Deck Panels	1	UNIT		
Galvanized Steel Stringer Bridge w/ TL-1 W Beam Rail	1	UNIT		
Erosion Control	1	UNIT		
Paving	1	UNIT		
			TOTAL	$253,512.00

- ☐ Price is based on Hansen Bridge to provide job supervision.
- ☐ Please allow 2-4 weeks for bridge fabrication and galvanizing.
- ☐ NHPE bridge design load rating HL-93 and TL-1 rail calculations by GM2 Engineers
- ☐ All steel beams, clips and diaphragm's to be hot dipped galvanized and delivered in kit form. A325 galvanized bolted connections only.
- ☐ Please allow 2-4 hrs for galvanized bridge superstructure and deck installation. 20 Ton excavator required.
- ☐ **Heaviest pick 5,000 lbs, Total number of trailer loads = 1.**

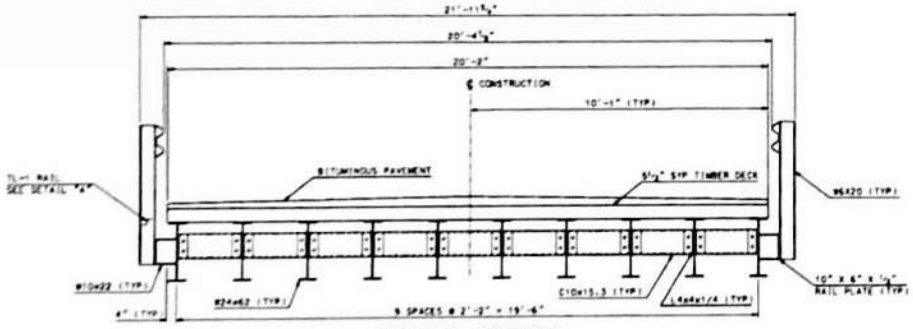
Acceptance of above contract by owner, _____ Date_____ Signature_____

Thank you,
Hansen Bridge LLC
Michael Hansen

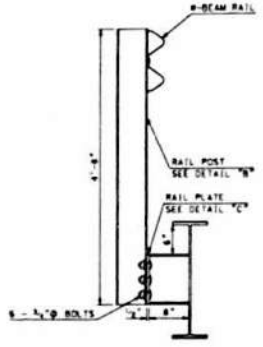
SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE



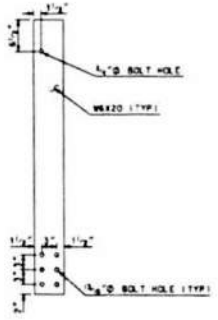
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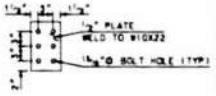
TYPICAL SECTION
SCALE: 1/2" = 1'-0"



DETAIL "A"
SCALE: 1" = 1'-0"



DETAIL "B"
SCALE: 1" = 1'-0"



DETAIL "C"
SCALE: 1" = 1'-0"

PRELIMINARY PLANS SUBJECT TO CHANGE
DATE 3/18/2022

GM2

TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

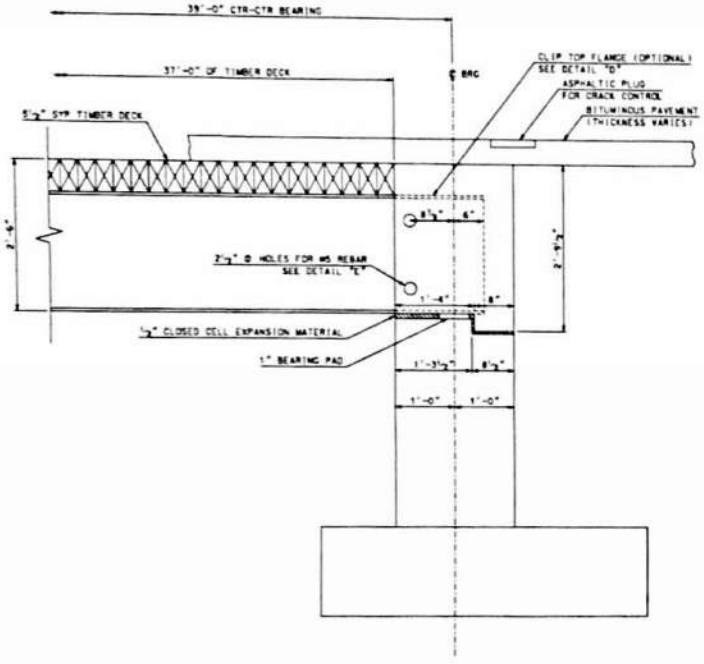
TOWN: ALEXANDRIA, NH | BRIDGE NO. 096/112 | STATE PROJECT: N/A
LOCATION: SMEM VALLEY ROAD OVER ABOCE BROOK

SUPERSTRUCTURE DETAILS

	BY	DATE		BY	DATE
DESIGNED	BJL	05/22	CHECKED	TPL	05/22
DRAWN	BJL	05/22	CHECKED	TPL	05/22
QUANTITIES	-	-	CHECKED	-	-
ISSUE DATE			PROJECT MGR.		
REV. DATE					

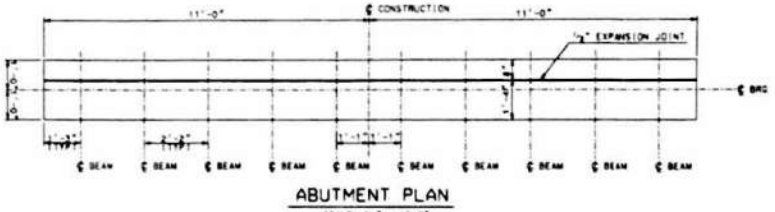
SHEET NO.: 1
TOTAL SHEETS: 3

SUBSTRUCTURE	SUPERSTRUCTURE	SHEET SCALE
BRIDGE ABUTMENTS	SEE DETAILS	AS NOTED



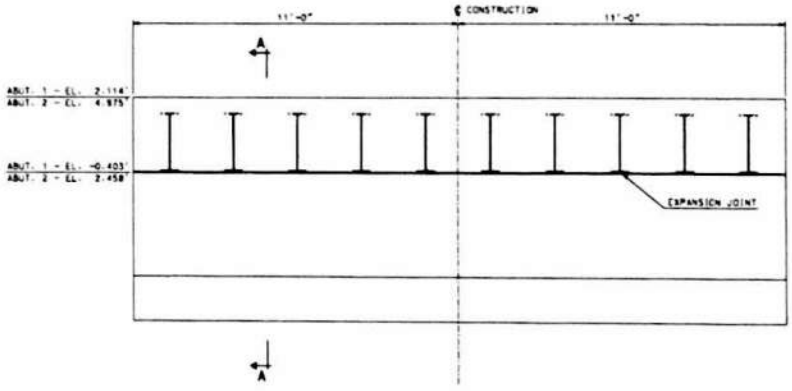
SECTION A-A

SCALE: 1" = 1'-0"



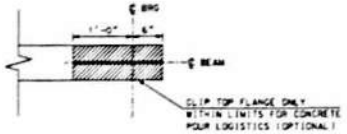
ABUTMENT PLAN

SCALE: ½" = 1'-0"



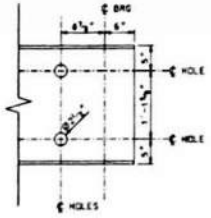
ABUTMENT ELEVATION

SCALE: ½" = 1'-0"



DETAIL "D"

SCALE: 1" = 1'-0"



DETAIL "E"

SCALE: 1" = 1'-0"

NOTE: ELEVATIONS ARE TAKEN RELATIVE TO THE ELEVATION AT STA. 0+00

SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE

PRELIMINARY PLANS
SUBJECT TO CHANGE
DATE 5/10/2022

GM2

TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

TOWN: ALEXANDRIA, NH — BRIDGE NO. 096/112 — STATE PROJECT: N/A

LOCATION: SIDEM VALLEY ROAD OVER BROCK BROOK

SUBSTRUCTURE DETAILS

	BY	DATE		BY	DATE
DESIGNED	BJL	05/22	CHECKED	TPL	05/22
DRAWN	BJL	05/22	CHECKED	TPL	05/22
QUANTITIES	–	–	CHECKED	–	–
ISSUED DATE					
REV. DATE					

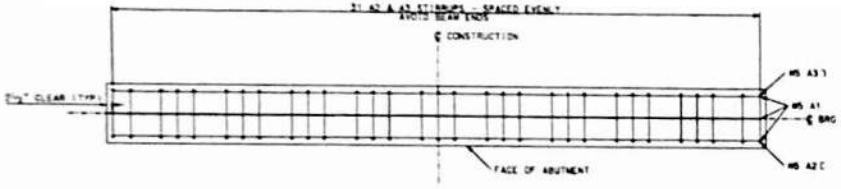
MASTER SHEET: 2 of 3
FILE NUMBER: –
SHEET NO.: 2
TOTAL SHEETS: 3

SUBDIRECTORY: BRNCADD 096-112 — DGN LOCATOR: Base Imagery — SHEET SCALE: DETAILS AS NOTED

ALEXANDRIA - Br. No. 000112

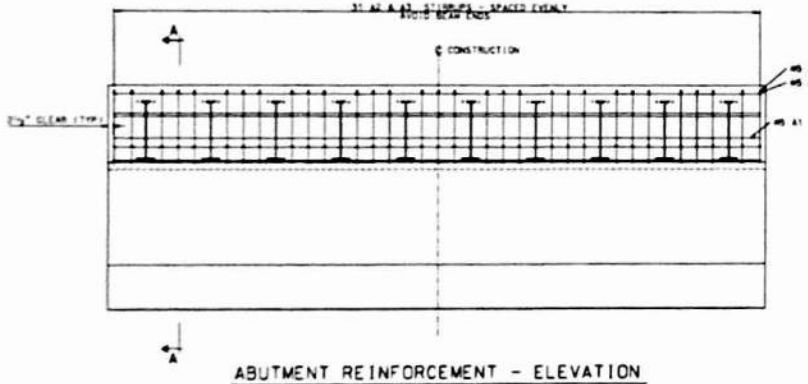
Mark	No.	Length	# Pieces	Type	A	B	C	D	E	F	G	H	J	K	R	O	NOTE
A1	4	21'-7"	16	-													
A2	5	8'-4"	82	17		1'-4"	2'-0"	1'-0"									EC
A3	5	9'-11"	82	17		1'-4"	2'-3"	0'-4"									EC

ITEM #	DESCRIPTION	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	TOTAL
544	REINFORCING STEEL												
544.01	MECH. CONNECTOR												
544.2	EPOXY COATED			894									
544.21	EPOXY MECH. CON.												894



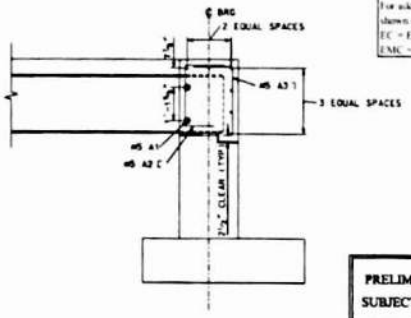
ABUTMENT REINFORCEMENT - PLAN

SCALE: 1/2" = 1'-0"



ABUTMENT REINFORCEMENT - ELEVATION

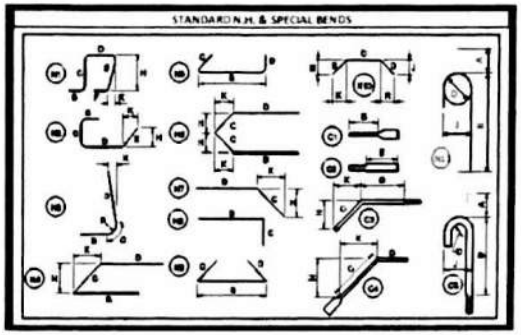
SCALE: 1/2" = 1'-0"



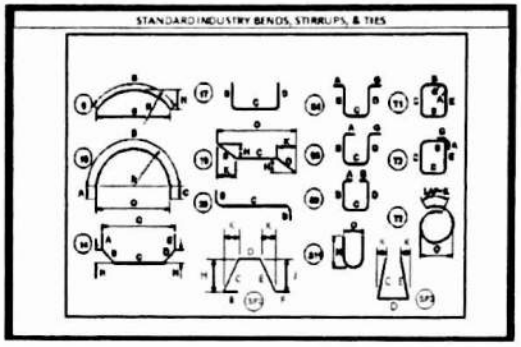
SECTION A-A

SCALE: 1/2" = 1'-0"

STANDARD N.H. & SPECIAL BENDS



STANDARD INDUSTRY BENDS, STIRRUPS, & TIES



NOTES:

1. FIGURES IN CIRCLE SHOW TYPE OF BEND.
2. UNLESS OTHERWISE DESIGNATED, ALL BAR REINFORCEMENT FOR CONCRETE IN SIZES UP TO AND INCLUDING NO. 18 SHALL CONFORM TO THE REQUIREMENTS OF THE "SPECIFICATIONS FOR DEFORMED BILLET - STEEL BARS FOR CONCRETE REINFORCEMENT," AASHTO M 31-94 (ASTM A615)
3. FOR TYPICAL BENDING DETAILS, RECOMMENDED PIN DIAMETER "D" OF BENDS AND HOOKS AND OTHER STANDARD PRACTICE REFER TO THE CURRENT CONCRETE REINFORCING STEEL INSTITUTE "MANUAL OF STANDARD PRACTICE."
4. BARS WHICH REQUIRE MORE ACCURATE BENDING THAN STANDARD PRACTICES SHOULD HAVE LIMITS INDICATED.
5. ALL DIMENSIONS ARE OUT TO OUT OF BAR EXCEPT "A" AND "B" ON STANDARD 180° AND 135° HOOKS.
6. "J" DIMENSION ON 180° HOOKS TO BE SHOWN ONLY WHEN NECESSARY TO RESTRICT HOOK SIZE. OTHERWISE STANDARD HOOKS ARE TO BE USED.
7. "H" DIMENSION ON STIRRUPS TO BE SHOWN ONLY WHEN NECESSARY TO MAINTAIN CLEARANCES.
8. WHERE SLOPE DIFFERS FROM 45° DIMENSIONS "H" AND "K" MUST BE SHOWN.

△ DENOTES BARS TO BE CUT IN FIELD, AS REQUIRED.
△ DENOTES BARS TO BE BENT IN FIELD.

RECOMMENDED END HOOKS, ALL GRADES (in.)				
BAR SIZE	180° HOOKS			90° HOOKS
	D(2)	A or G	J	A or G
#3	2 1/4	5	3	6
#4	3	6	4	8
#5	3 3/4	7	5	10
#6	4 1/2	8	6	12
#7	5 1/4	10	7	14
#8	6	11	8	16
#9	9 1/2	15	11 3/4	19
#10	10 3/4	17	13 1/4	22
#11	12	19	14 3/4	24
#14	18 1/4	27	21 3/4	31

ASTM STANDARD REINFORCING BARS (in.)			
BAR SIZE	WEIGHT (lb/ft)	NOM. DIMEN.	
		DIA. (in.)	AREA (in²)
#3	0.376	0.375	0.11
#4	0.668	0.500	0.20
#5	1.043	0.625	0.31
#6	1.502	0.750	0.44
#7	2.044	0.875	0.60
#8	2.670	1.000	0.79
#9	3.400	1.128	1.00
#10	4.303	1.270	1.27
#11	5.313	1.410	1.56
#14	7.650	1.693	2.25
#18	13.600	2.257	4.00

STIRRUP & TIE HOOK DIMENSIONS, GRADES 40-50-60 (in.)				
BAR SIZE	D(2)	90° HOOKS	135° HOOKS	
		A or G	A or G	H (approx.)
#3	1 1/2	4	4	2 1/2
#4	2	4 1/2	4 1/2	3
#5	2 1/2	6	5 1/2	3 3/4
#6	4 1/2	12	7 3/4	4 1/4
#7	5 1/4	14	9	5 1/4
#8	6	16	10 1/4	6

NOTE:
D = finished inside bend d of hook.
See 1997 CRSI Manual p.6-3&4.
For additional data on standard bar bends not shown on this sheet see 1997 CRSI Manual p.6-3&4.
EC = EPOXY COATED
EMC = EPOXY MECHANICAL CONNECTOR

PRELIMINARY PLANS
SUBJECT TO CHANGE
DATE 5/10/2022

GM2

SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE

SUBDIRECTORY	DGN LOCATOR	SHEET SCALE
BRDCADD (06/11) Bentley Systems, Inc.		1" = 1'-0"

TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

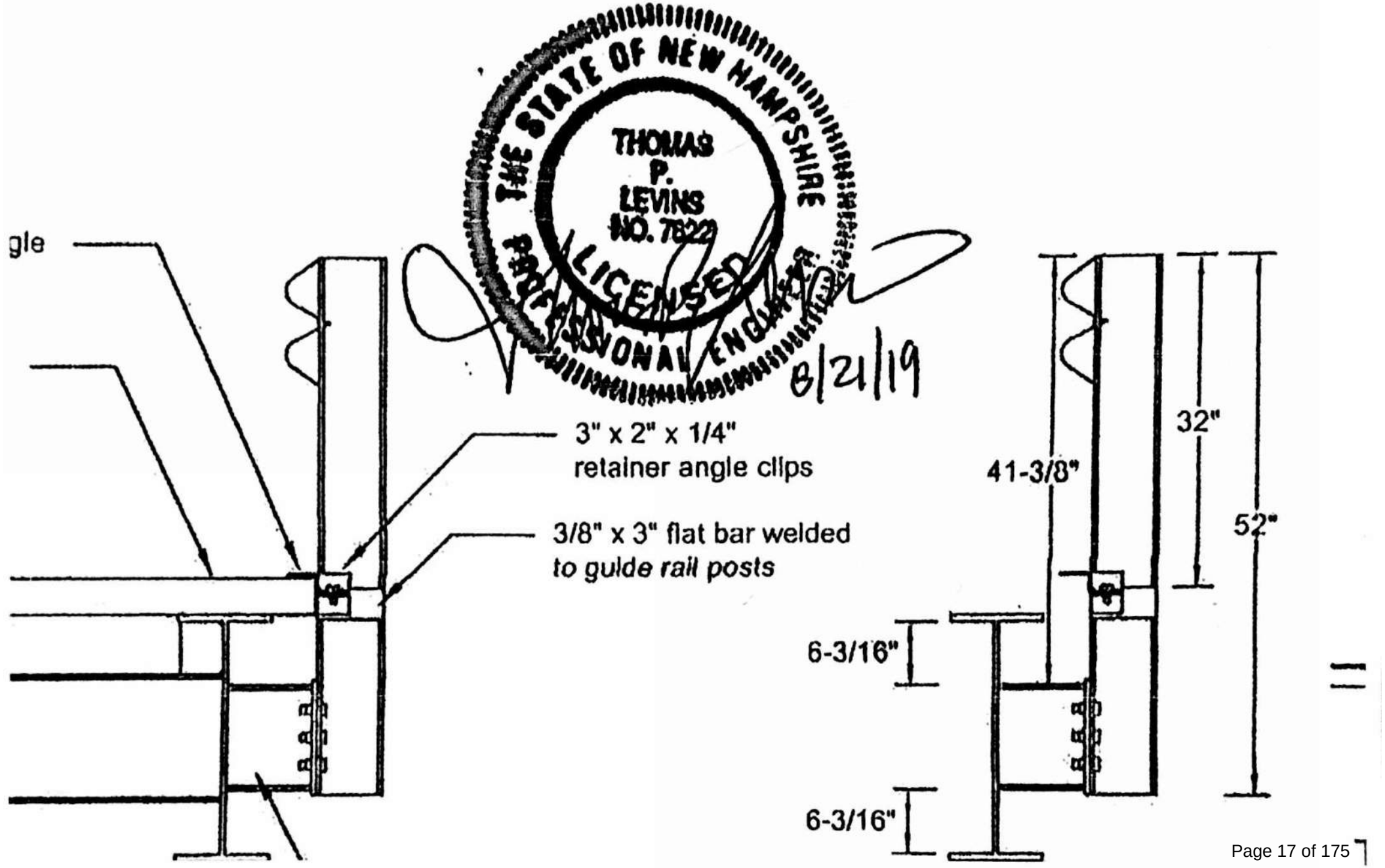
TOWN: ALEXANDRIA, NH | BRIDGE NO.: 000112 | STATE PROJECT: N/A

LOCATION: MIDDLE VALLEY ROAD OVER BROOK BRIDGE

ABUTMENT BACKWALL REINFORCEMENT

DESIGNED	BJL	05/22	CHECKED	TPL	05/22
DRAWN	BJL	05/22	CHECKED	TPL	05/22
QUANTITIES	-	-	CHECKED	-	-
ISSUE DATE			REVISION NO.	SHEET NO.	TOTAL SHEETS
REV. DATE			-	3	3

SHEET NUMBER: 3 OF 3
FILE NUMBER: -


gle
THE STATE OF NEW HAMPSHIRE
THOMAS
P.
LEVINS
NO. 7822
LICENSED
PROFESSIONAL ENGINEER
8/21/19
3" x 2" x 1/4"
retainer angle clips
3/8" x 3" flat bar welded
to gulde rail posts
41-3/8"
32"
52"
6-3/16"
6-3/16"

SAMPLE CALCULATIONS

Form 4
N.H. D.O.T.

TOWN: SWANZEY, NH

BRIDGE CAPACITY SUMMARY

BRIDGE NUMBER: ONE LANE Temporary Bridge

DESIGN LOAD: HL-93 *DESIGN METHOD:* LOAD & RESISTANCE FACTOR *RATED BY:* TEM *DATE:* 9/4/2019

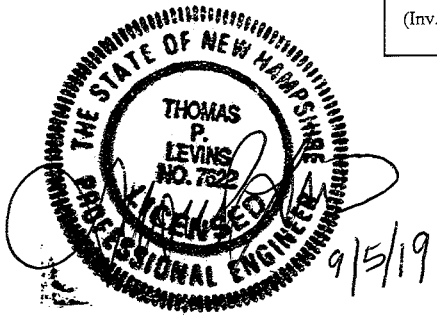
RATING METHOD: LRFR *PLAN FILE:* --- *CHECK BY:* TPL *DATE:* 9/4/2019

ROUTE: RABBIT HOLLOW ROAD *OVER:* PERRY BROOK

RATED MEMBER	LONGITUD. EFFECTIVE SPAN LENGTH	REQUIRED CAPACITY (HS Tons)			AVAILABLE CAPACITY (HS Tons)					
		CURRENT LEGAL LOADS	CERTIFIED VEHICLES		MULTIPLE LANES LOADED			SINGLE LANES LOADED		
			SINGLE UNIT	MULTIPLE UNIT	INVENTORY	OPERATING	POSTING	INVENTORY	OPERATING	POSTING
TIMBER DECK & CENTER PANEL										
POS. MOMENT	1'-0"	HS 14.0	HS 15.4	HS 15.4	HS	HS	HS	HS 16.5	HS 21.4	HS 20.2
SHEAR	1'-0"	HS 14.0	HS 15.4	HS 15.4	HS	HS	HS	HS 19.3	HS 25.0	HS 23.6
INTERIOR GIRDER										
POS. MOMENT	47'-0"	HS 20.4	HS 25.4	HS 22.8	HS	HS	HS	HS 29.0	HS 37.6	HS 35.4
SHEAR	47'-0"	HS 19.8	HS 23.1	HS 22.6	HS	HS	HS	HS 135.1	HS 175.1	HS 165.1
EXTERIOR GIRDER										
POS. MOMENT	47'-0"	HS 20.4	HS 25.4	HS 22.8	HS	HS	HS	HS 51.4	HS 66.8	HS 63.0
SHEAR	47'-0"	HS 19.8	HS 23.1	HS 22.6	HS	HS	HS	HS 239.1	HS 309.9	HS 292.2

	Rating Method	English Tons	Metric Tons
(Op.) 63.	LRFR	64. (Op.) 38.6	35.0
(Inv.) 65.	LRFR	66. (Inv.) 29.7	27.0

RECOMMENDED POSTING: NO POSTING REQUIRED



REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
MIN	-	Minnesota LRFD Bridge Design Manual (2016)
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)
AWC/NDS	-	American Wood Council NDS Supplement

Unit Definition:

$$kcf := 1000\ pcf$$

Unit weight of hard-wood: $\gamma_{DCHW} := 0.060\ kcf$ LRFD TABLE 3.5.1-1

Depth of Deck: $d_{deck} := 3.5\ in$ PLANS

Width of Decking: $b_{decking} := 6\ in$ PLANS

Weight of deck: $wt_{deck} := \gamma_{DCHW} \cdot d_{deck} \cdot b_{decking} = 0.0088\ \frac{kip}{ft}$ AISC

Spacing of girders $S_{girder} := 26.5\ in$ PLANS

Dead Load Bending Moment $M_{deck} := \frac{0.8\ wt_{deck} \cdot S_{girder}^{2}}{8} = 0.0043\ kip \cdot ft$

Weight of HL-93 Truck Wheel Loads $P_{wheel} := 4\ kip$ (Front Axle Wheel)

Note: Girder Spacing is such that rear axle wheel will always be partially supported by a girder

Moment Diagram (from truck):



Live Load Moment per wheel (from diagram) $M_{truck} := 1.7\ kip \cdot ft$

Wheel load distribution on deck plank (per foot) $E_s := \frac{b_{decking}}{10\ in} = 0.60$ LRFD 4.6.2.1.3

Maximum Number of lanes	$N_{lanes} := \text{round}\left(\frac{16.416\ ft}{12\ ft}\right) = 1.0$	
Multiple presence factor (for Number of Lanes = 1)	$m := 1.20$	LRFD TABLE 3.6.1.1.2-1
Live Load Bending Moment	$M_{LLmax} := m \cdot M_{truck} \cdot E_s = 1.22\ kip \cdot ft$	
Factored Moment:	$M_u := 1.25 \cdot M_{deck} + 1.75 \cdot M_{LLmax} = 2.15\ kip \cdot ft$	
	$\phi_{flexure} := 0.85$	LRFD 8.5.2.2
Required Nominal Capacity	$M_{n\_reqd} := \frac{M_u}{\phi_{flexure}} = 2.53\ kip \cdot ft$	
Format Conversion Factor	$C_{KF} := \frac{2.5}{\phi_{flexure}} = 2.94$	LRFD 8.4.4.2
Wet Service Factor	$C_M := 0.85$	LRFD Table 8.4.4.3-1
Size Factor	$C_F := 1.3$	LRFD 8.4.4.4
Flat-Use Factor	$C_{fu} := 1.10$	LRFD 8.4.4.6-1
Incising Factor	$C_i := 0.80$	LRFD 8.4.4.7
Deck Factor for Plank Decks	$C_d := 1.10$	LRFD Table 8.4.4.8-2
Time Effect Factor	$C_\lambda := 0.8$	LRFD Table 8.4.4.9-1
Bending Strength	$F_{bo} := 1.20\ ksi$	AWC/NDS
	$F_b := F_{bo} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_d \cdot C_\lambda = 3.02\ ksi$	
Required Depth of Rectangular Section	$d_{req} := \sqrt{\frac{6 \cdot M_{n\_reqd}}{b_{decking} \cdot F_b \cdot 1}} = 3.17\ in$	LRFD 8.6.2-1

Is depth given from plans greater than required depth?

$$\text{if}\left(d_{deck} > d_{req}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Nominal Capacity of section given in plans:

$$M_n := F_b \cdot \frac{b_{decking} \cdot d_{deck}^{\,2}}{6} = 3.08\ kip \cdot ft$$

Deck Flexure Rating Factor:

$$RF_{flexure} := \frac{\phi_{flexure} \cdot M_n - 1.25 \cdot M_{deck}}{1.75 \cdot M_{LLmax}} = 1.22$$

W24x76 girder flange $b_f := 8.99\ in$

1/2 of the girder flange $b_{length} := 0.5 \cdot b_f = 4.5\ in$

Distance to the left part of distributed load from left girder $b_{lefttruck} := b_{length} + d_{deck} = 8.00\ in$

Distance to the right part of distributed load from middle girder $b_{righttruck} := \left(b_{lefttruck} + 20\ in\right) - S_{girder} = 1.50\ in$

Distributed load from rear wheel $P_{truck} := \frac{16\ kip}{20\ in} = 9.60\ \frac{kip}{ft}$

Shear Diagram (from truck):



Maximum Shear from Truck (with multiple presence factor and distribution factor): $V_{truck} := m \cdot 4\ kip \cdot E_s = 2.88\ kip$

Shear from deck load $V_{deck} := \frac{wt_{deck} \cdot S_{girder}}{2} = 0.010\ kip$

$\phi_{shear} := 0.75$ LRFD 8.5.2.2

$F_{vo} := 0.220\ ksi$ AWC/NDS

Format Conversion Factor (for shear)

$$C_{KF\_shear} := \frac{2.5}{\phi_{shear}} = 3.33$$

LRFD 8.4.4.2

Wet Service Factor (for shear)

$$C_{M\_shear} := 0.97$$

LRFD Table 8.4.4.3-1

$$F_v := F_{vo} \cdot C_{KF\_shear} \cdot C_{M\_shear} \cdot C_i \cdot C_\lambda = 0.455 \ ksi$$

LRFD 8.4.4.1-2

Nominal shear resistance:

$$V_n := \frac{F_v \cdot b_{decking} \cdot d_{deck}}{1.5} = 6.37 \ kip$$

LRFD 8.7-2

Deck Shear Rating Factor:

$$RF_{shear} := \frac{\phi_{shear} \cdot V_n - 1.25 \cdot V_{deck}}{1.75 \cdot V_{truck}} = 0.946$$

REFERENCES:

LRFD - AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS - ADM Welding and Fabrication Plans
AISC - AISC Manual of Steel Construction, 15th ed. (2017)

Unit Definition:

$$kcf := 1000\ pcf$$

Unit weight of steel: $\gamma_{DCSTEEL} := 0.490\ kcf$ LRFD TABLE 3.5.1-1

Depth of Flat Bar $d_{flatbar} := 3.5\ in$ PLANS

Thickness of Flat Bar $t_{flatbar} := 0.25\ in$ PLANS

Spacing of flat bars $S_{flatbars} := 6\ in$ PLANS

Weight of top panel: $wt_{toppanel} := \gamma_{DCSTEEL} \cdot \frac{3}{16}\ in \cdot S_{flatbars}$

Length of flat bars $L_{flatbar} := 24.75\ in$ PLANS

Dead Load Bending Moment $M_{deck} := \frac{wt_{toppanel} \cdot L_{flatbar}^{2}}{8} = 0.002\ kip \cdot ft$

Weight of HL-93 Truck Wheel Loads $P_{wheel} := 4\ kip$ (Front Axle Wheel)

Note: Girder Spacing is such that rear axle wheel will always be partially supported by a girder

Wheel load distribution on middle section $E_s := \frac{S_{flatbars}}{10\ in} = 0.60$ LRFD 4.6.2.1.3

Maximum Number of lanes $N_{lanes} := \text{round}\left(\frac{16.416\ ft}{12\ ft}\right) = 1.0$

Multiple presence factor (for Number of Lanes = 1) $m := 1.20$ LRFD TABLE 3.6.1.1.2-1

Live Load Bending Moment $M_{LLmax} := E_s \cdot m \cdot \frac{0.8\ P_{wheel} \cdot L_{flatbar}}{4} = 1.19\ kip \cdot ft$

Factored Moment: $M_u := 1.25 \cdot M_{deck} + 1.75 \cdot M_{LLmax} = 2.08\ kip \cdot ft$

$\phi_{flexure} := 1.00$ LRFD 6.5.4.2

Yield Strength of Flat Bars $F_y := 50\ ksi$

Plastic Moment Capacity of flat bar (rectangular section) $M_p := \frac{t_{flatbar} \cdot d_{flatbar}^{\ 2}}{4} \cdot F_y = 3.19\ kip \cdot ft$ LRFD 6.12.2.2.7

Moment of Inertia $I_{xx} := \frac{t_{flatbar} \cdot d_{flatbar}^{\ 3}}{12} = 0.89\ in^4$

Center of gravity $y_{cog} := \frac{d_{flatbar}}{2} = 1.75\ in$

Elastic Section Modulus $S_{xx} := \frac{I_{xx}}{y_{cog}} = 0.51\ in^3$

Modulus of Elasticity of Steel $E_{steel} := 29000\ ksi$

Moment Gradient Modifier determined as specified in Article A6.3.3 (when M2=0, Cb = 1.0) $C_b := 1.0$ LRFD A6.3.3

$$M_n := min\left(\frac{1.9 \cdot E_{steel} \cdot 1.0}{\frac{L_{flatbar} \cdot d_{flatbar}}{t_{flatbar}^{\ 2}}} \cdot S_{xx}, M_p\right) = 1.69\ kip \cdot ft$$ LRFD 6.12.2.2.7-3

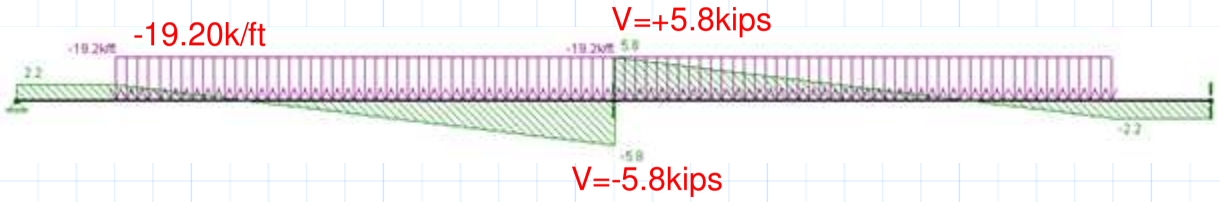
Middle Section of Deck Flexure Operating Rating Factor:

$$RF_{flexure} := \frac{\phi_{flexure} \cdot M_n - 1.25 \cdot M_{deck}}{1.35 \cdot M_{LLmax}} = 1.05$$

Distributed load from rear wheel

$$P_{truck} := \frac{16\ kip}{10\ in} = 19.20\ \frac{kip}{ft}$$

Shear Diagram (from truck):



Maximum Shear from Truck (with multiple presence factor and distribution factor):

$$V_{truck} := m \cdot (5.8\ kip + 5.8\ kip) \cdot E_s = 8.35\ kip$$

Shear from deck load

$$V_{deck} := \frac{wt_{toppanel} \cdot S_{flatbars}}{2} = 0.001\ kip$$

$$\phi_{shear} := 1.0$$ LRFD 6.5.4.2

Gross area of the connection element subject to shear

$$A_{vg} := d_{flatbar} \cdot t_{flatbar} = 0.88\ in^2$$

Nominal shear resistance:

$$R_r := 0.58 \cdot F_y \cdot A_{vg} = 25.38\ kip$$ LRFD 6.13.5.3

Deck Shear Inventory Rating Factor:

$$RF_{shear} := \frac{R_r - 1.25 \cdot V_{deck}}{1.75 \cdot V_{truck}} = 1.736$$

REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)

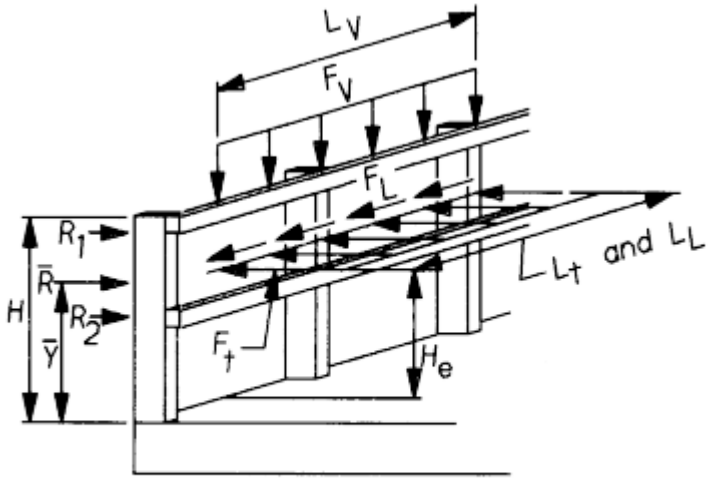


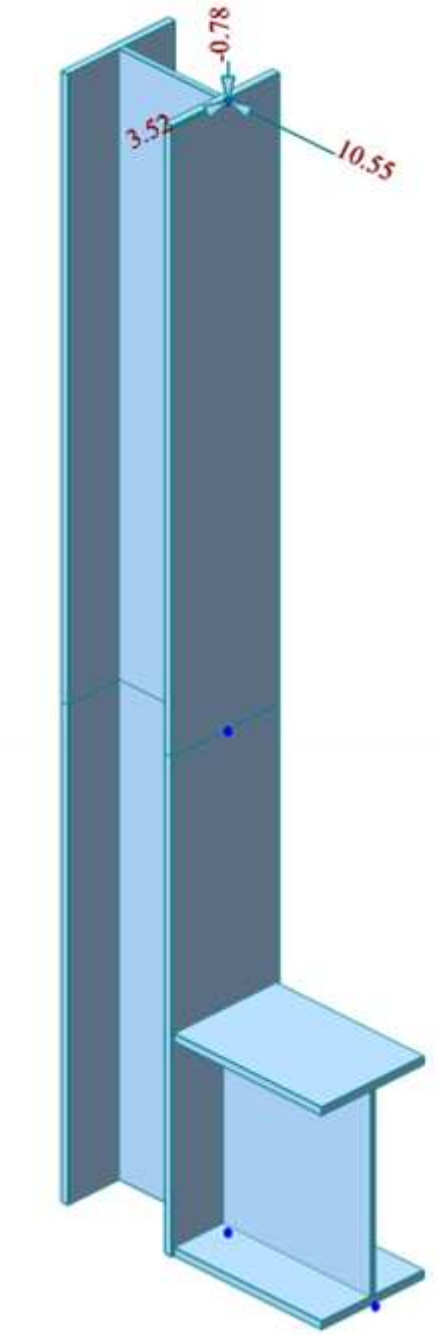
Figure A13.2-1—Metal Bridge Railing Design Forces, Vertical Location, and Horizontal Distribution Length

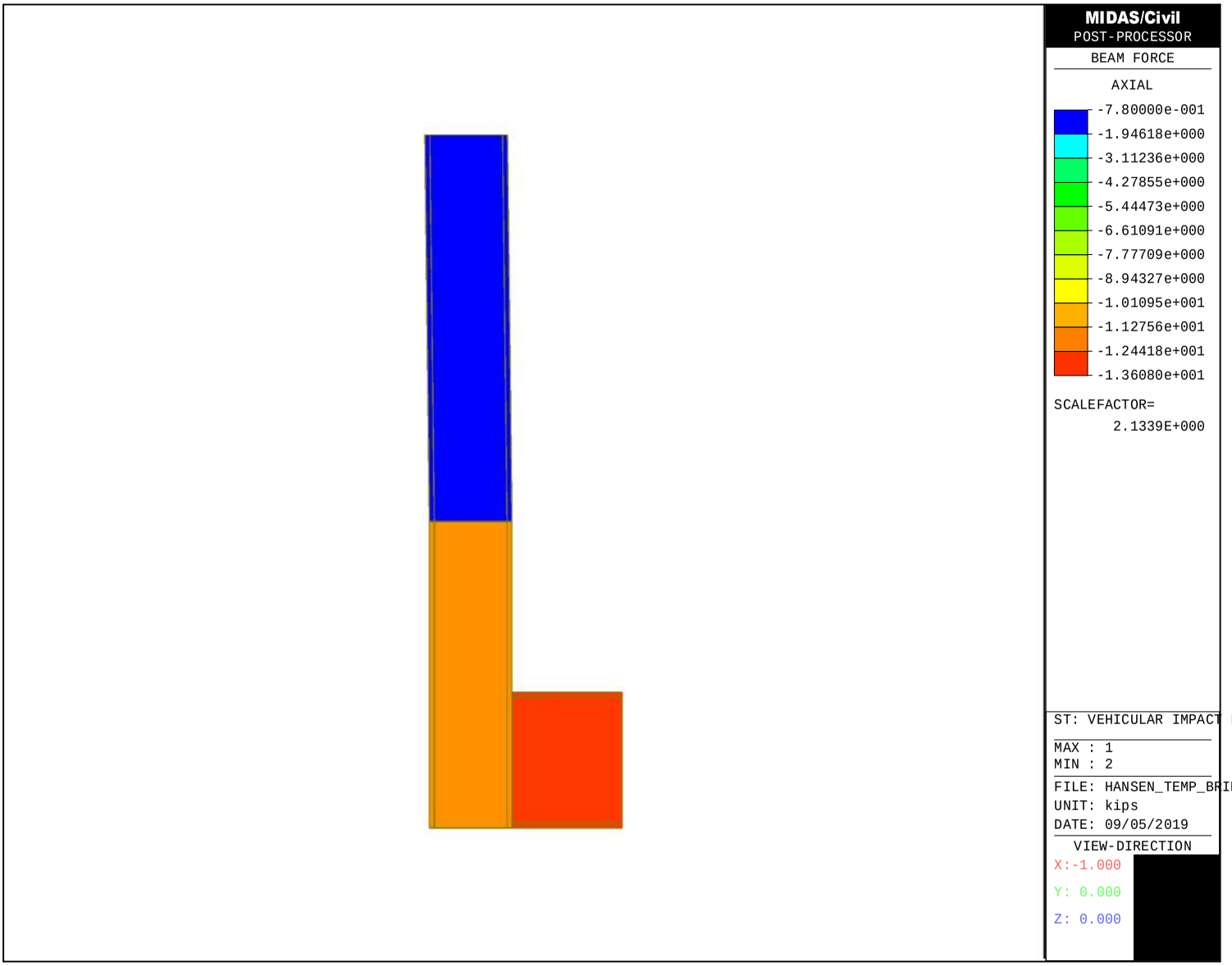
Table A13.2-1—Design Forces for Traffic Railings

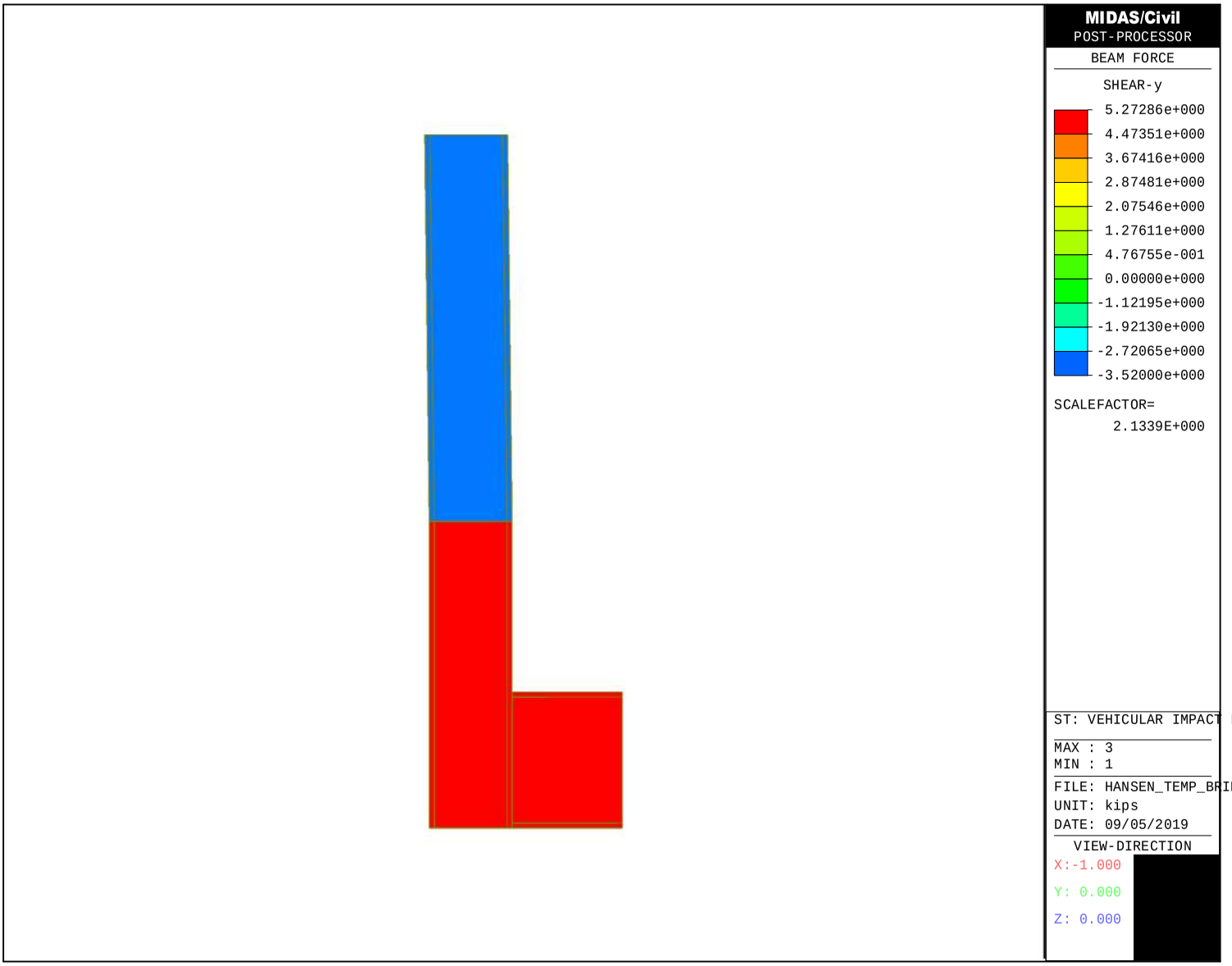
Design Forces and Designations	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips) Down	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum H Height of Rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0

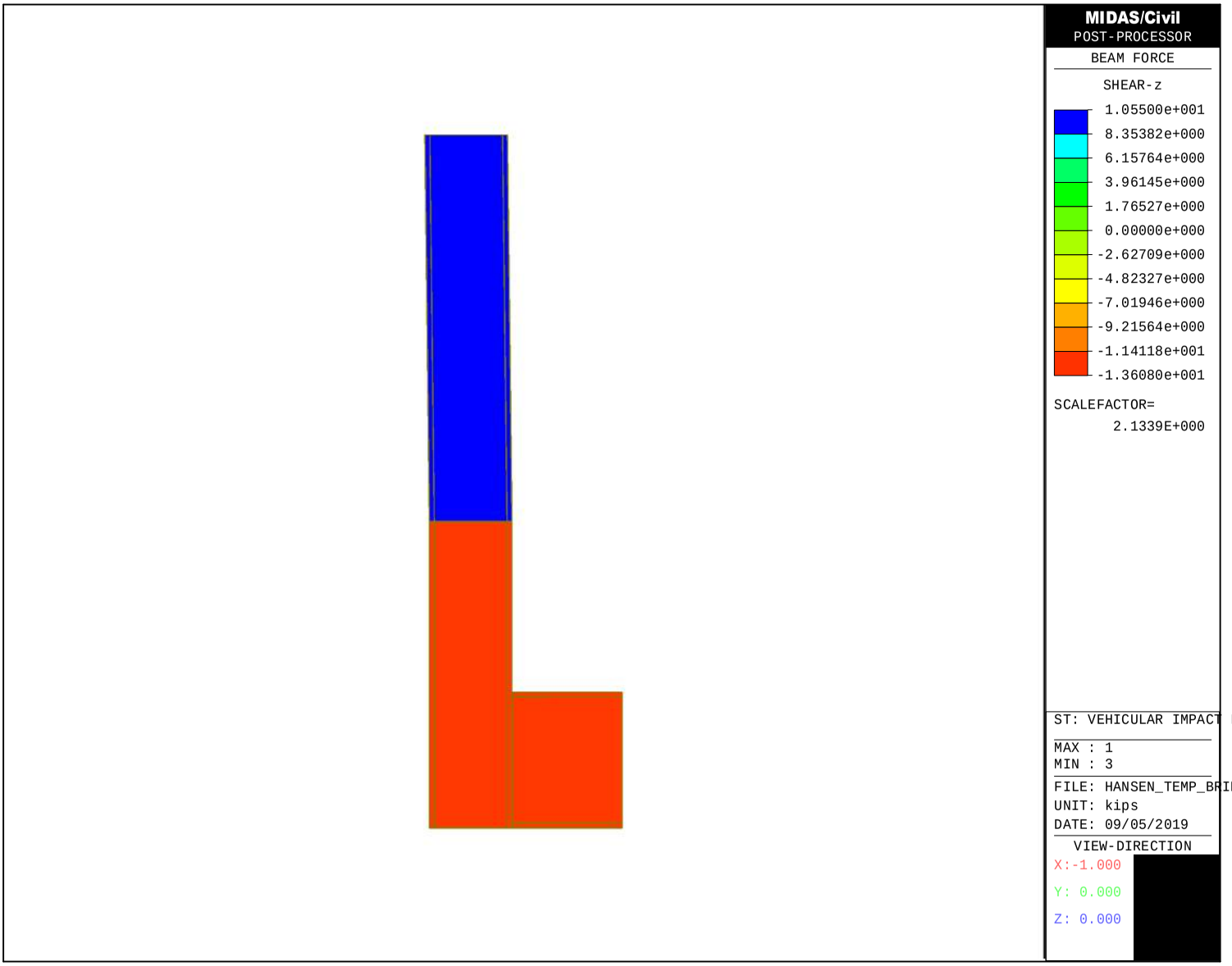
Transverse Force	$F_t := 13.5\ kip$	LRFD TABLE A13.2-1
Longitudinal Force	$F_L := 4.5\ kip$	LRFD TABLE A13.2-1
Vertical Force	$F_v := 4.5\ kip$	LRFD TABLE A13.2-1
Length of Significant contact in Transverse Direction	$L_t := 4\ ft$	LRFD TABLE A13.2-1
Length of Significant contact in Longitudinal Direction	$L_L := 4\ ft$	LRFD TABLE A13.2-1
Length of Significant contact in Vertical Direction	$L_v := 18\ ft$	LRFD TABLE A13.2-1
Transverse Force per unit length	$F_{tunit} := \frac{F_t}{L_t} = 3.38\ \frac{kip}{ft}$	
Longitudinal Force per unit length	$F_{Lunit} := \frac{F_L}{L_L} = 1.13\ \frac{kip}{ft}$	
Vertical Force per unit length	$F_{vunit} := \frac{F_v}{L_v} = 0.25\ \frac{kip}{ft}$	
Spacing of posts	$S_{post} := 37.5\ in$	
Transverse Force applied at top of railing post	$F_{transverse} := F_{tunit} \cdot S_{post} = 10.55\ kip$	
Longitudinal Force applied at top of railing post	$F_{longitudinal} := F_{Lunit} \cdot S_{post} = 3.52\ kip$	
Transverse Force applied at top of railing post	$F_{vertical} := F_{vunit} \cdot S_{post} = 0.78\ kip$	

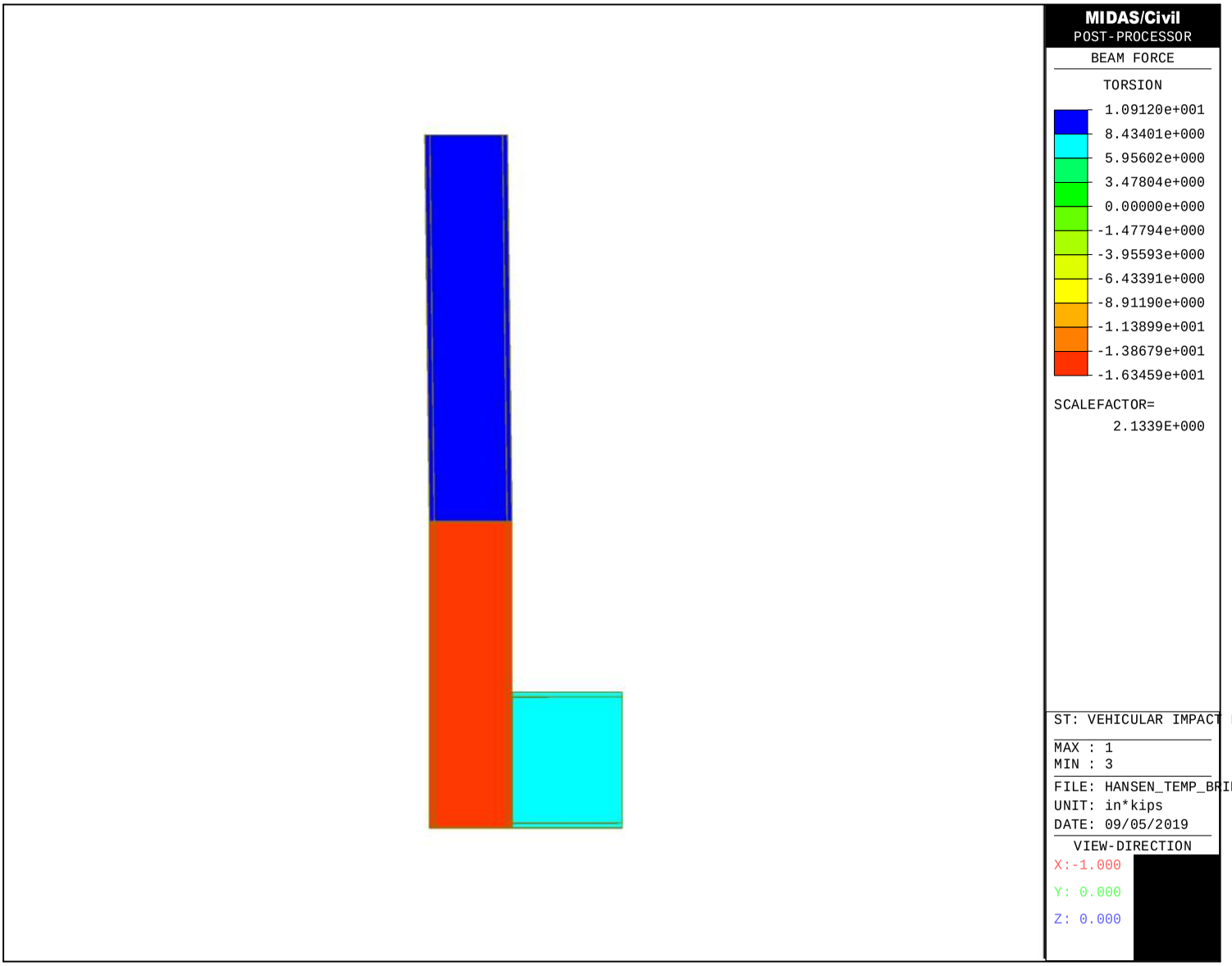
Loading condition for railing subject to vehicular impact force (for TL-1 Railing)

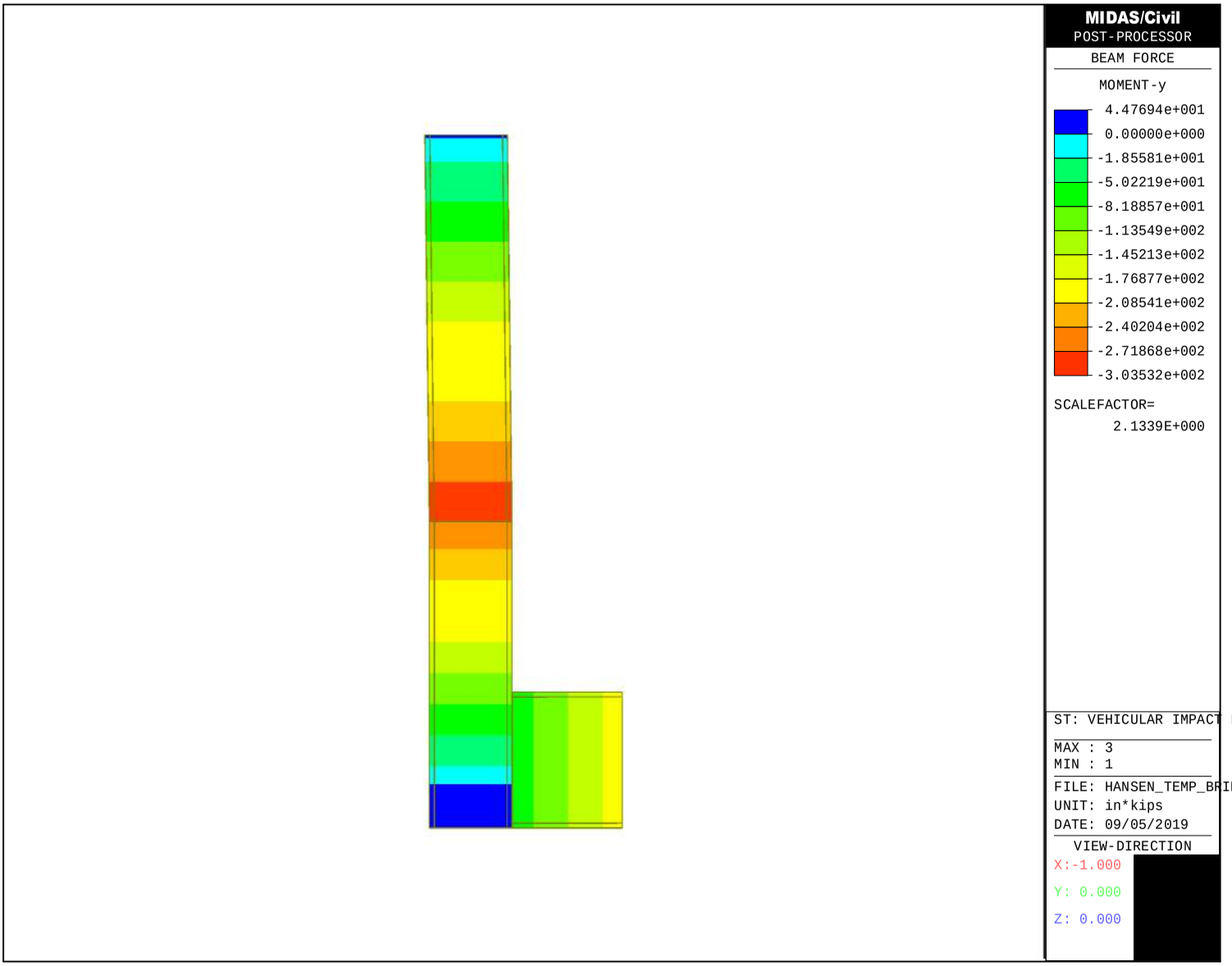


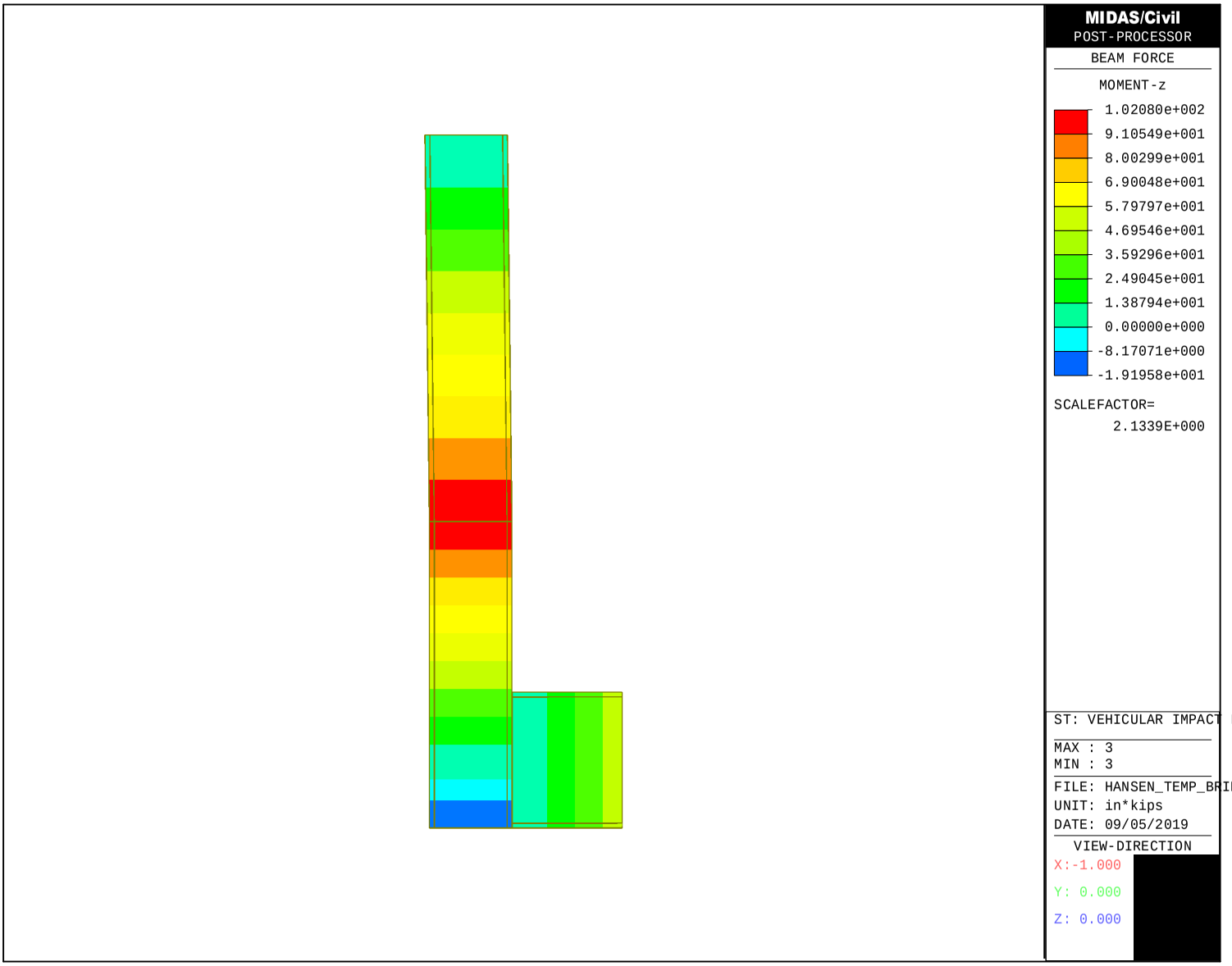

MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
AXIAL
-7.80000e-001
-1.94618e+000
-3.11236e+000
-4.27855e+000
-5.44473e+000
-6.61091e+000
-7.77709e+000
-8.94327e+000
-1.01095e+001
-1.12756e+001
-1.24418e+001
-1.36080e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 2
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
SHEAR-y
5.27286e+000
4.47351e+000
3.67416e+000
2.87481e+000
2.07546e+000
1.27611e+000
4.76755e-001
0.00000e+000
-1.12195e+000
-1.92130e+000
-2.72065e+000
-3.52000e+000
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 1
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
SHEAR-z
1.05500e+001
8.35382e+000
6.15764e+000
3.96145e+000
1.76527e+000
0.00000e+000
-2.62709e+000
-4.82327e+000
-7.01946e+000
-9.21564e+000
-1.14118e+001
-1.36080e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
TORSION
1.09120e+001
8.43401e+000
5.95602e+000
3.47804e+000
0.00000e+000
-1.47794e+000
-3.95593e+000
-6.43391e+000
-8.91190e+000
-1.13899e+001
-1.38679e+001
-1.63459e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
MOMENT-y
4.47694e+001
0.00000e+000
-1.85581e+001
-5.02219e+001
-8.18857e+001
-1.13549e+002
-1.45213e+002
-1.76877e+002
-2.08541e+002
-2.40204e+002
-2.71868e+002
-3.03532e+002
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 1
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
MOMENT-z
1.02080e+002
9.10549e+001
8.00299e+001
6.90048e+001
5.79797e+001
4.69546e+001
3.59296e+001
2.49045e+001
1.38794e+001
0.00000e+000
-8.17071e+000
-1.91958e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                Version 8.5.6
================================================================================


       +============================================================+
       |  MIDAS(Modeling, Integrated Design & Analysis Software)    |
       |  MIDAS/Civil - Design & checking system for windows        |
       +============================================================+
       |  Steel Member Applicable Code Checking                     |
       |  Based On  AASHTO-LRFD12, AASHTO-LRFD02, AASHTO-LFD96,     |
       |            AASHTO-ASD96, AISC-LRFD2K, AISC-LRFD93,         |
       |            AISC-ASD89, Eurocode3-2:05, BS5950-90,          |
       |            IS:800-2007, IS:800-1984                        |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                          (c)SINCE 1989     |
       +============================================================+
       |  MIDAS Information Technology Co.,Ltd.         (MIDAS IT)  |
       |  MIDAS IT Design Development Team                          |
       +============================================================+
       |            HomePage : www.MidasUser.com                    |
       +============================================================+
       |  MIDAS/Civil  Version 8.5.6                                |
       +============================================================+


    *. DEFINITION OF LOAD COMBINATIONS WITH SCALING UP FACTORS.
--------------------------------------------------------------------------------
     LCB  C    Loadcase Name(Factor) + Loadcase Name(Factor) + Loadcase Name(Factor)
--------------------------------------------------------------------------------
       1  1   Vehicular Impact Loa(-1.000) +   Self-Weight(-1.000)
--------------------------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      *. PROJECT     :
      *. MEMBER NO   =      1,  ELEMENT TYPE = Beam
      *. LOADCOMB NO =      1,  MATERIAL NO  =     1,  SECTION NO =     1
      *. UNIT SYSTEM : kips, in

      *. SECTION PROPERTIES : Designation = W6X20
         Shape     = I - Section. (Rolled)
         Depth     =      6.200,  Top F Width =      6.020,  Bot.F Width =      6.020
         Web Thick =      0.260,  Top F Thick =      0.365,  Bot.F Thick =      0.365

         Area = 5.87000e+000,  Asy  = 2.92973e+000,  Asz  = 1.61200e+000
         Ybar = 3.01000e+000,  Zbar = 3.10000e+000,  Qyb  = 2.83963e+001,  Qzb  = 4.53005e+000
         Syy  = 1.34000e+001,  Szz  = 4.41000e+000,  Zyy  = 1.50000e+001,  Zzz  = 6.72000e+000
         Iyy  = 4.14000e+001,  Izz  = 1.33000e+001,  Iyz  = 0.00000e+000
         ry   = 2.66000e+000,  rz   = 1.50000e+000
         J    = 2.40000e-001,  Cwp  = 1.12967e+002

      *. DESIGN PARAMETERS FOR STRENGTH EVALUATION :
         Ly   = 2.90000e+001,  Lz   = 2.90000e+001,  Lb   = 2.90000e+001
         Ky   = 1.00000e+000,  Kz   = 1.00000e+000

      *. MATERIAL PROPERTIES :
         Fy   = 5.00000e+001,  Es   = 2.90000e+004,  MATERIAL NAME  = A709-50W


================================================================================
   [[[*]]]   COMPUTE MOMENT MAGNIFICATION FACTORS AND MAGNIFIED MOMENTS.
================================================================================

      ( ). Compute moment magnification factors(B1y,B1z).
           -. If tension or bending member.
           -. Assumed   B1y  =  1.00
           -. Assumed   B1z  =  1.00

      ( ). Magnification factors for sidesway moments(B2y,B2z).
           -. B2y   =  1.00 (Default value)
           -. B2z   =  1.00 (Default value)

      ( ). Given factored axial forces and moments at <J>.
           ------------------------------------------------------------
            Load Case             Pu                 My                 Mz
           ------------------------------------------------------------
            DL                  0.05              0.00               0.00
            LL                  0.78            303.53            -102.08
            DL+LL               0.83            303.53            -102.08
            WL or EL            0.00              0.00               0.00
           ------------------------------------------------------------
            DL+LL+WL(EL)        0.83            303.53            -102.08
           ------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Compute magnified moments.
           -. Muy   =  B1y*My(DL+LL) + B2y*My(WL(EL)) =      303.53 in-kips.
           -. Muz   =  B1z*Mz(DL+LL) + B2z*Mz(WL(EL)) =     -102.08 in-kips.

================================================================================
   [[[*]]]   CHECK AXIAL STRENGTH.
================================================================================

      ( ). Check slenderness ratio of axial tension member (l/r).
           [ AASHTO-LRFD12 Specification 6.8.4 ]
           -. l/r  =    19.3  <   200.0  --->  O.K.

      ( ). Calculate axial tensile strength (phiPn).
           [ AASHTO-LRFD12 Specification 6.8.2.1-1 ]
           -. Resistance factor for tension  :  phi = 0.95
           -. phiPn  =  phi*Fy*Ag  =        278.82 kips.

      ( ). Check ratio of axial strength (Pu/phiPn).
                Pu                0.83
           -. -------  =  --------------  =  0.003  <  1.000  --->  O.K.
               phiPn             278.82

================================================================================
   [[[*]]]   CHECK SECTION PROPORTION LIMITS
================================================================================

      ( ). Check Web Proportions (AASHTO-LRFD12 Eq.6.10.2.1.1-1)

           -. Limit Value  = 150
           -. D/tw  =   21.04  <  Limit Value  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-1)

           -. bf/2tf  =    8.25  <     12.00  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-2)

           -. bf  >  D/6 =    0.91  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-3)

           -. tf > 1.1*tw =    0.29  --->  O.K.

      ( ). Check general proportions
                      Izc               6.6359
           -. 0.1  <  ------  =  ----------------  =  1.000  <  10    --->  O.K.
                      Izt               6.6359
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      => Flextural Resistance is Checked by AASHTO-LRFD12 Specification

================================================================================
   [[[*]]]   CHECK SECTION SLENDERNESS
================================================================================

      ( ). Check Flange ratio
                Izc               6.6359
           -. ------  =  ----------------  =  1.000  >  0.3
                Izt               6.6359

      ( ). Check web slenderness
           -. Limit Value  =  5.7*SQRT[E/Fyc]  =  137.27

           -. 2Dc/tw  =   21.04  <  Limit Value

           -. Fyc  =50ksi  <  70.0 ksi

      => Section is Checking by AASHTO-LRFD12 Appendix 6

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MAJOR AXIS.
================================================================================

      ( ). Check Web Compactness Length
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Dcp   =    2.74 in.
           -. Dc    =    2.74 in.
           -. Mp    =  739.77 in-kips.
           -. My    =  667.74 in-kips.
           -. Rh    =    1.00(homogenous section)
           -. Lambda_rw = 5.7*SQRT[Es/Fyc] =   137.27

                                          SQRT[Es/Fyc]
           -. Lambda_pw(Dcp)_cal = ------------------------ =    93.23
                                   (0.54*Mp/(Rh*My)-0.09)^2

           -. Lambda_pw(Dcp)_lim = Lambda_rw * (Dcp/Dc)    =   137.27
           -. Lambda_pw(Dcp)     = min[Lambda_pw(Dcp)_cal, Lambda_pw(Dcp)_lim ] =    93.23
           -. Lambda_w(Dcp)      = 2*Dcp/tw =    21.04

           -. Lambda_w(Dcp) < Lambda_pw(Dcp)  --->  compact web

      ( ). Web Plastification Factor
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Myc  = Fyc*Iyy/yc =  667.74 in-kips.
           -. Myt  = Fyt*Iyy/yt =  667.74 in-kips.
           -. Rpc  =  Mp/Myc  =    1.11
           -. Rpt  =  Mp/Myt  =    1.11
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check compression-flange compactness
           -. Lambda_pf  =  0.38*SQRT[E/Fyc]  =    9.15
           -. Lambda_f = bf/2tf  =    8.25  <  Lambda_pf  --->  Compact.

      ( ). Compute nominal flexural strength of the Compression Flange (Mnc).
           [ AASHTO-LRFD12 Specification A6.3.2]
           -. Rpc =    1.11
           -. Myc =  667.74 in-kips.
           -. Mnc_FLB  =  Rpc*Myc  =  739.77 in-kips.

      ( ). Effective Radius of Gyration for Lateral Torsional Buckling (rt)
           -. depth of the web in compression(Dc)  =    2.74 in.
                                 bfc
           -. rt = ------------------------------------ =     1.65 in.
                      [    (    1    Dc*tw   ) ]
                 SQRT[ 12*(1 + - * -------- ) ]
                      [    (    3   bfc*tfc  ) ]

      ( ). Unbraced Length(Lb) Check
           -. Lb  =   29.00 in.
           -. Lp  = 1.0*rt*SQRT(Es/Fyc) =   39.76 in.

           -. Lb < Lp  --->  compact unbraced length

      ( ). Compute Lateral Torsional Buckling Resistance of Compression Flange (Mnc_LTB).
           [ AASHTO-LRFD12 Specification A6.3.3]
           -. Rpc  =    1.11
           -. Myc  =  667.74 in-kips.
           -. Mnc_LTB =  Rpc*Myc  =  739.77 in-kips.

      ( ). Compute nominal flexural strength of Compression Flange (Mnc)
           -. Mnc  =  min[Mnc_FLB, Mnc_LTB] =  739.77 in-kips.

      ( ). Compute nominal flexural strength of Tension Flange (Mnt).
           [ AASHTO-LRFD12 Specification A6.4]
           -. Rpt  =    1.11
           -. Myt  =        667.74 in-kips.
           -. Mnt  =  Rpt*Myt  =        739.77 in-kips.

           -. Mn  =  min(Mnc, Mnt)  =        739.77 in-kips.

      ( ). Compute flexural strength about major axis (phiMny).
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMny  =  phi*Mny  =        739.77 in-kips.
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check ratio of flexural strength (Muy/phiMny).
                Muy               303.53
           -. --------  =  ---------------  =  0.410  <  1.000  --->  O.K.
               phiMny             739.77

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MINOR AXIS.
================================================================================

      ( ). Slenderness ratio for flange (lamda_f)
           [ AASHTO-LRFD12 Specification 6.12.2.2]
           -. lamda_f   = bf/(2*tf) =     8.25
           -. lamda_pf  = 0.038*SQRT(Es/Fyf) =     0.92
           -. lamda_rf  = 0.83*SQRT(Es/Fyf) =    19.99

           -. lamda_pf < lamda_f < lamda_rf  --->  noncompact flange

      ( ). Compute nominal flexural strength of the flange (Mn).
           [ AASHTO-LRFD12 Specification 6.12.2.2]

                   [    (     Szz )   ( Lambda_f  - Lambda_pf )]
           -. Mn = [1 - (1 - -----) * (-----------------------)]*Fy*Zzz =   257.87 in-kips.
                   [    (     Zzz )   (    0.45*SQRT[Es/Fy]   )]

      ( ). Compute flexural strength about minor axis (phiMnz).
           -. Mnz  =        257.87 in-kips.
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMnz  =  phi*Mnz  =        257.87 in-kips.

      ( ). Check ratio of flexural strength (Muz/phiMnz).
                Muz               102.08
           -. --------  =  ---------------  =  0.396  <  1.000  --->  O.K.
               phiMnz             257.87

================================================================================
   [[[*]]]   CHECK INTERACTION OF COMBINED STRENGTH.
================================================================================

      ( ). Check interaction ratio of combined strength.
           [ AASHTO-LRFD12 Specification 6.9.2.2 ]
           -. Pu/phiPn  <  0.20  --->  Formula(6.9.2.2-2)
                            Pu       [    Muy        Muz   ]
           -. ComRat  =  --------- + [ -------- + -------- ]
                          2*phiPn    [  phiMny     phiMnz  ]
                      =  0.001 + [  0.410  +  0.396 ]
                      =  0.808  <  1.000  --->  O.K.

================================================================================
   [[[*]]]   CHECK SHEAR STRENGTH.
================================================================================
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


     ( ). Calculate nominal shear strength in local-y direction (Vny).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asy  =         84.96 kips.
          -. Vny =  C*Vp  =         84.96 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuy =          3.52 kips.
                Vuy                3.52
          -. --------  =  --------------  =  0.041  <  1.000  --->  O.K.
              phiVny              84.96

     ( ). Calculate nominal shear strength in local-z direction (Vnz).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asz  =         46.75 kips.
          -. Vnz =  C*Vp  =         46.75 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuz =         10.55 kips.
                Vuz               10.55
          -. --------  =  --------------  =  0.226  <  1.000  --->  O.K.
              phiVnz              46.75
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                Version 8.5.6
================================================================================


       +============================================================+
       |  MIDAS(Modeling, Integrated Design & Analysis Software)    |
       |  MIDAS/Civil - Design & checking system for windows        |
       +============================================================+
       |  Steel Member Applicable Code Checking                     |
       |  Based On  AASHTO-LRFD12, AASHTO-LRFD02, AASHTO-LFD96,     |
       |            AASHTO-ASD96, AISC-LRFD2K, AISC-LRFD93,         |
       |            AISC-ASD89, Eurocode3-2:05, BS5950-90,          |
       |            IS:800-2007, IS:800-1984                        |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                          (c)SINCE 1989     |
       +============================================================+
       |  MIDAS Information Technology Co.,Ltd.         (MIDAS IT)  |
       |  MIDAS IT Design Development Team                          |
       +============================================================+
       |            HomePage : www.MidasUser.com                    |
       +============================================================+
       |  MIDAS/Civil  Version 8.5.6                                |
       +============================================================+


    *. DEFINITION OF LOAD COMBINATIONS WITH SCALING UP FACTORS.
--------------------------------------------------------------------------------
     LCB  C    Loadcase Name(Factor) + Loadcase Name(Factor) + Loadcase Name(Factor)
--------------------------------------------------------------------------------
       1  1   Vehicular Impact Loa(-1.000) +   Self-Weight(-1.000)
--------------------------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      *. PROJECT     :
      *. MEMBER NO   =      2,  ELEMENT TYPE = Beam
      *. LOADCOMB NO =      1,  MATERIAL NO  =     1,  SECTION NO =     2
      *. UNIT SYSTEM : kips, in

      *. SECTION PROPERTIES : Designation = W10X22
         Shape     = I - Section. (Rolled)
         Depth     =     10.200,  Top F Width =      5.750,  Bot.F Width =      5.750
         Web Thick =      0.240,  Top F Thick =      0.360,  Bot.F Thick =      0.360

         Area = 6.49000e+000,  Asy  = 2.76000e+000,  Asz  = 2.44800e+000
         Ybar = 2.87500e+000,  Zbar = 5.10000e+000,  Qyb  = 5.36688e+001,  Qzb  = 4.13281e+000
         Syy  = 2.32000e+001,  Szz  = 3.97000e+000,  Zyy  = 2.60000e+001,  Zzz  = 6.10000e+000
         Iyy  = 1.18000e+002,  Izz  = 1.14000e+001,  Iyz  = 0.00000e+000
         ry   = 4.27000e+000,  rz   = 1.33000e+000
         J    = 2.39000e-001,  Cwp  = 2.76112e+002

      *. DESIGN PARAMETERS FOR STRENGTH EVALUATION :
         Ly   = 8.28125e+000,  Lz   = 8.28125e+000,  Lb   = 8.28125e+000
         Ky   = 1.00000e+000,  Kz   = 1.00000e+000

      *. MATERIAL PROPERTIES :
         Fy   = 5.00000e+001,  Es   = 2.90000e+004,  MATERIAL NAME  = A709-50W


================================================================================
   [[[*]]]   COMPUTE MOMENT MAGNIFICATION FACTORS AND MAGNIFIED MOMENTS.
================================================================================

      ( ). Compute moment magnification factors(B1y,B1z).
           -. If tension or bending member.
           -. Assumed   B1y  =  1.00
           -. Assumed   B1z  =  1.00

      ( ). Magnification factors for sidesway moments(B2y,B2z).
           -. B2y   =  1.00 (Default value)
           -. B2z   =  1.00 (Default value)

      ( ). Given factored axial forces and moments at <I>.
           ---------------------------------------------------------------
            Load Case             Pu                My                Mz
           ---------------------------------------------------------------
            DL                  0.01              0.20              0.00
            LL                 13.61            163.17            -43.67
            DL+LL              13.62            163.37            -43.67
            WL or EL            0.00              0.00              0.00
           ---------------------------------------------------------------
            DL+LL+WL(EL)       13.62            163.37            -43.67
           ---------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Compute magnified moments.
           -. Muy   =  B1y*My(DL+LL) + B2y*My(WL(EL)) =      163.37 in-kips.
           -. Muz   =  B1z*Mz(DL+LL) + B2z*Mz(WL(EL)) =      -43.67 in-kips.

================================================================================
   [[[*]]]   CHECK AXIAL STRENGTH.
================================================================================

      ( ). Check slenderness ratio of axial tension member (l/r).
           [ AASHTO-LRFD12 Specification 6.8.4 ]
           -. l/r  =     6.2  <   200.0  --->  O.K.

      ( ). Calculate axial tensile strength (phiPn).
           [ AASHTO-LRFD12 Specification 6.8.2.1-1 ]
           -. Resistance factor for tension  :  phi = 0.95
           -. phiPn  =  phi*Fy*Ag  =        308.28 kips.

      ( ). Check ratio of axial strength (Pu/phiPn).
                Pu                13.62
           -. -------  =  --------------  =  0.044  <  1.000  --->  O.K.
               phiPn             308.28

================================================================================
   [[[*]]]   CHECK SECTION PROPORTION LIMITS
================================================================================

      ( ). Check Web Proportions (AASHTO-LRFD12 Eq.6.10.2.1.1-1)

           -. Limit Value  = 150
           -. D/tw  =   39.50  <  Limit Value  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-1)

           -. bf/2tf  =    7.99  <     12.00  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-2)

           -. bf  >  D/6 =    1.58  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-3)

           -. tf > 1.1*tw =    0.26  --->  O.K.

      ( ). Check general proportions
                      Izc               5.7033
           -. 0.1  <  ------  =  ----------------  =  1.000  <  10    --->  O.K.
                      Izt               5.7033
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      => Flextural Resistance is Checked by AASHTO-LRFD12 Specification

================================================================================
   [[[*]]]   CHECK SECTION SLENDERNESS
================================================================================

      ( ). Check Flange ratio
                Izc               5.7033
           -. ------  =  ----------------  =  1.000  >  0.3
                Izt               5.7033

      ( ). Check web slenderness
           -. Limit Value  =  5.7*SQRT[E/Fyc]  =  137.27

           -. 2Dc/tw  =   39.50  <  Limit Value

           -. Fyc  =50ksi  <  70.0 ksi

      => Section is Checking by AASHTO-LRFD12 Appendix 6

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MAJOR AXIS.
================================================================================

      ( ). Check Web Compactness Length
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Dcp   =    4.74 in.
           -. Dc    =    4.74 in.
           -. Mp    = 1292.01 in-kips.
           -. My    = 1156.86 in-kips.
           -. Rh    =    1.00(homogenous section)
           -. Lambda_rw = 5.7*SQRT[Es/Fyc] =   137.27

                                          SQRT[Es/Fyc]
           -. Lambda_pw(Dcp)_cal = ------------------------ =    91.48
                                   (0.54*Mp/(Rh*My)-0.09)^2

           -. Lambda_pw(Dcp)_lim = Lambda_rw * (Dcp/Dc)    =   137.27
           -. Lambda_pw(Dcp)     = min[Lambda_pw(Dcp)_cal, Lambda_pw(Dcp)_lim ] =    91.48
           -. Lambda_w(Dcp)      = 2*Dcp/tw =    39.50

           -. Lambda_w(Dcp) < Lambda_pw(Dcp)  --->  compact web

      ( ). Web Plastification Factor
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Myc  = Fyc*Iyy/yc = 1156.86 in-kips.
           -. Myt  = Fyt*Iyy/yt = 1156.86 in-kips.
           -. Rpc  =  Mp/Myc  =    1.12
           -. Rpt  =  Mp/Myt  =    1.12
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check compression-flange compactness
           -. Lambda_pf  =  0.38*SQRT[E/Fyc]  =    9.15
           -. Lambda_f = bf/2tf  =    7.99  <  Lambda_pf  --->  Compact.

      ( ). Compute nominal flexural strength of the Compression Flange (Mnc).
           [ AASHTO-LRFD12 Specification A6.3.2]
           -. Rpc =    1.12
           -. Myc = 1156.86 in-kips.
           -. Mnc_FLB  =  Rpc*Myc  = 1292.01 in-kips.

      ( ). Effective Radius of Gyration for Lateral Torsional Buckling (rt)
           -. depth of the web in compression(Dc)  =    4.74 in.
                                 bfc
           -. rt = ------------------------------------ =     1.53 in.
                      [    (    1    Dc*tw   ) ]
                 SQRT[ 12*(1 + - * -------- ) ]
                      [    (    3   bfc*tfc  ) ]

      ( ). Unbraced Length(Lb) Check
           -. Lb  =    8.28 in.
           -. Lp  = 1.0*rt*SQRT(Es/Fyc) =   36.75 in.

           -. Lb < Lp  --->  compact unbraced length

      ( ). Compute Lateral Torsional Buckling Resistance of Compression Flange (Mnc_LTB).
           [ AASHTO-LRFD12 Specification A6.3.3]
           -. Rpc  =    1.12
           -. Myc  = 1156.86 in-kips.
           -. Mnc_LTB =  Rpc*Myc  = 1292.01 in-kips.

      ( ). Compute nominal flexural strength of Compression Flange (Mnc)
           -. Mnc  =  min[Mnc_FLB, Mnc_LTB] = 1292.01 in-kips.

      ( ). Compute nominal flexural strength of Tension Flange (Mnt).
           [ AASHTO-LRFD12 Specification A6.4]
           -. Rpt  =    1.12
           -. Myt  =       1156.86 in-kips.
           -. Mnt  =  Rpt*Myt  =       1292.01 in-kips.

           -. Mn  =  min(Mnc, Mnt)  =       1292.01 in-kips.

      ( ). Compute flexural strength about major axis (phiMny).
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMny  =  phi*Mny  =       1292.01 in-kips.
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check ratio of flexural strength (Muy/phiMny).
                Muy               163.37
           -. --------  =  ---------------  =  0.126  <  1.000  --->  O.K.
               phiMny            1292.01

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MINOR AXIS.
================================================================================

      ( ). Slenderness ratio for flange (lamda_f)
           [ AASHTO-LRFD12 Specification 6.12.2.2]
           -. lamda_f   = bf/(2*tf) =     7.99
           -. lamda_pf  = 0.038*SQRT(Es/Fyf) =     0.92
           -. lamda_rf  = 0.83*SQRT(Es/Fyf) =    19.99

           -. lamda_pf < lamda_f < lamda_rf  --->  noncompact flange

      ( ). Compute nominal flexural strength of the flange (Mn).
           [ AASHTO-LRFD12 Specification 6.12.2.2]

                   [    (     Szz )   ( Lambda_f  - Lambda_pf )]
           -. Mn = [1 - (1 - -----) * (-----------------------)]*Fy*Zzz =   235.51 in-kips.
                   [    (     Zzz )   (    0.45*SQRT[Es/Fy]   )]

      ( ). Compute flexural strength about minor axis (phiMnz).
           -. Mnz  =        235.51 in-kips.
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMnz  =  phi*Mnz  =        235.51 in-kips.

      ( ). Check ratio of flexural strength (Muz/phiMnz).
                Muz                43.67
           -. --------  =  ---------------  =  0.185  <  1.000  --->  O.K.
               phiMnz             235.51

================================================================================
   [[[*]]]   CHECK INTERACTION OF COMBINED STRENGTH.
================================================================================

      ( ). Check interaction ratio of combined strength.
           [ AASHTO-LRFD12 Specification 6.9.2.2 ]
           -. Pu/phiPn  <  0.20  --->  Formula(6.9.2.2-2)
                            Pu       [    Muy        Muz   ]
           -. ComRat  =  --------- + [ -------- + -------- ]
                          2*phiPn    [  phiMny     phiMnz  ]
                      =  0.022 + [  0.126  +  0.185 ]
                      =  0.334  <  1.000  --->  O.K.

================================================================================
   [[[*]]]   CHECK SHEAR STRENGTH.
================================================================================
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


     ( ). Calculate nominal shear strength in local-y direction (Vny).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asy  =         80.04 kips.
          -. Vny =  C*Vp  =         80.04 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuy =          5.27 kips.
                Vuy                5.27
          -. --------  =  --------------  =  0.066  <  1.000  --->  O.K.
              phiVny              80.04

     ( ). Calculate nominal shear strength in local-z direction (Vnz).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asz  =         70.99 kips.
          -. Vnz =  C*Vp  =         70.99 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    I )
          -. Applied shear force : Vuz =         12.20 kips.
                Vuz                12.20
          -. --------  =  --------------  =  0.172  <  1.000  --->  O.K.
              phiVnz              70.99
```

REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)
MIDAS	-	Midas Civil Model for TL-1 Rail Loads

Unit Definition:

$$kcf := 1000\ pcf$$

Phi factor for weld metal in fillet welds	$\phi_{e2} := 0.80$	LRFD 6.5.4.2
Classification strength of weld metal (ksi)	$F_{exx} := 70\ ksi$	Assumed
Factored shear resistance of the weld metal (for Fillet-Welded Connections)	$R_r := 0.6 \cdot \phi_{e2} \cdot F_{exx} = 33.6\ ksi$	LRFD 6.13.3.2.4
Vector addition of the shear forces on the weld		LRFD C6.13.3.2.4 MIDAS

$$F_{resultant} := \sqrt{13.61^2 + 5.28^2 + 13.61^2}\ kip = 20.0\ kip$$

Length of weld along flange	$L_{flange} := 5\ in$	Assumed
Number of welds along both flanges	$N_{flange} := 2$	Assumed
Length of weld along web	$L_{web} := 8.25\ in$	Assumed
Number of welds along web	$N_{web} := 2$	Assumed
Thickness of welds	$t_{weld} := 0.25\ in$	PLANS
Factored capacity of shear resistance of weld metal	$R_{r\_weld} := R_r \cdot 0.707 \cdot t_{weld} \cdot \left(L_{web} \cdot N_{web} + L_{flange} \cdot N_{flange}\right) = 157.4\ kip$	
Is resistance from fillet weld greater than the resultant force applied?	$\text{if}\left(R_{r\_weld} > F_{resultant}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$	

1/2" x 7" x 13-1/4" Plate

Phi factor for shear	$\phi_v := 1.00$	LRFD 6.5.4.2
Phi factor for shear, rupture in connection element	$\phi_{vu} := 0.80$	LRFD 6.5.4.2
Yield strength of connection element	$F_y := 50\ ksi$	
Tensile strength of connection element	$F_u := 65.3\ ksi$	
Gross area of connection element (in global y direction)	$A_{vgply} := 0.5\ in \cdot 13.5\ in = 6.75\ in^2$	
Gross area of connection element (in global z direction)	$A_{vgplz} := 0.5\ in \cdot 7\ in = 3.5\ in^2$	
Factored shear yielding resistance of the connection element (in global y direction)	$R_{r\_yieldingply} := \phi_v \cdot 0.58 \cdot F_y \cdot A_{vgply} = 195.8\ kip$	LRFD 6.13.5.3-1
Factored shear yielding resistance of the connection element (in global z direction)	$R_{r\_yieldingplz} := \phi_v \cdot 0.58 \cdot F_y \cdot A_{vgplz} = 101.5\ kip$	LRFD 6.13.5.3-1
Reduction factor for bolt holes punched full size	$R_p := 0.90$	LRFD 6.13.5.3
Net area of connection element (in global y direction)	$A_{vny} := A_{vgply} - 3 \cdot \frac{13}{16}\ in \cdot 0.5\ in = 5.53\ in^2$	
Net area of connection element (in global z direction)	$A_{vnz} := A_{vgplz} - 2 \cdot \frac{13}{16}\ in \cdot 0.5\ in = 2.69\ in^2$	
Factored shear rupture resistance of the connection element (in global y direction)	$R_{r\_shear\_rupturey} := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot A_{vny} = 150.8\ kip$	
Factored shear rupture resistance of the connection element (in global z direction)	$R_{r\_shear\_rupturez} := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot A_{vnz} = 73.3\ kip$	

Global y direction factored shear force

$F_{force\_y} := 5.27\ kip$

Global z direction factored shear force

$F_{force\_z} := 13.62\ kip$

Minimum of Factored shear yielding and shear rupture resistance of the connection element (in global y direction)

$R_{r\_yielding\_shear\_rupturey} := min\left(R_{r\_yieldingply}, R_{r\_shear\_rupturey}\right) = 150.8\ kip$

Minimum of Factored shear yielding and shear rupture resistance of the connection element (in global z direction)

$R_{r\_yielding\_shear\_rupturez} := min\left(R_{r\_yieldingplz}, R_{r\_shear\_rupturez}\right) = 73.3\ kip$

Is shear yielding/shear rupture resistance greater than the resultant force applied? (in global y direction)

$\text{if}\left(R_{r\_yielding\_shear\_rupturey} > F_{force\_y}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Is shear yielding/shear rupture resistance greater than the resultant force applied? (in global z direction)

$\text{if}\left(R_{r\_yielding\_shear\_rupturez} > F_{force\_z}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

3/4" Bolts

Area of bolts

$A_b := \frac{\pi}{4} \cdot \left(\frac{3}{4}\ in\right)^2 = 0.44\ in^2$

Specified minimum tensile strength of the bolt (assume A307 bolts)

$F_{ub} := 60\ ksi$

Number of shear planes per bolt

$N_s := 1$

Number of bolts

$N_{bolts} := 6$

Nominal Shear Resistance of A307 bolts

$R_{n\_shear\_bolts} := 0.45 \cdot A_b \cdot F_{ub} \cdot N_s \cdot N_{bolts} = 71.57\ kip$ LRFD 6.13.2.7-2

Maximum shear force applied to bolts

$F_{shear\_bolts} := 13.61\ kip$

Is nominal shear resistance of A307 bolts greater than the maximum force applied?

$\text{if}\left(R_{n\_shear\_bolts} > F_{shear\_bolts}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Hole size factor specified in LRFD Table 6.13.2.8-2

$K_h := 1$

Surface condition factor specified in LRFD Table 6.13.2.8-3 (assumed)

$K_s := 0.30$

Number of shear planes per bolt

$N_s := 1$

Miminum required bolt tension, A307 bolts (assumed)

$P_t := 11\ kip$

Slip resistance of A307 bolts

$R_{n\_slip\_resistance} := K_h \cdot K_s \cdot N_s \cdot P_t \cdot N_{bolts} = 19.8\ kip$ LRFD 6.13.2.8-1

Maximum shear force applied to bolts

$F_{shear\_bolts} := 13.61\ kip$

Is nominal slip resistance of A307 bolts greater than the maximum force applied?

$\text{if}\left(R_{n\_shear\_bolts} > F_{shear\_bolts}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

1

```
                  ************************************
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                                                                      INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
150'x17'-Swanzey\M


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

                        DESCRIPTION                                      DATE
                        -----------                                      ----
     42095 Temporary Bridge                                          8/14/2019

     CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
     ---------------     ------    --------  -BY- -BY- ------------
                                           TEM  TPL


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

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

         1       10           2       AASHTO  2019     0     2            3
      ---------------------------------------------------------------------------

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

      * span interval   : maximum = 20
                          default = 10

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

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

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

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

      * EFFECTIVE FLANGE WITH OPTION =    0
         0 - DEFAULT (2008)
         1 - "PRIOR TO 2007" WIDTH IS USED
1
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     TABLE 0.0.3.1 STRUCTURAL DETAILS
                   ******************

                BEAM     WIDTH            EDGE OF               COMPOSITE  STEEL
RIGHT    R. EDGE OF
                POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
OVERHANG   SLAB TO
               NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
WIDTH    CURB
       OF    1=INT.   BARRIER  -------- ------- DEPTH WIDTH.   REGION    FACTOR ----
---- ----------
     GIRDERS 2,3=EXT.  (ft)      (ft)    (ft)   (in)  (in)      (%)     >= 1.0
(ft)     (ft)
     ------------------------------------------------------------------------------
---------------

        8     1        16.42     1.27    0.79   0.00   0.00   100.00     1.00
0.00      0.00

     ------------------------------------------------------------------------------
---------------
      * 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

      * EXT=2 for the same left and right exterior overhang information or
              only left exterior overhang information

      * EXT=3 for the right exterior overhang information



     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
     ------,------,------,------,------,------,------,------,------,------,

      47.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00



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

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

       2.21   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00

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350'x17'-Swanzey\M


     TABLE 0.0.4.1 DEFINITION OF SECTIONS
                   **********************

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

        1  W     24      76                 0.00 0.0000 0.00 0.0000

      ---------------------------------------------------------------------------
      NOTE: [1] maximum allowable section number is 70

            [2] For design option (flow 4, 5 or 6) this card need not be input
1
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     TABLE 0.0.5.1 DEFINITION OF MEMBERS
                   **********************

        MEMBER     MEMB END  MEMBER DESCRIPTN  PARAMETERS FOR     YIELD STRESS
```

```
        NUMBER     SECT  ID  ---------------- NONPRISMATIC MEMB      (KSI)
      ---------- ---------- LNGTH -->TYPE<-- ----------------- ---------------
     (IN ORDER) LEFT RIGHT  (ft)  0=PRISMAT    S(0)    S(1)     WEB  TOP  BOT
     ------------------------------------------------------------------------

          1       1    1    47.00                                50.  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
1
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                                                                      INPUT
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     TABLE 0.0.6.1 AASHTO LIVE LOADING -  LOAD TYPE (A)
                   ************************************



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

          HL-93             1          3      1000           0       0.00

          HL-93 VEHICLE X FACTOR OF  0.00

     ------------------------------------------------------------------------
     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  1000
1
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     TABLE 0.8.1.2 SPECIFICATION OF IMPACT AND DISTRIBUTION FACTORS
                   ************************************



        IMPACT  FACTOR  TO  OVERRIDE  THE  AASHTO  FORMULA
     ------------------------------------------------------
                                                                    OPTIONAL
               OPTIONAL CALCULATION OF FACTOR 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   0.33   0.33    0.33  15.00    1 1 1 1 1   0.33    1 1 1 1 1
      1  0.00  0   0.33   0.33    0.33   0.00    0 0 0 0 0   0.33    1 1 1 1 1

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


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

      1   1  Timber Deck     0.032    0.00   47.00      0.00        0.00

      2   1  Railing         0.013    0.00   47.00      0.00        0.00

      3   0  Intermediate    0.000    0.00    0.00      0.07        3.50

      4   0  Intermediate    0.000    0.00    0.00      0.07       13.50

      5   0  Intermediate    0.000    0.00    0.00      0.07       23.50

      6   0  Intermediate    0.000    0.00    0.00      0.07       33.50

      7   0  Intermediate    0.000    0.00    0.00      0.07       43.50


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

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

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

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

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

                       2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
                           where N = modulus ratio = Es/Ec
1
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          TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
                         ***********************************************


            L O C A T I O N          YIELD             SPACING
           --------------------      STRESS         OF LATERAL BRACING
           DISTANCE    DISTANCE   ----------------  --------------------
             FROM        TO        Fy    Fy (WEB)          Lb
             (ft)        (ft)     (ksi)  (ksi)            (ft)
           ------------------------------------------------------------

              0.00       47.00     50.0  50.0            10.00

      ** WARNING: LATERAL BRACING DISTANCE CAN NOT BE EVENLY DIVIDED **
           ------------------------------------------------------------
           NOTE: [1] default Fy = 36 ksi

                 [2] default spacing of lateral bracing = 25 feet

                     Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

                 [3] The spacing of lateral bracing is also assumed to
                     be the diaphragm spacing which is used for the
                     calculation of wind effect (code check only).
1
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     TABLE 2.2.1.1=PROGRAM ASSUMPTIONS
                   *******************

      NO.          D E S C R I P T I O N S
     ----- ------------------------------------------------

       1    Small deflection theory

       2    Material is elastic

       3    Beam length is much greater than lateral dimensions

       4    Torsional effects are neglected

       5    Shear deformations are neglected

       6    Two kinematic degree-of-freedom are assumed 'at each
            joint (vertical deflection and bending rotation)

       7    Concentrated joint loads

       8    Uniform member loads

       9    Transformed sections are used for composite sections
            .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b

      10    Sections symmetrical about vertical, principal axis

      11    Unshored construction

      12    Hinged bridge ends

     ------------------------------------------------------------
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     TABLE 2.2.2.1=LOADING INFORMATION
                   *******************

          AVERAGE DEAD LOAD INTENSITIES
          *****************************

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

```
       1     0.000       0.0762     0.0762


   NONCOMPOSITE SUPERIMPOSED DEAD LOADS
   ************************************


  LOAD       INTENSITY     DIST        DIST
                           FROM         TO
          (K or K/Ft)      (Ft)        (Ft)
  --------------------------------------------


     1          0.032       0.000     47.000

     2          0.013       0.000     47.000
1
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    TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
                  ********************************


           C O N C R E T E    D E C K          S T E E L  SUPERSTRUCTURE
    ----------------------------------------  ----------  TOTAL  WEIGHT
                   V O L U M E      TOTAL      TOTAL     ---------------
     UNIT WEIGHT  -----------------  WEIGHT     WEIGHT        (kip)
       (pcf)      (ft**3) (yard**3)  (kip)      (kip)
    ---------------------------------------------------------------------

        150.00         0.0      0.0      0.0       28.6        28.65

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

           [2] Superimposed dead load not included


     1          0.072       3.500

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

     2          0.072      13.500

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

     3          0.072      23.500

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

     4          0.072      33.500

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

     5          0.072      43.500

  --------------------------------------------
  *** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
          Ts >12.0 In (300 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
          S > 16.0 Ft (4900 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
          Ts >12.0 In (300 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
          S > 16.0 Ft (4900 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
          Ts >12.0 In (300 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
          S > 16.0 Ft (4900 mm) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
  *** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
          Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
1
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    TABLE 2.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS
                  ********************************


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

      1       0.330     0.330     0.251     0.251   FOR STRENGTH POSITIVE MOMENT
              0.330                                 FOR STRENGTH POSITIVE SHEAR
      1       0.330                                 FOR FATIGUE POSITIVE MOMENT
              0.330                                 FOR FATIGUE POSITIVE SHEAR
1
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 TABLE 2.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
               *********************************************



              MOMENT OF    WEB    LOCATION OF N.A.  ELASTIC SECTION MODULUS
```

```
 SP IN D FROM  INERTIA   DEPTH  FROM BOT OF STEEL -----------------------
 NO NO L SUPT --------- ------- -----------------          STEEL
                 Ix        D          Y(BS)          BOT.        TOP.
               (in**4)   (in)          (in)             (in**3)
 -------------------------------------------------------------------------

  1  0   0.00    2100.0   23.9         11.94          175.8        175.8
  1  1   4.70    2100.0   23.9         11.94          175.8        175.8
  1  2   9.40    2100.0   23.9         11.94          175.8        175.8
  1  3  14.10    2100.0   23.9         11.94          175.8        175.8
  1  4  18.80    2100.0   23.9         11.94          175.8        175.8
  1  5  23.50    2100.0   23.9         11.94          175.8        175.8
  1  6  28.20    2100.0   23.9         11.94          175.8        175.8
  1  7  32.90    2100.0   23.9         11.94          175.8        175.8
  1  8  37.60    2100.0   23.9         11.94          175.8        175.8
  1  9  42.30    2100.0   23.9         11.94          175.8        175.8
  1 10  47.00    2100.0   23.9         11.94          175.8        175.8

 -------------------------------------------------------------------------
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 TABLE 2.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
               *********************************************

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

  1  0   0.00      0.0        0.0         0.0        0.0          0.0
  1  1   4.70      7.6        0.0         0.8        4.4         12.8
  1  2   9.40     13.5        0.0         1.3        7.9         22.6
  1  3  14.10     17.7        0.0         1.7       10.4         29.8
  1  4  18.80     20.2        0.0         1.9       11.8         33.9
  1  5  23.50     21.0        0.0         2.1       12.3         35.5
  1  6  28.20     20.2        0.0         1.9       11.8         33.9
  1  7  32.90     17.7        0.0         1.7       10.4         29.8
  1  8  37.60     13.5        0.0         1.3        7.9         22.6
  1  9  42.30      7.6        0.0         0.8        4.4         12.8
  1 10  47.00      0.0        0.0         0.0        0.0          0.0

 -------------------------------------------------------------------------
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1550'x17'-Swanzey\M


 TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
               ***********************************************

               SIDEWALK (MAXIMUM)     LL+I ,(k-ft), LOAD TYPE= HL -93
 SP IN D FROM  ------------------  --------------------------------------
 NO NO L SUPT  POSITIVE  NEGATIVE  MAXIMUM   GOVERN.   MAXIMUM   GOVERN.
        (ft)    (k-ft)    (k-ft)   POSITIVE LOAD TYPE  NEGATIVE LOAD TYPE
 -------------------------------------------------------------------------

  1  0   0.00     0.0       0.0        0.0   HL-93        0.0    HL-93
  1  1   4.70     0.0       0.0      125.2   HL-93        0.0    HL-93
  1  2   9.40     0.0       0.0      216.0   HL-93        0.0    HL-93
  1  3  14.10     0.0       0.0      272.4   HL-93        0.0    HL-93
  1  4  18.80     0.0       0.0      304.3   HL-93        0.0    HL-93
  1  5  23.50     0.0       0.0      306.7   HL-93        0.0    HL-93
  1  6  28.20     0.0       0.0      304.3   HL-93        0.0    HL-93
  1  7  32.90     0.0       0.0      272.4   HL-93        0.0    HL-93
  1  8  37.60     0.0       0.0      216.0   HL-93        0.0    HL-93
  1  9  42.30     0.0       0.0      125.2   HL-93        0.0    HL-93
  1 10  47.00     0.0       0.0        0.0   HL-93        0.0    HL-93


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


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

  1  0   0.00    0.     0.     0.
  1  1   4.70   66.     0.    66.
  1  2   9.40  110.     0.   110.
  1  3  14.10  131.     0.   131.
  1  4  18.80  154.     0.   154.
  1  5  23.50  157.     0.   157.
  1  6  28.20  154.     0.   154.
  1  7  32.90  131.     0.   131.
  1  8  37.60  110.     0.   110.
  1  9  42.30   66.     0.    66.
  1 10  47.00    0.     0.     0.


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

SP NO	IN NO	D L	FROM SUPP (ft)	DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1		4.70	12.8	125.2	HL-93	0.0	HL-93	137.9	12.8
1	2		9.40	22.6	216.0	HL-93	0.0	HL-93	238.6	22.6
1	3		14.10	29.8	272.4	HL-93	0.0	HL-93	302.2	29.8
1	4		18.80	33.9	304.3	HL-93	0.0	HL-93	338.2	33.9
1	5		23.50	35.5	306.7	HL-93	0.0	HL-93	342.2	35.5
1	6		28.20	33.9	304.3	HL-93	0.0	HL-93	338.2	33.9
1	7		32.90	29.8	272.4	HL-93	0.0	HL-93	302.2	29.8
1	8		37.60	22.6	216.0	HL-93	0.0	HL-93	238.6	22.6
1	9		42.30	12.8	125.2	HL-93	0.0	HL-93	137.9	12.8
1	10		47.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)

SP NO	IN NO	D L	FROM SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0		0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1		4.70	137.9	175.5	235.0	185.0	19.2	53.0
1	2		9.40	238.6	303.4	406.2	319.8	33.9	87.8
1	3		14.10	302.2	383.9	513.9	405.0	44.7	104.5
1	4		18.80	338.2	429.5	574.9	453.2	50.9	123.4
1	5		23.50	342.2	434.2	581.1	458.4	53.2	125.7
1	6		28.20	338.2	429.5	574.9	453.2	50.9	123.4
1	7		32.90	302.2	383.9	513.9	405.0	44.7	104.5
1	8		37.60	238.6	303.4	406.2	319.8	33.9	87.8
1	9		42.30	137.9	175.5	235.0	185.0	19.2	53.0
1	10		47.00	0.0	0.0	0.0	0.0	0.0	0.0

1

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

SP NO	IN NO	D L	FROM SUPT (ft)	DEAD LOAD BEAM (k)	DEAD LOAD SLAB (k)	CONCENTRATED LOADS (k)	UNIFORM LOADS (k)	TOTAL NONCOMPOSITE DEAD LOAD (k)
1	0		0.00	1.79	0.00	0.2	1.05	3.02
1	1		4.70	1.43	0.00	0.1	0.84	2.38
1	2		9.40	1.07	0.00	0.1	0.63	1.81
1	3		14.10	0.72	0.00	0.0	0.42	1.17
1	4		18.80	0.36	0.00	0.0	0.21	0.60
1	5		23.50	0.00	0.00	0.0	0.00	0.04
1	6		28.20	-0.36	0.00	0.0	-0.21	-0.60
1	7		32.90	-0.72	0.00	0.0	-0.42	-1.17
1	8		37.60	-1.07	0.00	-0.1	-0.63	-1.81
1	9		42.30	-1.43	0.00	-0.1	-0.84	-2.38
1	10		47.00	-1.79	0.00	-0.2	-1.05	-3.02

1

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

SP NO	IN NO	D L	FROM SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0		0.00	0.0	0.0	30.28	HL-93	0.00	HL-93
1	1		4.70	0.0	0.0	26.18	HL-93	-1.45	HL-93
1	2		9.40	0.0	0.0	22.17	HL-93	-3.65	HL-93
1	3		14.10	0.0	0.0	18.27	HL-93	-6.09	HL-93
1	4		18.80	0.0	0.0	14.46	HL-93	-8.63	HL-93
1	5		23.50	0.0	0.0	11.27	HL-93	-11.27	HL-93
1	6		28.20	0.0	0.0	8.63	HL-93	-14.46	HL-93
1	7		32.90	0.0	0.0	6.09	HL-93	-18.27	HL-93
1	8		37.60	0.0	0.0	3.65	HL-93	-22.17	HL-93
1	9		42.30	0.0	0.0	1.45	HL-93	-26.18	HL-93
1	10		47.00	0.0	0.0	0.00	HL-93	-30.28	HL-93

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

SP NO	IN NO	D L	FROM SUPT (ft)	T R U C K POS	NEG	RANGE
1	0		0.00	17.	0.	17.
1	1		4.70	14.	-1.	15.
1	2		9.40	12.	-2.	14.
1	3		14.10	9.	-4.	13.
1	4		18.80	8.	-5.	13.
1	5		23.50	7.	-7.	13.
1	6		28.20	5.	-8.	13.
1	7		32.90	4.	-9.	13.
1	8		37.60	2.	-12.	14.
1	9		42.30	1.	-14.	15.
1	10		47.00	0.	-17.	17.

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MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.6.2B=MINIMUM WELD SIZE

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	16.7	.36962E-01	0.31	0.2500*	.36962E-01	0.31	0.2500*
1	1		4.70	15.3	.36962E-01	0.28	0.2500*	.36962E-01	0.28	0.2500*
1	2		9.40	14.1	.36962E-01	0.26	0.2500*	.36962E-01	0.26	0.2500*
1	3		14.10	12.9	.36962E-01	0.24	0.2500*	.36962E-01	0.24	0.2500*
1	4		18.80	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	5		23.50	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	6		28.20	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	7		32.90	12.9	.36962E-01	0.24	0.2500*	.36962E-01	0.24	0.2500*
1	8		37.60	14.1	.36962E-01	0.26	0.2500*	.36962E-01	0.26	0.2500*
1	9		42.30	15.3	.36962E-01	0.28	0.2500*	.36962E-01	0.28	0.2500*
1	10		47.00	16.7	.36962E-01	0.31	0.2500*	.36962E-01	0.31	0.2500*

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

[2] For non-composite construction:

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

For composite construction:

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

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

* -- minimum weld size governs

1

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TABLE 2.2.6.2C=WELD DESIGN BY SHEAR FORCE

SP NO	IN NO	D L	FROM SUPT (ft)	SHEAR FLOW St [1] (kip/in)	SHEAR FLOW Sb [2] (kip/in)	SHEAR FLOW Sg [3] (kip/in)	MIN SIZE WELD (in)	ALLOWABLE SHEAR FLOW Vw [4] (kip/in)	ALLOWABLE SHEAR FLOW Vc [5] (kip/in)	ALLOWABLE SHEAR FLOW Vg [6] (kip/in)	FL AG
1	0		0.00	1.96	1.96	1.96	0.2500	11.88	12.73	11.88	
1	1		4.70	1.69	1.69	1.69	0.2500	11.88	12.73	11.88	
1	2		9.40	1.43	1.43	1.43	0.2500	11.88	12.73	11.88	
1	3		14.10	1.18	1.18	1.18	0.2500	11.88	12.73	11.88	
1	4		18.80	0.94	0.94	0.94	0.2500	11.88	12.73	11.88	
1	5		23.50	0.73	0.73	0.73	0.2500	11.88	12.73	11.88	
1	6		28.20	0.94	0.94	0.94	0.2500	11.88	12.73	11.88	
1	7		32.90	1.18	1.18	1.18	0.2500	11.88	12.73	11.88	
1	8		37.60	1.43	1.43	1.43	0.2500	11.88	12.73	11.88	
1	9		42.30	1.69	1.69	1.69	0.2500	11.88	12.73	11.88	
1	10		47.00	1.96	1.96	1.96	0.2500	11.88	12.73	11.88	

NOTE: [1] St = top flange shear flow
= VQ/I
where : V = shear force
Q = statical moment of the area about neutral axis
I = moment of inertia

[2] Sb = bottom flange shear flow

[3] Sg = govern. shear flow (max. of St and Sb)

[4] Vw = allowable shear flow for min. size weld
= 0.6 * (phi)e2 * Fexx * 0.707 * min. weld size * 2
where: (Phi)e2 = weld resistance factor (0.8)

Fexx = class. strength of the weld metal (70 ksi)

[5] Vc = allowable shear flow on the connected material
= (Phi)v * (0.58 * Ag * Fy)
where: (Phi)v = resistance factor for shear (1.0)
Ag = gross area of the connection element
Fy = min. yield strength of connection element

[6] Vg = govern. allow. shear flow (min. of Vw and Vc)

If [6] is less than [3], then the flag ** will show up.

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D L NO	FROM SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0	0.00	3.0	30.3	HL-93	0.0	HL-93	33.3	3.0
1	1	4.70	2.4	26.2	HL-93	-1.5	HL-93	28.6	0.9
1	2	9.40	1.8	22.2	HL-93	-3.6	HL-93	24.0	-1.8
1	3	14.10	1.2	18.3	HL-93	-6.1	HL-93	19.4	-4.9
1	4	18.80	0.6	14.5	HL-93	-8.6	HL-93	15.1	-8.0
1	5	23.50	0.0	11.3	HL-93	-11.3	HL-93	11.3	-11.2
1	6	28.20	-0.6	8.6	HL-93	-14.5	HL-93	-15.1	8.0
1	7	32.90	-1.2	6.1	HL-93	-18.3	HL-93	-19.4	4.9
1	8	37.60	-1.8	3.6	HL-93	-22.2	HL-93	-24.0	1.8
1	9	42.30	-2.4	1.5	HL-93	-26.2	HL-93	-28.6	-0.9
1	10	47.00	-3.0	0.0	HL-93	-30.3	HL-93	-33.3	-3.0

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)

SP NO	IN D L NO	FROM SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	33.3	42.4	56.8	44.7	4.5	13.4
1	1	4.70	28.6	36.4	48.8	38.3	3.6	12.3
1	2	9.40	24.0	30.6	41.1	32.2	2.7	11.3
1	3	14.10	19.4	24.9	33.4	26.1	1.8	10.3
1	4	18.80	15.1	19.4	26.1	20.3	0.9	10.7
1	5	23.50	11.3	14.7	19.8	15.3	0.1	10.7
1	6	28.20	-15.1	-19.4	-26.1	-20.3	-0.9	10.7
1	7	32.90	-19.4	-24.9	-33.4	-26.1	-1.8	10.3
1	8	37.60	-24.0	-30.6	-41.1	-32.2	-2.7	11.3
1	9	42.30	-28.6	-36.4	-48.8	-38.3	-3.6	12.3
1	10	47.00	-33.3	-42.4	-56.8	-44.7	-4.5	13.4

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM
1	3.02	0.00	HL-93	30.28	HL-93 ST1	2.72	56.77
					ST2	2.72	44.66
					ST4	4.53	4.53
					SE1	3.02	33.30
					SE2	3.02	42.39
2	3.02	0.00	HL-93	30.28	HL-93 ST1	2.72	56.77
					ST2	2.72	44.66
					ST4	4.53	4.53
					SE1	3.02	33.30
					SE2	3.02	42.39

NOTE: [1] " - " Indicates Uplift

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

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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

SP NO	IN D L NO	FROM SUPT (ft)	DEAD LOAD BEAM (in)	DEAD LOAD SLAB (in)	CONCENTRATED LOADS (in)	UNIFORM LOADS (in)	TOTAL DEAD LOAD +=UP , -=DOWN
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	4.70	-0.0431	0.0000	-0.0041	-0.0253	-0.0726
1	2	9.40	-0.0816	0.0000	-0.0078	-0.0479	-0.1373
1	3	14.10	-0.1117	0.0000	-0.0107	-0.0655	-0.1879
1	4	18.80	-0.1308	0.0000	-0.0125	-0.0767	-0.2201
1	5	23.50	-0.1374	0.0000	-0.0132	-0.0806	-0.2311
1	6	28.20	-0.1308	0.0000	-0.0125	-0.0767	-0.2201
1	7	32.90	-0.1117	0.0000	-0.0107	-0.0655	-0.1879
1	8	37.60	-0.0816	0.0000	-0.0078	-0.0479	-0.1373
1	9	42.30	-0.0431	0.0000	-0.0041	-0.0253	-0.0726
1	10	47.00	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " indicates downward deflection

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ANALYSIS

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TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)

SP NO	IN D L NO	FROM SUPT (FT)	NONCOMPOSITE STEEL DEFL.	NONCOMPOSITE CAMBER (IN) SIZE	DEAD LOADS SLAB DEFL.	DEAD LOADS CAMBER (IN) SIZE	OTHER DL DEFL.	OTHER DL CAMBER (IN) SIZE	T O T A L DEFL.	T O T A L CAMBER (IN) SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	4.7	-0.043	0 1/16	0.000	0 0/16	-0.029	0 1/16	-0.073	0 1/ 8
1	2	9.4	-0.082	0 1/ 8	0.000	0 0/16	-0.056	0 1/16	-0.137	0 3/16
1	3	14.1	-0.112	0 1/ 8	0.000	0 0/16	-0.076	0 1/ 8	-0.188	0 1/ 4
1	4	18.8	-0.131	0 3/16	0.000	0 0/16	-0.089	0 1/ 8	-0.220	0 1/ 4
1	5	23.5	-0.137	0 3/16	0.000	0 0/16	-0.094	0 1/ 8	-0.231	0 1/ 4
1	6	28.2	-0.131	0 3/16	0.000	0 0/16	-0.089	0 1/ 8	-0.220	0 1/ 4
1	7	32.9	-0.112	0 1/ 8	0.000	0 0/16	-0.076	0 1/ 8	-0.188	0 1/ 4
1	8	37.6	-0.082	0 1/ 8	0.000	0 0/16	-0.056	0 1/16	-0.137	0 3/16
1	9	42.3	-0.043	0 1/16	0.000	0 0/16	-0.029	0 1/16	-0.073	0 1/ 8
1	10	47.0	0.000	--	0.000	--	0.000	--	0.000	--

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

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	23.50	1 0.125	-0.593 MAX	HL-93	0.70	0.02005
			0.065 MIN			
			-0.144 MAX	LANE		
			0.065 MIN	LANE		
		1 0.125	-0.373 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) ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

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

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

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

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

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D L NO	FROM SUPT (ft)	N.COMP DEAD LOAD STEEL BEAM TOP (ksi)	N.COMP DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.70	-0.52	0.52	-0.36	0.36	-0.87	0.87
1	2	9.40	-0.92	0.92	-0.63	0.63	-1.54	1.54
1	3	14.10	-1.21	1.21	-0.83	0.83	-2.03	2.03
1	4	18.80	-1.38	1.38	-0.94	0.94	-2.32	2.32

1	5	23.50	-1.44	1.44	-0.98	0.98	-2.42	2.42
1	6	28.20	-1.38	1.38	-0.94	0.94	-2.32	2.32
1	7	32.90	-1.21	1.21	-0.83	0.83	-2.03	2.03
1	8	37.60	-0.92	0.92	-0.63	0.63	-1.54	1.54
1	9	42.30	-0.52	0.52	-0.36	0.36	-0.87	0.87
1	10	47.00	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D L (ft)	FROM SUPT	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		0.00	0.00	0.00	0.00
1	1	4.70		-8.54	8.54	0.00	0.00
1	2	9.40		-14.74	14.74	0.00	0.00
1	3	14.10		-18.59	18.59	0.00	0.00
1	4	18.80		-20.77	20.77	0.00	0.00
1	5	23.50		-20.94	20.94	0.00	0.00
1	6	28.20		-20.77	20.77	0.00	0.00
1	7	32.90		-18.59	18.59	0.00	0.00
1	8	37.60		-14.74	14.74	0.00	0.00
1	9	42.30		-8.54	8.54	0.00	0.00
1	10	47.00		0.00	0.00	0.00	0.00

1

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ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 3250'x17'-Swanzey\M

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

SP NO	IN NO	D L (ft)	FROM SUPT	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0	0.00		0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1	4.70		4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	2	9.40		7.5	0.0	7.5	C	HL	7.5	0.0	7.5	T	HL
1	3	14.10		8.9	0.0	8.9	C	HL	8.9	0.0	8.9	T	HL
1	4	18.80		10.5	0.0	10.5	C	HL	10.5	0.0	10.5	T	HL
1	5	23.50		10.7	0.0	10.7	C	HL	10.7	0.0	10.7	T	HL
1	6	28.20		10.5	0.0	10.5	C	HL	10.5	0.0	10.5	T	HL
1	7	32.90		8.9	0.0	8.9	C	HL	8.9	0.0	8.9	T	HL
1	8	37.60		7.5	0.0	7.5	C	HL	7.5	0.0	7.5	T	HL
1	9	42.30		4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	10	47.00		0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.9.3A=SERVICE I TOTAL (DC+DW+(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-9.42	9.42	-0.87	0.87
1	2	9.40		50.	50.	-16.29	16.29	-1.54	1.54
1	3	14.10		50.	50.	-20.63	20.63	-2.03	2.03
1	4	18.80		50.	50.	-23.09	23.09	-2.32	2.32
1	5	23.50		50.	50.	-23.36	23.36	-2.42	2.42
1	6	28.20		50.	50.	-23.09	23.09	-2.32	2.32
1	7	32.90		50.	50.	-20.63	20.63	-2.03	2.03
1	8	37.60		50.	50.	-16.29	16.29	-1.54	1.54
1	9	42.30		50.	50.	-9.42	9.42	-0.87	0.87
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-11.98	11.98	-0.87	0.87
1	2	9.40		50.	50.	-20.71	20.71	-1.54	1.54
1	3	14.10		50.	50.	-26.20	26.20	-2.03	2.03
1	4	18.80		50.	50.	-29.32	29.32	-2.32	2.32
1	5	23.50		50.	50.	-29.64	29.64	-2.42	2.42
1	6	28.20		50.	50.	-29.32	29.32	-2.32	2.32
1	7	32.90		50.	50.	-26.20	26.20	-2.03	2.03
1	8	37.60		50.	50.	-20.71	20.71	-1.54	1.54
1	9	42.30		50.	50.	-11.98	11.98	-0.87	0.87
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-14.52	14.52	0.44	-0.44
1	2	9.40		50.	50.	-25.03	25.03	0.77	-0.77
1	3	14.10		50.	50.	-31.52	31.52	1.02	-1.02
1	4	18.80		50.	50.	-35.19	35.19	1.16	-1.16
1	5	23.50		50.	50.	-35.43	35.43	1.21	-1.21
1	6	28.20		50.	50.	-35.19	35.19	1.16	-1.16
1	7	32.90		50.	50.	-31.52	31.52	1.02	-1.02
1	8	37.60		50.	50.	-25.03	25.03	0.77	-0.77
1	9	42.30		50.	50.	-14.52	14.52	0.44	-0.44
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-14.21	14.21	0.74	-0.74
1	2	9.40		50.	50.	-24.48	24.48	1.31	-1.31
1	3	14.10		50.	50.	-30.81	30.81	1.73	-1.73
1	4	18.80		50.	50.	-34.38	34.38	1.97	-1.97
1	5	23.50		50.	50.	-34.58	34.58	2.06	-2.06
1	6	28.20		50.	50.	-34.38	34.38	1.97	-1.97
1	7	32.90		50.	50.	-30.81	30.81	1.73	-1.73
1	8	37.60		50.	50.	-24.48	24.48	1.31	-1.31
1	9	42.30		50.	50.	-14.21	14.21	0.74	-0.74
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-12.62	12.62	-1.09	1.09
1	2	9.40		50.	50.	-21.83	21.83	-1.93	1.93
1	3	14.10		50.	50.	-27.64	27.64	-2.54	2.54
1	4	18.80		50.	50.	-30.94	30.94	-2.90	2.90
1	5	23.50		50.	50.	-31.29	31.29	-3.02	3.02
1	6	28.20		50.	50.	-30.94	30.94	-2.90	2.90
1	7	32.90		50.	50.	-27.64	27.64	-2.54	2.54
1	8	37.60		50.	50.	-21.83	21.83	-1.93	1.93
1	9	42.30		50.	50.	-12.62	12.62	-1.09	1.09
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00		50.	50.	0.00	0.00	0.00	0.00
1	1	4.70		50.	50.	-12.32	12.32	-0.78	0.78
1	2	9.40		50.	50.	-21.29	21.29	-1.39	1.39
1	3	14.10		50.	50.	-26.93	26.93	-1.83	1.83
1	4	18.80		50.	50.	-30.13	30.13	-2.09	2.09
1	5	23.50		50.	50.	-30.44	30.44	-2.18	2.18
1	6	28.20		50.	50.	-30.13	30.13	-2.09	2.09
1	7	32.90		50.	50.	-26.93	26.93	-1.83	1.83
1	8	37.60		50.	50.	-21.29	21.29	-1.39	1.39
1	9	42.30		50.	50.	-12.32	12.32	-0.78	0.78
1	10	47.00		50.	50.	0.00	0.00	0.00	0.00

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH THICK	COVER PLATE -- (in) BOT WIDTH THICK
1	47.00	W	9.0	0.6800	22.5	0.4390	9.0	0.6800	0.00 0.0000	0.00 0.0000

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

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.22.4=DEPTH RATIOS
****

** [1] = Span length (ft) = 47.0

MEMBER NO	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D. [2]/(12*[1]) (in)	<=>	LIMITATION 1/25 1/30	STATUS
1	47.00	23.89	[2] does not apply			
			[3]= 23.89	>	0.03333	OK

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

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

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

[3] -- same as NOTE [2]

** -- cover plates not taken into account

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

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

SP NO	IN NO	D L [1]	FROM SUPT	Lo ng [1]	Co mp [1]	Web depth	Web thick	D/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0		0.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	1		4.70	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	2		9.40	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	3		14.10	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	4		18.80	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	5		23.50	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	6		28.20	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	7		32.90	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	8		37.60	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	9		42.30	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	10		47.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0

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

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 1.2.22.5A=DEPTH/THICKNESS RATIOS (N = inf.)
****

SP NO	IN NO	D L [1]	FROM SUPT	Lo ng [1]	Co mp [1]	Web depth	Web thick	2Dc/tw [2]	Web Category [3]
1	0		0.00	0	1	22.53	0.439	51.32	137.27	2
1	1		4.70	0	1	22.53	0.439	51.32	137.27	2
1	2		9.40	0	1	22.53	0.439	51.32	137.27	2
1	3		14.10	0	1	22.53	0.439	51.32	137.27	2
1	4		18.80	0	1	22.53	0.439	51.32	137.27	2
1	5		23.50	0	1	22.53	0.439	51.32	137.27	2
1	6		28.20	0	1	22.53	0.439	51.32	137.27	2
1	7		32.90	0	1	22.53	0.439	51.32	137.27	2
1	8		37.60	0	1	22.53	0.439	51.32	137.27	2
1	9		42.30	0	1	22.53	0.439	51.32	137.27	2
1	10		47.00	0	1	22.53	0.439	51.32	137.27	2

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.22.6=FLANGE PROPORTIONS CHECK
****

SP NO	D NO	L SUPT	bf/2tf [1] (ft)	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	1	4.70	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	2	9.40	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	3	14.10	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	4	18.80	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	5	23.50	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	6	28.20	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	7	32.90	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	8	37.60	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	9	42.30	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	10	47.00	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0

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

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

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING
****

LATERAL BRACING NO	DIST. FROM LEFT SUPT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	10.0	0.000	28.667	16.788	4.909	1.579
2	10.0	20.0	28.667	39.351	35.876	32.401	1.089
3	20.0	30.0	37.649	39.351	39.530	39.708	1.000
4	30.0	40.0	21.761	37.649	31.794	25.940	1.169

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

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES
****

SP NO	IN NO	D L SUPT (ft)	mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	FLB CAT	LTB Lb (ft)	LTB Lp (ft)	LTB Lr (ft)	GOV CAT	GOV CAT
1	0	0.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	1	4.70	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	2	9.40	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	3	14.10	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	4	18.80	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	5	23.50	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	6	28.20	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	7	32.90	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	8	37.60	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	9	42.30	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	10	47.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.58	50.00	50.00	50.00
1	1		4.70	1	1.000	1.000	1.58	50.00	50.00	50.00
1	2		9.40	1	1.000	1.000	1.58	50.00	50.00	50.00
1	3		14.10	1	1.000	1.000	1.09	50.00	47.54	47.54
1	4		18.80	1	1.000	1.000	1.09	50.00	47.54	47.54
1	5		23.50	1	1.000	1.000	1.00	50.00	43.66	43.66
1	6		28.20	1	1.000	1.000	1.00	50.00	43.66	43.66
1	7		32.90	1	1.000	1.000	1.17	50.00	50.00	50.00
1	8		37.60	1	1.000	1.000	1.17	50.00	50.00	50.00
1	9		42.30	1	1.000	1.000	1.00	50.00	43.66	43.66
1	10		47.00	1	1.000	1.000	1.00	50.00	43.66	43.66

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

1

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TABLE 2.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Category Non-Comp.
1	0		0.00	1	2	2			2
1	1		4.70	1	2	2			2
1	2		9.40	1	2	2			2
1	3		14.10	1	2	2			2
1	4		18.80	1	2	2			2
1	5		23.50	1	2	2			2
1	6		28.20	1	2	2			2
1	7		32.90	1	2	2			2
1	8		37.60	1	2	2			2
1	9		42.30	1	2	2			2
1	10		47.00	1	2	2			2

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

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

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

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

1

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TABLE 2.2.22.10=CONSTRUCTIBILITY CHECK (STRENGTH IV)

SP NO	IN NO	D L	FROM SUPT (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	1		4.70	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	2		9.40	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	3		14.10	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.5	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	4		18.80	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.5	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	5		23.50	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	6		28.20	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	7		32.90	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	8		37.60	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	9		42.30	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	10		47.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0

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

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

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

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

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

1

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TABLE 2.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY

SP NO	IN NO	D L	FROM SUPT (ft)	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0		0.00	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	1		4.70	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	2		9.40	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	3		14.10	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	4		18.80	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	5		23.50	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	6		28.20	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	7		32.90	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	8		37.60	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	9		42.30	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	10		47.00	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3

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

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

1

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TABLE 2.2.22.14=STRENGTH LIMIT STATE CHECK

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	-	-	0.0	-	0.0	30.0	0.0	50.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-
1	1		4.70	0	-	-	16.0	-	0.0	30.0	16.0	50.0	-0--
					-	-	16.0	-	0.0	30.0	16.0	50.0	--0-
1	2		9.40	0	-	-	27.7	-	0.0	30.0	27.7	50.0	-0--
					-	-	27.7	-	0.0	30.0	27.7	50.0	--0-
1	3		14.10	0	-	-	35.1	-	0.0	30.0	35.1	47.5	-0--
					-	-	35.1	-	0.0	30.0	35.1	50.0	--0-
1	4		18.80	0	-	-	39.2	-	0.0	30.0	39.2	47.5	-0--
					-	-	39.2	-	0.0	30.0	39.2	50.0	--0-
1	5		23.50	0	-	-	39.7	-	0.0	30.0	39.7	43.7	-0--
					-	-	39.7	-	0.0	30.0	39.7	50.0	--0-
1	6		28.20	0	-	-	39.2	-	0.0	30.0	39.2	43.7	-0--
					-	-	39.2	-	0.0	30.0	39.2	50.0	--0-
1	7		32.90	0	-	-	35.1	-	0.0	30.0	35.1	50.0	-0--
					-	-	35.1	-	0.0	30.0	35.1	50.0	--0-
1	8		37.60	0	-	-	27.7	-	0.0	30.0	27.7	50.0	-0--
					-	-	27.7	-	0.0	30.0	27.7	50.0	--0-
1	9		42.30	0	-	-	16.0	-	0.0	30.0	16.0	43.7	-0--
					-	-	16.0	-	0.0	30.0	16.0	50.0	--0-
1	10		47.00	0	-	-	0.0	-	0.0	30.0	0.0	45.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-

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

         "-" is N.A.

         Under FLAG Column, 0 = OK; 1= NG
         M = Moment; C = Comp. Flange; T = Tension Flange
         D = Ductility; Mu = maximum of Strengh I, II and IV

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

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

         The value of fbu is the actual stress in this table,
         the users can use the maximum value within the
         unbraced length to do their own check
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   TABLE 2.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                  ***********************************************

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

    1  0   0.00  0                              0.000      0.000 0.000     4
                                                0.000      0.000 0.000     4
    1  1   4.70  0                              0.000      0.321 0.321     4
                                                0.000      0.321 0.321     4
    1  2   9.40  0                              0.000      0.555 0.555     4
                                                0.000      0.555 0.555     4
    1  3  14.10  0                              0.000      0.738 0.738     4
                                                0.000      0.702 0.702     4
    1  4  18.80  0                              0.000      0.825 0.825     4
                                                0.000      0.785 0.785     4
    1  5  23.50  0                              0.000      0.908 0.908     4
                                                0.000      0.793 0.793     4
    1  6  28.20  0                              0.000      0.899 0.899     4
                                                0.000      0.785 0.785     4
    1  7  32.90  0                              0.000      0.702 0.702     4
                                                0.000      0.702 0.702     4
    1  8  37.60  0                              0.000      0.555 0.555     4
                                                0.000      0.555 0.555     4
    1  9  42.30  0                              0.000      0.367 0.367     4
                                                0.000      0.321 0.321     4
    1 10  47.00  0                              0.000      0.000 0.000     4
                                                0.000      0.000 0.000     4


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

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

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

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 TABLE 2.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
                 ***********************************

 SP IN D FROM  Fy     k     Art.6.10.9.2         C     Vp        SHEAR CAPACITY
 NO NO L SUPT ----- ----- -------------- D/tw  ----- ----- -----  -Eq.6.10.9.2--
        (ft)  (ksi)  [1]    [2]    [3]          [4]  (ksi)        Vn = C x Vp
 ------------------------------------------------------------------------------

  1  0   0.00  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  1   4.70  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  2   9.40  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  3  14.10  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  4  18.80  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  5  23.50  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  6  28.20  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  7  32.90  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  8  37.60  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  9  42.30  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1 10  47.00  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8


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

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

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

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

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

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

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


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

                    Vu = CVp

               where D = clear, unsupported distance between flange components

                    d0 = distance between transverse stiffeners

            ** This Article (6.10.9.2) indicates that a designer cannot
               count on post-buckling shear resistance from tension-
               field action for an unstiffened girder.
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      TABLE 2.2.22.16=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             56.8   <      286.8       2          2
       1  1   4.70       2             48.8   <      286.8       2          2
       1  2   9.40       2             41.1   <      286.8       2          2
       1  3  14.10       2             33.4   <      286.8       2          2
       1  4  18.80       2             26.1   <      286.8       2          2
       1  5  23.50       2             19.8   <      286.8       2          2
       1  6  28.20       2             26.1   <      286.8       2          2
       1  7  32.90       2             33.4   <      286.8       2          2
       1  8  37.60       2             41.1   <      286.8       2          2
       1  9  42.30       2             48.8   <      286.8       2          2
       1 10  47.00       2             56.8   <      286.8       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 <= 150 (Eq. 6.10.2.1.1-1)

                  w / long. stiff.

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

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

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

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

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


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

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

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

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

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

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

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

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

               * STRENGTH CATEGORY; 0 = compact section
                                    2 = braced non-compact section
                                    3 = slender section
1
```

TABLE 2.2.22.17=TRANSVERSE STIFFENER SPACING

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF SHEAR CAPACITY (k)	MAX. ALLOWABLE TRANS. STIFFENERS TRANS. STIFFENERS 1=YES , 0=NO	SPACING (ft-in)
1	0	0.00	50.	56.77	286.83	0	
1	1	4.70	50.	48.79	286.83	0	
1	2	9.40	50.	41.07	286.83	0	
1	3	14.10	50.	33.44	286.83	0	
1	4	18.80	50.	26.07	286.83	0	
1	5	23.50	50.	19.78	286.83	0	
1	6	28.20	50.	26.07	286.83	0	
1	7	32.90	50.	33.44	286.83	0	
1	8	37.60	50.	41.07	286.83	0	
1	9	42.30	50.	48.79	286.83	0	
1	10	47.00	50.	56.77	286.83	0	

TABLE 2.2.22.19A=SPECIAL FATIGUE REQUIREMENTS FOR WEBS

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR DL (k)	SDL (k)	FACTORED FATIGUE PERM. V (k)	<=>	Vcr (k)	<=>	UNF. PERM + FAC FATIG V (k)
1	0	0.00	3.02	0.00	16.72	3.78 <	286.83	>	32.29
1	1	4.70	2.38	0.00	14.10	2.98 <	286.83	>	27.06
1	2	9.40	1.81	0.00	11.67	2.27 <	286.83	>	22.24
1	3	14.10	1.17	0.00	9.24	1.47 <	286.83	>	17.35
1	4	18.80	0.60	0.00	8.20	0.76 <	286.83	>	14.95
1	5	23.50	0.04	0.00	6.68	0.05 <	286.83	>	11.73
1	6	28.20	0.60	0.00	8.20	0.76 <	286.83	>	14.95
1	7	32.90	1.17	0.00	9.24	1.47 <	286.83	>	17.35
1	8	37.60	1.81	0.00	11.67	2.27 <	286.83	>	22.24
1	9	42.30	2.38	0.00	14.10	2.98 <	286.83	>	27.06
1	10	47.00	3.02	0.00	16.72	3.78 <	286.83	>	32.29

NOTE: This table checks Art. 6.10.3.3 and Art. 6.10.5.3.
Vcr is the shear-buckling resistance determined from Eq. 6.10.9.3.3-1.

TABLE 2.2.22.21=SERVICE LIMIT STATE CHECK

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1] (ksi)	RATIO1	BOT. FLANGE ff+fl/2 (ksi)	[2] (ksi)	RATIO2	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0	0.00	-	0.0	40.0	0.000	0.0	40.0	Infin	0.0	50.0	000
1	1	4.70	-	12.0	40.0	0.299	12.0	40.0	Infin	12.0	50.0	000
1	2	9.40	-	20.7	40.0	0.518	20.7	40.0	Infin	20.7	50.0	000
1	3	14.10	-	26.2	40.0	0.655	26.2	40.0	Infin	26.2	50.0	000
1	4	18.80	-	29.3	40.0	0.733	29.3	40.0	Infin	29.3	50.0	000
1	5	23.50	-	29.6	40.0	0.741	29.6	40.0	Infin	29.6	50.0	000
1	6	28.20	-	29.3	40.0	0.733	29.3	40.0	Infin	29.3	50.0	000
1	7	32.90	-	26.2	40.0	0.655	26.2	40.0	Infin	26.2	50.0	000
1	8	37.60	-	20.7	40.0	0.518	20.7	40.0	Infin	20.7	50.0	000
1	9	42.30	-	12.0	40.0	0.299	12.0	40.0	Infin	12.0	50.0	000
1	10	47.00	-	0.0	40.0	0.000	0.0	40.0	NaN	0.0	50.0	000

NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

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

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

RATIO1 = ff / [1] or (ff + fl/2) / [1] (top flange)
RATIO2 = (ff + fl/2) / [2] (bot. flange)

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING	ACCEPTABLE RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING	ACCEPTABLE RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.70	TR	-7.9	A B B^C^C D E E^	TR	7.9	A B B^C^C
1	2	9.40	TR	-13.1	A B B^C^C D E E^	TR	13.1	A B
1	3	14.10	TR	-15.6	A B B^C^C D E E^	TR	15.6	A B
1	4	18.80	TR	-18.4	A B B^C^C D E E^	TR	18.4	A
1	5	23.50	TR	-18.8	A B B^C^C D E E^	TR	18.8	A
1	6	28.20	TR	-18.4	A B B^C^C D E E^	TR	18.4	A
1	7	32.90	TR	-15.6	A B B^C^C D E E^	TR	15.6	A B
1	8	37.60	TR	-13.1	A B B^C^C D E E^	TR	13.1	A B
1	9	42.30	TR	-7.9	A B B^C^C D E E^	TR	7.9	A B B^C^C
1	10	47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.75 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

TABLE 2.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING	ACCEPTABLE RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING	ACCEPTABLE RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.70	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C D
1	2	9.40	TR	-6.0	A B B^C^C D E E^	TR	6.0	A B B^
1	3	14.10	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B
1	4	18.80	TR	-8.4	A B B^C^C D E E^	TR	8.4	A
1	5	23.50	TR	-8.6	A B B^C^C D E E^	TR	8.6	A
1	6	28.20	TR	-8.4	A B B^C^C D E E^	TR	8.4	A
1	7	32.90	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B
1	8	37.60	TR	-6.0	A B B^C^C D E E^	TR	6.0	A B B^
1	9	42.30	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C D
1	10	47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.80 load factor.
Design for Finite Life w/ ADTT Single Lane= 1000 & No. of Cycles= 27375000
* 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

TABLE 1.2.30.1=AASHTO CODE CHECK STATUS SUMMARY

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

```
 OK     1.2.22.21=SERVICE LIMIT STATE CHECK



        MORE TABLES TO BE INSPECTED ...

        1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

        1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

        1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

        1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
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 TABLE 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
               ***********************************

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

  1  0   0.00    50.00  TOP          0.00      0.00     0.00 99.99  HL-93 99.99  0
                50.00  BOT          0.00      0.00     0.00 99.99  HL-93 99.99  0
  1  1   4.70    50.00  TOP         -0.87     -8.54     0.00  4.24  HL-93  3.27  0
                50.00  BOT          0.87      8.54     0.00  4.24  HL-93  3.27  0
  1  2   9.40    50.00  TOP         -1.54    -14.74     0.00  2.42  HL-93  1.86  0
                50.00  BOT          1.54     14.74     0.00  2.42  HL-93  1.86  0
  1  3  14.10    47.54  TOP         -2.03    -18.59     0.00  1.79  HL-93  1.38  0
                50.00  BOT          2.03     18.59     0.00  1.89  HL-93  1.46  0
  1  4  18.80    47.54  TOP         -2.32    -20.77     0.00  1.59  HL-93  1.23  0
                50.00  BOT          2.32     20.77     0.00  1.68  HL-93  1.30  0
  1  5  23.50    43.66  TOP         -2.42    -20.94     0.00  1.44  HL-93  1.11  0
                50.00  BOT          2.42     20.94     0.00  1.66  HL-93  1.28  0
  1  6  28.20    43.66  TOP         -2.32    -20.77     0.00  1.45  HL-93  1.12  0
                50.00  BOT          2.32     20.77     0.00  1.68  HL-93  1.30  0
  1  7  32.90    50.00  TOP         -2.03    -18.59     0.00  1.89  HL-93  1.46  0
                50.00  BOT          2.03     18.59     0.00  1.89  HL-93  1.46  0
  1  8  37.60    50.00  TOP         -1.54    -14.74     0.00  2.42  HL-93  1.86  0
                50.00  BOT          1.54     14.74     0.00  2.42  HL-93  1.86  0
  1  9  42.30    43.66  TOP         -0.87     -8.54     0.00  3.69  HL-93  2.85  0
                50.00  BOT          0.87      8.54     0.00  4.24  HL-93  3.27  0
  1 10  47.00    44.99  TOP          0.00      0.00     0.00 99.99  HL-93 99.99  0
                50.00  BOT          0.00      0.00     0.00 99.99  HL-93 99.99  0


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

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

               [2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

               [3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
                    SECTION IS NOT GOVERNING AT ALL.

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

               [4]:  LRFD SECTION IS RATED BY
                       RF=(Mu-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
                     FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
                       RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
                     WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
                          STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
                     DRI: Factor related to Ductility, Redundancy and
                          operational Importance
                          Maximum for Strength and Minimum for Other Limit State
                          (AASHTO LRFD Art. 1.3.2.1)

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

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 TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
               ********************************

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

  1  0   0.00  40.0   0.00   0.00   0.00  0.00  0.00   0.00 99.99   HL-93  76.92
  1  1   4.70  40.0  -0.87   0.87  -8.54  8.54  0.00   0.00  4.58   HL-93   3.52
  1  2   9.40  40.0  -1.54   1.54 -14.74 14.74  0.00   0.00  2.61   HL-93   2.01
  1  3  14.10  40.0  -2.03   2.03 -18.59 18.59  0.00   0.00  2.04   HL-93   1.57
  1  4  18.80  40.0  -2.32   2.32 -20.77 20.77  0.00   0.00  1.81   HL-93   1.40
  1  5  23.50  40.0  -2.42   2.42 -20.94 20.94  0.00   0.00  1.79   HL-93   1.38
  1  6  28.20  40.0  -2.32   2.32 -20.77 20.77  0.00   0.00  1.81   HL-93   1.40
  1  7  32.90  40.0  -2.03   2.03 -18.59 18.59  0.00   0.00  2.04   HL-93   1.57
  1  8  37.60  40.0  -1.54   1.54 -14.74 14.74  0.00   0.00  2.61   HL-93   2.01
  1  9  42.30  40.0  -0.87   0.87  -8.54  8.54  0.00   0.00  4.58   HL-93   3.52
  1 10  47.00  40.0   0.00   0.00   0.00  0.00  0.00   0.00 99.99   HL-93  76.92
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         NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
                   SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
                   ---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA


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


               [3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
                    SECTION IS NOT GOVERNING AT ALL.

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

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

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 TABLE 2.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
                 ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                          GOVERNING
 ------------ ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    RATING     TYPE
         (ft)         (ft)
 ------------------------------------------------------------------------------

   1    47.00        23.50       STRENGTH L.S.  1.44      1.11     HL-93
   1    47.00        23.50       SERVICEBILITY  1.79      1.38     HL-93

 ------------------------------------------------------------------------------
     NOTE: Control Rating Factor for Current Bridge:
           STRENGTH L.S.
                Operating= 1.438
                Inventory= 1.109



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

     SPAN       CRITICAL LOCATION GOVERNING
 ------------ ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    LL MOMENT
         (ft)         (ft)                               (k-ft)
 ------------------------------------------------------------------

   1    47.00        23.50       STRENGTH L.S.  2.52        236.17
   1    47.00        23.50       SERVICEBILITY  2.33

 ------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1000
                Legal Load Rating= 2.331
1
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 TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
               ***********************************

                              UNFACTORED   UNFACTORED L+I
                              DEAD LOAD    --------------  OPERAT.       INVENT.
 SP IN D FROM  SHEAR           SHEAR        MAX.  SHEAR    RATING  LIVE  RATING
 NO NO L SUPT  CAP.                                       ---------- FACTOR  LOAD  FACTOR
        (ft)  (kips)          D1; kips  D2     POS.     NEG.  [3]   TYPE
 ------------------------------------------------------------------------------

  1  0   0.00    286.8       0.0      3.0     30.3      0.0  6.91  HL-93  5.33
  1  1   4.70    286.8       0.0      2.4     26.2     -1.5  8.02  HL-93  6.18
  1  2   9.40    286.8       0.0      1.8     22.2     -3.6  9.49  HL-93  7.32
  1  3  14.10    286.8       0.0      1.2     18.3     -6.1 11.56  HL-93  8.92
  1  4  18.80    286.8       0.0      0.6     14.5     -8.6 14.64  HL-93 11.30
  1  5  23.50    286.8       0.0      0.0     11.3    -11.3 18.84  HL-93 14.53
  1  6  28.20    286.8       0.0     -0.6      8.6    -14.5 14.64  HL-93 11.30
  1  7  32.90    286.8       0.0     -1.2      6.1    -18.3 11.56  HL-93  8.92
  1  8  37.60    286.8       0.0     -1.8      3.6    -22.2  9.49  HL-93  7.32
  1  9  42.30    286.8       0.0     -2.4      1.5    -26.2  8.02  HL-93  6.18
  1 10  47.00    286.8       0.0     -3.0      0.0    -30.3  6.91  HL-93  5.33


1
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                DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING          NONCOMPOSITE
                        UNIVERSITY OF MARYLAND                    LRFD- 8TH ED
                                                                   RATING
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6550'x17'-Swanzey\M



 TABLE 2.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
                 ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                          GOVERNING
 ------------ ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    RATING     TYPE
         (ft)         (ft)
 ------------------------------------------------------------------------------

   1    47.00        47.00       STRENGTH L.S.  6.91      5.33     HL-93
```

```
 ---------------------------------------------------------------------------
     NOTE: Control Rating Factor for Current Bridge:
           STRENGTH L.S.
                Operating= 6.906
                Inventory= 5.327


 TABLE 2.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK     FA
                 ****************************************************

     SPAN       CRITICAL LOCATION GOVERNING
 ------------ ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    LL SHEAR
        (ft)         (ft)                                 (kips)
 ---------------------------------------------------------------------

   1    47.00        47.00       STRENGTH L.S. 11.23         25.14

 ---------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.300 & ADTT = 1000
                Legal Load Rating=11.227
1
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 TABLE 2.2.34.1=RATING; LIVE LOAD DEFLECTION
                 ****************************

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

       1    23.50    1   0.125       -0.59    HL-93            1.19

                     1   0.125       -0.59                     1.19

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

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

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

Exterior Beam

1

```
                  ************************************
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                                                                       INPUT
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150'x17'\Merlin-Da


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

                        DESCRIPTION                                      DATE
                        -----------                                      ----
     42095 Temporary Bridge                                          8/14/2019

     CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
     ---------------     ------    --------  -BY- -BY- ------------
                                           TEM  TPL


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

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

         1       10           2       AASHTO  2019     0     2            3
      ---------------------------------------------------------------------------

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

      * span interval   : maximum = 20
                          default = 10

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

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

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

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

      * EFFECTIVE FLANGE WITH OPTION =    0
         0 - DEFAULT (2008)
         1 - "PRIOR TO 2007" WIDTH IS USED
1
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                                                                       INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
250'x17'\Merlin-Da


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

                BEAM     WIDTH            EDGE OF               COMPOSITE  STEEL
RIGHT    R. EDGE OF
                POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
OVERHANG   SLAB TO
               NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
WIDTH    CURB
       OF    1=INT.   BARRIER  -------- ------- DEPTH WIDTH.   REGION    FACTOR ----
---- ----------
     GIRDERS 2,3=EXT.  (ft)      (ft)    (ft)   (in)  (in)      (%)     >= 1.0
(ft)     (ft)
     ------------------------------------------------------------------------------
---------------

        8     2        16.42     1.27    0.79   0.00   0.00   100.00     1.00
0.00      0.00

     ------------------------------------------------------------------------------
---------------
      * 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

      * EXT=2 for the same left and right exterior overhang information or
              only left exterior overhang information

      * EXT=3 for the right exterior overhang information


     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
     ------,------,------,------,------,------,------,------,------,------,

      47.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00



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

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

       2.21   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00

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                                                                       INPUT
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350'x17'\Merlin-Da


     TABLE 0.0.4.1 DEFINITION OF SECTIONS
                   **********************

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

        1  W     24      76                0.00 0.0000 0.00 0.0000

      ---------------------------------------------------------------------------
      NOTE: [1] maximum allowable section number is 70

            [2] For design option (flow 4, 5 or 6) this card need not be input
1
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                                                                       INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
450'x17'\Merlin-Da


     TABLE 0.0.5.1 DEFINITION OF MEMBERS
                   **********************

        MEMBER     MEMB END  MEMBER DESCRIPTN  PARAMETERS FOR     YIELD STRESS
```

```
        NUMBER     SECT  ID  ---------------- NONPRISMATIC MEMB      (KSI)
       ---------- ---------- LNGTH -->TYPE<-- ----------------- ---------------
      (IN ORDER) LEFT RIGHT  (ft)  0=PRISMAT    S(0)    S(1)     WEB  TOP  BOT
      ------------------------------------------------------------------------

           1       1    1    47.00                                50.  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
1
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                                                                        INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
550'x17'\Merlin-Da



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



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

           HL-93             1          3      1000           0       0.00

           HL-93 VEHICLE X FACTOR OF  0.00

      ------------------------------------------------------------------------
      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  1000
1
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                            UNIVERSITY OF MARYLAND                     LRFD- 8TH ED
                                                                        INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
650'x17'\Merlin-Da



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


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

       1   1  Timber Deck     0.024    0.00   47.00      0.00        0.00

       2   1  Railing         0.013    0.00   47.00      0.00        0.00

       3   0  Intermediate    0.000    0.00    0.00      0.04        3.50

       4   0  Intermediate    0.000    0.00    0.00      0.04       13.50

       5   0  Intermediate    0.000    0.00    0.00      0.04       23.50

       6   0  Intermediate    0.000    0.00    0.00      0.04       33.50

       7   0  Intermediate    0.000    0.00    0.00      0.04       43.50


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

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

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

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

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

                       2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
                           where N = modulus ratio = Es/Ec
1
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                                                                        INPUT
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750'x17'\Merlin-Da



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


          L O C A T I O N          YIELD             SPACING
         -------------------       STRESS         OF LATERAL BRACING
        DISTANCE    DISTANCE   ----------------  --------------------
          FROM        TO        Fy    Fy (WEB)          Lb
          (ft)        (ft)     (ksi)  (ksi)            (ft)
        ------------------------------------------------------------

           0.00       47.00     50.0  50.0             10.00

       ** WARNING: LATERAL BRACING DISTANCE CAN NOT BE EVENLY DIVIDED **
        ------------------------------------------------------------
        NOTE: [1] default Fy = 36 ksi

              [2] default spacing of lateral bracing = 25 feet

                  Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

              [3] The spacing of lateral bracing is also assumed to
                  be the diaphragm spacing which is used for the
                  calculation of wind effect (code check only).
1
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                                                                        ANALYSIS
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
850'x17'\Merlin-Da

        TABLE 2.2.1.1=PROGRAM ASSUMPTIONS
                      ********************

        NO.          D E S C R I P T I O N S
        ----- ------------------------------------------------

          1    Small deflection theory

          2    Material is elastic

          3    Beam length is much greater than lateral dimensions

          4    Torsional effects are neglected

          5    Shear deformations are neglected

          6    Two kinematic degree-of-freedom are assumed 'at each
               joint (vertical deflection and bending rotation)

          7    Concentrated joint loads

          8    Uniform member loads

          9    Transformed sections are used for composite sections
               .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b

         10    Sections symmetrical about vertical, principal axis

         11    Unshored construction

         12    Hinged bridge ends

        ------------------------------------------------------------
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                                                                        ANALYSIS
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
950'x17'\Merlin-Da

        TABLE 2.2.2.1=LOADING INFORMATION
                      *******************

              AVERAGE DEAD LOAD INTENSITIES
              *****************************

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

             1      0.000       0.0762     0.0762


       NONCOMPOSITE SUPERIMPOSED DEAD LOADS
       ************************************


      LOAD       INTENSITY     DIST        DIST
                               FROM         TO
                (K or K/Ft)     (Ft)        (Ft)
      ------------------------------------------

          1          0.024       0.000     47.000

          2          0.013       0.000     47.000
1
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                                                                        ANALYSIS
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1050'x17'\Merlin-Da

        TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
                      ********************************

              C O N C R E T E    D E C K          S T E E L  SUPERSTRUCTURE
        ------------------------------------------  ----------  TOTAL  WEIGHT
                          V O L U M E      TOTAL      TOTAL     ----------------
         UNIT WEIGHT  -----------------  WEIGHT     WEIGHT        (kip)
            (pcf)     (ft**3) (yard**3)  (kip)       (kip)
        ------------------------------------------------------------------------

            150.00        0.0      0.0      0.0       28.6        28.65

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

               [2] Superimposed dead load not included


        1          0.036       3.500
```

2	0.036	13.500	
3	0.036	23.500	
4	0.036	33.500	
5	0.036	43.500	

TABLE 2.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.187	0.187	0.187	0.187	FOR STRENGTH POSITIVE MOMENT
	0.187				FOR STRENGTH POSITIVE SHEAR
1	0.246				FOR FATIGUE POSITIVE MOMENT
	0.246				FOR FATIGUE POSITIVE SHEAR

TABLE 2.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY

SP NO	IN D FROM NO L SUPT	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS STEEL BOT. (in**3)	TOP. (in**3)
1	0 0.00	2100.0	23.9	11.94	175.8	175.8
1	1 4.70	2100.0	23.9	11.94	175.8	175.8
1	2 9.40	2100.0	23.9	11.94	175.8	175.8
1	3 14.10	2100.0	23.9	11.94	175.8	175.8
1	4 18.80	2100.0	23.9	11.94	175.8	175.8
1	5 23.50	2100.0	23.9	11.94	175.8	175.8
1	6 28.20	2100.0	23.9	11.94	175.8	175.8
1	7 32.90	2100.0	23.9	11.94	175.8	175.8
1	8 37.60	2100.0	23.9	11.94	175.8	175.8
1	9 42.30	2100.0	23.9	11.94	175.8	175.8
1	10 47.00	2100.0	23.9	11.94	175.8	175.8

TABLE 2.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)

SP NO	IN D FROM NO L SUPT	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
	(ft)						
1	0 0.00	0.0	0.0	0.0	0.0	0.0	
1	1 4.70	7.6	0.0	0.4	3.6	11.6	
1	2 9.40	13.5	0.0	0.6	6.4	20.5	
1	3 14.10	17.7	0.0	0.9	8.4	26.9	
1	4 18.80	20.2	0.0	1.0	9.6	30.7	
1	5 23.50	21.0	0.0	1.0	10.0	32.1	
1	6 28.20	20.2	0.0	1.0	9.6	30.7	
1	7 32.90	17.7	0.0	0.9	8.4	26.9	
1	8 37.60	13.5	0.0	0.6	6.4	20.5	
1	9 42.30	7.6	0.0	0.4	3.6	11.6	
1	10 47.00	0.0	0.0	0.0	0.0	0.0	

TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D FROM NO L SUPT	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	SIDEWALK (MAXIMUM) NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	LOAD TYPE	MAXIMUM NEGATIVE	LOAD TYPE
	(ft)						
1	0 0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1 4.70	0.0	0.0	70.8	HL-93	0.0	HL-93
1	2 9.40	0.0	0.0	122.2	HL-93	0.0	HL-93
1	3 14.10	0.0	0.0	154.2	HL-93	0.0	HL-93
1	4 18.80	0.0	0.0	172.2	HL-93	0.0	HL-93
1	5 23.50	0.0	0.0	173.6	HL-93	0.0	HL-93
1	6 28.20	0.0	0.0	172.2	HL-93	0.0	HL-93
1	7 32.90	0.0	0.0	154.2	HL-93	0.0	HL-93
1	8 37.60	0.0	0.0	122.2	HL-93	0.0	HL-93
1	9 42.30	0.0	0.0	70.8	HL-93	0.0	HL-93
1	10 47.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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

SP NO	IN D FROM NO L SUPT	TRUCK (ft)	POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	4.70	49.	0.	49.
1	2	9.40	82.	0.	82.
1	3	14.10	97.	0.	97.
1	4	18.80	115.	0.	115.
1	5	23.50	117.	0.	117.
1	6	28.20	115.	0.	115.
1	7	32.90	97.	0.	97.
1	8	37.60	82.	0.	82.
1	9	42.30	49.	0.	49.
1	10	47.00	0.	0.	0.

TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D FROM NO L SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0 0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1 4.70	11.6	70.8	HL-93	0.0	HL-93	82.4	11.6
1	2 9.40	20.5	122.2	HL-93	0.0	HL-93	142.7	20.5
1	3 14.10	26.9	154.2	HL-93	0.0	HL-93	181.1	26.9
1	4 18.80	30.7	172.2	HL-93	0.0	HL-93	203.0	30.7
1	5 23.50	32.1	173.6	HL-93	0.0	HL-93	205.7	32.1
1	6 28.20	30.7	172.2	HL-93	0.0	HL-93	203.0	30.7
1	7 32.90	26.9	154.2	HL-93	0.0	HL-93	181.1	26.9
1	8 37.60	20.5	122.2	HL-93	0.0	HL-93	142.7	20.5
1	9 42.30	11.6	70.8	HL-93	0.0	HL-93	82.4	11.6
1	10 47.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)

SP NO	IN D FROM NO L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0 0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1 4.70	82.4	103.7	138.4	110.1	17.3	39.6
1	2 9.40	142.7	179.4	239.5	190.6	30.7	65.5
1	3 14.10	181.1	227.4	303.5	241.8	40.4	77.9
1	4 18.80	203.0	254.6	339.8	270.9	46.1	92.0
1	5 23.50	205.7	257.8	343.9	274.5	48.1	93.7
1	6 28.20	203.0	254.6	339.8	270.9	46.1	92.0
1	7 32.90	181.1	227.4	303.5	241.8	40.4	77.9
1	8 37.60	142.7	179.4	239.5	190.6	30.7	65.5
1	9 42.30	82.4	103.7	138.4	110.1	17.3	39.6
1	10 47.00	0.0	0.0	0.0	0.0	0.0	0.0

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

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

1	0	0.00	1.79	0.00	0.1	0.85	2.73
1	1	4.70	1.43	0.00	0.1	0.68	2.17
1	2	9.40	1.07	0.00	0.1	0.51	1.64
1	3	14.10	0.72	0.00	0.0	0.34	1.07
1	4	18.80	0.36	0.00	0.0	0.17	0.55
1	5	23.50	0.00	0.00	0.0	0.00	0.02
1	6	28.20	-0.36	0.00	0.0	-0.17	-0.55
1	7	32.90	-0.72	0.00	0.0	-0.34	-1.07
1	8	37.60	-1.07	0.00	-0.1	-0.51	-1.64
1	9	42.30	-1.43	0.00	-0.1	-0.68	-2.17
1	10	47.00	-1.79	0.00	-0.1	-0.85	-2.73

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

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	17.14	HL-93	0.00	HL-93
1	1	4.70	0.0	0.0	14.82	HL-93	-0.82	HL-93
1	2	9.40	0.0	0.0	12.55	HL-93	-2.07	HL-93
1	3	14.10	0.0	0.0	10.34	HL-93	-3.45	HL-93
1	4	18.80	0.0	0.0	8.19	HL-93	-4.89	HL-93
1	5	23.50	0.0	0.0	6.38	HL-93	-6.38	HL-93
1	6	28.20	0.0	0.0	4.89	HL-93	-8.19	HL-93
1	7	32.90	0.0	0.0	3.45	HL-93	-10.34	HL-93
1	8	37.60	0.0	0.0	2.07	HL-93	-12.55	HL-93
1	9	42.30	0.0	0.0	0.82	HL-93	-14.82	HL-93
1	10	47.00	0.0	0.0	0.00	HL-93	-17.14	HL-93

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

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	12.	0.	12.
1	1	4.70	11.	-1.	11.
1	2	9.40	9.	-2.	11.
1	3	14.10	7.	-3.	10.
1	4	18.80	6.	-4.	10.
1	5	23.50	5.	-5.	10.
1	6	28.20	4.	-6.	10.
1	7	32.90	3.	-7.	10.
1	8	37.60	2.	-9.	11.
1	9	42.30	1.	-11.	11.
1	10	47.00	0.	-12.	12.

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TABLE 2.2.6.2B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	12.5	.36962E-01	0.23	0.2500*	.36962E-01	0.23	0.2500*
1	1	4.70	11.4	.36962E-01	0.21	0.2500*	.36962E-01	0.21	0.2500*
1	2	9.40	10.5	.36962E-01	0.19	0.2500*	.36962E-01	0.19	0.2500*
1	3	14.10	9.6	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	4	18.80	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	5	23.50	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	6	28.20	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	7	32.90	9.6	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	8	37.60	10.5	.36962E-01	0.19	0.2500*	.36962E-01	0.19	0.2500*
1	9	42.30	11.4	.36962E-01	0.21	0.2500*	.36962E-01	0.21	0.2500*
1	10	47.00	12.5	.36962E-01	0.23	0.2500*	.36962E-01	0.23	0.2500*

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

[2] For non-composite construction:

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

For composite construction:

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

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

* -- minimum weld size governs

1

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TABLE 2.2.6.2C=WELD DESIGN BY SHEAR FORCE
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR FLOW St [1] (kip/in)	SHEAR FLOW Sb [2] (kip/in)	SHEAR FLOW Sg [3] (kip/in)	MIN SIZE WELD (in)	ALLOWABLE SHEAR FLOW Vw [4] (kip/in)	ALLOWABLE SHEAR FLOW Vc [5] (kip/in)	ALLOWABLE SHEAR FLOW Vg [6] (kip/in)	FL AG
1	0	0.00	1.11	1.11	1.11	0.2500	11.88	12.73	11.88	
1	1	4.70	0.96	0.96	0.96	0.2500	11.88	12.73	11.88	
1	2	9.40	0.81	0.81	0.81	0.2500	11.88	12.73	11.88	
1	3	14.10	0.67	0.67	0.67	0.2500	11.88	12.73	11.88	
1	4	18.80	0.53	0.53	0.53	0.2500	11.88	12.73	11.88	
1	5	23.50	0.41	0.41	0.41	0.2500	11.88	12.73	11.88	
1	6	28.20	0.53	0.53	0.53	0.2500	11.88	12.73	11.88	
1	7	32.90	0.67	0.67	0.67	0.2500	11.88	12.73	11.88	
1	8	37.60	0.81	0.81	0.81	0.2500	11.88	12.73	11.88	
1	9	42.30	0.96	0.96	0.96	0.2500	11.88	12.73	11.88	
1	10	47.00	1.11	1.11	1.11	0.2500	11.88	12.73	11.88	

NOTE: [1] St = top flange shear flow
= VQ/I
where : V = shear force
Q = statical moment of the area about neutral axis
I = moment of inertia

[2] Sb = bottom flange shear flow

[3] Sg = govern. shear flow (max. of St and Sb)

[4] Vw = allowable shear flow for min. size weld
= 0.6 * (phi)e2 * Fexx * 0.707 * min. weld size * 2
where: (Phi)e2 = weld resistance factor (0.8)
Fexx = class. strength of the weld metal (70 ksi)

[5] Vc = allowable shear flow on the connected material
= (Phi)v * (0.58 * Ag * Fy)
where: (Phi)v = resistance factor for shear (1.0)
Ag = gross area of the connection element
Fy = min. yield strength of connection element

[6] Vg = govern. allow. shear flow (min. of Vw and Vc)

If [6] is less than [3], then the flag ** will show up.

1

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TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0	0.00	2.7	17.1	HL-93	0.0	HL-93	19.9	2.7
1	1	4.70	2.2	14.8	HL-93	-0.8	HL-93	17.0	1.3
1	2	9.40	1.6	12.6	HL-93	-2.1	HL-93	14.2	-0.4
1	3	14.10	1.1	10.3	HL-93	-3.4	HL-93	11.4	-2.4
1	4	18.80	0.5	8.2	HL-93	-4.9	HL-93	8.7	-4.3
1	5	23.50	0.0	6.4	HL-93	-6.4	HL-93	6.4	-6.4
1	6	28.20	-0.5	4.9	HL-93	-8.2	HL-93	-8.7	4.3
1	7	32.90	-1.1	3.4	HL-93	-10.3	HL-93	-11.4	2.4
1	8	37.60	-1.6	2.1	HL-93	-12.6	HL-93	-14.2	0.4
1	9	42.30	-2.2	0.8	HL-93	-14.8	HL-93	-17.0	-1.3
1	10	47.00	-2.7	0.0	HL-93	-17.1	HL-93	-19.9	-2.7

1

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TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	19.9	25.0	33.4	26.6	4.1	10.0
1	1	4.70	17.0	21.4	28.6	22.7	3.3	9.1
1	2	9.40	14.2	18.0	24.0	19.0	2.5	8.4
1	3	14.10	11.4	14.5	19.4	15.3	1.6	7.7
1	4	18.80	8.7	11.2	15.0	11.7	0.8	8.0
1	5	23.50	6.4	8.3	11.2	8.6	0.0	8.0
1	6	28.20	-8.7	-11.2	-15.0	-11.7	-0.8	8.0
1	7	32.90	-11.4	-14.5	-19.4	-15.3	-1.6	7.7
1	8	37.60	-14.2	-18.0	-24.0	-19.0	-2.5	8.4

```
  1  9  42.30     -17.0       -21.4      -28.6       -22.7        -3.3       9.1
  1 10  47.00     -19.9       -25.0      -33.4       -26.6        -4.1      10.0
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TABLE 2.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
****

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D) MINIMUM	MAXIMUM
1	2.73	0.00	HL-93	17.14	HL-93 ST1	2.46	33.41
					ST2	2.46	26.55
					ST4	4.10	4.10
					SE1	2.73	19.87
					SE2	2.73	25.01
2	2.73	0.00	HL-93	17.14	HL-93 ST1	2.46	33.41
					ST2	2.46	26.55
					ST4	4.10	4.10
					SE1	2.73	19.87
					SE2	2.73	25.01

NOTE: [1] " - " Indicates Uplift

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

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

SP NO	IN NO	D L	FROM SUPT (ft)	DEAD LOAD BEAM (in)	DEAD LOAD SLAB (in)	CONCENTRATED LOADS (in)	UNIFORM LOADS (in)	T O T A L +=UP , -=DOWN
1	0		0.00	0.0000	0.0000	0.0000	0.0000	0.0000
1	1		4.70	-0.0431	0.0000	-0.0021	-0.0205	-0.0657
1	2		9.40	-0.0816	0.0000	-0.0039	-0.0388	-0.1242
1	3		14.10	-0.1117	0.0000	-0.0054	-0.0531	-0.1701
1	4		18.80	-0.1308	0.0000	-0.0063	-0.0622	-0.1992
1	5		23.50	-0.1374	0.0000	-0.0066	-0.0653	-0.2092
1	6		28.20	-0.1308	0.0000	-0.0063	-0.0622	-0.1992
1	7		32.90	-0.1117	0.0000	-0.0054	-0.0531	-0.1701
1	8		37.60	-0.0816	0.0000	-0.0039	-0.0388	-0.1242
1	9		42.30	-0.0431	0.0000	-0.0021	-0.0205	-0.0657
1	10		47.00	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " indicates downward deflection

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TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (FT)	NONCOMPOSITE DEAD LOADS (IN) STEEL DEFL.	CAMBER SIZE	SLAB DEFL.	CAMBER SIZE	OTHER DL (IN) DEFL.	CAMBER SIZE	T O T A L (IN) DEFL.	CAMBER SIZE
1	0		0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1		4.7	-0.043	0 1/16	0.000	0 0/16	-0.023	0 1/16	-0.066	0 1/ 8
1	2		9.4	-0.082	0 1/ 8	0.000	0 0/16	-0.043	0 1/16	-0.124	0 1/ 8
1	3		14.1	-0.112	0 1/ 8	0.000	0 0/16	-0.058	0 1/16	-0.170	0 3/16
1	4		18.8	-0.131	0 3/16	0.000	0 0/16	-0.068	0 1/ 8	-0.199	0 1/ 4
1	5		23.5	-0.137	0 3/16	0.000	0 0/16	-0.072	0 1/ 8	-0.209	0 1/ 4
1	6		28.2	-0.131	0 3/16	0.000	0 0/16	-0.068	0 1/ 8	-0.199	0 1/ 4
1	7		32.9	-0.112	0 1/ 8	0.000	0 0/16	-0.058	0 1/16	-0.170	0 3/16
1	8		37.6	-0.082	0 1/ 8	0.000	0 0/16	-0.043	0 1/16	-0.124	0 1/ 8
1	9		42.3	-0.043	0 1/16	0.000	0 0/16	-0.023	0 1/16	-0.066	0 1/ 8
1	10		47.0	0.000	--	0.000	--	0.000	--	0.000	--

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

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TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
****

(UNFACTORED)

SPAN NO.	L SUPT (ft)	DIST. FACTOR FOR LL	NUMBER OF LANE AND DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	23.50	1	0.125	-0.593 MAX	HL-93	0.70	0.02005
				0.065 MIN			
				-0.144 MAX	LANE		
				0.065 MIN	LANE		
			1	0.125	-0.373 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) ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

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

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

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

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

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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1
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TABLE 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPT (ft)	N.COMP DEAD LOAD STEEL BEAM TOP (ksi)	N.COMP DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0		0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1		4.70	-0.52	0.52	-0.27	0.27	-0.79	0.79
1	2		9.40	-0.92	0.92	-0.48	0.48	-1.40	1.40
1	3		14.10	-1.21	1.21	-0.63	0.63	-1.84	1.84
1	4		18.80	-1.38	1.38	-0.72	0.72	-2.10	2.10
1	5		23.50	-1.44	1.44	-0.75	0.75	-2.19	2.19
1	6		28.20	-1.38	1.38	-0.72	0.72	-2.10	2.10
1	7		32.90	-1.21	1.21	-0.63	0.63	-1.84	1.84
1	8		37.60	-0.92	0.92	-0.48	0.48	-1.40	1.40
1	9		42.30	-0.52	0.52	-0.27	0.27	-0.79	0.79
1	10		47.00	0.00	0.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D L	FROM SUPT (ft)	MAXIMUM POSITIVE,(ksi) STEEL TOP	MAXIMUM POSITIVE,(ksi) STEEL BOT	MAXIMUM NEGATIVE,(ksi) STEEL TOP	MAXIMUM NEGATIVE,(ksi) STEEL BOT
1	0		0.00	0.00	0.00	0.00	0.00
1	1		4.70	-4.84	4.84	0.00	0.00
1	2		9.40	-8.34	8.34	0.00	0.00
1	3		14.10	-10.52	10.52	0.00	0.00
1	4		18.80	-11.76	11.76	0.00	0.00
1	5		23.50	-11.85	11.85	0.00	0.00
1	6		28.20	-11.76	11.76	0.00	0.00
1	7		32.90	-10.52	10.52	0.00	0.00
1	8		37.60	-8.34	8.34	0.00	0.00
1	9		42.30	-4.84	4.84	0.00	0.00
1	10		47.00	0.00	0.00	0.00	0.00

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

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	LOAD TP	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	LOAD TP
1	0		0.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL
1	1		4.70	3.4	0.0	3.4	C HL	3.4	0.0	3.4	T HL
1	2		9.40	5.6	0.0	5.6	C HL	5.6	0.0	5.6	T HL
1	3		14.10	6.6	0.0	6.6	C HL	6.6	0.0	6.6	T HL
1	4		18.80	7.8	0.0	7.8	C HL	7.8	0.0	7.8	T HL
1	5		23.50	8.0	0.0	8.0	C HL	8.0	0.0	8.0	T HL
1	6		28.20	7.8	0.0	7.8	C HL	7.8	0.0	7.8	T HL
1	7		32.90	6.6	0.0	6.6	C HL	6.6	0.0	6.6	T HL
1	8		37.60	5.6	0.0	5.6	C HL	5.6	0.0	5.6	T HL
1	9		42.30	3.4	0.0	3.4	C HL	3.4	0.0	3.4	T HL
1	10		47.00	0.0	0.0	0.0	T HL	0.0	0.0	0.0	T HL

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

1

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TABLE 2.2.9.3A=SERVICE I TOTAL (DC+DW+(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-5.62	5.62	-0.79	0.79
1	2	9.40	50.	50.	-9.74	9.74	-1.40	1.40
1	3	14.10	50.	50.	-12.36	12.36	-1.84	1.84
1	4	18.80	50.	50.	-13.85	13.85	-2.10	2.10
1	5	23.50	50.	50.	-14.04	14.04	-2.19	2.19
1	6	28.20	50.	50.	-13.85	13.85	-2.10	2.10
1	7	32.90	50.	50.	-12.36	12.36	-1.84	1.84
1	8	37.60	50.	50.	-9.74	9.74	-1.40	1.40
1	9	42.30	50.	50.	-5.62	5.62	-0.79	0.79
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.08	7.08	-0.79	0.79
1	2	9.40	50.	50.	-12.25	12.25	-1.40	1.40
1	3	14.10	50.	50.	-15.52	15.52	-1.84	1.84
1	4	18.80	50.	50.	-17.38	17.38	-2.10	2.10
1	5	23.50	50.	50.	-17.59	17.59	-2.19	2.19
1	6	28.20	50.	50.	-17.38	17.38	-2.10	2.10
1	7	32.90	50.	50.	-15.52	15.52	-1.84	1.84
1	8	37.60	50.	50.	-12.25	12.25	-1.40	1.40
1	9	42.30	50.	50.	-7.08	7.08	-0.79	0.79
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-8.07	8.07	0.39	-0.39
1	2	9.40	50.	50.	-13.90	13.90	0.70	-0.70
1	3	14.10	50.	50.	-17.50	17.50	0.92	-0.92
1	4	18.80	50.	50.	-19.52	19.52	1.05	-1.05
1	5	23.50	50.	50.	-19.64	19.64	1.09	-1.09
1	6	28.20	50.	50.	-19.52	19.52	1.05	-1.05
1	7	32.90	50.	50.	-17.50	17.50	0.92	-0.92
1	8	37.60	50.	50.	-13.90	13.90	0.70	-0.70
1	9	42.30	50.	50.	-8.07	8.07	0.39	-0.39
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.79	7.79	0.67	-0.67
1	2	9.40	50.	50.	-13.41	13.41	1.19	-1.19
1	3	14.10	50.	50.	-16.85	16.85	1.56	-1.56
1	4	18.80	50.	50.	-18.79	18.79	1.78	-1.78
1	5	23.50	50.	50.	-18.88	18.88	1.86	-1.86
1	6	28.20	50.	50.	-18.79	18.79	1.78	-1.78
1	7	32.90	50.	50.	-16.85	16.85	1.56	-1.56
1	8	37.60	50.	50.	-13.41	13.41	1.19	-1.19
1	9	42.30	50.	50.	-7.79	7.79	0.67	-0.67
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.51	7.51	-0.99	0.99
1	2	9.40	50.	50.	-13.01	13.01	-1.75	1.75
1	3	14.10	50.	50.	-16.51	16.51	-2.30	2.30
1	4	18.80	50.	50.	-18.49	18.49	-2.62	2.62
1	5	23.50	50.	50.	-18.73	18.73	-2.74	2.74
1	6	28.20	50.	50.	-18.49	18.49	-2.62	2.62
1	7	32.90	50.	50.	-16.51	16.51	-2.30	2.30
1	8	37.60	50.	50.	-13.01	13.01	-1.75	1.75
1	9	42.30	50.	50.	-7.51	7.51	-0.99	0.99
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.24	7.24	-0.71	0.71
1	2	9.40	50.	50.	-12.52	12.52	-1.26	1.26
1	3	14.10	50.	50.	-15.86	15.86	-1.65	1.65
1	4	18.80	50.	50.	-17.76	17.76	-1.89	1.89
1	5	23.50	50.	50.	-17.97	17.97	-1.97	1.97
1	6	28.20	50.	50.	-17.76	17.76	-1.89	1.89
1	7	32.90	50.	50.	-15.86	15.86	-1.65	1.65
1	8	37.60	50.	50.	-12.52	12.52	-1.26	1.26
1	9	42.30	50.	50.	-7.24	7.24	-0.71	0.71
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

1

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TABLE 2.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

MEMBER NO	LENGTH (ft)	TOP FLANGE TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	47.00	W	9.0	0.6800	22.5	0.4390	9.0	0.6800	0.00	0.0000	0.00	0.0000

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

1

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TABLE 2.2.22.4=DEPTH RATIOS

** [1] = Span length (ft) = 47.0

MEMBER NO	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D.	[2]/(12*[1]) [3]/(12*[1])	LIMITATION <=>	1/25 1/30	STATUS
1	47.00		23.89	[2] does not apply			
			[3]= 23.89	0.04236	>	0.03333	OK

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

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

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

[3] -- same as NOTE [2]

** -- cover plates not taken into account

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

1

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

SP NO	IN NO	D L (ft)	FROM SUPT	Lo ng [1]	Co mp	Web depth	Web thick	D/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	1	4.70	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	2	9.40	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	3	14.10	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	4	18.80	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	5	23.50	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	6	28.20	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	7	32.90	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	8	37.60	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	9	42.30	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	10	47.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0

NOTE:

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

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

1

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

SP NO	IN NO	D L	FROM SUPT	Lo [1]	Co ng	mp	Web depth	thick	2Dc/tw [2]	[3]	Web Category
1	0		0.00	0	1		22.53	0.439	51.32	137.27	2
1	1		4.70	0	1		22.53	0.439	51.32	137.27	2
1	2		9.40	0	1		22.53	0.439	51.32	137.27	2
1	3		14.10	0	1		22.53	0.439	51.32	137.27	2
1	4		18.80	0	1		22.53	0.439	51.32	137.27	2
1	5		23.50	0	1		22.53	0.439	51.32	137.27	2
1	6		28.20	0	1		22.53	0.439	51.32	137.27	2
1	7		32.90	0	1		22.53	0.439	51.32	137.27	2
1	8		37.60	0	1		22.53	0.439	51.32	137.27	2
1	9		42.30	0	1		22.53	0.439	51.32	137.27	2
1	10		47.00	0	1		22.53	0.439	51.32	137.27	2

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

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

1

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TABLE 2.2.22.6=FLANGE PROPORTIONS CHECK

SP NO	IN NO	D L	FROM SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0		0.00	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	1		4.70	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	2		9.40	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	3		14.10	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	4		18.80	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	5		23.50	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	6		28.20	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	7		32.90	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	8		37.60	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	9		42.30	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	10		47.00	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0

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

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

1

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	10.0	0.000	16.908	9.889	2.870	1.580
2	10.0	20.0	16.908	23.267	21.190	19.113	1.090
3	20.0	30.0	22.246	23.267	23.386	23.504	1.000
4	30.0	40.0	12.826	22.246	18.765	15.284	1.170

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

1

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	LMDApf	LMDArf	LTB CAT	Lb (ft)	Lp (ft)	Lr (ft)	GOV CAT	CAT
1	0		0.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	1		4.70	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	2		9.40	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	3		14.10	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	4		18.80	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	5		23.50	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	6		28.20	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	7		32.90	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	8		37.60	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	9		42.30	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	10		47.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

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

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

1

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.58	50.00	50.00	50.00
1	1		4.70	1	1.000	1.000	1.58	50.00	50.00	50.00
1	2		9.40	1	1.000	1.000	1.58	50.00	50.00	50.00
1	3		14.10	1	1.000	1.000	1.09	50.00	47.59	47.59
1	4		18.80	1	1.000	1.000	1.09	50.00	47.59	47.59
1	5		23.50	1	1.000	1.000	1.00	50.00	43.66	43.66
1	6		28.20	1	1.000	1.000	1.00	50.00	43.66	43.66
1	7		32.90	1	1.000	1.000	1.17	50.00	50.00	50.00
1	8		37.60	1	1.000	1.000	1.17	50.00	50.00	50.00
1	9		42.30	1	1.000	1.000	1.00	50.00	43.66	43.66
1	10		47.00	1	1.000	1.000	1.00	50.00	43.66	43.66

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

1

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TABLE 2.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.
1	0		0.00	1	2	2			2
1	1		4.70	1	2	2			2
1	2		9.40	1	2	2			2
1	3		14.10	1	2	2			2
1	4		18.80	1	2	2			2
1	5		23.50	1	2	2			2
1	6		28.20	1	2	2			2
1	7		32.90	1	2	2			2
1	8		37.60	1	2	2			2
1	9		42.30	1	2	2			2
1	10		47.00	1	2	2			2

Please read NOTE on the following page

1

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

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

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

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

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

SP NO	IN D L	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	1	4.70	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	2	9.40	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	3	14.10	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.6	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	4	18.80	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.6	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	5	23.50	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	6	28.20	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	7	32.90	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	8	37.60	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	9	42.30	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
			0.0	-	0.0	-	0.0	50.0	-	-	0
1	10	47.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
			0.0	-	0.0	-	0.0	50.0	-	-	0

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

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

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

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

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

TABLE 2.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP NO	IN D L	FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	1	4.70	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	2	9.40	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	3	14.10	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	4	18.80	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	5	23.50	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	6	28.20	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	7	32.90	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	8	37.60	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	9	42.30	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3
1	10	47.00	0.000	0.000	0.000	0.000	0.000	1
					0.000		0.000	3

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

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

TABLE 2.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0	0.00	0	-	-	0.0	-	0.0	30.0	0.0	50.0	-0--
				-	-	0.0	-	0.0	30.0	0.0	50.0	--0-
1	1	4.70	0	-	-	9.4	-	0.0	30.0	9.4	50.0	-0--
				-	-	9.4	-	0.0	30.0	9.4	50.0	--0-
1	2	9.40	0	-	-	16.4	-	0.0	30.0	16.4	50.0	-0--
				-	-	16.4	-	0.0	30.0	16.4	50.0	--0-
1	3	14.10	0	-	-	20.7	-	0.0	30.0	20.7	47.6	-0--
				-	-	20.7	-	0.0	30.0	20.7	50.0	--0-
1	4	18.80	0	-	-	23.2	-	0.0	30.0	23.2	47.6	-0--
				-	-	23.2	-	0.0	30.0	23.2	50.0	--0-
1	5	23.50	0	-	-	23.5	-	0.0	30.0	23.5	43.7	-0--
				-	-	23.5	-	0.0	30.0	23.5	50.0	--0-
1	6	28.20	0	-	-	23.2	-	0.0	30.0	23.2	43.7	-0--
				-	-	23.2	-	0.0	30.0	23.2	50.0	--0-
1	7	32.90	0	-	-	20.7	-	0.0	30.0	20.7	50.0	-0--
				-	-	20.7	-	0.0	30.0	20.7	50.0	--0-
1	8	37.60	0	-	-	16.4	-	0.0	30.0	16.4	50.0	-0--
				-	-	16.4	-	0.0	30.0	16.4	50.0	--0-
1	9	42.30	0	-	-	9.4	-	0.0	30.0	9.4	43.7	-0--
				-	-	9.4	-	0.0	30.0	9.4	50.0	--0-
1	10	47.00	0	-	-	0.0	-	0.0	30.0	0.0	45.0	-0--
				-	-	0.0	-	0.0	30.0	0.0	50.0	--0-

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

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility; Mu = maximum of Strengh I, II and IV

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

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

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

TABLE 2.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP NO	IN D L	FROM	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00	0		0.000	0.000	0.000	0.000	4
					0.000	0.000	0.000	0.000	4
1	1	4.70	0		0.000	0.000	0.189	0.189	4
					0.000	0.000	0.189	0.189	4
1	2	9.40	0		0.000	0.000	0.327	0.327	4
					0.000	0.000	0.327	0.327	4
1	3	14.10	0		0.000	0.000	0.435	0.435	4
					0.000	0.000	0.414	0.414	4
1	4	18.80	0		0.000	0.000	0.487	0.487	4
					0.000	0.000	0.464	0.464	4
1	5	23.50	0		0.000	0.000	0.538	0.538	4
					0.000	0.000	0.469	0.469	4
1	6	28.20	0		0.000	0.000	0.531	0.531	4
					0.000	0.000	0.464	0.464	4
1	7	32.90	0		0.000	0.000	0.414	0.414	4
					0.000	0.000	0.414	0.414	4
1	8	37.60	0		0.000	0.000	0.327	0.327	4
					0.000	0.000	0.327	0.327	4
1	9	42.30	0		0.000	0.000	0.216	0.216	4
					0.000	0.000	0.189	0.189	4
1	10	47.00	0		0.000	0.000	0.000	0.000	4
					0.000	0.000	0.000	0.000	4

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

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

1

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TABLE 2.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY

SP NO	IN NO	D L (ft)	FROM SUPT	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	Art.6.10.9.2 [3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	1	4.70	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	2	9.40	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	3	14.10	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	4	18.80	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	5	23.50	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	6	28.20	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	7	32.90	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	8	37.60	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	9	42.30	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	10	47.00	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8

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1

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

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

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

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

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

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

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

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

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

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

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TABLE 2.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY

SP NO	IN NO	D L (ft)	FROM SUPT	WEB STABILITY UNSTIFFENED,[1] D/tw		LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	OVERALL CATEGORY WEB
1	0	0.00		2		33.4	<	286.8	2	2
1	1	4.70		2		28.6	<	286.8	2	2
1	2	9.40		2		24.0	<	286.8	2	2
1	3	14.10		2		19.4	<	286.8	2	2
1	4	18.80		2		15.0	<	286.8	2	2
1	5	23.50		2		11.2	<	286.8	2	2
1	6	28.20		2		15.0	<	286.8	2	2
1	7	32.90		2		19.4	<	286.8	2	2
1	8	37.60		2		24.0	<	286.8	2	2
1	9	42.30		2		28.6	<	286.8	2	2
1	10	47.00		2		33.4	<	286.8	2	2

Please read NOTE on the following page
1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK
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NOTE [1]: WEB STABILITY, D/tw ratio ..... buckling consideration

For compact section (composite sections in positive flexure)

w/o long. stiff.

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

w / long. stiff.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK
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TABLE 2.2.22.17=TRANSVERSE STIFFENER SPACING

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	LFD/LRFD UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF SHEAR CAPACITY (k)	MAX. ALLOWABLE TRANS. STIFFENERS 1=YES , 0=NO	SPACING (ft-in)
1	0	0.00		50.	33.41	286.83	0	
1	1	4.70		50.	28.64	286.83	0	
1	2	9.40		50.	24.01	286.83	0	
1	3	14.10		50.	19.44	286.83	0	
1	4	18.80		50.	15.01	286.83	0	
1	5	23.50		50.	11.19	286.83	0	
1	6	28.20		50.	15.01	286.83	0	
1	7	32.90		50.	19.44	286.83	0	
1	8	37.60		50.	24.01	286.83	0	
1	9	42.30		50.	28.64	286.83	0	
1	10	47.00		50.	33.41	286.83	0	

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK
FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5250'x17'\Merlin-Da

TABLE 2.2.22.19A=SPECIAL FATIGUE REQUIREMENTS FOR WEBS

SP NO	IN NO	D L (ft)	FROM SUPT	SHEAR DL (k)	SDL (k)	FACTORED FATIGUE PERM. V (k)	<=>	Vcr (k)	<=>	UNF. PERM + FAC FATIG V (k)
1	0	0.00		2.73	0.00	12.47	3.41 <	286.83	>	24.55
1	1	4.70		2.17	0.00	10.52	2.71 <	286.83	>	20.57
1	2	9.40		1.64	0.00	8.70	2.05 <	286.83	>	16.87
1	3	14.10		1.07	0.00	6.89	1.34 <	286.83	>	13.14
1	4	18.80		0.55	0.00	6.11	0.68 <	286.83	>	11.24
1	5	23.50		0.02	0.00	4.98	0.02 <	286.83	>	8.74
1	6	28.20		0.55	0.00	6.11	0.68 <	286.83	>	11.24
1	7	32.90		1.07	0.00	6.89	1.34 <	286.83	>	13.14
1	8	37.60		1.64	0.00	8.70	2.05 <	286.83	>	16.87
1	9	42.30		2.17	0.00	10.52	2.71 <	286.83	>	20.57
1	10	47.00		2.73	0.00	12.47	3.41 <	286.83	>	24.55

NOTE: This table checks Art. 6.10.3.3 and Art. 6.10.5.3.
Vcr is the shear-buckling resistance determined from Eq. 6.10.9.3.3-1.
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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK
FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5350'x17'\Merlin-Da

TABLE 2.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	RATIO1	BOT. FLANGE ff+fl/2 (ksi)	[2]	RATIO2	COMP. fc (ksi)	FLG fcrw (ksi)	[3]	FLAG TBC
1	0		0.00	-	0.0	40.0	0.000	0.0	40.0	Infin	0.0	50.0	000	
1	1		4.70	-	7.1	40.0	0.177	7.1	40.0	Infin	7.1	50.0	000	
1	2		9.40	-	12.2	40.0	0.306	12.2	40.0	Infin	12.2	50.0	000	
1	3		14.10	-	15.5	40.0	0.388	15.5	40.0	Infin	15.5	50.0	000	
1	4		18.80	-	17.4	40.0	0.435	17.4	40.0	Infin	17.4	50.0	000	
1	5		23.50	-	17.6	40.0	0.440	17.6	40.0	Infin	17.6	50.0	000	
1	6		28.20	-	17.4	40.0	0.435	17.4	40.0	Infin	17.4	50.0	000	
1	7		32.90	-	15.5	40.0	0.388	15.5	40.0	Infin	15.5	50.0	000	
1	8		37.60	-	12.2	40.0	0.306	12.2	40.0	Infin	12.2	50.0	000	
1	9		42.30	-	7.1	40.0	0.177	7.1	40.0	Infin	7.1	50.0	000	
1	10		47.00	-	0.0	40.0	0.000	0.0	40.0	NaN	0.0	50.0	000	

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

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5450'x17'\Merlin-Da

NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

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

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

RATIO1 = ff / [1] or (ff + fl/2) / [1] (top flange)
RATIO2 = (ff + fl/2) / [2] (bot. flange)

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
5550'x17'\Merlin-Da

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY
1	0		0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1		4.70	TR	-5.9	A B B^C^C D E E^	TR	5.9	A B B^C^C D
1	2		9.40	TR	-9.8	A B B^C^C D E E^	TR	9.8	A B B^C^C
1	3		14.10	TR	-11.6	A B B^C^C D E E^	TR	11.6	A B B^C^
1	4		18.80	TR	-13.7	A B B^C^C D E E^	TR	13.7	A B
1	5		23.50	TR	-14.0	A B B^C^C D E E^	TR	14.0	A B
1	6		28.20	TR	-13.7	A B B^C^C D E E^	TR	13.7	A B
1	7		32.90	TR	-11.6	A B B^C^C D E E^	TR	11.6	A B B^C^
1	8		37.60	TR	-9.8	A B B^C^C D E E^	TR	9.8	A B B^C^C
1	9		42.30	TR	-5.9	A B B^C^C D E E^	TR	5.9	A B B^C^C D
1	10		47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.75 load factor.
Design for Infinite Life

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

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
5650'x17'\Merlin-Da

TABLE 2.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY
1	0		0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1		4.70	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	2		9.40	TR	-4.5	A B B^C^C D E E^	TR	4.5	A B B^C^C
1	3		14.10	TR	-5.3	A B B^C^C D E E^	TR	5.3	A B B^C^C
1	4		18.80	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B
1	5		23.50	TR	-6.4	A B B^C^C D E E^	TR	6.4	A B
1	6		28.20	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B
1	7		32.90	TR	-5.3	A B B^C^C D E E^	TR	5.3	A B B^C^C
1	8		37.60	TR	-4.5	A B B^C^C D E E^	TR	4.5	A B B^C^C
1	9		42.30	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	10		47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.80 load factor.
Design for Finite Life w/ ADTT Single Lane= 1000 & No. of Cycles= 27375000
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
5750'x17'\Merlin-Da

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

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

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
5850'x17'\Merlin-Da

TABLE 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	ksi	D2	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING FACTOR	LIVE [3]	INVENT. RATING FACTOR	C A T TYPE
1	0		0.00	50.00	TOP	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
				50.00	BOT	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
1	1		4.70	50.00	TOP	-0.79		-4.84	0.00	0.00	7.51	HL-93	5.79	0
				50.00	BOT	0.79		4.84	0.00	0.00	7.51	HL-93	5.79	0
1	2		9.40	50.00	TOP	-1.40		-8.34	0.00	0.00	4.28	HL-93	3.30	0
				50.00	BOT	1.40		8.34	0.00	0.00	4.28	HL-93	3.30	0
1	3		14.10	47.59	TOP	-1.84		-10.52	0.00	0.00	3.19	HL-93	2.46	0
				50.00	BOT	1.84		10.52	0.00	0.00	3.36	HL-93	2.59	0
1	4		18.80	47.59	TOP	-2.10		-11.76	0.00	0.00	2.83	HL-93	2.19	0
				50.00	BOT	2.10		11.76	0.00	0.00	2.99	HL-93	2.30	0
1	5		23.50	43.66	TOP	-2.19		-11.85	0.00	0.00	2.56	HL-93	1.97	0
				50.00	BOT	2.19		11.85	0.00	0.00	2.95	HL-93	2.28	0
1	6		28.20	43.66	TOP	-2.10		-11.76	0.00	0.00	2.59	HL-93	1.99	0
				50.00	BOT	2.10		11.76	0.00	0.00	2.99	HL-93	2.30	0
1	7		32.90	50.00	TOP	-1.84		-10.52	0.00	0.00	3.36	HL-93	2.59	0
				50.00	BOT	1.84		10.52	0.00	0.00	3.36	HL-93	2.59	0
1	8		37.60	50.00	TOP	-1.40		-8.34	0.00	0.00	4.28	HL-93	3.30	0
				50.00	BOT	1.40		8.34	0.00	0.00	4.28	HL-93	3.30	0
1	9		42.30	43.66	TOP	-0.79		-4.84	0.00	0.00	6.54	HL-93	5.04	0
				50.00	BOT	0.79		4.84	0.00	0.00	7.51	HL-93	5.79	0
1	10		47.00	44.99	TOP	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
				50.00	BOT	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0

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1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
5950'x17'\Merlin-Da

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6050'x17'\Merlin-Da

TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	INVENT. LIVE LOAD TYPE	RATING FACTOR
1	0	0.00	40.0	0.00	0.00	0.00	0.00	99.99	HL-93	76.92
1	1	4.70	40.0	-0.79	0.79	-4.84	4.84	0.00	0.00	8.11	HL-93	6.24
1	2	9.40	40.0	-1.40	1.40	-8.34	8.34	0.00	0.00	4.63	HL-93	3.56
1	3	14.10	40.0	-1.84	1.84	-10.52	10.52	0.00	0.00	3.63	HL-93	2.79
1	4	18.80	40.0	-2.10	2.10	-11.76	11.76	0.00	0.00	3.22	HL-93	2.48
1	5	23.50	40.0	-2.19	2.19	-11.85	11.85	0.00	0.00	3.19	HL-93	2.45
1	6	28.20	40.0	-2.10	2.10	-11.76	11.76	0.00	0.00	3.22	HL-93	2.48
1	7	32.90	40.0	-1.84	1.84	-10.52	10.52	0.00	0.00	3.63	HL-93	2.79
1	8	37.60	40.0	-1.40	1.40	-8.34	8.34	0.00	0.00	4.63	HL-93	3.56
1	9	42.30	40.0	-0.79	0.79	-4.84	4.84	0.00	0.00	8.11	HL-93	6.24
1	10	47.00	40.0	0.00	0.00	0.00	0.00	0.00	0.00	99.99	HL-93	76.92

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

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6150'x17'\Merlin-Da

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

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

[3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6250'x17'\Merlin-Da

TABLE 2.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	47.00	23.50	STRENGTH L.S.	2.56	1.97	HL-93
1	47.00	23.50	SERVICEBILITY	3.19	2.45	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.558
Inventory= 1.974

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

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL MOMENT (k-ft)
1	47.00	23.50	STRENGTH L.S.	4.49	133.68
1	47.00	23.50	SERVICEBILITY	4.14	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1000
Legal Load Rating= 4.144

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6350'x17'\Merlin-Da

TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	UNFACTORED L+I MAX. SHEAR D2	POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	286.8	0.0	2.7	17.1	0.0	12.22	HL-93	9.43
1	1	4.70	286.8	0.0	2.2	14.8	-0.8	14.18	HL-93	10.94
1	2	9.40	286.8	0.0	1.6	12.6	-2.1	16.78	HL-93	12.95
1	3	14.10	286.8	0.0	1.1	10.3	-3.4	20.43	HL-93	15.76
1	4	18.80	286.8	0.0	0.5	8.2	-4.9	25.88	HL-93	19.96
1	5	23.50	286.8	0.0	0.0	6.4	-6.4	33.29	HL-93	25.68
1	6	28.20	286.8	0.0	-0.5	4.9	-8.2	25.88	HL-93	19.96
1	7	32.90	286.8	0.0	-1.1	3.4	-10.3	20.43	HL-93	15.76
1	8	37.60	286.8	0.0	-1.6	2.1	-12.6	16.78	HL-93	12.95
1	9	42.30	286.8	0.0	-2.2	0.8	-14.8	14.18	HL-93	10.94
1	10	47.00	286.8	0.0	-2.7	0.0	-17.1	12.22	HL-93	9.43

1

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UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6450'x17'\Merlin-Da

TABLE 2.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	RATING CRITERION	OPERATING RATING	INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	47.00	47.00	STRENGTH L.S.	12.22	9.43	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating=12.219
Inventory= 9.426

TABLE 2.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	RATING CRITERION	LEGAL LOAD RATING	MAXIMUM LL SHEAR (kips)
1	47.00	47.00	STRENGTH L.S.	19.87	14.23

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.300 & ADTT = 1000
Legal Load Rating=19.866

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
6550'x17'\Merlin-Da

TABLE 2.2.34.1=RATING; LIVE LOAD DEFLECTION

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ I. DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	23.50	1	0.125	-0.59	HL-93	1.19
		1	0.125	-0.59		1.19

NOTE: [1] "-" indicates downward deflection

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

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

TEMPORARY BRIDGE SUBSTRUCTURE DESIGN

Swanzey, NH (Rabbit Hollow Road over Perry Brook - Hansen Construction)

Notes
No borings were taken at temporary bridge abutment locations
The temporary bridge abutments will be set at the top of the embankments and will not impact the brook.
Use boring information supplied in plans prepared for proposed bridge design.

Boring B-1: Avg. blow count for 0' - 7' depth

$BC_{B1} := 4$

Boring B-2: Avg. blow count for 0' - 7' depth

$BC_{B2} := \frac{(3+4+4+3+4+4+3+3)}{8} = 3.5$

Controlling blow count for design:

$BC := min(BC_{B1}, BC_{B2}) = 3.5$

Conservatively use:

$\phi_f := 28 \cdot deg$ Friction Angle

Allowable Bearing Capacity (Strip footing):
Using Terzaghi Bearing Capcacity:

$\gamma_c := 100\ pcf$ $D_f := 2\ ft$ $q_{bar} := \gamma_c \cdot D_f = 200\ psf$

$N_q := 14.7$ $B := 6\ ft$ $L := 20\ ft$

$N_\gamma := 16.7$ $S_\gamma := 1 - 0.4 \cdot \frac{B}{L} = 0.88$ LRFD Table 10.6.3.1.2a-3

$S_q := 1 + \frac{B}{L} \cdot \tan(\phi_f) = 1.16$

$c_{wq} := 1$ $c_{w\gamma} := 1$ LRFD Table 10.6.3.1.2a-2

for cohesionless soil:
ultimate bearing capacity $q_{ult} := q_{bar} \cdot N_q \cdot S_q \cdot c_{wq} + 0.5 \cdot \gamma_c \cdot B \cdot N_\gamma \cdot S_\gamma \cdot c_{w\gamma}$ LRFD 10.6.3.1.2a-1

safety factor for cohesionless soils = 3 $q_{ult} = 7.82\ ksf$

Use $SF := 3.5$ since no actual boring data at temporary abutment locations

allowable bearing capacity $q_{all} := \frac{q_{ult}}{SF} = 2.23\ ksf$

Superstructure Dead Load (from Merlin-Dash):

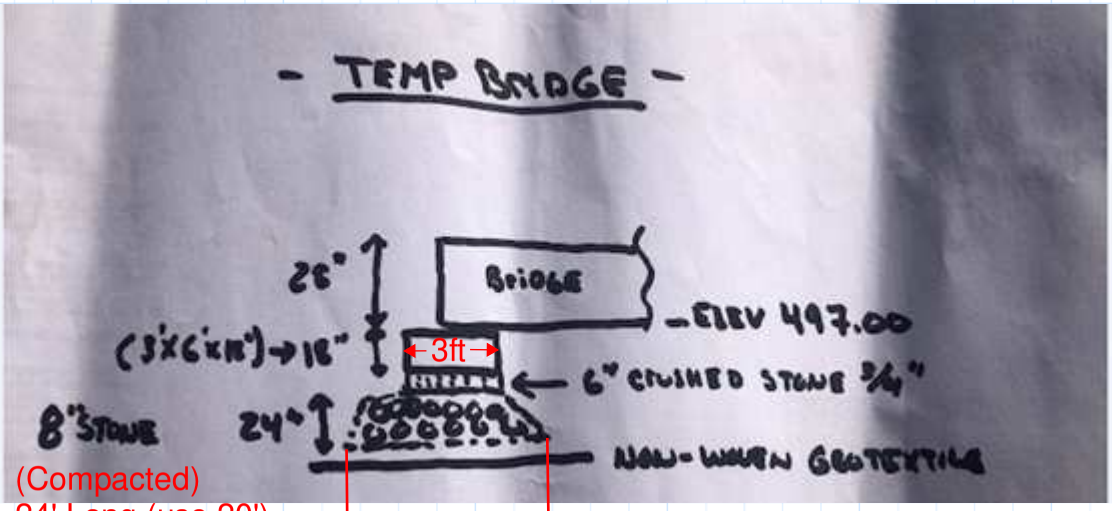
$$DL_{super} := 3.02\ kip \cdot 6 + 2.73\ kip \cdot 2 = 23.58\ kip$$

Live Load (from Appendix A AASHTO 17th Edition: HS20-44), governing reaction for L = 47' plus HL-93 Lane Load

$$LL_{super} := 72.7\ kip$$

Total Factored Superstructure Load on Abutment:

$$TL_{super} := 1.25 \cdot DL_{super} + 1.75 \cdot LL_{super} = 156.7\ kip$$



Abutment DL (factored)

$$DL_{abutment} := (18\ ft \cdot 3\ ft \cdot 1.5\ ft \cdot 150\ pcf + 20\ ft \cdot 3\ ft \cdot 0.5\ ft \cdot 170\ pcf + 7\ ft \cdot 2\ ft \cdot 22\ ft \cdot 170\ pcf) \cdot 1.25 = 87.01\ kip$$

$$DL_{abutment} = 87.01\ kip$$

Actual bearing pressure:

$$q_{act} := \frac{TL_{super} + DL_{abutment}}{B \cdot L} = 2.03\ ksf$$

Is actual bearing pressure less than allowable bearing pressure?

$$\text{if}\left(q_{all} > q_{act}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

R.M. PIPER

ADM Welding & Fabrication, LLC
37 Broadhead Street
Warren, PA 16365
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E: admwelding@verizon.net

November 21, 2022

Temporary Bridge
Thornton Ferry Road
Over Beaver Brook
Town of Amherst, NH

Technical Proposal

<u>Bridge Specifications</u> [clarifications to and/or deviations from project specifications in bold]:

- Either 12' or 22' clear travel width [per the Town's selection].
- Overall width no more than 24" wider than clear width.
- Maximum structure depth of 32" [bottom of stringer to top of running plank].
- Nominal 50' overall length.
- Capacity for the following:
  - HL93 loading with no speed restriction [full impact loading].
  - **Live load deflection limited to L/800.**
- New Hampshire PE stamped drawings and calculations for the following:
  - Stringers
  - Timber deck
  - Guardrail
- Tapered beam ends, pipe towbars, and additional reinforcing for ease of handling.
  - Designed to withstand the rigors of relocation between multiple sites.
  - Bridges can be "skidded" utilizing typical excavating/forestry equipment.
- Full-dimension steel end plates [backwalls], above towbars.
- "Strong-post" guardrail system consisting of posts spaced @ 3' – 1.5" c-c and standard galvanized w-rail.
  - Guardrail system will be shown to have TL-1 equivalency by calculation.
- **Two coats epoxy primer/urethane topcoat over SP3 prep.***
- 56-mil waterproofing membrane on 4-mil carrier film, slit to overhang top flanges of stringers by 1/4" – ½" on either side, pre-installed on top flanges between steel and timber deck.
- Timber deck system:
  - Treated, nail-laminated, 2"x4" SYP transverse deck system.
  - Treated, nominal 2" thick, SYP longitudinal timber "running plank" installed over top of transverse deck.
  - Transverse deck boards to be #1 grade. **Running plank to be #2 grade**.
  - All lumber to be MCA treated to UC4A "ground contact". **No additional waterproofing is included as the lumber is already treated.**

***Both a DOT-type paint system and/or a galvanized finish are available as well, at additional cost and lead time.**

Certification of Design

The bridge will be designed/fabricated specifically for this project based on the unique project characteristics/specs however, as noted, PE stamped drawings and calculations for the stringers/decking/guardrail will be provided showing adequacy for the above criteria. ADM has previously fabricated bridges to these same criteria and has no concerns about our ability to meet them.

Description of Bridge System

Single-Lane Bridge

The bridge would consist of two modular steel sections each nominally 8' wide x 50' long x 27,000# [each modular section would consist of three stringer beams with pre-welded bracing]. Each modular section has the guardrail system and timber deck pre-installed. Each section would transport as a legal-dimension/weight load on a standard flatbed trailer.

The sections require a level/uniform bearing surface on each end, preferably a minimum of 2' wide. No special bearing plates are required, simplifying the installation process. Assuming a 2' wide bearing surface at each end, and taking into account the 6" tapered beam ends, maximum clear span is approximately 45'.

The tapered beam ends, towbars, and additional reinforcing facilitate handling with excavating/forestry equipment – the sections can be "skidded" into sites that are not accessible to the delivery vehicle and can often be slid into place with excavators in lieu of a crane. Integrated steel end plates [backwalls] eliminate the need to install a backwall on the temporary abutment.

The sections are set in place over the crossing one at a time, aligned, and bolted together. Lastly a series of steel "center panels" are set in place to cover the bolt-splice location. Typical installation requires only a few hours with two laborers/ironworkers plus equipment/operators. The process is intuitive and requires only basic rigging and hand tools. Dis-assembly is essentially the reverse process, and requires similar resources/timeframe.

Although designed for temporary use in different locations, the bridge is also well-suited for installation as a permanent structure.

Two-Lane Bridge

The two-lane bridge would be conceptually similar to the single-lane structure, except that it would consist of four modular units.

Proposed Project Timeline

- Design Generation [ADM] – 4 weeks
- Design Review/Seal [NH PE] – 2 weeks
- Material Procurement – 2-6 weeks [depending on whether beams can be found in stock or have to be rolled to order]
- Fabrication
    - Single-Lane Bridge – 3 weeks
    - Two-Lane Bridge – 4 weeks

*The above timeline is based on the assumption that a single/complete NH PE sealed submittal package would be provided to HoyleTanner. It may be possible to expedite if there was a willingness to do either/both of the following:

- Review the stringer calculations in advance of the other bridge submittals so that beam procurement could start earlier.
- Review submittals without the NH PE stamp and "approve as noted" subject to final submission including NH seal.

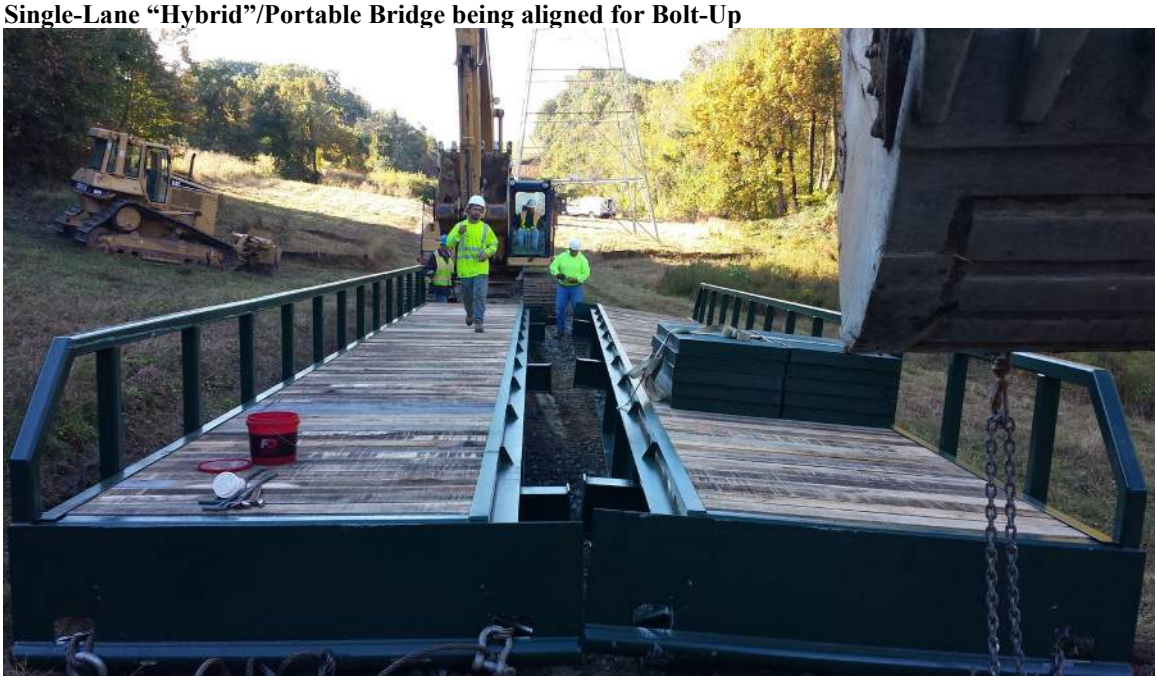
“Hybrid”/Portable Bridge Section being “Skidded” along utility ROW



Single-Lane “Hybrid”/Portable Bridge Section set in place over existing/deteriorated structure [awaiting second section]



Single-Lane “Hybrid”/Portable Bridge being aligned for Bolt-Up



Single-Lane “Hybrid”/Portable Bridge with Guardrail System similar to Proposed Guardrail



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November 21, 2022

Temporary Bridge
Thornton Ferry Road
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Technical Proposal

Bridge Specifications [clarifications to and/or deviations from project specifications in bold]:

- Either 12' or 22' clear travel width [pcr thc Town's seleetion].
- Overall width no more than 6' wider than clear width.
- Maximum structure depth of 32" [bottom of stringer to top of running plank].
- Nominal 50' overall length.
- Capacity for the following:
  - HL93 loading with no speed restriction [full impact loading].
  - **Livc load deflection limited to L/800.**
- New Hampshire PE stamped drawings and calculations for the following:
  - Stringers
  - Timber deck
- Tapered beam ends, pipe towbars, and additional reinforcing for ease of handling.
  - Designed to withstand the rigors of relocation between multiple sites.
  - Bridges can be "skidded" utilizing typical excavating/forestry equipment.
- Full-dimension steel end plates [backwalls], above towbars.
- **Two coats cpoxy primer/urethane topcoat ovcr SP3 prep.***
- 56-mil waterproofing membrane on 4-mil carrier film, slit to overhang top flanges of stringers by 1/4" – ½" on either side, pre-installed on top flanges between steel and timber deck.
- Timber deck system:
  - Treated, nail-laminated, 2"x4" SYP transverse deck system.
  - Treated, nominal 2" thick, SYP longitudinal timber "running plank" installed over top of transverse deck.
  - Transverse deck boards to be #1 grade. **Running plank to be #2 grade.**
  - All lumber to be MCA treated to UC4A "ground contact". **No additional waterproofing is included as the lumber is already treated.**

***Both a DOT-type paint system and/or a galvanizcd finish are available as well, at additional cost and lead time.**

Certification of Design

The bridge will be designed/fabricated specifically for this project based on the unique project characteristics/specs however, as noted, PE stamped drawings and calculations for the stringers and decking will be provided showing adequacy for the above criteria. ADM has previously fabricated bridges to these same criteria and has no concerns about our ability to meet them.

Description of Bridge System

Single-Lane Bridge

The bridge would consist of two modular steel sections each nominally 8.5' wide x 50' long x 31,000# [each modular section would consist of four stringer beams with pre-welded bracing]. Each modular section has the timber deck pre-installed. Each section would transport as a legal-dimension/weight load on a standard flatbed trailer.

The sections require a level/uniform bearing surface on each end, preferably a minimum of 2' wide. No special bearing plates are required, simplifying the installation process. Assuming a 2' wide bearing surface at each end, and taking into account the 6" tapered beam ends, maximum clear span is approximately 45'.

The tapered beam ends, towbars, and additional reinforcing facilitate handling with excavating/forestry equipment – the sections can be "skidded" into sites that are not accessible to the delivery vehicle and can often be slid into place with excavators in lieu of a crane. Integrated steel end plates [backwalls] eliminate the need to install a backwall on the temporary abutment.

The sections are set in place over the crossing one at a time, aligned, and bolted together. Lastly a series of steel "center panels" are set in place to cover the bolt-splice location. Typical installation requires only a few hours with two laborers/ironworkers plus equipment/operators. The process is intuitive and requires only basic rigging and hand tools. Dis-assembly is essentially the reverse process, and requires similar resources/timeframe.

Pre-cast concrete jersey barriers will be installed on the modular steel sections after they are set in place to serve as the guardrail system.

Although designed for temporary use in different locations, the bridge is also well-suited for installation as a permanent structure.

Two-Lane Bridge

The two-lane bridge would be conceptually similar to the single-lane structure, except that it would consist of four modular units each nominally 7' wide x 50' long x 27,000#.

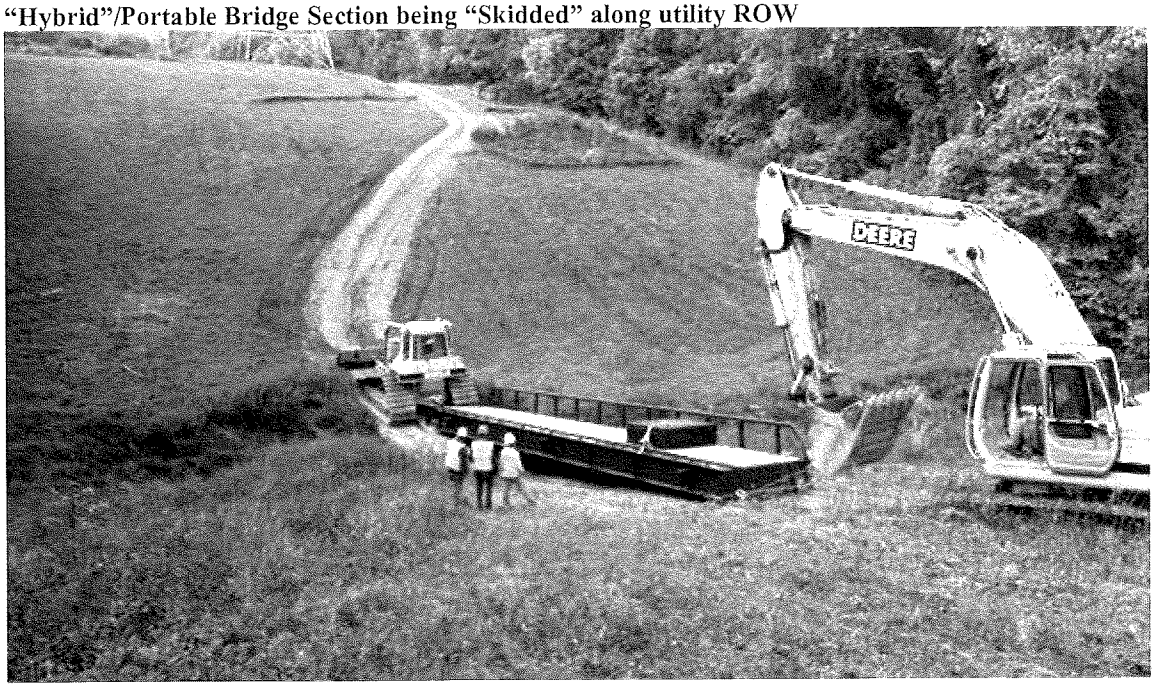
Proposed Project Timeline

- Design Generation [ADM] – 4 weeks
- Design Review/Seal [NH PE] – 2 weeks
- Material Procurement – 2-6 weeks [depending on whether beams can be found in stock or have to be rolled to order]
- Fabrication
    - Single-Lane Bridge – 3 weeks
    - Two-Lane Bridge – 4 weeks

*The above timeline is based on the assumption that a single/complete NH PE sealed submittal package would be provided to HoyleTanner. It may be possible to expedite if there was a willingness to do either/both of the following:

- Review the stringer calculations in advance of the other bridge submittals so that beam procurement could start earlier.
- Review submittals without the NH PE stamp and "approve as noted" subject to final submission including NH seal.

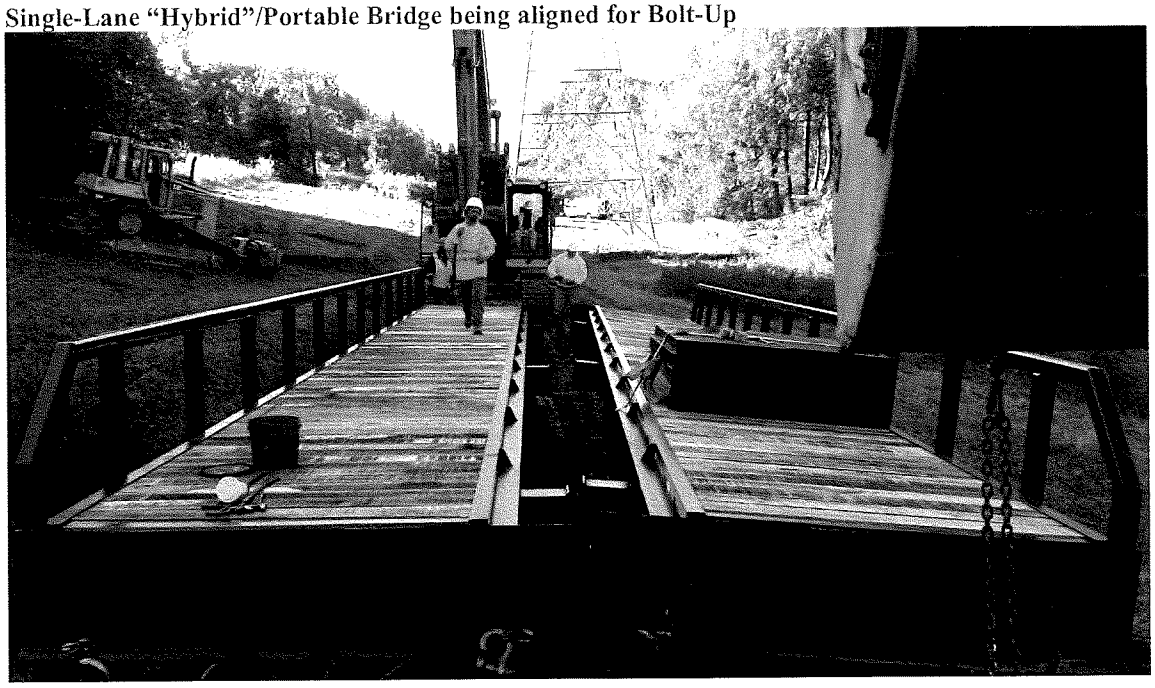
"Hybrid"/Portable Bridge Section being "Skidded" along utility ROW



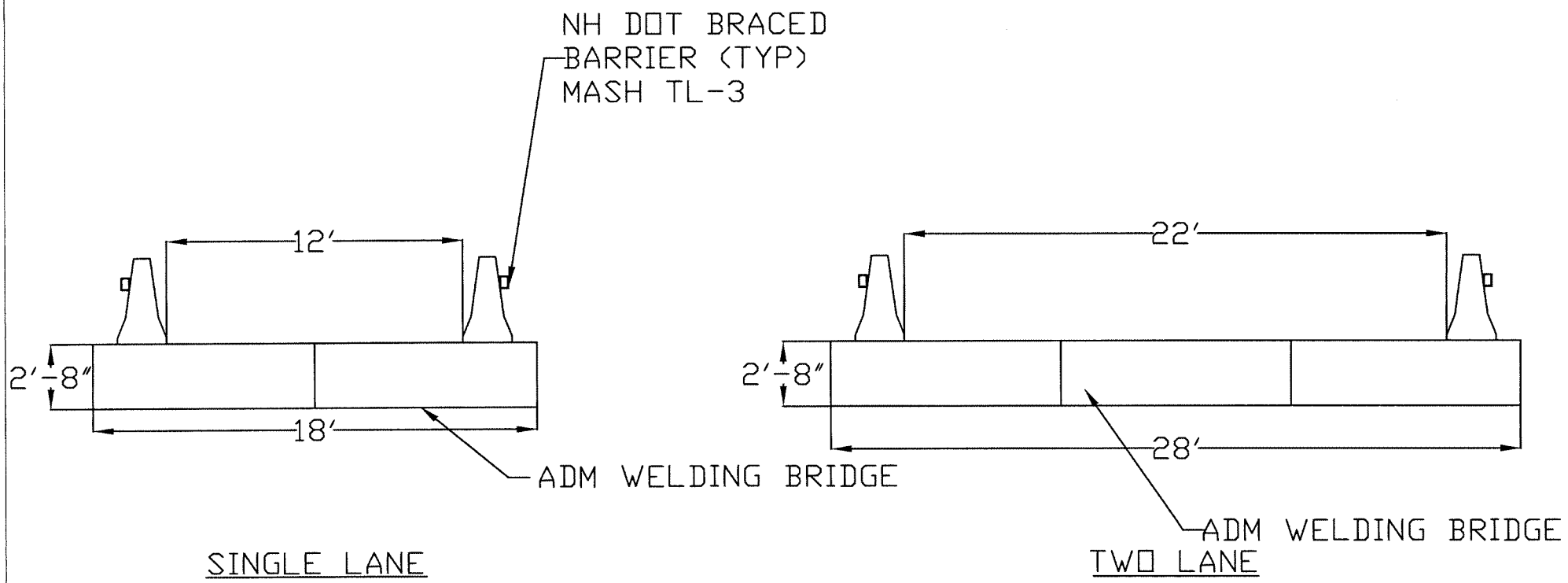
Single-Lane "Hybrid"/Portable Bridge Section set in place over existing/deteriorated structure [awaiting second section]



Single-Lane "Hybrid"/Portable Bridge being aligned for Bolt-Up



Note: Photos depict a pre-installed steel guardrail different from the proposed pre-cast concrete jersey barrier guardrail [to be installed on top of the bridge deck].



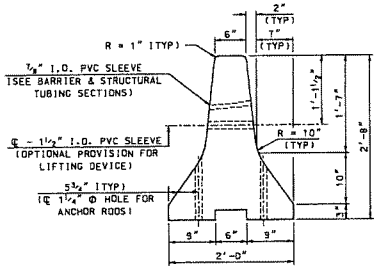
COLD RIVER BRIDGES, LLC
10 LANBRO LANE
TEL 603-756-9300 WALPOLE, NH FAX 603-756-9303

TOWN OF AMHERST NH

TYPICAL SECTION

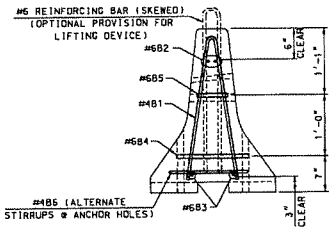
SHEET NUMBER
TB

11/22/22



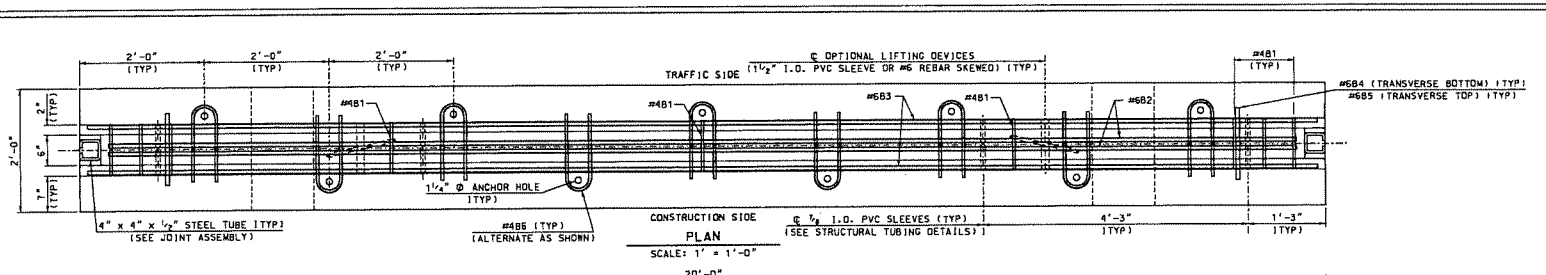
TYPICAL SECTION

SCALE: 1" = 1'-0"



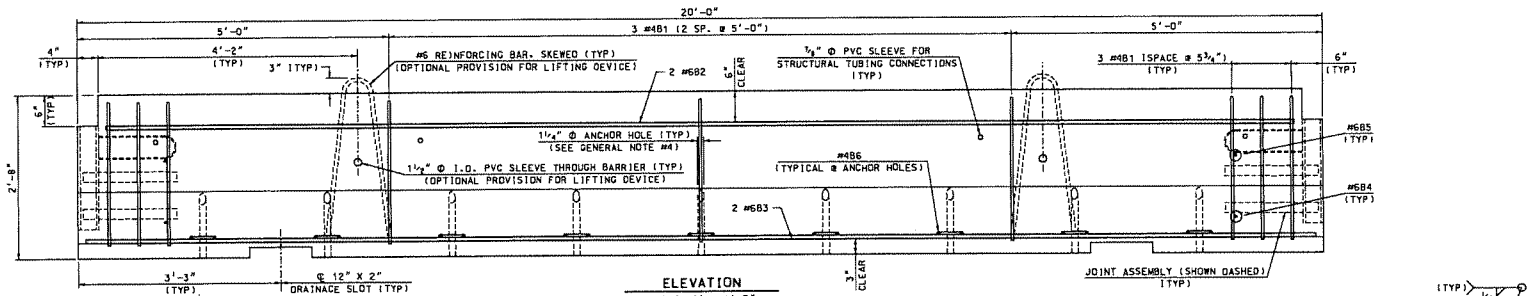
TYPICAL SECTION

SCALE: 1" = 1'-0"



PLAN

SCALE: 1' = 1'-0"



ELEVATION

SCALE: 1' = 1'-0"

GENERAL NOTES

(1) PORTABLE CONCRETE BARRIER SHALL BE FURNISHED BY THE CONTRACTOR AND PAID FOR AS ITEM 606.41741 PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL (BRIDGE). CONCRETE BARRIER AND ALL ATTACHMENTS SHALL BE FABRICATED IN ACCORDANCE WITH SPECIAL PROVISIONS. ALL BARRIER UNITS FOR BRACED SYSTEMS SHALL BE 20' LONG.

(2) PORTABLE CONCRETE BARRIER DETAILS, AS SHOWN IN THESE PLANS, ARE IN COMPLIANCE WITH REQUIREMENTS PER UPDATED NCHRP REPORT 350 FOR TEST NO. 3-11 (MASH TEST LEVEL 3). CRASH TESTED BY MIDWEST ROADSIDE SAFETY: NY BOX BEAM STIFFENING OF UNANCHORED TCB, MARCH 2008, AND ACCEPTED PER FHWA LETTER B-239 (11/1/2012). THE BARRIER SYSTEM HAS BEEN CRASH TESTED WITH A 27.6" DYNAMIC DEFLECTION WHICH WILL ALLOW BRACED BARRIER TO BE PLACED A MINIMUM 12" FROM THE EDGE OF BRIDGE DECK.

(3) A MINIMUM OF TWO BARRIER UNITS, WITH BRACED JOINTS ARE REQUIRED TO BE PLACED BEYOND BOTH ENDS OF THE BRIDGE WORK AREA. FOR SPEEDS GREATER THAN 45 MPH. FOR SPEEDS ≤ 45 MPH, A MINIMUM OF ONE BRACED BARRIER IS REQUIRED TO BE FULLY SET BEYOND EACH END OF BRIDGE WORK AREA.

(4) THE LAST CONCRETE BARRIER UNIT, AT EACH END OF BRACED BARRIER LAYOUT, SHALL BE ANCHORED A MINIMUM 18" BELOW THE ROADWAY SURFACE. REQUIRED 1" Ø ANCHOR RODS (A36 STEEL) SHALL BE INSTALLED WITH 5 ANCHORS ON THE TRAFFIC SIDE OF BARRIER AND 4 ON THE CONSTRUCTION SIDE. IF THE END(S) OF THE BRACED CONCRETE BARRIER SYSTEM EXTENDS 50' OR MORE BEYOND LIMITS OF BRIDGE WORK THE LAST BARRIER UNIT DOES NOT REQUIRE ANCHORAGE.

(5) PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL (BRIDGE), ITEM 606.41741, MAY BE INSTALLED WITH A 230' MINIMUM RADIUS. GAPS CREATED BETWEEN STRUCTURAL TUBES AND CONCRETE BARRIER, DURING A RADIAL LAYOUT, SHALL BE SHIMMED WITH 8" x 8" x 1/2" PLATES & FENDER WASHERS TO FIRMLY ATTACH STRUCTURAL TUBING TO BARRIER.

(6) THE CONTRACTOR SHALL FURNISH AND INSTALL APPROVED RETROREFLECTIVE DELINEATORS AT 25-FOOT INTERVALS ALONG TOP AND/OR ONE FOOT DOWN THE SIDE OF PORTABLE CONCRETE BARRIER, SUBSIDIARY TO ITEM 606.41741 (SEE STANDARD NO. DL-1 OF NHDOT STANDARD PLANS FOR ROAD CONSTRUCTION). THE COLOR OF DELINEATORS SHALL, IN ALL INSTANCES, CONFORM TO THE COLOR OF EDGE LINE MARKINGS. DELINEATORS SUPPLEMENT, BUT DO NOT REPLACE, THE NEED FOR RETROREFLECTIVE SOLID EDGE LINE MARKINGS.

MATERIAL NOTES

(1) BARRIERS SHALL BE LIGHT COLORED CLASS AA CONCRETE, WITH MINIMUM COMPRESSIVE STRENGTH OF 4000 psi, AND SHALL HAVE A SMOOTH UNIFORM SURFACE FREE OF DEFECTS AND IRREGULARITIES. CASTING DATE SHALL BE SHOWN ON BARRIER. ALL EXPOSED EDGES OF CONCRETE SHALL BE CHAMFERED 3/4", UNLESS OTHERWISE NOTED.

(2) ALL REINFORCING STEEL SHALL BE AASHTO M31 (ASTM A615) GRADE 60. ALL REINFORCEMENT SHALL HAVE 1 1/2" MINIMUM CLEAR COVER, UNLESS OTHERWISE NOTED.

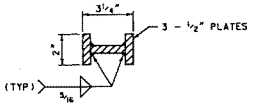
(3) STRUCTURAL STEEL, EXCEPT THE STEEL TUBES, SHALL BE ASTM A36 OR A572. ALL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH SECTION 550.

(4) STEEL TUBES, 6" x 6" x 3/16" & 4" x 4" x 1/2", SHALL BE ASTM A 500 GRADE B OR C. ALL TUBES SHALL BE GALVANIZED IN ACCORDANCE WITH SECTION 550.

(5) ALL STEEL FOR CONNECTION KEY AND TRANSITION KEY ASSEMBLIES SHALL BE GALVANIZED IN ACCORDANCE WITH SECTION 550.

(6) A MINIMUM OF 2 LIFTING DEVICES, EACH WITH THE CAPACITY TO LIFT A MASS OF 6 TONS (MINIMUM), SHALL BE INSTALLED TO EACH BARRIER UNIT. TWENTY FOOT LONG CONCRETE BARRIER UNITS ARE APPROXIMATELY 400 LBS./FT.

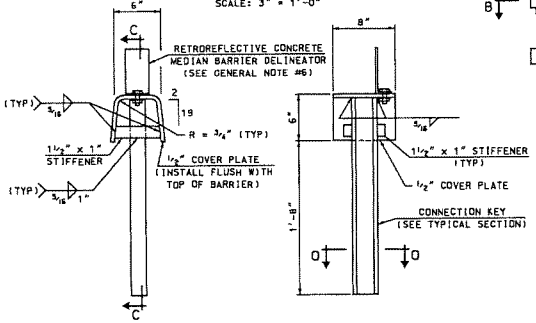
(7) DELINEATORS SHALL BE ATTACHED TO BARRIER USING AN APPROVED ADHESIVE MATERIAL OR AS SHOWN ON THIS SHEET.



SECTION O-O

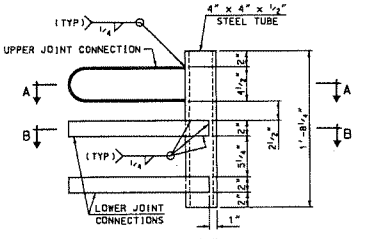
(CONNECTION KEY)

SCALE: 3" = 1'-0"



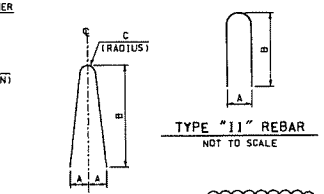
CONNECTION KEY ASSEMBLY DETAILS

SCALE: 1 1/2" = 1'-0"



JOINT ASSEMBLY

SCALE: 1 1/2" = 1'-0"



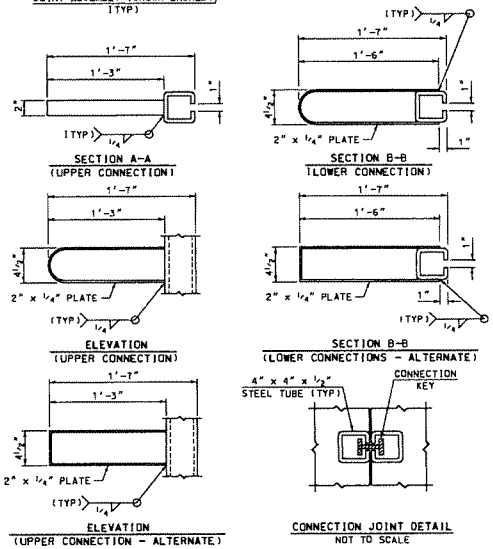
TYPE "1" REBAR

NOT TO SCALE

TYPE "11" REBAR

NOT TO SCALE

NO MODIFICATIONS SHALL BE MADE TO THIS SHEET



JOINT CONNECTION DETAILS

SCALE: 1 1/2" = 1'-0" (EXCEPT AS NOTED)

REINFORCING SCHEDULE (PER 20' BARRIER UNIT)

MARK	SIZE	LENGTH	# PIECES	TYPE	A	B	C	LOCATION
B1	#4	4'-10"	9	1	5"	2'-4"	1"	STIRRUPS
B2	#6	19'-1"	2	—				LONGITUDINAL (TOP)
B3	#6	19'-9"	2	—				LONGITUDINAL (BOTTOM)
B4	#6	1'-2"	2	—				TRANSVERSE (BOTTOM)
B5	#6	6"	2	—				TRANSVERSE (TOP)
B6	#4	2'-9"	9	11	5"	1'-3"		STIRRUPS

SUBDIRECTORY	DGN LOCATOR	SHEET SCALE
English/BARRIER	PCB-BRACED	AS NOTED

STATE OF NEW HAMPSHIRE

DEPARTMENT OF TRANSPORTATION * BUREAU OF BRIDGE DESIGN

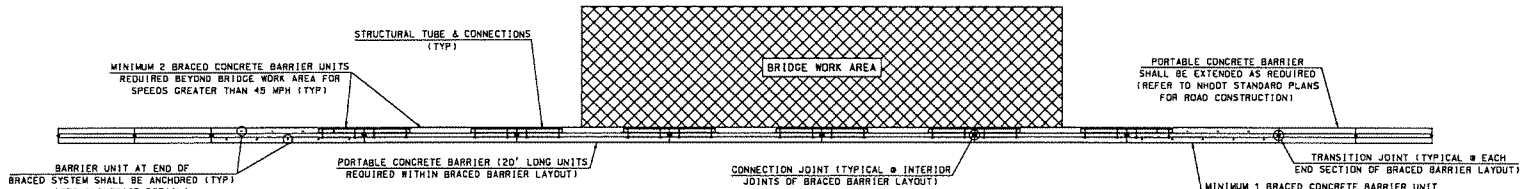
TOWN	BRIDGE NO.	STATE PROJECT
LOCATION		

PORTABLE CONCRETE BARRIER - BRACED (1 OF 2)

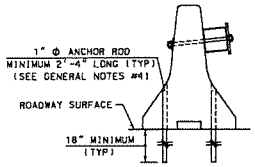
BRIDGE SHEET OF TOTAL SHEETS

REVISIONS AFTER PROJECT	BY	DATE

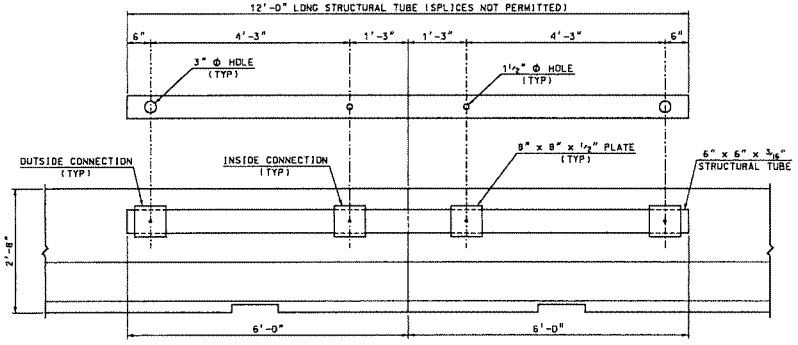
DESIGNED	NHDOT	7/12	CHECKED	ABII	8/12
DRAWN	PJP	8/12	CHECKED	ABII	8/12
QUANTITIES			CHECKED		
ISSUE DATE	8/15/12	FEDERAL PROJECT NO.		SHEET NO.	
REV. DATE	5/15/18				



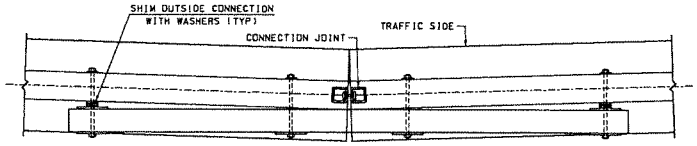
PLAN - BARRIER LAYOUT
NOT TO SCALE



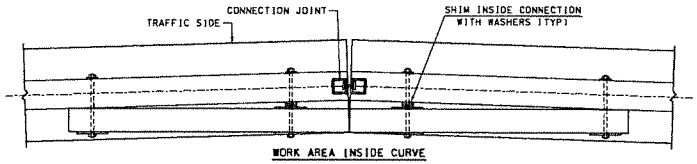
ANCHORAGE DETAIL
SCALE: 3/4" = 1'-0"



STRUCTURAL TUBING DETAILS
SCALE: 3/4" = 1'-0"



WORK AREA OUTSIDE CURVE

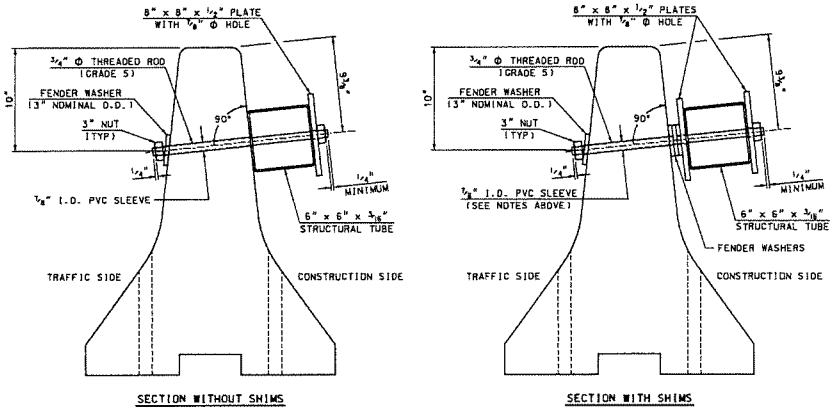


WORK AREA INSIDE CURVE

TUBE CONNECTIONS (CURVED ALIGNMENT)
SCALE: 3/4" = 1'-0"

PVC SLEEVE OPENINGS SHALL BE MODIFIED/DRILLED AS REQUIRED TO PROPERLY ALIGN STRUCTURAL TUBE BRACING UNITS FOR CURVED ALIGNMENTS

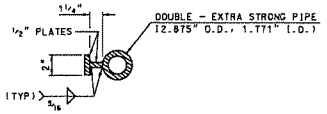
THE PRESENCE OF NORMAL HOLES WHICH HAVE BEEN MODIFIED/DRILLED WILL NOT AFFECT THE REUSE OF CONCRETE BARRIER UNITS



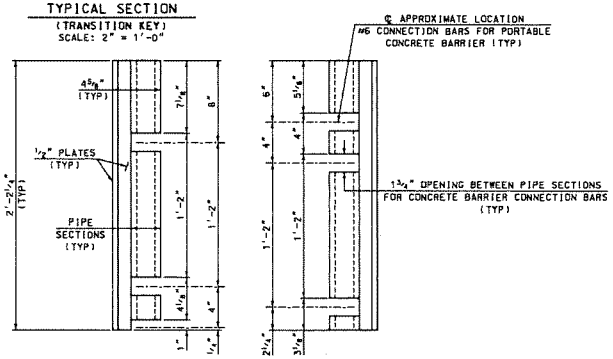
SECTION WITHOUT SHIMS

SECTION WITH SHIMS

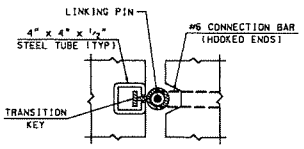
BARRIER & STRUCTURAL TUBING SECTIONS
SCALE: 2" = 1'-0"



TYPICAL SECTION
(TRANSITION KEY)
SCALE: 2" = 1'-0"



TRANSITION KEY ELEVATIONS
SCALE: 2" = 1'-0"



TRANSITION JOINT DETAILS
NOT TO SCALE

NO MODIFICATIONS SHALL BE MADE TO THIS SHEET

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION * BUREAU OF BRIDGE DESIGN

TOWN	BRIDGE NO.	STATE PROJECT
LOCATION		

PORTABLE CONCRETE BARRIER - BRACED (2 OF 2)

BRIDGE SHEET OF / TITLE SHEET

REVISIONS AFTER PROPOSAL	BY	DATE		BY	DATE
	DESIGNED	NHDOT	7/12	CHECKED	ABH 8/12
	DRAWN	DJP	8/12	CHECKED	ABH 8/12
	QUANTITIES			CHECKED	
	ISSUE DATE	8/15/12	FEDERAL PROJECT NO.		SHEET NO. / TOTAL SHEETS
	REV. DATE	5/15/18	----------		

SUBDIRECTORY	.DGN LOCATOR	SHEET SCALE
English/BARRIER	PCB-BRACED	AS NOTED

Bid Tabulation
Construction of the Temporary Bridge Installation - Thornton Ferry Road I Over Beaver Brook
Town of Amherst, NH
11/22/2022 at 02:00 PM EST

				Engineer Estimate		Hansen Bridge LLC		R.M. Piper, Inc.		Cold River Bridges. LLC	
Item Code	Item Description	UofM	Quantity	Unit Price	Extension	Unit Price	Extension	Unit Price	Extension	Unit Price	Extension
201.1	CLEARING AND GRUBBING (F)	A	0.15	$10,000.00	$1,500.00	$10,000.00	$1,500.00	$10,000.00	$1,500.00	$10,000.00	$1,500.00
201.881	INVASIVE SPECIES CONTROL TYPE I	SY	20	$30.00	$600.00	$100.00	$2,000.00	$10.00	$200.00	$25.00	$500.00
201.882	INVASIVE SPECIES CONTROL TYPE II	SY	20	$30.00	$600.00	$100.00	$2,000.00	$10.00	$200.00	$50.00	$1,000.00
202.7	REMOVAL OF GUARDRAIL	LF	245	$5.00	$1,225.00	$20.00	$4,900.00	$5.00	$1,225.00	$3.00	$735.00
203.1	COMMON EXCAVATION	CY	150	$30.00	$4,500.00	$30.00	$4,500.00	$20.00	$3,000.00	$40.00	$6,000.00
203.601	EMBANKMENT-IN-PLACE	CY	15	$30.00	$450.00	$20.00	$300.00	$25.00	$375.00	$100.00	$1,500.00
214	FINE GRADING	U	1	$3,000.00	$3,000.00	$2,000.00	$2,000.00	$5,000.00	$5,000.00	$5,000.00	$5,000.00
304.301	CRUSHED GRAVEL	CY	610	$45.00	$27,450.00	$40.00	$24,400.00	$50.00	$30,500.00	$55.00	$33,550.00
403.11033	HBP- 3/4" WINTER BINDER, MACHINE METHOD	TON	110	$180.00	$19,800.00	$200.00	$22,000.00	$360.00	$39,600.00	$325.00	$35,750.00
403.11043	HBP-1/2" SURFACE MIX, MACHINE METHOD	TON	70	$200.00	$14,000.00	$200.00	$14,000.00	$180.00	$12,600.00	$325.00	$22,750.00
410.22	ASPHALT EMULSION FOR TACK COAT	GAL	25	$10.00	$250.00	$50.00	$1,250.00	$100.00	$2,500.00	$50.00	$1,250.00
417	COLD PLANING BITUMINOUS SURFACES	SY	25	$40.00	$1,000.00	$40.00	$1,000.00	$50.00	$1,250.00	$100.00	$2,500.00
501.191	TEMPORARY BIRDGE (TWO-LANE) LEFT IN PLACE	U	1	$160,000.00	$160,000.00	$131,437.00	$131,437.00	$285,000.00	$285,000.00	$355,000.00	$355,000.00
592.9	MODULAR CONCRETE BLOCK RETAINING WALL	LF	279	$75.00	$20,925.00	$60.00	$16,740.00	$245.00	$68,355.00	$100.00	$27,900.00
606.4171	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL - LEFT IN PLACE	LF	300	$50.00	$15,000.00	$80.00	$24,000.00	$80.00	$24,000.00	$75.00	$22,500.00
606.9522	TEMP. IMPACT ATTENUATION DEVICE (NON-REDIRECTIVE) TEST LEVEL 2	U	4	$5,000.00	$20,000.00	$500.00	$2,000.00	$4,500.00	$18,000.00	$3,000.00	$12,000.00
615.0301	TRAFFIC SIGN TYPE C	SF	18	$100.00	$1,800.00	$10.00	$180.00	$80.00	$1,440.00	$75.00	$1,350.00
615.0601	TRAFFIC SIGN TYPE CC	SF	5	$40.00	$200.00	$10.00	$50.00	$80.00	$400.00	$100.00	$500.00
628.2	SAWED BITUMINOUS PAVEMENT	LF	95	$6.00	$570.00	$3.00	$285.00	$3.00	$285.00	$5.00	$475.00
645.44	TEMPORARY SLOPE MATTING TYPE D (WILDLIFE FRIENDLY)	SY	210	$5.00	$1,050.00	$20.00	$4,200.00	$5.00	$1,050.00	$5.00	$1,050.00
645.512	COMPOST SOCK FOR PERIMETER BERM	LF	735	$5.00	$3,675.00	$10.00	$7,350.00	$8.00	$5,880.00	$5.00	$3,675.00
646.51	TURF ESTABLISHMENT WITH MULCH, TACKIFIERS AND LOAM	SY	210	$5.00	$1,050.00	$2.00	$420.00	$7.50	$1,575.00	$5.00	$1,050.00
692	MOBILIZATION	U	1	$30,200.00	$30,200.00	$20,000.00	$20,000.00	$50,000.00	$50,000.00	$75,000.00	$75,000.00
697.11	INVASIVE SPECIES CONTROL AND MANAGEMENT PLAN	U	1	$3,000.00	$3,000.00	$2,000.00	$2,000.00	$1,200.00	$1,200.00	$1,000.00	$1,000.00
			Alternative A Totals		**$331,845.00**		**$288,512.00**		**$555,135.00**		**$613,535.00**

				Engineer Estimate		Hansen Bridge LLC		R.M. Piper, Inc.		Cold River Bridges. LLC	
Item Code	Item Description	UofM	Quantity	Unit Price	Extension	Unit Price	Extension	Unit Price	Extension	Unit Price	Extension
201.1	CLEARING AND GRUBBING (F)	A	0.1	$10,000.00	$1,000.00	$10,000.00	$1,000.00	$10,000.00	$1,000.00	$10,000.00	$1,000.00
201.881	INVASIVE SPECIES CONTROL TYPE I	SY	20	$30.00	$600.00	$100.00	$2,000.00	$10.00	$200.00	$25.00	$500.00
201.882	INVASIVE SPECIES CONTROL TYPE II	SY	20	$30.00	$600.00	$100.00	$2,000.00	$10.00	$200.00	$50.00	$1,000.00
202.7	REMOVAL OF GUARDRAIL	LF	245	$5.00	$1,225.00	$20.00	$4,900.00	$5.00	$1,225.00	$3.00	$735.00
203.1	COMMON EXCAVATION	CY	150	$30.00	$4,500.00	$30.00	$4,500.00	$20.00	$3,000.00	$40.00	$6,000.00
203.601	EMBANKMENT-IN-PLACE	CY	15	$30.00	$450.00	$20.00	$300.00	$25.00	$375.00	$100.00	$1,500.00
214	FINE GRADING	U	1	$3,000.00	$3,000.00	$2,000.00	$2,000.00	$5,000.00	$5,000.00	$5,000.00	$5,000.00
304.301	CRUSHED GRAVEL	CY	555	$45.00	$24,975.00	$40.00	$22,200.00	$50.00	$27,750.00	$55.00	$30,525.00
403.11033	HBP- 3/4" WINTER BINDER, MACHINE METHOD	TON	90	$180.00	$16,200.00	$200.00	$18,000.00	$360.00	$32,400.00	$350.00	$31,500.00
403.11043	HBP-1/2" SURFACE MIX, MACHINE METHOD	TON	60	$200.00	$12,000.00	$200.00	$12,000.00	$180.00	$10,800.00	$350.00	$21,000.00
410.22	ASPHALT EMULSION FOR TACK COAT	GAL	21	$10.00	$210.00	$50.00	$1,050.00	$100.00	$2,100.00	$50.00	$1,050.00
417	COLD PLANING BITUMINOUS SURFACES	SY	25	$40.00	$1,000.00	$40.00	$1,000.00	$50.00	$1,250.00	$100.00	$2,500.00
501.192	TEMPORARY BIRDGE (ONE-LANE) LEFT IN PLACE	U	1	$120,000.00	$120,000.00	$123,267.00	$123,267.00	$158,500.00	$158,500.00	$276,000.00	$276,000.00
592.9	MODULAR CONCRETE BLOCK RETAINING WALL	LF	93	$75.00	$6,975.00	$60.00	$5,580.00	$260.00	$24,180.00	$100.00	$9,300.00
606.4171	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL - LEFT IN PLACE	LF	140	$50.00	$7,000.00	$80.00	$11,200.00	$70.00	$9,800.00	$75.00	$10,500.00
606.9522	TEMP. IMPACT ATTENUATION DEVICE (NON-REDIRECTIVE) TEST LEVEL 2	U	4	$5,000.00	$20,000.00	$500.00	$2,000.00	$4,500.00	$18,000.00	$3,000.00	$12,000.00
615.0301	TRAFFIC SIGN TYPE C	SF	49	$100.00	$4,900.00	$10.00	$490.00	$80.00	$3,920.00	$75.00	$3,675.00
615.0601	TRAFFIC SIGN TYPE CC	SF	17	$40.00	$680.00	$10.00	$170.00	$80.00	$1,360.00	$100.00	$1,700.00
628.2	SAWED BITUMINOUS PAVEMENT	LF	95	$6.00	$570.00	$3.00	$285.00	$3.00	$285.00	$5.00	$475.00
632.3118	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 18" LINE	LF	22	$40.00	$880.00	$20.00	$440.00	$25.00	$550.00	$100.00	$2,200.00
645.44	TEMPORARY SLOPE MATTING TYPE D (WILDLIFE FRIENDLY)	SY	440	$5.00	$2,200.00	$20.00	$8,800.00	$5.00	$2,200.00	$5.00	$2,200.00
645.512	COMPOST SOCK FOR PERIMETER BERM	LF	745	$5.00	$3,725.00	$10.00	$7,450.00	$8.00	$5,960.00	$5.00	$3,725.00
646.51	TURF ESTABLISHMENT WITH MULCH, TACKIFIERS AND LOAM	SY	440	$5.00	$2,200.00	$2.00	$880.00	$7.50	$3,300.00	$5.00	$2,200.00
692	MOBILIZATION	U	1	$23,800.00	$23,800.00	$20,000.00	$20,000.00	$50,000.00	$50,000.00	$70,000.00	$70,000.00
697.11	INVASIVE SPECIES CONTROL AND MANAGEMENT PLAN	U	1	$3,000.00	$3,000.00	$2,000.00	$2,000.00	$1,200.00	$1,200.00	$1,000.00	$1,000.00
			Alternative B Totals		**$261,690.00**		**$253,512.00**		**$364,555.00**		**$497,285.00**

BID SUBMITTAL CERTIFICATION

BIDDER hereby submits this Bid as set forth above:

Bidder: Hansen Bridge LLC
(typed or printed name of organization)

By: _____
(individual's signature)

Name: Michael Hansen
(typed or printed)

Title: Manager/Owner
(typed or printed)

Date: November 20, 2022
(typed or printed)

If Bidder is a corporation, a partnership, or a joint venture, attach evidence of authority to sign.

Attest: _____
(individual's signature)

Name: Michael Hansen
(typed or printed)

Title: Manager
(typed or printed)

Date: 11/20/22
(typed or printed)

Address for giving notices:
1325 County Road
New London, NH 03257

Bidder's Contact:

Name: Michael Hansen
(typed or printed)

Title: Manager/Owner
(typed or printed)

Phone: 603-491-5295

Email: hansenbridge@tds.net

Address:
1325 County Road
New London, NH 03257

Bidder's Contractor License No.: (if applicable) _____

END OF SECTION

Hansen Bridge LLC

I, Michael C. Hansen, hereby certify that I am duly elected sole director of Hansen Bridge LLC.

I hereby certify the following is a true copy of a vote taken at a meeting of the Board of Directors of the Corporation, duly called and held on the 15th day of July, 2022, at which a quorum of the Board was present and voting.

VOTED:

That Michael C. Hansen as sole director is duly authorized to execute any documents which may be in his judgement desirable or necessary to effect the purpose of this vote.

I hereby certify that said vote has not been amended or repealed and remains in full force and affects as of November 20, 2022, and that Michael C. Hansen is the duly elected sole director of this corporation.

Date: 11/20/22

Attest: Beth Hansen

Secretary: Michael C Hansen

HANSEN BRIDGE

1325 County Road, New London, NH 03257
603-491- 5295
hansenbridge@tds.net

22' x 50' GALVANIZED STEEL STRINGER TEMPORARY BRIDGE ESTIMATE

November 20, 2022

CLIENT:
Town Of Amherst NH
Attn: Town Administrator

RE: Temporary Bridge Construction- Thornton Ferry Road Bridge

Description: Construct precast modular abutments for a 22'-0" x 50'-0" galvanized steel stringer bridge with removable laminated timber deck panels and TL-1 crash rated W Beam bridge rail. Installation per HTA plans and specifications.

Payment Schedule : #1 $144,256.00 deposit prior to construction.
#2 $144,256.00 due after bridge is complete.

BRIDGE ITEMS

Description	Estimated Qty	Unit		
Modular Precast Abutments	1	UNIT		
Laminated Pressure Treated Timber Deck Panels	1	UNIT		
Galvanized Steel Stringer Bridge w/ TL-1 W Beam Rail	1	UNIT		
Erosion Control	1	UNIT		
Paving	1	UNIT		
			TOTAL	$288,512.00

- ☐ Price is based on Hansen Bridge to provide job supervision.
- ☐ Please allow 2-4 weeks for bridge fabrication and galvanizing.
- ☐ NHPE bridge design load rating HL-93 and TL-1 rail calculations by GM2 Engineers
- ☐ All steel beams, clips and diaphragm's to be hot dipped galvanized and delivered in kit form. A325 galvanized bolted connections only.
- ☐ Please allow 4-6 hrs for galvanized bridge superstructure and deck installation. 20 Ton excavator required.
- ☐ **Heaviest pick 5,000 lbs, Total number of trailer loads = 1.**

Acceptance of above contract by owner, _____ Date_____ Signature_____

Thank you,
Hansen Bridge LLC
Michael Hansen

HANSEN BRIDGE

1325 County Road, New London, NH 03257
603-491- 5295
hansenbridge@tds.net

12' x 50' GALVANIZED STEEL STRINGER TEMPORARY BRIDGE ESTIMATE

November 20, 2022

CLIENT:
Town Of Amherst NH
Attn: Town Administrator

RE: Temporary Bridge Construction- Thornton Ferry Road Bridge

Description: Construct precast modular abutments for a 12'-0" x 50'-0" galvanized steel stringer bridge with removable laminated timber deck panels and TL-1 crash rated W Beam bridge rail. Installation per HTA plans and specifications.

Payment Schedule : #1 $126,756.00 deposit prior to construction.
#2 $126,756.00 due after bridge is complete.

BRIDGE ITEMS

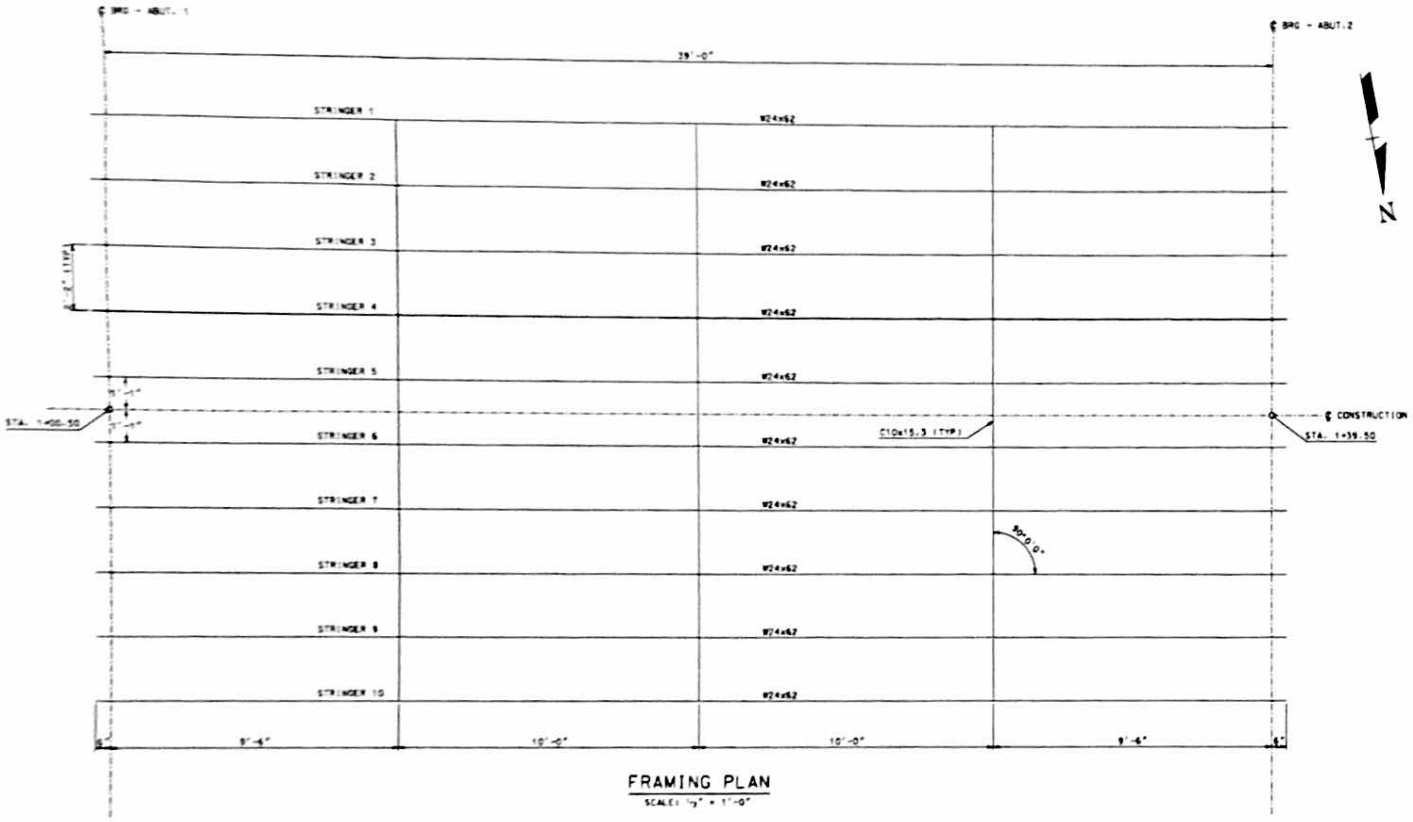
Description	Estimated Qty	Unit		
Modular Precast Abutments	1	UNIT		
Laminated Pressure Treated Timber Deck Panels	1	UNIT		
Galvanized Steel Stringer Bridge w/ TL-1 W Beam Rail	1	UNIT		
Erosion Control	1	UNIT		
Paving	1	UNIT		
			TOTAL	$253,512.00

- ☐ Price is based on Hansen Bridge to provide job supervision.
- ☐ Please allow 2-4 weeks for bridge fabrication and galvanizing.
- ☐ NHPE bridge design load rating HL-93 and TL-1 rail calculations by GM2 Engineers
- ☐ All steel beams, clips and diaphragm's to be hot dipped galvanized and delivered in kit form. A325 galvanized bolted connections only.
- ☐ Please allow 2-4 hrs for galvanized bridge superstructure and deck installation. 20 Ton excavator required.
- ☐ **Heaviest pick 5,000 lbs, Total number of trailer loads = 1.**

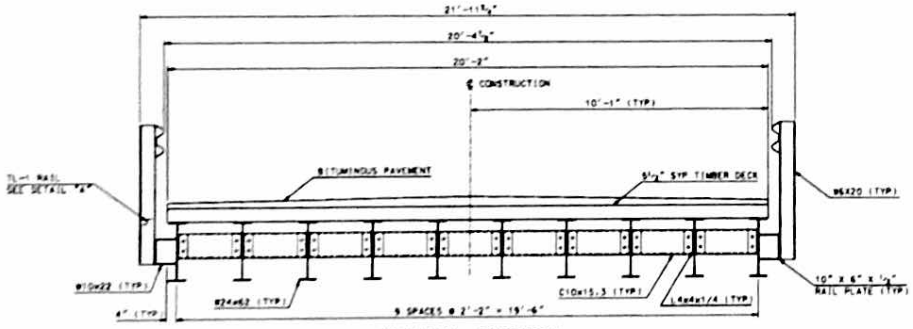
Acceptance of above contract by owner, _____ Date_____ Signature_____

Thank you,
Hansen Bridge LLC
Michael Hansen

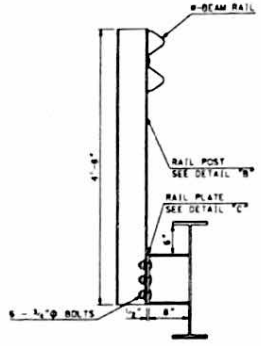
SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE



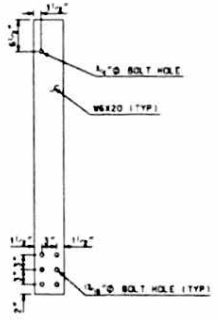
FRAMING PLAN
SCALE: 1/2" = 1'-0"



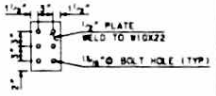
TYPICAL SECTION
SCALE: 1/2" = 1'-0"



DETAIL "A"
SCALE: 1" = 1'-0"



DETAIL "B"
SCALE: 1" = 1'-0"



DETAIL "C"
SCALE: 1" = 1'-0"

PRELIMINARY PLANS SUBJECT TO CHANGE
DATE 3/18/2022

GM2

SUBSTRUCTURE	ITEM LOCATOR	SHEET SCALE
BRIDCADD	696.113 Superstructure	AS NOTED

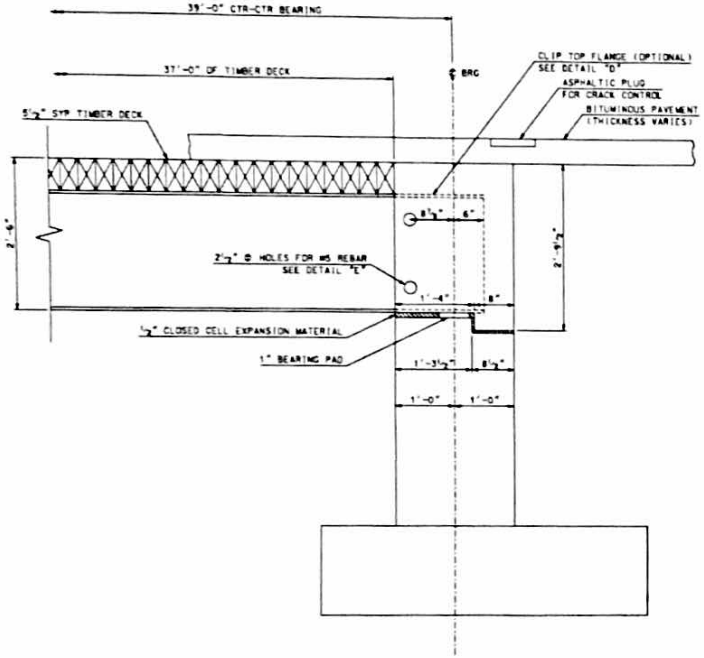
TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

TOWN: ALEXANDRIA, NH — BRIDGE NO.: 096/112 — STATE PROJECT: N/A
LOCATION: SALEM VALLEY ROAD OVER ABOOS BROOK

SUPERSTRUCTURE DETAILS

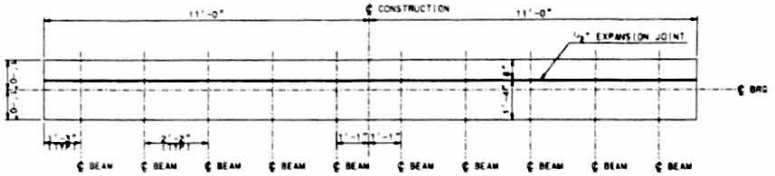
	BY	DATE		BY	DATE
DESIGNED	BJL	05/22	CHECKED	TPL	05/22
DRAWN	BJL	05/22	CHECKED	TPL	05/22
QUANTITIES	–	–	CHECKED	–	–
ISSUE DATE			FINAL REVIEW		
REV DATE			–		

SHEET NO.: 1 — TOTAL SHEETS: 3



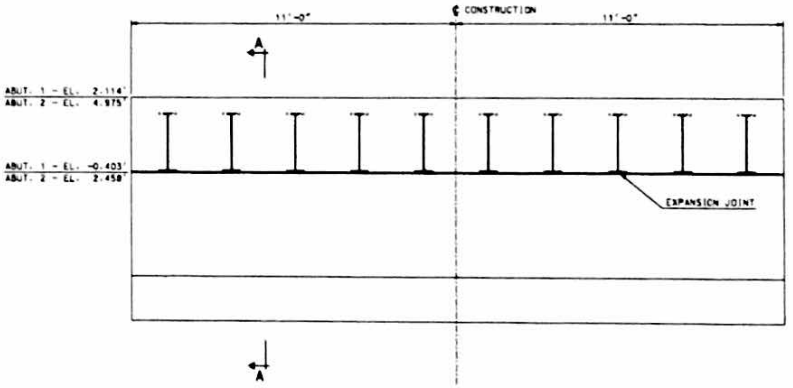
SECTION A-A

SCALE: 1" = 1'-0"



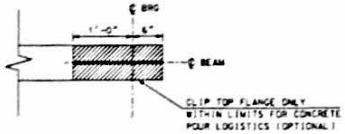
ABUTMENT PLAN

SCALE: ½" = 1'-0"



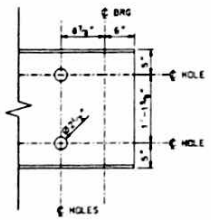
ABUTMENT ELEVATION

SCALE: ½" = 1'-0"



DETAIL "D"

SCALE: 1" = 1'-0"



DETAIL "E"

SCALE: 1" = 1'-0"

NOTE: ELEVATIONS ARE TAKEN RELATIVE TO THE ELEVATION AT STA. 0+00

SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE

PRELIMINARY PLANS
SUBJECT TO CHANGE
DATE 5/10/2022

GM2

TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

TOWN ALEXANDRIA, NH BRIDGE NO. 096/112 STATE PROJECT N/A

LOCATION SKEW VALLEY ROAD OVER BROCK BROOK

SUBSTRUCTURE DETAILS

REVISIONS AFTER PROPOSAL	BY	DATE		BY	DATE
	DESIGNED	BJL	05/22	CHECKED	TPL 05/22
	DRAWN	BJL	05/22	CHECKED	TPL 05/22
	QUANTITIES	–	–	CHECKED	–
	ISSUE DATE			FINAL REVIEW NO.	SHEET NO.
	REV. DATE			–	2

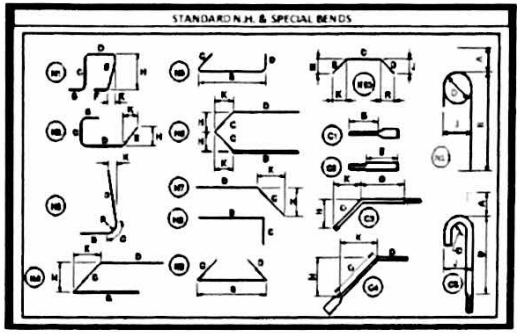
SHEET SERIES: 2 of 3
PLAN NUMBER: –
TOTAL SHEETS: 3

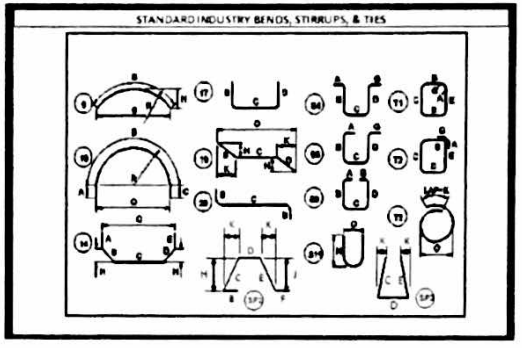
SUBDIRECTORY: BRINCADD 096-112 | DGN LOCATOR: Some Imagery | SHEET SCALE: DETAILS AS NOTED

ALEXANDRIA - Br. No. 096112

Mark	No.	Length	# Pieces	Type	A	B	C	D	E	F	G	H	J	K	R	O	NOTE
A1	5	21'-7"	16	-													EC
A2	5	4'-4"	62	17		1'-4"	2'-0"	1'-0"									EC
A3	5	3'-11"	62	17		1'-4"	2'-3"	0'-4"									EC

ITEM #	DESCRIPTION	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	TOTAL
544	REINFORCING STEEL												
544.01	MECH. CONNECTOR												
544.2	EPOXY COATED			894									
544.21	EPOXY MECH. CON.												894

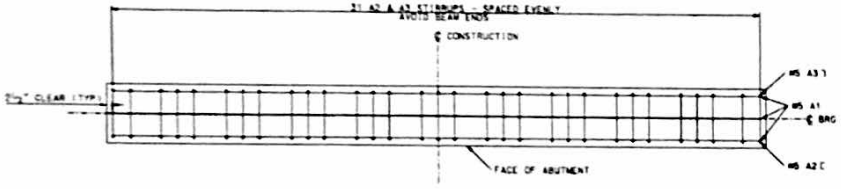




NOTES:

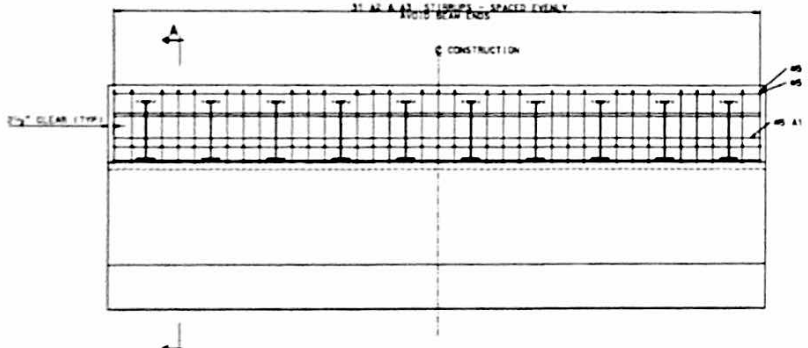
1. FIGURES IN CIRCLE SHOW TYPE OF BEND.
2. UNLESS OTHERWISE DESIGNATED, ALL BAR REINFORCEMENT FOR CONCRETE IN SIZES UP TO AND INCLUDING NO. 18 SHALL CONFORM TO THE REQUIREMENTS OF THE "SPECIFICATIONS FOR DEFORMED BILLET - STEEL BARS FOR CONCRETE REINFORCEMENT," AASHTO M 31-94 (ASTM A615)
3. FOR TYPICAL BENDING DETAILS, RECOMMENDED PIN DIAMETER "D" OF BENDS AND HOOKS AND OTHER STANDARD PRACTICE REFER TO THE CURRENT CONCRETE REINFORCING STEEL INSTITUTE "MANUAL OF STANDARD PRACTICE."
4. BARS WHICH REQUIRE MORE ACCURATE BENDING THAN STANDARD PRACTICES SHOULD HAVE LIMITS INDICATED.
5. ALL DIMENSIONS ARE OUT TO OUT OF BAR EXCEPT "A" AND "B" ON STANDARD 180° AND 135° HOOKS.
6. "J" DIMENSION ON 180° HOOKS TO BE SHOWN ONLY WHEN NECESSARY TO RESTRICT HOOK SIZE. OTHERWISE STANDARD HOOKS ARE TO BE USED.
7. "H" DIMENSION ON STIRRUPS TO BE SHOWN ONLY WHEN NECESSARY TO MAINTAIN CLEARANCES.
8. WHERE SLOPE DIFFERS FROM 45° DIMENSIONS "H" AND "K" MUST BE SHOWN.

△ DENOTES BARS TO BE CUT IN FIELD, AS REQUIRED.
△ DENOTES BARS TO BE BENT IN FIELD.



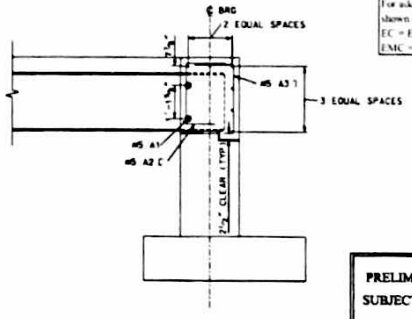
ABUTMENT REINFORCEMENT - PLAN

SCALE: ½" = 1'-0"



ABUTMENT REINFORCEMENT - ELEVATION

SCALE: ½" = 1'-0"



SECTION A-A

SCALE: ½" = 1'-0"

NOTE:
D = finished inside bend d of hook
See 1997 CRSI Manual p 6-3&4.
For additional data on standard bar bends not shown on this sheet see 1997 CRSI Manual p 6-3&4.
EC = EPOXY COATED
EMC = EPOXY MECHANICAL CONNECTOR

RECOMMENDED END HOOKS, ALL GRADES (in.)				
BAR SIZE	180° HOOKS			90° HOOKS
	D(2)	A or G	J	A or G
#3	2 1/4	5	3	6
#4	3	6	4	8
#5	3 3/4	7	5	10
#6	4 1/2	8	6	12
#7	5 1/4	10	7	14
#8	6	11	8	16
#9	9 1/2	15	11 3/4	19
#10	10 3/4	17	13 1/4	22
#11	12	19	14 3/4	24
#14	18 1/4	27	21 3/4	31

ASTM STANDARD REINFORCING BARS (in.)			
BAR SIZE	WEIGHT (lb/ft)	NOM. DIMEN.	
		DIA. (in.)	AREA (in²)
#3	0.376	0.375	0.11
#4	0.668	0.500	0.20
#5	1.043	0.625	0.31
#6	1.502	0.750	0.44
#7	2.044	0.875	0.60
#8	2.670	1.000	0.79
#9	3.400	1.128	1.00
#10	4.303	1.270	1.27
#11	5.313	1.410	1.56
#14	7.650	1.693	2.25
#18	13.600	2.257	4.00

STIRRUP & TIE HOOK DIMENSIONS, GRADES 40-50-60 ksi (in.)				
BAR SIZE	D(2)	90° HOOKS	135° HOOKS	
		A or G	A or G	H (approx.)
#3	1 1/2	4	4	2 1/2
#4	2	4 1/2	4 1/2	3
#5	2 1/2	6	5 1/2	3 3/4
#6	4 1/2	12	7 3/4	4 1/4
#7	5 1/4	14	9	5 1/4
#8	6	16	10 1/4	6

PRELIMINARY PLANS
SUBJECT TO CHANGE
DATE 5/10/2022

SAMPLE DESIGN ONLY:
GALVANIZED STEEL STRINGER BRIDGE

SUBDIRECTORY	DGN LOCATOR	SHEET SCALE
BRDCADD 096112 Semi Integral REBAR		1" = 1'-0"

TOWN OF ALEXANDRIA
PUBLIC WORKS DEPARTMENT

TOWN: ALEXANDRIA, NH | BRIDGE NO. 096112 | STATE PROJECT: N/A

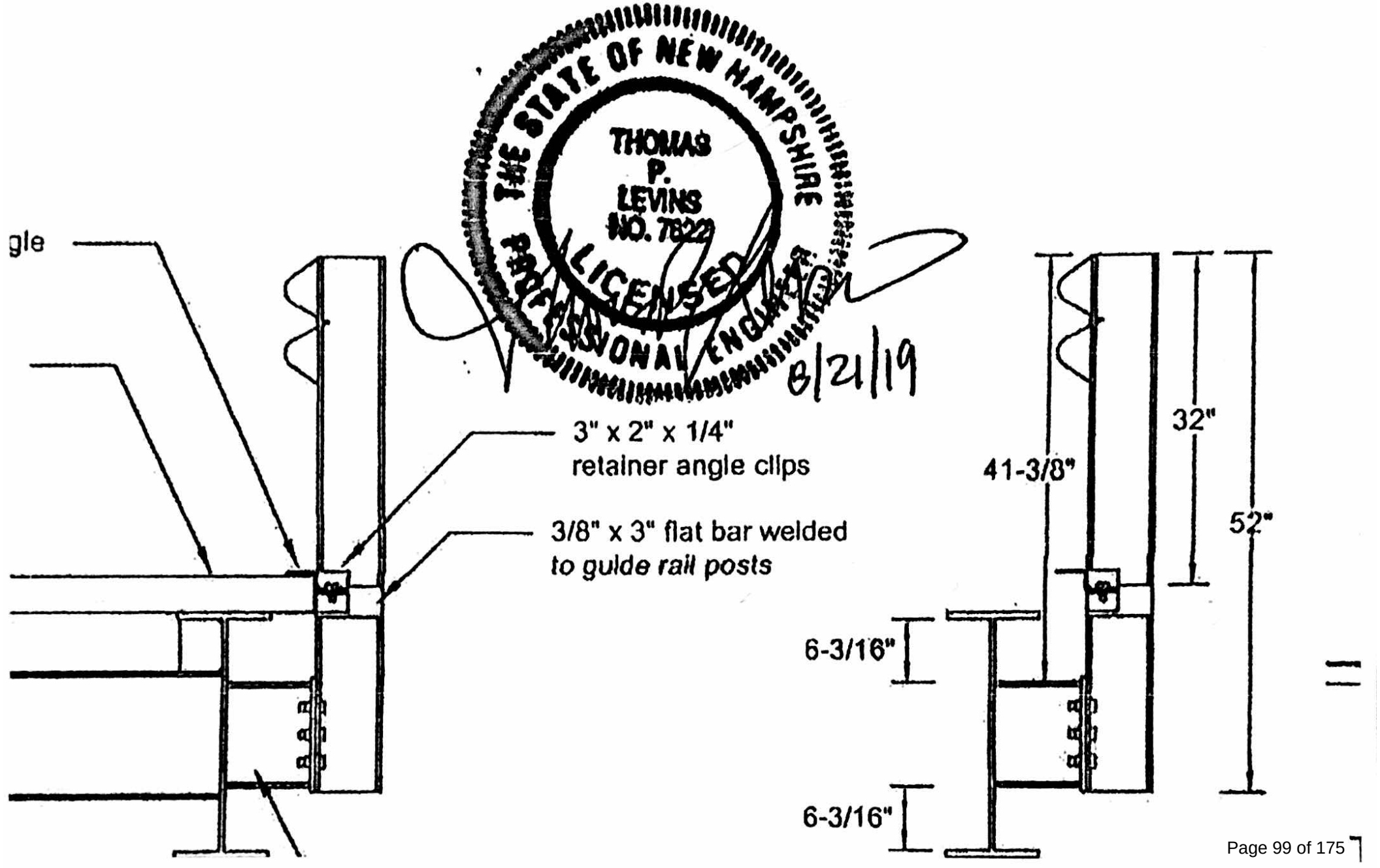
LOCATION: MEDW VALLEY ROAD OVER BROCK BROOK

ABUTMENT BACKWALL REINFORCEMENT

	BY	DATE		BY	DATE
DESIGNED	BJL	05/22	CHECKED	TPL	05/22
DRAWN	BJL	05/22	CHECKED	TPL	05/22
QUANTITIES	-	-	CHECKED	-	-
ISSUE DATE			ORIGINAL PROJECT NO.	SHEET NO.	TOTAL SHEETS
REV. DATE			-	3	3

SHEET NUMBER: 3 of 3

FILE NUMBER: -


THE STATE OF NEW HAMPSHIRE
THOMAS
P.
LEVINS
NO. 7822
LICENSED
PROFESSIONAL ENGINEER
8/21/19
gle
3" x 2" x 1/4"
retainer angle clips
3/8" x 3" flat bar welded
to gulde rail posts
41-3/8"
32"
52"
6-3/16"
6-3/16"

SAMPLE CALCULATIONS

Form 4
N.H. D.O.T.

TOWN: SWANZEY, NH

BRIDGE CAPACITY SUMMARY

BRIDGE NUMBER: ONE LANE Temporary Bridge

DESIGN LOAD: HL-93 *DESIGN METHOD:* LOAD & RESISTANCE FACTOR *RATED BY:* TEM *DATE:* 9/4/2019

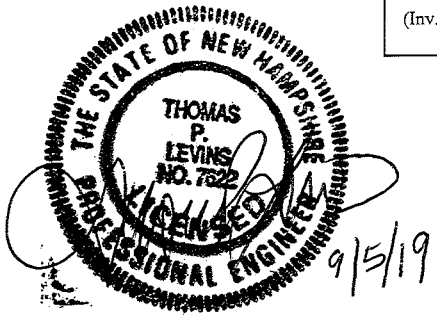
RATING METHOD: LRFR *PLAN FILE:* --- *CHECK BY:* TPL *DATE:* 9/4/2019

ROUTE: RABBIT HOLLOW ROAD *OVER:* PERRY BROOK

RATED MEMBER	LONGITUD. EFFECTIVE SPAN LENGTH	REQUIRED CAPACITY (HS Tons)			AVAILABLE CAPACITY (HS Tons)					
		CURRENT LEGAL LOADS	CERTIFIED VEHICLES		MULTIPLE LANES LOADED			SINGLE LANES LOADED		
			SINGLE UNIT	MULTIPLE UNIT	INVENTORY	OPERATING	POSTING	INVENTORY	OPERATING	POSTING
TIMBER DECK & CENTER PANEL										
POS. MOMENT	1'-0"	HS 14.0	HS 15.4	HS 15.4	HS	HS	HS	HS 16.5	HS 21.4	HS 20.2
SHEAR	1'-0"	HS 14.0	HS 15.4	HS 15.4	HS	HS	HS	HS 19.3	HS 25.0	HS 23.6
INTERIOR GIRDER										
POS. MOMENT	47'-0"	HS 20.4	HS 25.4	HS 22.8	HS	HS	HS	HS 29.0	HS 37.6	HS 35.4
SHEAR	47'-0"	HS 19.8	HS 23.1	HS 22.6	HS	HS	HS	HS 135.1	HS 175.1	HS 165.1
EXTERIOR GIRDER										
POS. MOMENT	47'-0"	HS 20.4	HS 25.4	HS 22.8	HS	HS	HS	HS 51.4	HS 66.8	HS 63.0
SHEAR	47'-0"	HS 19.8	HS 23.1	HS 22.6	HS	HS	HS	HS 239.1	HS 309.9	HS 292.2

RECOMMENDED POSTING: NO POSTING REQUIRED

Rating Method		English Tons		Metric Tons
(Op.) 63.	LRFR	64. (Op.)	38.6	35.0
(Inv.) 65.	LRFR	66. (Inv.)	29.7	27.0



9/5/19

REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
MIN	-	Minnesota LRFD Bridge Design Manual (2016)
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)
AWC/NDS	-	American Wood Council NDS Supplement

Unit Definition:

$$kcf := 1000\ pcf$$

Unit weight of hard-wood: $\gamma_{DCHW} := 0.060\ kcf$ LRFD TABLE 3.5.1-1

Depth of Deck: $d_{deck} := 3.5\ in$ PLANS

Width of Decking: $b_{decking} := 6\ in$ PLANS

Weight of deck: $wt_{deck} := \gamma_{DCHW} \cdot d_{deck} \cdot b_{decking} = 0.0088\ \frac{kip}{ft}$ AISC

Spacing of girders $S_{girder} := 26.5\ in$ PLANS

Dead Load Bending Moment $M_{deck} := \frac{0.8\ wt_{deck} \cdot S_{girder}^{2}}{8} = 0.0043\ kip \cdot ft$

Weight of HL-93 Truck Wheel Loads $P_{wheel} := 4\ kip$ (Front Axle Wheel)

Note: Girder Spacing is such that rear axle wheel will always be partially supported by a girder

Moment Diagram (from truck):



Live Load Moment per wheel (from diagram) $M_{truck} := 1.7\ kip \cdot ft$

Wheel load distribution on deck plank (per foot) $E_s := \frac{b_{decking}}{10\ in} = 0.60$ LRFD 4.6.2.1.3

Maximum Number of lanes	$N_{lanes} := \text{round}\left(\frac{16.416\ ft}{12\ ft}\right) = 1.0$	
Multiple presence factor (for Number of Lanes = 1)	$m := 1.20$	LRFD TABLE 3.6.1.1.2-1
Live Load Bending Moment	$M_{LLmax} := m \cdot M_{truck} \cdot E_s = 1.22\ kip \cdot ft$	
Factored Moment:	$M_u := 1.25 \cdot M_{deck} + 1.75 \cdot M_{LLmax} = 2.15\ kip \cdot ft$	
	$\phi_{flexure} := 0.85$	LRFD 8.5.2.2
Required Nominal Capacity	$M_{n\_reqd} := \frac{M_u}{\phi_{flexure}} = 2.53\ kip \cdot ft$	
Format Conversion Factor	$C_{KF} := \frac{2.5}{\phi_{flexure}} = 2.94$	LRFD 8.4.4.2
Wet Service Factor	$C_M := 0.85$	LRFD Table 8.4.4.3-1
Size Factor	$C_F := 1.3$	LRFD 8.4.4.4
Flat-Use Factor	$C_{fu} := 1.10$	LRFD 8.4.4.6-1
Incising Factor	$C_i := 0.80$	LRFD 8.4.4.7
Deck Factor for Plank Decks	$C_d := 1.10$	LRFD Table 8.4.4.8-2
Time Effect Factor	$C_\lambda := 0.8$	LRFD Table 8.4.4.9-1
Bending Strength	$F_{bo} := 1.20\ ksi$	AWC/NDS
	$F_b := F_{bo} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_d \cdot C_\lambda = 3.02\ ksi$	
Required Depth of Rectangular Section	$d_{req} := \sqrt{\frac{6 \cdot M_{n\_reqd}}{b_{decking} \cdot F_b \cdot 1}} = 3.17\ in$	LRFD 8.6.2-1

Is depth given from plans greater than required depth?

$$\text{if}\left(d_{deck} > d_{req}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Nominal Capacity of section given in plans:

$$M_n := F_b \cdot \frac{b_{decking} \cdot d_{deck}^{\,2}}{6} = 3.08\ kip \cdot ft$$

Deck Flexure Rating Factor:

$$RF_{flexure} := \frac{\phi_{flexure} \cdot M_n - 1.25 \cdot M_{deck}}{1.75 \cdot M_{LLmax}} = 1.22$$

W24x76 girder flange

$$b_f := 8.99\ in$$

1/2 of the girder flange

$$b_{length} := 0.5 \cdot b_f = 4.5\ in$$

Distance to the left part of distributed load from left girder

$$b_{lefttruck} := b_{length} + d_{deck} = 8.00\ in$$

Distance to the right part of distributed load from middle girder

$$b_{righttruck} := \left(b_{lefttruck} + 20\ in\right) - S_{girder} = 1.50\ in$$

Distributed load from rear wheel

$$P_{truck} := \frac{16\ kip}{20\ in} = 9.60\ \frac{kip}{ft}$$

Shear Diagram (from truck):



Maximum Shear from Truck (with multiple presence factor and distribution factor):

$$V_{truck} := m \cdot 4\ kip \cdot E_s = 2.88\ kip$$

Shear from deck load

$$V_{deck} := \frac{wt_{deck} \cdot S_{girder}}{2} = 0.010\ kip$$

$$\phi_{shear} := 0.75$$ LRFD 8.5.2.2

$$F_{vo} := 0.220\ ksi$$ AWC/NDS

Format Conversion Factor (for shear)

$$C_{KF\_shear} := \frac{2.5}{\phi_{shear}} = 3.33$$

LRFD 8.4.4.2

Wet Service Factor (for shear)

$$C_{M\_shear} := 0.97$$

LRFD Table 8.4.4.3-1

$$F_v := F_{vo} \cdot C_{KF\_shear} \cdot C_{M\_shear} \cdot C_i \cdot C_\lambda = 0.455 \ ksi$$

LRFD 8.4.4.1-2

Nominal shear resistance:

$$V_n := \frac{F_v \cdot b_{decking} \cdot d_{deck}}{1.5} = 6.37 \ kip$$

LRFD 8.7-2

Deck Shear Rating Factor:

$$RF_{shear} := \frac{\phi_{shear} \cdot V_n - 1.25 \cdot V_{deck}}{1.75 \cdot V_{truck}} = 0.946$$

REFERENCES:

LRFD - AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS - ADM Welding and Fabrication Plans
AISC - AISC Manual of Steel Construction, 15th ed. (2017)

Unit Definition:

$$kcf := 1000\ pcf$$

Unit weight of steel: $\gamma_{DCSTEEL} := 0.490\ kcf$ LRFD TABLE 3.5.1-1

Depth of Flat Bar $d_{flatbar} := 3.5\ in$ PLANS

Thickness of Flat Bar $t_{flatbar} := 0.25\ in$ PLANS

Spacing of flat bars $S_{flatbars} := 6\ in$ PLANS

Weight of top panel: $wt_{toppanel} := \gamma_{DCSTEEL} \cdot \frac{3}{16}\ in \cdot S_{flatbars}$

Length of flat bars $L_{flatbar} := 24.75\ in$ PLANS

Dead Load Bending Moment $M_{deck} := \frac{wt_{toppanel} \cdot L_{flatbar}^{2}}{8} = 0.002\ kip \cdot ft$

Weight of HL-93 Truck Wheel Loads $P_{wheel} := 4\ kip$ (Front Axle Wheel)

Note: Girder Spacing is such that rear axle wheel will always be partially supported by a girder

Wheel load distribution on middle section $E_s := \frac{S_{flatbars}}{10\ in} = 0.60$ LRFD 4.6.2.1.3

Maximum Number of lanes $N_{lanes} := \text{round}\left(\frac{16.416\ ft}{12\ ft}\right) = 1.0$

Multiple presence factor (for Number of Lanes = 1) $m := 1.20$ LRFD TABLE 3.6.1.1.2-1

Live Load Bending Moment $M_{LLmax} := E_s \cdot m \cdot \frac{0.8\ P_{wheel} \cdot L_{flatbar}}{4} = 1.19\ kip \cdot ft$

Factored Moment: $M_u := 1.25 \cdot M_{deck} + 1.75 \cdot M_{LLmax} = 2.08\ kip \cdot ft$

$\phi_{flexure} := 1.00$ LRFD 6.5.4.2

Yield Strength of Flat Bars $F_y := 50\ ksi$

Plastic Moment Capacity of flat bar (rectangular section) $M_p := \frac{t_{flatbar} \cdot d_{flatbar}^{\ 2}}{4} \cdot F_y = 3.19\ kip \cdot ft$ LRFD 6.12.2.2.7

Moment of Inertia $I_{xx} := \frac{t_{flatbar} \cdot d_{flatbar}^{\ 3}}{12} = 0.89\ in^4$

Center of gravity $y_{cog} := \frac{d_{flatbar}}{2} = 1.75\ in$

Elastic Section Modulus $S_{xx} := \frac{I_{xx}}{y_{cog}} = 0.51\ in^3$

Modulus of Elasticity of Steel $E_{steel} := 29000\ ksi$

Moment Gradient Modifier determined as specified in Article A6.3.3 (when M2=0, Cb = 1.0) $C_b := 1.0$ LRFD A6.3.3

$$M_n := min\left(\frac{1.9 \cdot E_{steel} \cdot 1.0}{\frac{L_{flatbar} \cdot d_{flatbar}}{t_{flatbar}^{\ 2}}} \cdot S_{xx}, M_p\right) = 1.69\ kip \cdot ft$$ LRFD 6.12.2.2.7-3

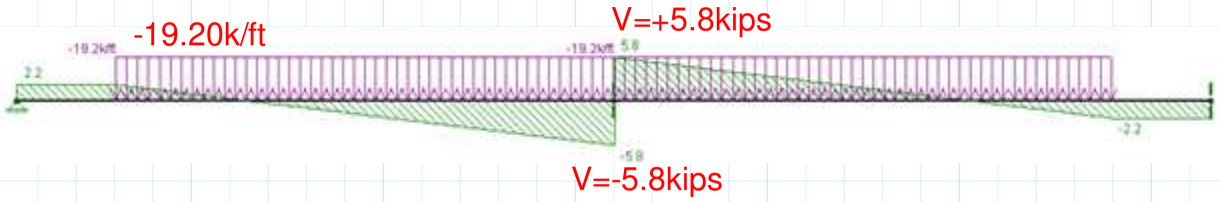
Middle Section of Deck Flexure Operating Rating Factor:

$$RF_{flexure} := \frac{\phi_{flexure} \cdot M_n - 1.25 \cdot M_{deck}}{1.35 \cdot M_{LLmax}} = 1.05$$

Distributed load from rear wheel

$$P_{truck} := \frac{16\ kip}{10\ in} = 19.20\ \frac{kip}{ft}$$

Shear Diagram (from truck):



Maximum Shear from Truck (with multiple presence factor and distribution factor):

$$V_{truck} := m \cdot (5.8\ kip + 5.8\ kip) \cdot E_s = 8.35\ kip$$

Shear from deck load

$$V_{deck} := \frac{wt_{toppanel} \cdot S_{flatbars}}{2} = 0.001\ kip$$

$$\phi_{shear} := 1.0$$ LRFD 6.5.4.2

Gross area of the connection element subject to shear

$$A_{vg} := d_{flatbar} \cdot t_{flatbar} = 0.88\ in^2$$

Nominal shear resistance:

$$R_r := 0.58 \cdot F_y \cdot A_{vg} = 25.38\ kip$$ LRFD 6.13.5.3

Deck Shear Inventory Rating Factor:

$$RF_{shear} := \frac{R_r - 1.25 \cdot V_{deck}}{1.75 \cdot V_{truck}} = 1.736$$

REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)

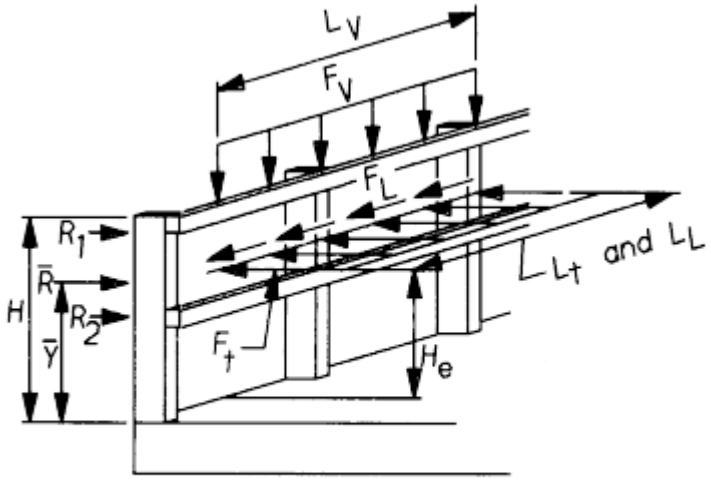


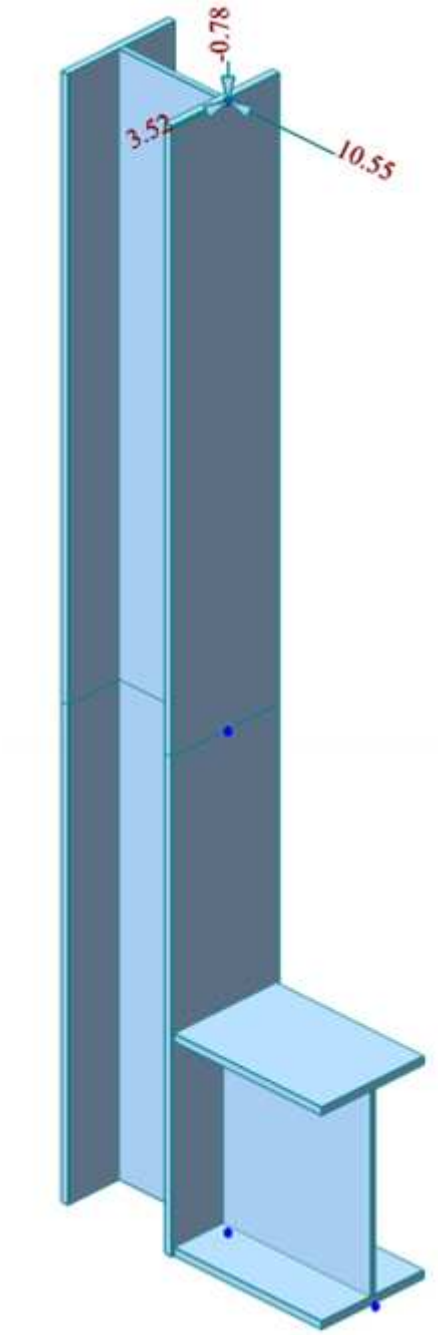
Figure A13.2-1—Metal Bridge Railing Design Forces, Vertical Location, and Horizontal Distribution Length

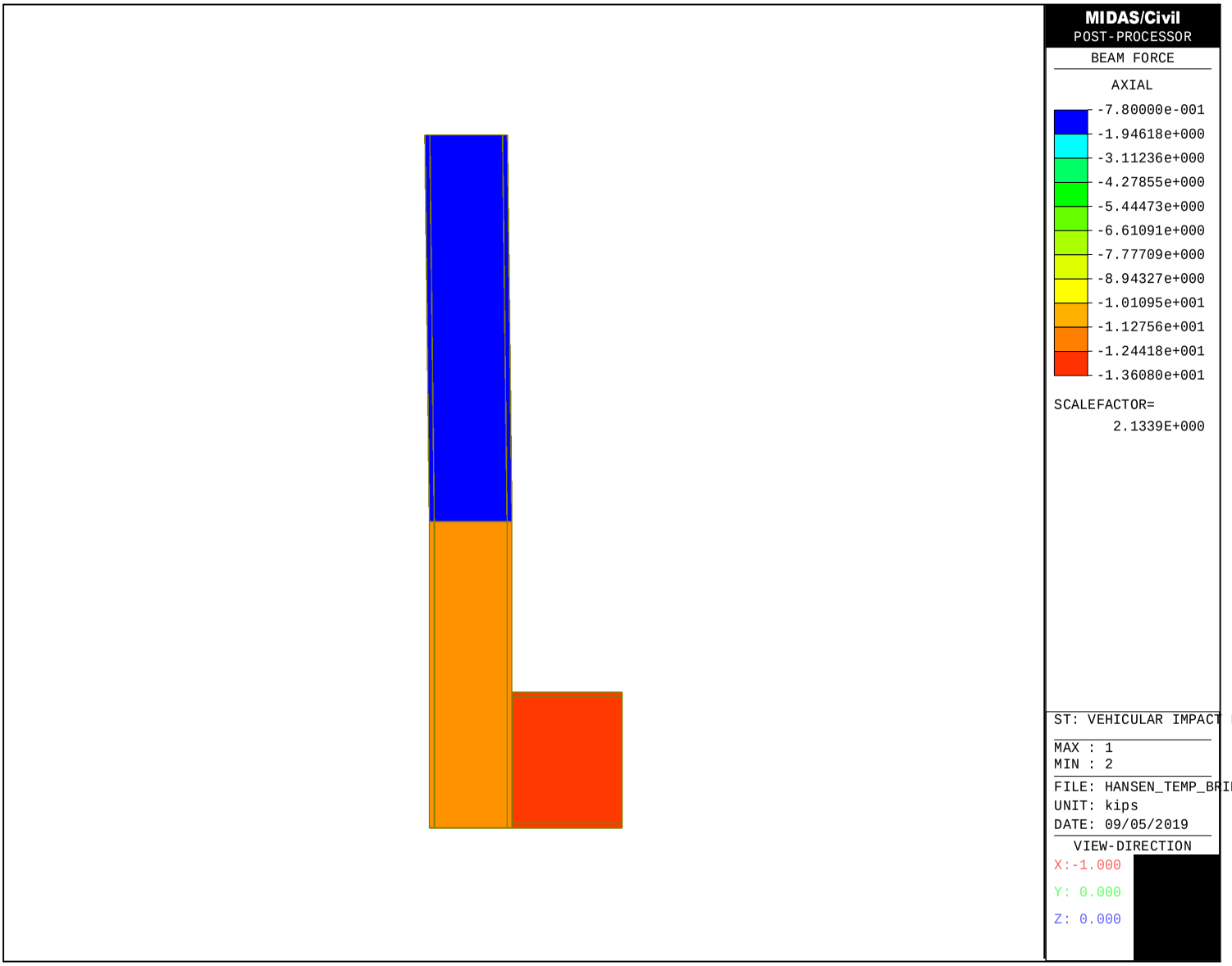
Table A13.2-1—Design Forces for Traffic Railings

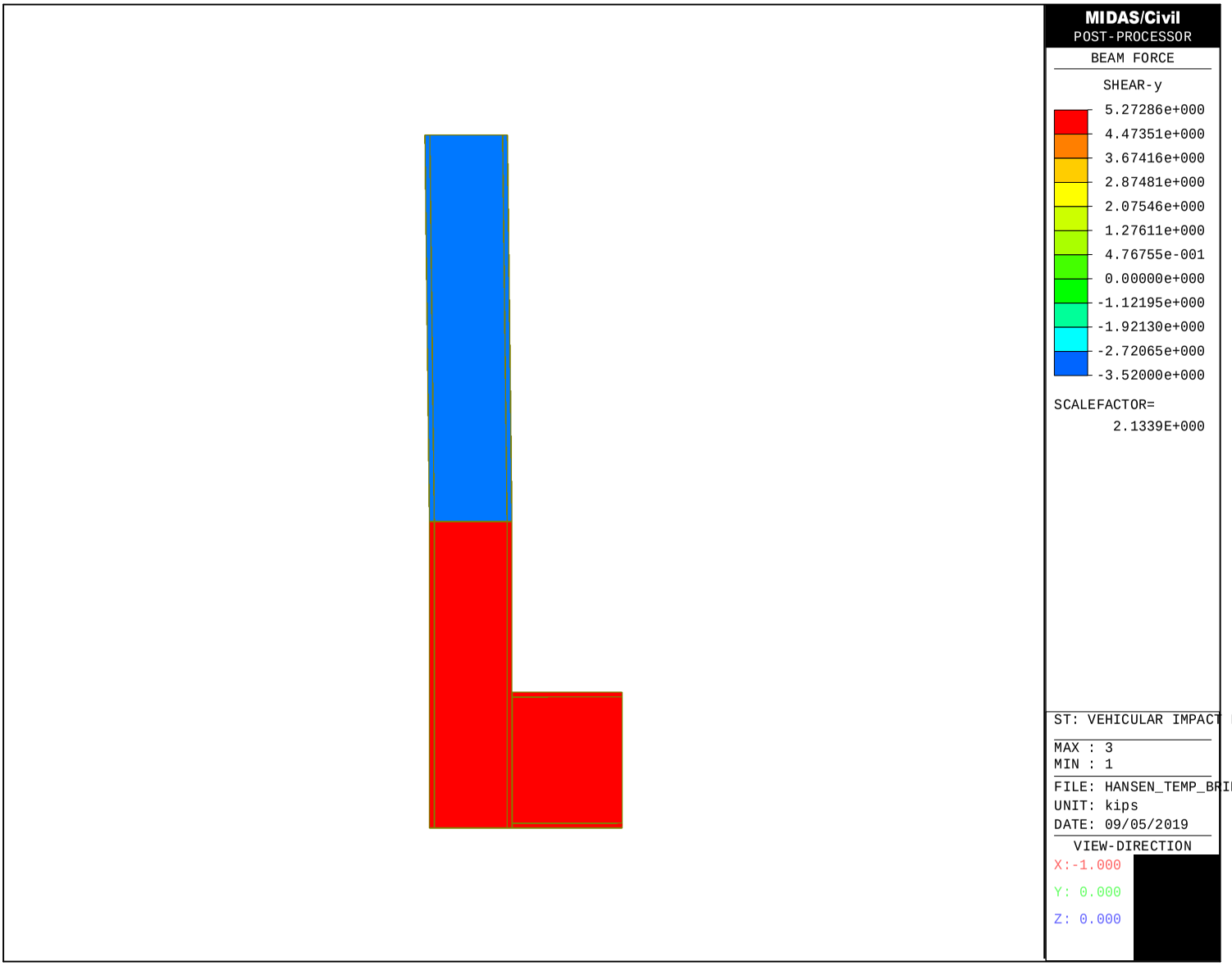
Design Forces and Designations	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips) Down	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum H Height of Rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0

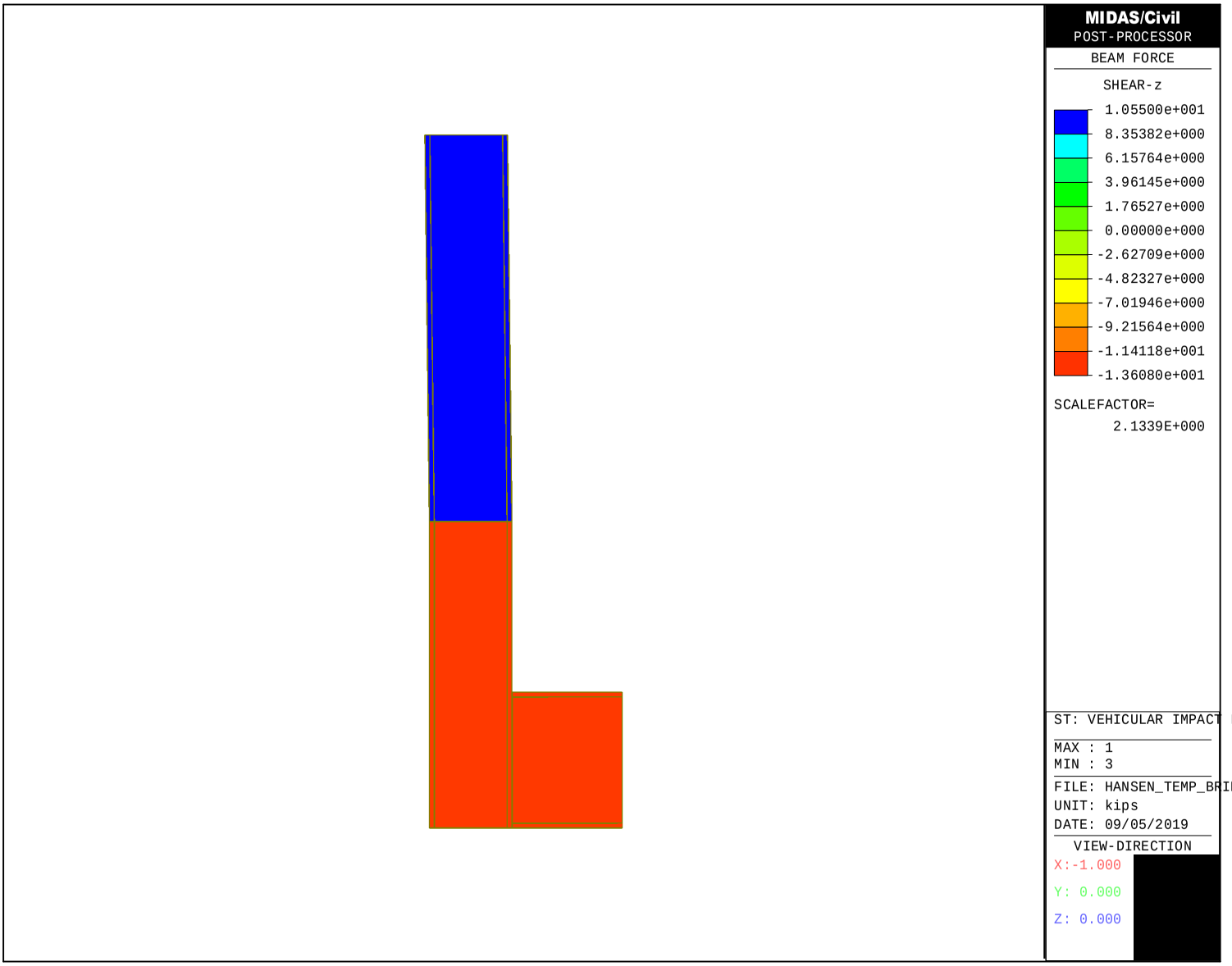
Transverse Force	$F_t := 13.5\ kip$	LRFD TABLE A13.2-1
Longitudinal Force	$F_L := 4.5\ kip$	LRFD TABLE A13.2-1
Vertical Force	$F_v := 4.5\ kip$	LRFD TABLE A13.2-1
Length of Significant contact in Transverse Direction	$L_t := 4\ ft$	LRFD TABLE A13.2-1
Length of Significant contact in Longitudinal Direction	$L_L := 4\ ft$	LRFD TABLE A13.2-1
Length of Significant contact in Vertical Direction	$L_v := 18\ ft$	LRFD TABLE A13.2-1
Transverse Force per unit length	$F_{tunit} := \frac{F_t}{L_t} = 3.38\ \frac{kip}{ft}$	
Longitudinal Force per unit length	$F_{Lunit} := \frac{F_L}{L_L} = 1.13\ \frac{kip}{ft}$	
Vertical Force per unit length	$F_{vunit} := \frac{F_v}{L_v} = 0.25\ \frac{kip}{ft}$	
Spacing of posts	$S_{post} := 37.5\ in$	
Transverse Force applied at top of railing post	$F_{transverse} := F_{tunit} \cdot S_{post} = 10.55\ kip$	
Longitudinal Force applied at top of railing post	$F_{longitudinal} := F_{Lunit} \cdot S_{post} = 3.52\ kip$	
Transverse Force applied at top of railing post	$F_{vertical} := F_{vunit} \cdot S_{post} = 0.78\ kip$	

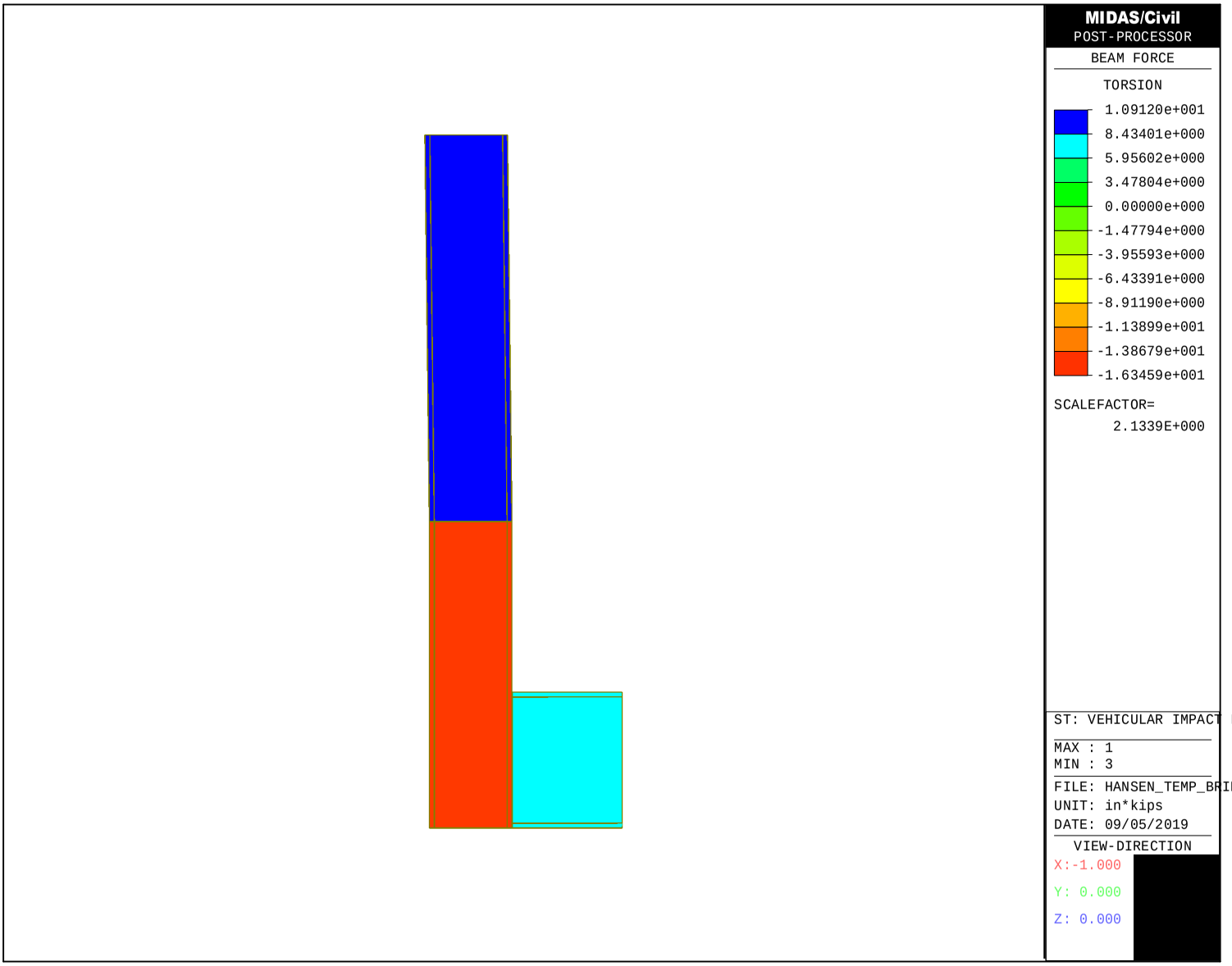
Loading condition for railing subject to vehicular impact force (for TL-1 Railing)

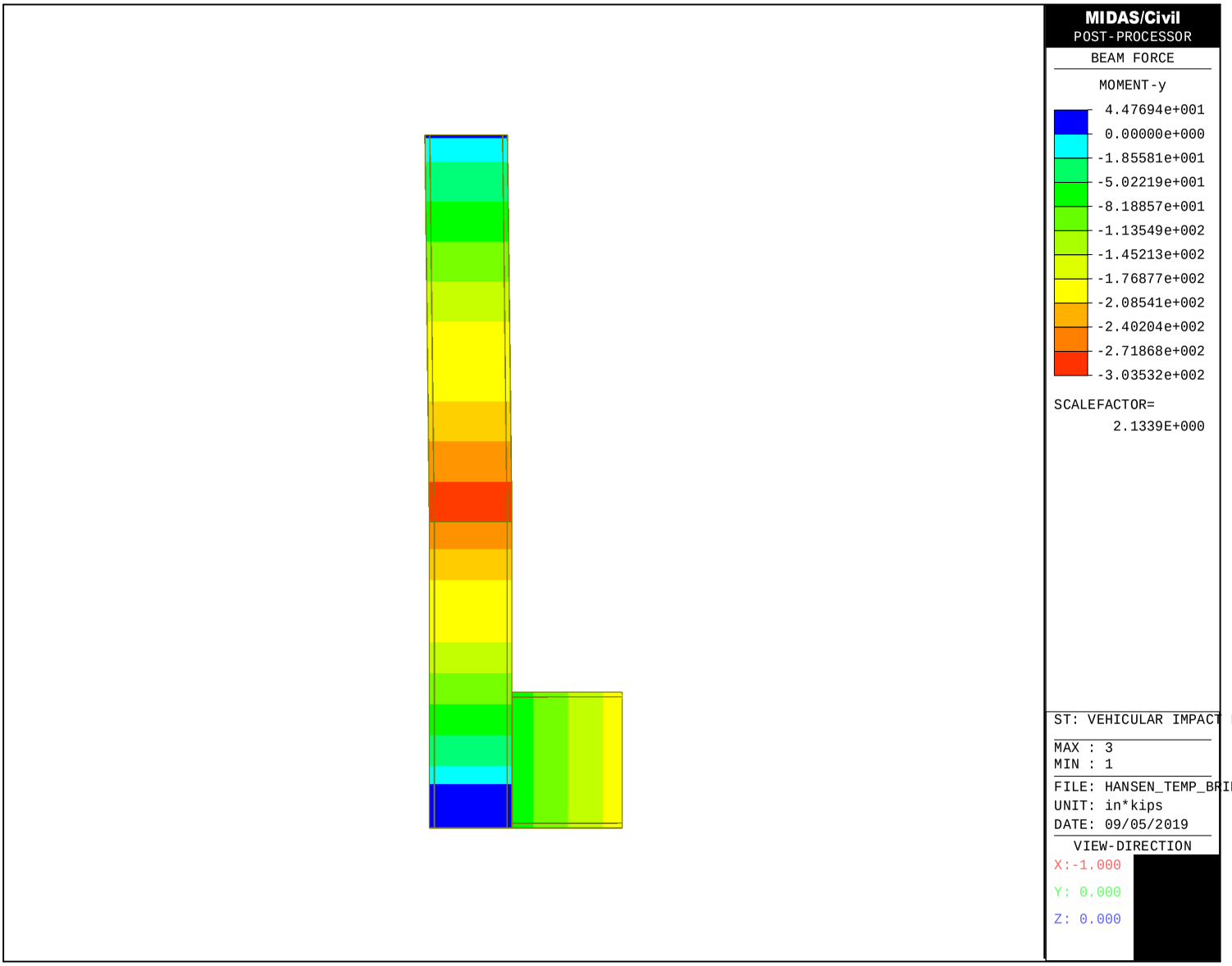


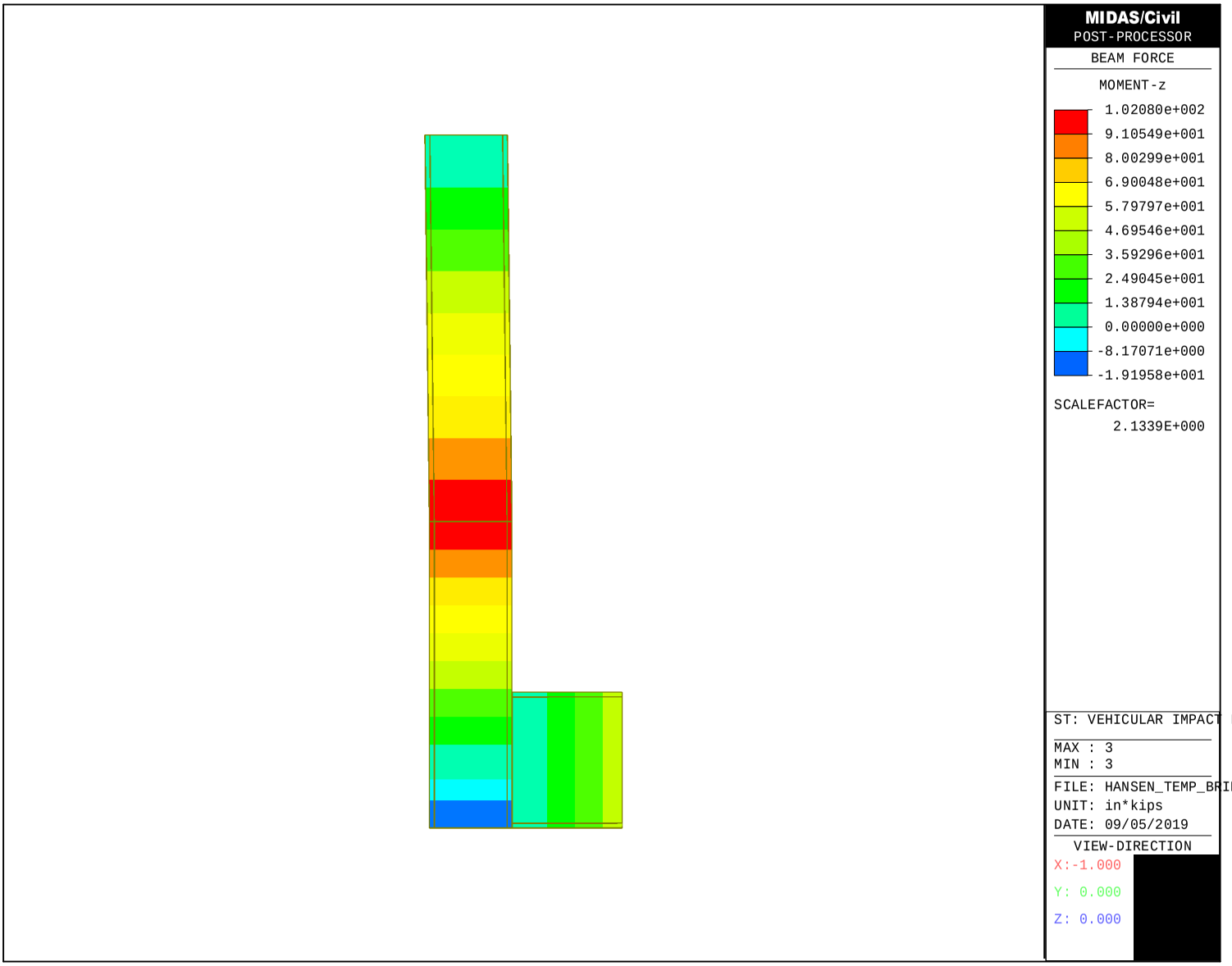

MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
AXIAL
-7.80000e-001
-1.94618e+000
-3.11236e+000
-4.27855e+000
-5.44473e+000
-6.61091e+000
-7.77709e+000
-8.94327e+000
-1.01095e+001
-1.12756e+001
-1.24418e+001
-1.36080e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 2
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
SHEAR-y
5.27286e+000
4.47351e+000
3.67416e+000
2.87481e+000
2.07546e+000
1.27611e+000
4.76755e-001
0.00000e+000
-1.12195e+000
-1.92130e+000
-2.72065e+000
-3.52000e+000
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 1
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
SHEAR-z
1.05500e+001
8.35382e+000
6.15764e+000
3.96145e+000
1.76527e+000
0.00000e+000
-2.62709e+000
-4.82327e+000
-7.01946e+000
-9.21564e+000
-1.14118e+001
-1.36080e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
TORSION
1.09120e+001
8.43401e+000
5.95602e+000
3.47804e+000
0.00000e+000
-1.47794e+000
-3.95593e+000
-6.43391e+000
-8.91190e+000
-1.13899e+001
-1.38679e+001
-1.63459e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 1
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
MOMENT-y
4.47694e+001
0.00000e+000
-1.85581e+001
-5.02219e+001
-8.18857e+001
-1.13549e+002
-1.45213e+002
-1.76877e+002
-2.08541e+002
-2.40204e+002
-2.71868e+002
-3.03532e+002
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 1
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000


MIDAS/Civil
POST-PROCESSOR
BEAM FORCE
MOMENT-z
1.02080e+002
9.10549e+001
8.00299e+001
6.90048e+001
5.79797e+001
4.69546e+001
3.59296e+001
2.49045e+001
1.38794e+001
0.00000e+000
-8.17071e+000
-1.91958e+001
SCALEFACTOR=
2.1339E+000
ST: VEHICULAR IMPACT LOA
MAX : 3
MIN : 3
FILE: HANSEN_TEMP_BRIDGE_RA
UNIT: in*kips
DATE: 09/05/2019
VIEW-DIRECTION
X:-1.000
Y: 0.000
Z: 0.000

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                Version 8.5.6
================================================================================


       +============================================================+
       |  MIDAS(Modeling, Integrated Design & Analysis Software)    |
       |  MIDAS/Civil - Design & checking system for windows        |
       +============================================================+
       |  Steel Member Applicable Code Checking                     |
       |  Based On  AASHTO-LRFD12, AASHTO-LRFD02, AASHTO-LFD96,     |
       |            AASHTO-ASD96, AISC-LRFD2K, AISC-LRFD93,         |
       |            AISC-ASD89, Eurocode3-2:05, BS5950-90,          |
       |            IS:800-2007, IS:800-1984                        |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                          (c)SINCE 1989     |
       +============================================================+
       |  MIDAS Information Technology Co.,Ltd.         (MIDAS IT)  |
       |  MIDAS IT Design Development Team                          |
       +============================================================+
       |            HomePage : www.MidasUser.com                    |
       +============================================================+
       |  MIDAS/Civil  Version 8.5.6                                |
       +============================================================+


    *. DEFINITION OF LOAD COMBINATIONS WITH SCALING UP FACTORS.
--------------------------------------------------------------------------------
     LCB  C    Loadcase Name(Factor) + Loadcase Name(Factor) + Loadcase Name(Factor)
--------------------------------------------------------------------------------
       1  1   Vehicular Impact Loa(-1.000) +   Self-Weight(-1.000)
--------------------------------------------------------------------------------
```

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--------------------------------------------------------------------------------
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      *. PROJECT     :
      *. MEMBER NO   =      1,  ELEMENT TYPE = Beam
      *. LOADCOMB NO =      1,  MATERIAL NO  =     1,  SECTION NO =     1
      *. UNIT SYSTEM : kips, in

      *. SECTION PROPERTIES : Designation = W6X20
         Shape     = I - Section. (Rolled)
         Depth     =      6.200,  Top F Width =      6.020,  Bot.F Width =      6.020
         Web Thick =      0.260,  Top F Thick =      0.365,  Bot.F Thick =      0.365

         Area = 5.87000e+000,  Asy  = 2.92973e+000,  Asz  = 1.61200e+000
         Ybar = 3.01000e+000,  Zbar = 3.10000e+000,  Qyb  = 2.83963e+001,  Qzb  = 4.53005e+000
         Syy  = 1.34000e+001,  Szz  = 4.41000e+000,  Zyy  = 1.50000e+001,  Zzz  = 6.72000e+000
         Iyy  = 4.14000e+001,  Izz  = 1.33000e+001,  Iyz  = 0.00000e+000
         ry   = 2.66000e+000,  rz   = 1.50000e+000
         J    = 2.40000e-001,  Cwp  = 1.12967e+002

      *. DESIGN PARAMETERS FOR STRENGTH EVALUATION :
         Ly   = 2.90000e+001,  Lz   = 2.90000e+001,  Lb   = 2.90000e+001
         Ky   = 1.00000e+000,  Kz   = 1.00000e+000

      *. MATERIAL PROPERTIES :
         Fy   = 5.00000e+001,  Es   = 2.90000e+004,  MATERIAL NAME  = A709-50W


================================================================================
   [[[*]]]   COMPUTE MOMENT MAGNIFICATION FACTORS AND MAGNIFIED MOMENTS.
================================================================================

      ( ). Compute moment magnification factors(B1y,B1z).
           -. If tension or bending member.
           -. Assumed   B1y  =  1.00
           -. Assumed   B1z  =  1.00

      ( ). Magnification factors for sidesway moments(B2y,B2z).
           -. B2y   =  1.00 (Default value)
           -. B2z   =  1.00 (Default value)

      ( ). Given factored axial forces and moments at <J>.
           ------------------------------------------------------------
            Load Case             Pu                 My                 Mz
           ------------------------------------------------------------
            DL                 0.05              0.00               0.00
            LL                 0.78            303.53            -102.08
            DL+LL              0.83            303.53            -102.08
            WL or EL           0.00              0.00               0.00
           ------------------------------------------------------------
            DL+LL+WL(EL)       0.83            303.53            -102.08
           ------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Compute magnified moments.
           -. Muy   =  B1y*My(DL+LL) + B2y*My(WL(EL)) =      303.53 in-kips.
           -. Muz   =  B1z*Mz(DL+LL) + B2z*Mz(WL(EL)) =     -102.08 in-kips.

================================================================================
   [[[*]]]   CHECK AXIAL STRENGTH.
================================================================================

      ( ). Check slenderness ratio of axial tension member (l/r).
           [ AASHTO-LRFD12 Specification 6.8.4 ]
           -. l/r  =    19.3  <   200.0  --->  O.K.

      ( ). Calculate axial tensile strength (phiPn).
           [ AASHTO-LRFD12 Specification 6.8.2.1-1 ]
           -. Resistance factor for tension  :  phi = 0.95
           -. phiPn  =  phi*Fy*Ag  =        278.82 kips.

      ( ). Check ratio of axial strength (Pu/phiPn).
                Pu                0.83
           -. -------  =  --------------  =  0.003  <  1.000  --->  O.K.
               phiPn             278.82

================================================================================
   [[[*]]]   CHECK SECTION PROPORTION LIMITS
================================================================================

      ( ). Check Web Proportions (AASHTO-LRFD12 Eq.6.10.2.1.1-1)

           -. Limit Value  = 150
           -. D/tw  =   21.04  <  Limit Value  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-1)

           -. bf/2tf  =    8.25  <     12.00  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-2)

           -. bf  >  D/6 =    0.91  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-3)

           -. tf > 1.1*tw =    0.29  --->  O.K.

      ( ). Check general proportions
                      Izc               6.6359
           -. 0.1  <  ------  =  ----------------  =  1.000  <  10    --->  O.K.
                      Izt               6.6359
```

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--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      => Flextural Resistance is Checked by AASHTO-LRFD12 Specification

================================================================================
   [[[*]]]   CHECK SECTION SLENDERNESS
================================================================================

      ( ). Check Flange ratio
                Izc               6.6359
           -. ------  =  ----------------  =  1.000  >  0.3
                Izt               6.6359

      ( ). Check web slenderness
           -. Limit Value  =  5.7*SQRT[E/Fyc]  =  137.27

           -. 2Dc/tw  =   21.04  <  Limit Value

           -. Fyc  =50ksi  <  70.0 ksi

      => Section is Checking by AASHTO-LRFD12 Appendix 6

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MAJOR AXIS.
================================================================================

      ( ). Check Web Compactness Length
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Dcp   =    2.74 in.
           -. Dc    =    2.74 in.
           -. Mp    =  739.77 in-kips.
           -. My    =  667.74 in-kips.
           -. Rh    =    1.00(homogenous section)
           -. Lambda_rw = 5.7*SQRT[Es/Fyc] =   137.27

                                          SQRT[Es/Fyc]
           -. Lambda_pw(Dcp)_cal = ------------------------ =    93.23
                                   (0.54*Mp/(Rh*My)-0.09)^2

           -. Lambda_pw(Dcp)_lim = Lambda_rw * (Dcp/Dc)    =   137.27
           -. Lambda_pw(Dcp)     = min[Lambda_pw(Dcp)_cal, Lambda_pw(Dcp)_lim ] =    93.23
           -. Lambda_w(Dcp)      = 2*Dcp/tw =    21.04

           -. Lambda_w(Dcp) < Lambda_pw(Dcp)  --->  compact web

      ( ). Web Plastification Factor
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Myc  = Fyc*Iyy/yc =  667.74 in-kips.
           -. Myt  = Fyt*Iyy/yt =  667.74 in-kips.
           -. Rpc  =  Mp/Myc  =    1.11
           -. Rpt  =  Mp/Myt  =    1.11
```

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 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check compression-flange compactness
           -. Lambda_pf  =  0.38*SQRT[E/Fyc]  =    9.15
           -. Lambda_f = bf/2tf  =    8.25  <  Lambda_pf  --->  Compact.

      ( ). Compute nominal flexural strength of the Compression Flange (Mnc).
           [ AASHTO-LRFD12 Specification A6.3.2]
           -. Rpc =    1.11
           -. Myc =  667.74 in-kips.
           -. Mnc_FLB  =  Rpc*Myc  =  739.77 in-kips.

      ( ). Effective Radius of Gyration for Lateral Torsional Buckling (rt)
           -. depth of the web in compression(Dc)  =    2.74 in.
                                 bfc
           -. rt = ------------------------------------ =     1.65 in.
                      [    (    1    Dc*tw   ) ]
                 SQRT[ 12*(1 + - * -------- ) ]
                      [    (    3   bfc*tfc  ) ]

      ( ). Unbraced Length(Lb) Check
           -. Lb  =   29.00 in.
           -. Lp  = 1.0*rt*SQRT(Es/Fyc) =   39.76 in.

           -. Lb < Lp  --->  compact unbraced length

      ( ). Compute Lateral Torsional Buckling Resistance of Compression Flange (Mnc_LTB).
           [ AASHTO-LRFD12 Specification A6.3.3]
           -. Rpc  =    1.11
           -. Myc  =  667.74 in-kips.
           -. Mnc_LTB =  Rpc*Myc  =  739.77 in-kips.

      ( ). Compute nominal flexural strength of Compression Flange (Mnc)
           -. Mnc  =  min[Mnc_FLB, Mnc_LTB] =  739.77 in-kips.

      ( ). Compute nominal flexural strength of Tension Flange (Mnt).
           [ AASHTO-LRFD12 Specification A6.4]
           -. Rpt  =    1.11
           -. Myt  =        667.74 in-kips.
           -. Mnt  =  Rpt*Myt  =        739.77 in-kips.

           -. Mn  =  min(Mnc, Mnt)  =        739.77 in-kips.

      ( ). Compute flexural strength about major axis (phiMny).
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMny  =  phi*Mny  =        739.77 in-kips.
```

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 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check ratio of flexural strength (Muy/phiMny).
                Muy               303.53
           -. --------  =  ---------------  =  0.410  <  1.000  --->  O.K.
               phiMny             739.77

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MINOR AXIS.
================================================================================

      ( ). Slenderness ratio for flange (lamda_f)
           [ AASHTO-LRFD12 Specification 6.12.2.2]
           -. lamda_f   = bf/(2*tf) =     8.25
           -. lamda_pf  = 0.038*SQRT(Es/Fyf) =     0.92
           -. lamda_rf  = 0.83*SQRT(Es/Fyf) =    19.99

           -. lamda_pf < lamda_f < lamda_rf  --->  noncompact flange

      ( ). Compute nominal flexural strength of the flange (Mn).
           [ AASHTO-LRFD12 Specification 6.12.2.2]

                   [    (     Szz )   ( Lambda_f  - Lambda_pf )]
           -. Mn = [1 - (1 - -----) * (-----------------------)]*Fy*Zzz =   257.87 in-kips.
                   [    (     Zzz )   (    0.45*SQRT[Es/Fy]   )]

      ( ). Compute flexural strength about minor axis (phiMnz).
           -. Mnz  =        257.87 in-kips.
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMnz  =  phi*Mnz  =        257.87 in-kips.

      ( ). Check ratio of flexural strength (Muz/phiMnz).
                Muz               102.08
           -. --------  =  ---------------  =  0.396  <  1.000  --->  O.K.
               phiMnz             257.87

================================================================================
   [[[*]]]   CHECK INTERACTION OF COMBINED STRENGTH.
================================================================================

      ( ). Check interaction ratio of combined strength.
           [ AASHTO-LRFD12 Specification 6.9.2.2 ]
           -. Pu/phiPn  <  0.20  --->  Formula(6.9.2.2-2)
                            Pu       [    Muy        Muz   ]
           -. ComRat  =  --------- + [ -------- + -------- ]
                          2*phiPn    [  phiMny     phiMnz  ]
                      =  0.001 + [  0.410  +  0.396 ]
                      =  0.808  <  1.000  --->  O.K.

================================================================================
   [[[*]]]   CHECK SHEAR STRENGTH.
================================================================================
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


     ( ). Calculate nominal shear strength in local-y direction (Vny).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asy  =         84.96 kips.
          -. Vny =  C*Vp  =         84.96 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuy =          3.52 kips.
                Vuy                3.52
          -. --------  =  --------------  =  0.041  <  1.000  --->  O.K.
              phiVny              84.96

     ( ). Calculate nominal shear strength in local-z direction (Vnz).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asz  =         46.75 kips.
          -. Vnz =  C*Vp  =         46.75 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuz =         10.55 kips.
                Vuz                10.55
          -. --------  =  --------------  =  0.226  <  1.000  --->  O.K.
              phiVnz              46.75
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                Version 8.5.6
================================================================================


       +============================================================+
       |  MIDAS(Modeling, Integrated Design & Analysis Software)    |
       |  MIDAS/Civil - Design & checking system for windows        |
       +============================================================+
       |  Steel Member Applicable Code Checking                     |
       |  Based On  AASHTO-LRFD12, AASHTO-LRFD02, AASHTO-LFD96,     |
       |            AASHTO-ASD96, AISC-LRFD2K, AISC-LRFD93,         |
       |            AISC-ASD89, Eurocode3-2:05, BS5950-90,          |
       |            IS:800-2007, IS:800-1984                        |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
       |                                                            |
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       |  MIDAS Information Technology Co.,Ltd.         (MIDAS IT)  |
       |  MIDAS IT Design Development Team                          |
       +============================================================+
       |            HomePage : www.MidasUser.com                    |
       +============================================================+
       |  MIDAS/Civil  Version 8.5.6                                |
       +============================================================+


    *. DEFINITION OF LOAD COMBINATIONS WITH SCALING UP FACTORS.
--------------------------------------------------------------------------------
     LCB  C    Loadcase Name(Factor) + Loadcase Name(Factor) + Loadcase Name(Factor)
--------------------------------------------------------------------------------
       1  1   Vehicular Impact Loa(-1.000) +   Self-Weight(-1.000)
--------------------------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      *. PROJECT     :
      *. MEMBER NO   =      2,  ELEMENT TYPE = Beam
      *. LOADCOMB NO =      1,  MATERIAL NO  =     1,  SECTION NO =     2
      *. UNIT SYSTEM : kips, in

      *. SECTION PROPERTIES : Designation = W10X22
         Shape     = I - Section. (Rolled)
         Depth     =     10.200,  Top F Width =      5.750,  Bot.F Width =      5.750
         Web Thick =      0.240,  Top F Thick =      0.360,  Bot.F Thick =      0.360

         Area = 6.49000e+000,  Asy  = 2.76000e+000,  Asz  = 2.44800e+000
         Ybar = 2.87500e+000,  Zbar = 5.10000e+000,  Qyb  = 5.36688e+001,  Qzb  = 4.13281e+000
         Syy  = 2.32000e+001,  Szz  = 3.97000e+000,  Zyy  = 2.60000e+001,  Zzz  = 6.10000e+000
         Iyy  = 1.18000e+002,  Izz  = 1.14000e+001,  Iyz  = 0.00000e+000
         ry   = 4.27000e+000,  rz   = 1.33000e+000
         J    = 2.39000e-001,  Cwp  = 2.76112e+002

      *. DESIGN PARAMETERS FOR STRENGTH EVALUATION :
         Ly   = 8.28125e+000,  Lz   = 8.28125e+000,  Lb   = 8.28125e+000
         Ky   = 1.00000e+000,  Kz   = 1.00000e+000

      *. MATERIAL PROPERTIES :
         Fy   = 5.00000e+001,  Es   = 2.90000e+004,  MATERIAL NAME  = A709-50W


================================================================================
   [[[*]]]   COMPUTE MOMENT MAGNIFICATION FACTORS AND MAGNIFIED MOMENTS.
================================================================================

      ( ). Compute moment magnification factors(B1y,B1z).
           -. If tension or bending member.
           -. Assumed   B1y  =  1.00
           -. Assumed   B1z  =  1.00

      ( ). Magnification factors for sidesway moments(B2y,B2z).
           -. B2y   =  1.00 (Default value)
           -. B2z   =  1.00 (Default value)

      ( ). Given factored axial forces and moments at <I>.
           ------------------------------------------------------------
            Load Case             Pu                 My                 Mz
           ------------------------------------------------------------
            DL                  0.01               0.20               0.00
            LL                 13.61             163.17             -43.67
            DL+LL              13.62             163.37             -43.67
            WL or EL            0.00               0.00               0.00
           ------------------------------------------------------------
            DL+LL+WL(EL)       13.62             163.37             -43.67
           ------------------------------------------------------------
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Compute magnified moments.
           -. Muy   =  B1y*My(DL+LL) + B2y*My(WL(EL)) =      163.37 in-kips.
           -. Muz   =  B1z*Mz(DL+LL) + B2z*Mz(WL(EL)) =      -43.67 in-kips.

================================================================================
   [[[*]]]   CHECK AXIAL STRENGTH.
================================================================================

      ( ). Check slenderness ratio of axial tension member (l/r).
           [ AASHTO-LRFD12 Specification 6.8.4 ]
           -. l/r  =     6.2  <   200.0  --->  O.K.

      ( ). Calculate axial tensile strength (phiPn).
           [ AASHTO-LRFD12 Specification 6.8.2.1-1 ]
           -. Resistance factor for tension  :  phi = 0.95
           -. phiPn  =  phi*Fy*Ag  =        308.28 kips.

      ( ). Check ratio of axial strength (Pu/phiPn).
                Pu                13.62
           -. -------  =  --------------  =  0.044  <  1.000  --->  O.K.
               phiPn             308.28

================================================================================
   [[[*]]]   CHECK SECTION PROPORTION LIMITS
================================================================================

      ( ). Check Web Proportions (AASHTO-LRFD12 Eq.6.10.2.1.1-1)

           -. Limit Value  = 150
           -. D/tw  =   39.50  <  Limit Value  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-1)

           -. bf/2tf  =    7.99  <     12.00  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-2)

           -. bf  >  D/6 =    1.58  --->  O.K.

      ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-3)

           -. tf > 1.1*tw =    0.26  --->  O.K.

      ( ). Check general proportions
                      Izc               5.7033
           -. 0.1  <  ------  =  ----------------  =  1.000  <  10    --->  O.K.
                      Izt               5.7033
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      => Flextural Resistance is Checked by AASHTO-LRFD12 Specification

================================================================================
   [[[*]]]   CHECK SECTION SLENDERNESS
================================================================================

      ( ). Check Flange ratio
                Izc               5.7033
           -. ------  =  ----------------  =  1.000  >  0.3
                Izt               5.7033

      ( ). Check web slenderness
           -. Limit Value  =  5.7*SQRT[E/Fyc]  =  137.27

           -. 2Dc/tw  =   39.50  <  Limit Value

           -. Fyc  =50ksi  <  70.0 ksi

      => Section is Checking by AASHTO-LRFD12 Appendix 6

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MAJOR AXIS.
================================================================================

      ( ). Check Web Compactness Length
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Dcp   =    4.74 in.
           -. Dc    =    4.74 in.
           -. Mp    = 1292.01 in-kips.
           -. My    = 1156.86 in-kips.
           -. Rh    =    1.00(homogenous section)
           -. Lambda_rw = 5.7*SQRT[Es/Fyc] =   137.27

                                             SQRT[Es/Fyc]
           -. Lambda_pw(Dcp)_cal = ------------------------ =    91.48
                                   (0.54*Mp/(Rh*My)-0.09)^2

           -. Lambda_pw(Dcp)_lim = Lambda_rw * (Dcp/Dc)    =   137.27
           -. Lambda_pw(Dcp)     = min[Lambda_pw(Dcp)_cal, Lambda_pw(Dcp)_lim ] =    91.48
           -. Lambda_w(Dcp)      = 2*Dcp/tw =    39.50

           -. Lambda_w(Dcp) < Lambda_pw(Dcp)  --->  compact web

      ( ). Web Plastification Factor
           [ AASHTO-LRFD12 Specification A6.2.1]
           -. Myc  = Fyc*Iyy/yc = 1156.86 in-kips.
           -. Myt  = Fyt*Iyy/yt = 1156.86 in-kips.
           -. Rpc  =  Mp/Myc  =    1.12
           -. Rpt  =  Mp/Myt  =    1.12
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check compression-flange compactness
           -. Lambda_pf  =  0.38*SQRT[E/Fyc]  =    9.15
           -. Lambda_f = bf/2tf  =    7.99  <  Lambda_pf  --->  Compact.

      ( ). Compute nominal flexural strength of the Compression Flange (Mnc).
           [ AASHTO-LRFD12 Specification A6.3.2]
           -. Rpc =    1.12
           -. Myc = 1156.86 in-kips.
           -. Mnc_FLB  =  Rpc*Myc  = 1292.01 in-kips.

      ( ). Effective Radius of Gyration for Lateral Torsional Buckling (rt)
           -. depth of the web in compression(Dc)  =    4.74 in.
                                 bfc
           -. rt = ------------------------------------ =     1.53 in.
                      [    (    1    Dc*tw   ) ]
                 SQRT[ 12*(1 + - * -------- ) ]
                      [    (    3   bfc*tfc  ) ]

      ( ). Unbraced Length(Lb) Check
           -. Lb  =    8.28 in.
           -. Lp  = 1.0*rt*SQRT(Es/Fyc) =   36.75 in.

           -. Lb < Lp  --->  compact unbraced length

      ( ). Compute Lateral Torsional Buckling Resistance of Compression Flange (Mnc_LTB).
           [ AASHTO-LRFD12 Specification A6.3.3]
           -. Rpc  =    1.12
           -. Myc  = 1156.86 in-kips.
           -. Mnc_LTB =  Rpc*Myc  = 1292.01 in-kips.

      ( ). Compute nominal flexural strength of Compression Flange (Mnc)
           -. Mnc  =  min[Mnc_FLB, Mnc_LTB] = 1292.01 in-kips.

      ( ). Compute nominal flexural strength of Tension Flange (Mnt).
           [ AASHTO-LRFD12 Specification A6.4]
           -. Rpt  =    1.12
           -. Myt  =       1156.86 in-kips.
           -. Mnt  =  Rpt*Myt  =       1292.01 in-kips.

           -. Mn  =  min(Mnc, Mnt)  =       1292.01 in-kips.

      ( ). Compute flexural strength about major axis (phiMny).
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMny  =  phi*Mny  =       1292.01 in-kips.
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


      ( ). Check ratio of flexural strength (Muy/phiMny).
                Muy               163.37
           -. --------  =  ---------------  =  0.126  <  1.000  --->  O.K.
               phiMny            1292.01

================================================================================
   [[[*]]]   CHECK FLEXURAL STRENGTH ABOUT MINOR AXIS.
================================================================================

      ( ). Slenderness ratio for flange (lamda_f)
           [ AASHTO-LRFD12 Specification 6.12.2.2]
           -. lamda_f   = bf/(2*tf) =     7.99
           -. lamda_pf  = 0.038*SQRT(Es/Fyf) =     0.92
           -. lamda_rf  = 0.83*SQRT(Es/Fyf) =    19.99

           -. lamda_pf < lamda_f < lamda_rf  --->  noncompact flange

      ( ). Compute nominal flexural strength of the flange (Mn).
           [ AASHTO-LRFD12 Specification 6.12.2.2]

                   [    (     Szz )   ( Lambda_f  - Lambda_pf )]
           -. Mn = [1 - (1 - -----) * (-----------------------)]*Fy*Zzz =   235.51 in-kips.
                   [    (     Zzz )   (    0.45*SQRT[Es/Fy]   )]

      ( ). Compute flexural strength about minor axis (phiMnz).
           -. Mnz  =        235.51 in-kips.
           -. Resistance factor for flexure  :  phi = 1.00
           -. phiMnz  =  phi*Mnz  =        235.51 in-kips.

      ( ). Check ratio of flexural strength (Muz/phiMnz).
                Muz                43.67
           -. --------  =  ---------------  =  0.185  <  1.000  --->  O.K.
               phiMnz             235.51

================================================================================
   [[[*]]]   CHECK INTERACTION OF COMBINED STRENGTH.
================================================================================

      ( ). Check interaction ratio of combined strength.
           [ AASHTO-LRFD12 Specification 6.9.2.2 ]
           -. Pu/phiPn  <  0.20  --->  Formula(6.9.2.2-2)
                            Pu       [    Muy        Muz   ]
           -. ComRat  =  --------- + [ -------- + -------- ]
                          2*phiPn    [  phiMny     phiMnz  ]
                      =  0.022 + [  0.126  +  0.185 ]
                      =  0.334  <  1.000  --->  O.K.

================================================================================
   [[[*]]]   CHECK SHEAR STRENGTH.
================================================================================
```

```
--------------------------------------------------------------------------------
 MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                  Version 8.5.6
================================================================================


     ( ). Calculate nominal shear strength in local-y direction (Vny).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asy  =         80.04 kips.
          -. Vny =  C*Vp  =         80.04 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    J )
          -. Applied shear force : Vuy =          5.27 kips.
                Vuy                5.27
          -. --------  =  --------------  =  0.066  <  1.000  --->  O.K.
              phiVny              80.04

     ( ). Calculate nominal shear strength in local-z direction (Vnz).
          [AASHTO-LRFD12 Specification 6.10.9.2]
          -. Resistance factor for shear  :  phi = 1.00
          -. C   =    1.00
          -. Vp  =  0.58*Fyw*Asz  =         70.99 kips.
          -. Vnz =  C*Vp  =         70.99 kips.

     ( ). Check ratio of shear strength (Vu/phiVn).
          ( LCB =    1, POS =    I )
          -. Applied shear force : Vuz =         12.20 kips.
                Vuz                12.20
          -. --------  =  --------------  =  0.172  <  1.000  --->  O.K.
              phiVnz              70.99
```

REFERENCES:

LRFD	-	AASHTO LRFD Bridge Design Specifications 8th ed.
PLANS	-	ADM Welding and Fabrication Plans
AISC	-	AISC Manual of Steel Construction, 15th ed. (2017)
MIDAS	-	Midas Civil Model for TL-1 Rail Loads

Unit Definition:

$$kcf := 1000\ pcf$$

Phi factor for weld metal in fillet welds	$\phi_{e2} := 0.80$	LRFD 6.5.4.2
Classification strength of weld metal (ksi)	$F_{exx} := 70\ ksi$	Assumed
Factored shear resistance of the weld metal (for Fillet-Welded Connections)	$R_r := 0.6 \cdot \phi_{e2} \cdot F_{exx} = 33.6\ ksi$	LRFD 6.13.3.2.4
Vector addition of the shear forces on the weld		LRFD C6.13.3.2.4 MIDAS

$$F_{resultant} := \sqrt{13.61^2 + 5.28^2 + 13.61^2}\ kip = 20.0\ kip$$

Length of weld along flange	$L_{flange} := 5\ in$	Assumed
Number of welds along both flanges	$N_{flange} := 2$	Assumed
Length of weld along web	$L_{web} := 8.25\ in$	Assumed
Number of welds along web	$N_{web} := 2$	Assumed
Thickness of welds	$t_{weld} := 0.25\ in$	PLANS
Factored capacity of shear resistance of weld metal	$R_{r\_weld} := R_r \cdot 0.707 \cdot t_{weld} \cdot \left(L_{web} \cdot N_{web} + L_{flange} \cdot N_{flange}\right) = 157.4\ kip$	
Is resistance from fillet weld greater than the resultant force applied?	$\text{if}\left(R_{r\_weld} > F_{resultant}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$	

1/2" x 7" x 13-1/4" Plate

Phi factor for shear	$\phi_v := 1.00$	LRFD 6.5.4.2
Phi factor for shear, rupture in connection element	$\phi_{vu} := 0.80$	LRFD 6.5.4.2
Yield strength of connection element	$F_y := 50\ ksi$	
Tensile strength of connection element	$F_u := 65.3\ ksi$	
Gross area of connection element (in global y direction)	$A_{vgply} := 0.5\ in \cdot 13.5\ in = 6.75\ in^2$	
Gross area of connection element (in global z direction)	$A_{vgplz} := 0.5\ in \cdot 7\ in = 3.5\ in^2$	
Factored shear yielding resistance of the connection element (in global y direction)	$R_{r\_yieldingply} := \phi_v \cdot 0.58 \cdot F_y \cdot A_{vgply} = 195.8\ kip$	LRFD 6.13.5.3-1
Factored shear yielding resistance of the connection element (in global z direction)	$R_{r\_yieldingplz} := \phi_v \cdot 0.58 \cdot F_y \cdot A_{vgplz} = 101.5\ kip$	LRFD 6.13.5.3-1
Reduction factor for bolt holes punched full size	$R_p := 0.90$	LRFD 6.13.5.3
Net area of connection element (in global y direction)	$A_{vny} := A_{vgply} - 3 \cdot \frac{13}{16}\ in \cdot 0.5\ in = 5.53\ in^2$	
Net area of connection element (in global z direction)	$A_{vnz} := A_{vgplz} - 2 \cdot \frac{13}{16}\ in \cdot 0.5\ in = 2.69\ in^2$	
Factored shear rupture resistance of the connection element (in global y direction)	$R_{r\_shear\_rupturey} := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot A_{vny} = 150.8\ kip$	
Factored shear rupture resistance of the connection element (in global z direction)	$R_{r\_shear\_rupturez} := \phi_{vu} \cdot 0.58 \cdot R_p \cdot F_u \cdot A_{vnz} = 73.3\ kip$	

Global y direction factored shear force

$F_{force\_y} := 5.27\ kip$

Global z direction factored shear force

$F_{force\_z} := 13.62\ kip$

Minimum of Factored shear yielding and shear rupture resistance of the connection element (in global y direction)

$R_{r\_yielding\_shear\_rupturey} := min\left(R_{r\_yieldingply}, R_{r\_shear\_rupturey}\right) = 150.8\ kip$

Minimum of Factored shear yielding and shear rupture resistance of the connection element (in global z direction)

$R_{r\_yielding\_shear\_rupturez} := min\left(R_{r\_yieldingplz}, R_{r\_shear\_rupturez}\right) = 73.3\ kip$

Is shear yielding/shear rupture resistance greater than the resultant force applied? (in global y direction)

$\text{if}\left(R_{r\_yielding\_shear\_rupturey} > F_{force\_y}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Is shear yielding/shear rupture resistance greater than the resultant force applied? (in global z direction)

$\text{if}\left(R_{r\_yielding\_shear\_rupturez} > F_{force\_z}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

3/4" Bolts

Area of bolts

$A_b := \frac{\pi}{4} \cdot \left(\frac{3}{4}\ in\right)^2 = 0.44\ in^2$

Specified minimum tensile strength of the bolt (assume A307 bolts)

$F_{ub} := 60\ ksi$

Number of shear planes per bolt

$N_s := 1$

Number of bolts

$N_{bolts} := 6$

Nominal Shear Resistance of A307 bolts

$R_{n\_shear\_bolts} := 0.45 \cdot A_b \cdot F_{ub} \cdot N_s \cdot N_{bolts} = 71.57\ kip$ LRFD 6.13.2.7-2

Maximum shear force applied to bolts

$F_{shear\_bolts} := 13.61\ kip$

Is nominal shear resistance of A307 bolts greater than the maximum force applied?

$\text{if}\left(R_{n\_shear\_bolts} > F_{shear\_bolts}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Hole size factor specified in LRFD Table 6.13.2.8-2

$K_h := 1$

Surface condition factor specified in LRFD Table 6.13.2.8-3 (assumed)

$K_s := 0.30$

Number of shear planes per bolt

$N_s := 1$

Miminum required bolt tension, A307 bolts (assumed)

$P_t := 11\ kip$

Slip resistance of A307 bolts

$R_{n\_slip\_resistance} := K_h \cdot K_s \cdot N_s \cdot P_t \cdot N_{bolts} = 19.8\ kip$ LRFD 6.13.2.8-1

Maximum shear force applied to bolts

$F_{shear\_bolts} := 13.61\ kip$

Is nominal slip resistance of A307 bolts greater than the maximum force applied?

$\text{if}\left(R_{n\_shear\_bolts} > F_{shear\_bolts}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

1

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          (D)  THE CUMULATIVE LIABILITY OF UMD TO LICENSEE FOR ALL
               CLAIMS, DEMANDS OR ACTIONS ARISING OUT OF OR RELATING
               TO THE LICENSED MATERIALS AND THIS AGREEMENT, SHALL
               NOT EXCEED THE TOTAL AMOUNT OF ALL LICENSE FEES PAID
               TO UMD HEREUNDER DURING THE TWELVE (12) MONTHS
               IMMEDIATELY PRECEDING SUCH CLAIM, DEMAND OR ACTION.
               THIS LIMITATION OF LIABILITY IS INTENDED TO APPLY
               WITHOUT REGARD TO WHICH OTHER PROVISIONS OF THIS
               AGREEMENT HAVE BEEN BREACHED OR HAVE PROVEN INEFFECTIVE.

          (E)  WITHOUT LIMITING THE FOREGOING, IN NO EVENT SHALL UMD
               BE LIABLE FOR ANY BUSINESS EXPENSE, MACHINE DOWN TIME,
               LOSS OF PROFITS, ANY INCIDENTAL, SPECIAL, EXEMPLARY OR
               CONSEQUENTIAL DAMAGES, OR ANY CLAIMS OR DEMANDS BROUGHT
               AGAINST LICENSEE OR LICENSEE'S CUSTOMERS, EVEN IF
               UMD HAS BEEN ADVISED OF THE POSSIBILITY OF SUCH CLAIMS
               OR DEMANDS. THIS LIMITATION UPON DAMAGES AND CLAIMS IS
               INTENDED TO APPLY WITHOUT REGARD TO WHICH OTHER
               PROVISIONS OF THIS AGREEMENT HAVE BEEN BREACHED OR
               HAVE PROVEN INEFFECTIVE.
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                                                                      INPUT
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150'x17'-Swanzey\M


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

                        DESCRIPTION                                      DATE
                        -----------                                      ----
     42095 Temporary Bridge                                           8/14/2019

     CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
     ---------------     ------    --------  -BY- -BY- ------------
                                           TEM  TPL


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

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

         1       10           2       AASHTO  2019     0     2            3
      ---------------------------------------------------------------------------

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

       * span interval   : maximum = 20
                           default = 10

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

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

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

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

       * EFFECTIVE FLANGE WITH OPTION =    0
          0 - DEFAULT (2008)
          1 - "PRIOR TO 2007" WIDTH IS USED
1
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                                                                      INPUT
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     TABLE 0.0.3.1 STRUCTURAL DETAILS
                   ******************

                  BEAM     WIDTH             EDGE OF               COMPOSITE  STEEL
RIGHT    R. EDGE OF
                  POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
OVERHANG   SLAB TO
                  NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
WIDTH    CURB
                     OF    1=INT.   BARRIER  -------- ------- DEPTH WIDTH.   REGION    FACTOR ----
---- ----------
     GIRDERS 2,3=EXT.  (ft)      (ft)    (ft)   (in)  (in)      (%)     >= 1.0
(ft)     (ft)
     ---------------------------------------------------------------------------------------
---------------

         8     1        16.42     1.27    0.79   0.00   0.00   100.00     1.00
0.00      0.00

     ---------------------------------------------------------------------------------------
---------------
     * 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

     * EXT=2 for the same left and right exterior overhang information or
             only left exterior overhang information

     * EXT=3 for the right exterior overhang information


     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
     ------,------,------,------,------,------,------,------,------,------,

      47.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00



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

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

       2.21   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00

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                                                                      INPUT
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     TABLE 0.0.4.1 DEFINITION OF SECTIONS
                   **********************

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

        1  W     24      76                 0.00 0.0000 0.00 0.0000

      ---------------------------------------------------------------------------
      NOTE: [1] maximum allowable section number is 70

            [2] For design option (flow 4, 5 or 6) this card need not be input
1
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                                                                      INPUT
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     TABLE 0.0.5.1 DEFINITION OF MEMBERS
                   **********************


        MEMBER     MEMB END  MEMBER DESCRIPTN  PARAMETERS FOR     YIELD STRESS
```

```
        NUMBER     SECT  ID  ---------------- NONPRISMATIC MEMB      (KSI)
      ---------- ---------- LNGTH -->TYPE<-- ----------------- ---------------
     (IN ORDER) LEFT RIGHT  (ft)  0=PRISMAT    S(0)    S(1)     WEB  TOP  BOT
     ------------------------------------------------------------------------

          1       1    1    47.00                                50.  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
1
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                                                                      INPUT
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     TABLE 0.0.6.1 AASHTO LIVE LOADING -  LOAD TYPE (A)
                   ************************************



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

          HL-93             1          3      1000           0       0.00

          HL-93 VEHICLE X FACTOR OF  0.00

     ------------------------------------------------------------------------
     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  1000
1
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                                                                      INPUT
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650'x17'-Swanzey\M



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



        IMPACT  FACTOR  TO  OVERRIDE  THE  AASHTO  FORMULA
     ------------------------------------------------------
                                                                    OPTIONAL
               OPTIONAL CALCULATION OF FACTOR 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   0.33   0.33    0.33  15.00    1 1 1 1 1   0.33    1 1 1 1 1
      1  0.00  0   0.33   0.33    0.33   0.00    0 0 0 0 0   0.33    1 1 1 1 1

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

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

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

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

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

1
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                                                                     INPUT
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     TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS
                    *******************************************


      LOAD IDENTIFICATION    UNIFORM LOAD DATA      CONCENTRATED LOAD DATA
     ---------------------- ---------------------   ----------------------
       LOAD   DESCRIPTION  INTENSITY  POSITION       INTENSITY    DISTANCE
     NO. TYPE                        FROM     TO                  FROM L SUPT
                            (k/ft)      (ft)         (Kips)        (ft)
     ------------------------------------------------------------------------
      1   1  Timber Deck     0.032    0.00   47.00      0.00        0.00

      2   1  Railing         0.013    0.00   47.00      0.00        0.00

      3   0  Intermediate    0.000    0.00    0.00      0.07        3.50

      4   0  Intermediate    0.000    0.00    0.00      0.07       13.50

      5   0  Intermediate    0.000    0.00    0.00      0.07       23.50

      6   0  Intermediate    0.000    0.00    0.00      0.07       33.50

      7   0  Intermediate    0.000    0.00    0.00      0.07       43.50


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

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

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

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

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

                       2 = Noncomposite Loads,(In LRFD, it is for DC1 load)
                           where N = modulus ratio = Es/Ec
1
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       TABLE 0.0.13.1 YIELD STRESS (Fy) AND LATERAL BRACING DATA (lb)
                      ***********************************************


         L O C A T I O N          YIELD             SPACING
       --------------------       STRESS         OF LATERAL BRACING
       DISTANCE    DISTANCE   ----------------  --------------------
         FROM        TO        Fy    Fy (WEB)          Lb
         (ft)        (ft)     (ksi)  (ksi)            (ft)
       ------------------------------------------------------------

          0.00       47.00     50.0  50.0            10.00

      ** WARNING: LATERAL BRACING DISTANCE CAN NOT BE EVENLY DIVIDED **
       ------------------------------------------------------------
       NOTE: [1] default Fy = 36 ksi

             [2] default spacing of lateral bracing = 25 feet

                 Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

             [3] The spacing of lateral bracing is also assumed to
                 be the diaphragm spacing which is used for the
                 calculation of wind effect (code check only).
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     TABLE 2.2.1.1=PROGRAM ASSUMPTIONS
                   *******************

      NO.          D E S C R I P T I O N S
     ----- ------------------------------------------------

       1    Small deflection theory

       2    Material is elastic

       3    Beam length is much greater than lateral dimensions

       4    Torsional effects are neglected

       5    Shear deformations are neglected

       6    Two kinematic degree-of-freedom are assumed 'at each
            joint (vertical deflection and bending rotation)

       7    Concentrated joint loads

       8    Uniform member loads

       9    Transformed sections are used for composite sections
            .... see AASHTO Art.10.38.1.4 or LRFD Art.6.10.1.1.1b

      10    Sections symmetrical about vertical, principal axis

      11    Unshored construction

      12    Hinged bridge ends

     ------------------------------------------------------
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     TABLE 2.2.2.1=LOADING INFORMATION
                   *******************

          AVERAGE DEAD LOAD INTENSITIES
          *****************************

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

1	0.000	0.0762	0.0762

NONCOMPOSITE SUPERIMPOSED DEAD LOADS
****

LOAD	INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	0.032	0.000	47.000
2	0.013	0.000	47.000

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TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES
****

C O N C R E T E D E C K				S T E E L	SUPERSTRUCTURE
UNIT WEIGHT (pcf)	V O L U M E (ft**3)	(yard**3)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)	TOTAL WEIGHT (kip)
150.00	0.0	0.0	0.0	28.6	28.65

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

[2] Superimposed dead load not included

1	0.072	3.500
2	0.072	13.500
3	0.072	23.500
4	0.072	33.500
5	0.072	43.500

```
*** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
       Ts >12.0 In (300 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
       S > 16.0 Ft (4900 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
       Ts >12.0 In (300 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
       S > 16.0 Ft (4900 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Ts =  0.0 < 4.5 In (110 mm) OR
       Ts >12.0 In (300 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  S =  2.2 < 3.5 Ft (1100 mm) OR
       S > 16.0 Ft (4900 mm) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
*** WARNING in LRFD Distrib Factor ***  Kg < 10,000 In4 (4 x 10^9 mm4) OR
       Kg > 7,000,000 In4 (3 x 10^12 mm4) (Pg. 4-31)
```

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

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.330	0.330	0.251	0.251	FOR STRENGTH POSITIVE MOMENT
	0.330				FOR STRENGTH POSITIVE SHEAR
1	0.330				FOR FATIGUE POSITIVE MOMENT
	0.330				FOR FATIGUE POSITIVE SHEAR

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TABLE 2.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY
****

MOMENT OF WEB LOCATION OF N.A. ELASTIC SECTION MODULUS

SP NO	IN NO	D FROM L SUPT	INERTIA Ix (in**4)	DEPTH D (in)	FROM BOT OF STEEL Y(BS) (in)	STEEL BOT. (in**3)	STEEL TOP. (in**3)
1	0	0.00	2100.0	23.9	11.94	175.8	175.8
1	1	4.70	2100.0	23.9	11.94	175.8	175.8
1	2	9.40	2100.0	23.9	11.94	175.8	175.8
1	3	14.10	2100.0	23.9	11.94	175.8	175.8
1	4	18.80	2100.0	23.9	11.94	175.8	175.8
1	5	23.50	2100.0	23.9	11.94	175.8	175.8
1	6	28.20	2100.0	23.9	11.94	175.8	175.8
1	7	32.90	2100.0	23.9	11.94	175.8	175.8
1	8	37.60	2100.0	23.9	11.94	175.8	175.8
1	9	42.30	2100.0	23.9	11.94	175.8	175.8
1	10	47.00	2100.0	23.9	11.94	175.8	175.8

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TABLE 2.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD LOAD
1	0	0.00	0.0	0.0	0.0	0.0	0.0
1	1	4.70	7.6	0.0	0.8	4.4	12.8
1	2	9.40	13.5	0.0	1.3	7.9	22.6
1	3	14.10	17.7	0.0	1.7	10.4	29.8
1	4	18.80	20.2	0.0	1.9	11.8	33.9
1	5	23.50	21.0	0.0	2.1	12.3	35.5
1	6	28.20	20.2	0.0	1.9	11.8	33.9
1	7	32.90	17.7	0.0	1.7	10.4	29.8
1	8	37.60	13.5	0.0	1.3	7.9	22.6
1	9	42.30	7.6	0.0	0.8	4.4	12.8
1	10	47.00	0.0	0.0	0.0	0.0	0.0

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TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	SIDEWALK (MAXIMUM) NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1	4.70	0.0	0.0	125.2	HL-93	0.0	HL-93
1	2	9.40	0.0	0.0	216.0	HL-93	0.0	HL-93
1	3	14.10	0.0	0.0	272.4	HL-93	0.0	HL-93
1	4	18.80	0.0	0.0	304.3	HL-93	0.0	HL-93
1	5	23.50	0.0	0.0	306.7	HL-93	0.0	HL-93
1	6	28.20	0.0	0.0	304.3	HL-93	0.0	HL-93
1	7	32.90	0.0	0.0	272.4	HL-93	0.0	HL-93
1	8	37.60	0.0	0.0	216.0	HL-93	0.0	HL-93
1	9	42.30	0.0	0.0	125.2	HL-93	0.0	HL-93
1	10	47.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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UNIVERSITY OF MARYLAND LRFD- 8TH ED
ANALYSIS
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1650'x17'-Swanzey\M

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

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	0.	0.	0.
1	1	4.70	66.	0.	66.
1	2	9.40	110.	0.	110.
1	3	14.10	131.	0.	131.
1	4	18.80	154.	0.	154.
1	5	23.50	157.	0.	157.
1	6	28.20	154.	0.	154.
1	7	32.90	131.	0.	131.
1	8	37.60	110.	0.	110.
1	9	42.30	66.	0.	66.
1	10	47.00	0.	0.	0.

1
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ANALYSIS
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TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D L	FROM SUPP (ft)	DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0		0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1		4.70	12.8	125.2	HL-93	0.0	HL-93	137.9	12.8
1	2		9.40	22.6	216.0	HL-93	0.0	HL-93	238.6	22.6
1	3		14.10	29.8	272.4	HL-93	0.0	HL-93	302.2	29.8
1	4		18.80	33.9	304.3	HL-93	0.0	HL-93	338.2	33.9
1	5		23.50	35.5	306.7	HL-93	0.0	HL-93	342.2	35.5
1	6		28.20	33.9	304.3	HL-93	0.0	HL-93	338.2	33.9
1	7		32.90	29.8	272.4	HL-93	0.0	HL-93	302.2	29.8
1	8		37.60	22.6	216.0	HL-93	0.0	HL-93	238.6	22.6
1	9		42.30	12.8	125.2	HL-93	0.0	HL-93	137.9	12.8
1	10		47.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

1

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

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
1850'x17'-Swanzey\M

TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D L	FROM SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0		0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1		4.70	137.9	175.5	235.0	185.0	19.2	53.0
1	2		9.40	238.6	303.4	406.2	319.8	33.9	87.8
1	3		14.10	302.2	383.9	513.9	405.0	44.7	104.5
1	4		18.80	338.2	429.5	574.9	453.2	50.9	123.4
1	5		23.50	342.2	434.2	581.1	458.4	53.2	125.7
1	6		28.20	338.2	429.5	574.9	453.2	50.9	123.4
1	7		32.90	302.2	383.9	513.9	405.0	44.7	104.5
1	8		37.60	238.6	303.4	406.2	319.8	33.9	87.8
1	9		42.30	137.9	175.5	235.0	185.0	19.2	53.0
1	10		47.00	0.0	0.0	0.0	0.0	0.0	0.0

1

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UNIVERSITY OF MARYLAND

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
1950'x17'-Swanzey\M

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

SP NO	IN NO	D L	FROM SUPT (ft)	DEAD LOAD BEAM (k)	DEAD LOAD SLAB (k)	CONCENTRATED LOADS (k)	UNIFORM LOADS (k)	TOTAL NONCOMPOSITE DEAD LOAD (k)
1	0		0.00	1.79	0.00	0.2	1.05	3.02
1	1		4.70	1.43	0.00	0.1	0.84	2.38
1	2		9.40	1.07	0.00	0.1	0.63	1.81
1	3		14.10	0.72	0.00	0.0	0.42	1.17
1	4		18.80	0.36	0.00	0.0	0.21	0.60
1	5		23.50	0.00	0.00	0.0	0.00	0.04
1	6		28.20	-0.36	0.00	0.0	-0.21	-0.60
1	7		32.90	-0.72	0.00	0.0	-0.42	-1.17
1	8		37.60	-1.07	0.00	-0.1	-0.63	-1.81
1	9		42.30	-1.43	0.00	-0.1	-0.84	-2.38
1	10		47.00	-1.79	0.00	-0.2	-1.05	-3.02

1

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UNIVERSITY OF MARYLAND

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
2050'x17'-Swanzey\M

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

SP NO	IN NO	D L	FROM SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0		0.00	0.0	0.0	30.28	HL-93	0.00	HL-93
1	1		4.70	0.0	0.0	26.18	HL-93	-1.45	HL-93
1	2		9.40	0.0	0.0	22.17	HL-93	-3.65	HL-93
1	3		14.10	0.0	0.0	18.27	HL-93	-6.09	HL-93
1	4		18.80	0.0	0.0	14.46	HL-93	-8.63	HL-93
1	5		23.50	0.0	0.0	11.27	HL-93	-11.27	HL-93
1	6		28.20	0.0	0.0	8.63	HL-93	-14.46	HL-93
1	7		32.90	0.0	0.0	6.09	HL-93	-18.27	HL-93
1	8		37.60	0.0	0.0	3.65	HL-93	-22.17	HL-93
1	9		42.30	0.0	0.0	1.45	HL-93	-26.18	HL-93
1	10		47.00	0.0	0.0	0.00	HL-93	-30.28	HL-93

1

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UNIVERSITY OF MARYLAND

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
2150'x17'-Swanzey\M

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

SP NO	IN NO	D L	FROM SUPT (ft)	T R U C K POS	NEG	RANGE
1	0		0.00	17.	0.	17.
1	1		4.70	14.	-1.	15.
1	2		9.40	12.	-2.	14.
1	3		14.10	9.	-4.	13.
1	4		18.80	8.	-5.	13.
1	5		23.50	7.	-7.	13.
1	6		28.20	5.	-8.	13.
1	7		32.90	4.	-9.	13.
1	8		37.60	2.	-12.	14.
1	9		42.30	1.	-14.	15.
1	10		47.00	0.	-17.	17.

1

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

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
2250'x17'-Swanzey\M

TABLE 2.2.6.2B=MINIMUM WELD SIZE
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0		0.00	16.7	.36962E-01	0.31	0.2500*	.36962E-01	0.31	0.2500*
1	1		4.70	15.3	.36962E-01	0.28	0.2500*	.36962E-01	0.28	0.2500*
1	2		9.40	14.1	.36962E-01	0.26	0.2500*	.36962E-01	0.26	0.2500*
1	3		14.10	12.9	.36962E-01	0.24	0.2500*	.36962E-01	0.24	0.2500*
1	4		18.80	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	5		23.50	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	6		28.20	13.4	.36962E-01	0.25	0.2500*	.36962E-01	0.25	0.2500*
1	7		32.90	12.9	.36962E-01	0.24	0.2500*	.36962E-01	0.24	0.2500*
1	8		37.60	14.1	.36962E-01	0.26	0.2500*	.36962E-01	0.26	0.2500*
1	9		42.30	15.3	.36962E-01	0.28	0.2500*	.36962E-01	0.28	0.2500*
1	10		47.00	16.7	.36962E-01	0.31	0.2500*	.36962E-01	0.31	0.2500*

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

[2] For non-composite construction:

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

For composite construction:

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

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

* -- minimum weld size governs

1

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

MERLIN V10.11
NONCOMPOSITE
LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
2350'x17'-Swanzey\M

TABLE 2.2.6.2C=WELD DESIGN BY SHEAR FORCE
****

SP NO	IN NO	D L	FROM SUPT (ft)	SHEAR FLOW St [1] (kip/in)	Sb [2] (kip/in)	Sg [3] (kip/in)	MIN SIZE WELD (in)	ALLOWABLE SHEAR FLOW Vw [4] (kip/in)	Vc [5] (kip/in)	Vg [6] (kip/in)	FL AG
1	0		0.00	1.96	1.96	1.96	0.2500	11.88	12.73	11.88	
1	1		4.70	1.69	1.69	1.69	0.2500	11.88	12.73	11.88	
1	2		9.40	1.43	1.43	1.43	0.2500	11.88	12.73	11.88	
1	3		14.10	1.18	1.18	1.18	0.2500	11.88	12.73	11.88	
1	4		18.80	0.94	0.94	0.94	0.2500	11.88	12.73	11.88	
1	5		23.50	0.73	0.73	0.73	0.2500	11.88	12.73	11.88	
1	6		28.20	0.94	0.94	0.94	0.2500	11.88	12.73	11.88	
1	7		32.90	1.18	1.18	1.18	0.2500	11.88	12.73	11.88	
1	8		37.60	1.43	1.43	1.43	0.2500	11.88	12.73	11.88	
1	9		42.30	1.69	1.69	1.69	0.2500	11.88	12.73	11.88	
1	10		47.00	1.96	1.96	1.96	0.2500	11.88	12.73	11.88	

NOTE: [1] St = top flange shear flow
= VQ/I
where : V = shear force
Q = statical moment of the area about neutral axis
I = moment of inertia

[2] Sb = bottom flange shear flow

[3] Sg = govern. shear flow (max. of St and Sb)

[4] Vw = allowable shear flow for min. size weld
= 0.6 * (phi)e2 * Fexx * 0.707 * min. weld size * 2
where: (Phi)e2 = weld resistance factor (0.8)

Fexx = class. strength of the weld metal (70 ksi)

[5] Vc = allowable shear flow on the connected material
= (Phi)v * (0.58 * Ag * Fy)
where: (Phi)v = resistance factor for shear (1.0)
Ag = gross area of the connection element
Fy = min. yield strength of connection element

[6] Vg = govern. allow. shear flow (min. of Vw and Vc)

If [6] is less than [3], then the flag ** will show up.

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 2450'x17'-Swanzey\M

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN NO	D FROM L SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0	0.00	3.0	30.3	HL-93	0.0	HL-93	33.3	3.0
1	1	4.70	2.4	26.2	HL-93	-1.5	HL-93	28.6	0.9
1	2	9.40	1.8	22.2	HL-93	-3.6	HL-93	24.0	-1.8
1	3	14.10	1.2	18.3	HL-93	-6.1	HL-93	19.4	-4.9
1	4	18.80	0.6	14.5	HL-93	-8.6	HL-93	15.1	-8.0
1	5	23.50	0.0	11.3	HL-93	-11.3	HL-93	11.3	-11.2
1	6	28.20	-0.6	8.6	HL-93	-14.5	HL-93	-15.1	8.0
1	7	32.90	-1.2	6.1	HL-93	-18.3	HL-93	-19.4	4.9
1	8	37.60	-1.8	3.6	HL-93	-22.2	HL-93	-24.0	1.8
1	9	42.30	-2.4	1.5	HL-93	-26.2	HL-93	-28.6	-0.9
1	10	47.00	-3.0	0.0	HL-93	-30.3	HL-93	-33.3	-3.0

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 2550'x17'-Swanzey\M

TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	33.3	42.4	56.8	44.7	4.5	13.4
1	1	4.70	28.6	36.4	48.8	38.3	3.6	12.3
1	2	9.40	24.0	30.6	41.1	32.2	2.7	11.3
1	3	14.10	19.4	24.9	33.4	26.1	1.8	10.3
1	4	18.80	15.1	19.4	26.1	20.3	0.9	10.7
1	5	23.50	11.3	14.7	19.8	15.3	0.1	10.7
1	6	28.20	-15.1	-19.4	-26.1	-20.3	-0.9	10.7
1	7	32.90	-19.4	-24.9	-33.4	-26.1	-1.8	10.3
1	8	37.60	-24.0	-30.6	-41.1	-32.2	-2.7	11.3
1	9	42.30	-28.6	-36.4	-48.8	-38.3	-3.6	12.3
1	10	47.00	-33.3	-42.4	-56.8	-44.7	-4.5	13.4

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 2650'x17'-Swanzey\M

TABLE 2.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)

SUPT NO.	TOTAL DEAD LOADS (K)	LL+I ,(K), LOAD TYPE = HL - 93 MINIMUM	GOVERN. LOAD TYPE	MAXIMUM	GOVERN. LOAD TYPE	TOTAL DL+LL+I (L R F D)	MINIMUM	MAXIMUM
1	3.02	0.00	HL-93	30.28	HL-93	ST1	2.72	56.77
						ST2	2.72	44.66
						ST4	4.53	4.53
						SE1	3.02	33.30
						SE2	3.02	42.39
2	3.02	0.00	HL-93	30.28	HL-93	ST1	2.72	56.77
						ST2	2.72	44.66
						ST4	4.53	4.53
						SE1	3.02	33.30
						SE2	3.02	42.39

NOTE: [1] " - " Indicates Uplift

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 2750'x17'-Swanzey\M

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

SP NO	IN NO	D FROM L SUPT (ft)	DEAD LOADS BEAM (in)	DEAD LOADS SLAB (in)	CONCENTRATED LOADS (in)	UNIFORM LOADS (in)	TOTAL DEAD LOAD +=UP , -=DOWN
1	0	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
1	1	4.70	-0.0431	0.0000	-0.0041	-0.0253	-0.0726
1	2	9.40	-0.0816	0.0000	-0.0078	-0.0479	-0.1373
1	3	14.10	-0.1117	0.0000	-0.0107	-0.0655	-0.1879
1	4	18.80	-0.1308	0.0000	-0.0125	-0.0767	-0.2201
1	5	23.50	-0.1374	0.0000	-0.0132	-0.0806	-0.2311
1	6	28.20	-0.1308	0.0000	-0.0125	-0.0767	-0.2201
1	7	32.90	-0.1117	0.0000	-0.0107	-0.0655	-0.1879
1	8	37.60	-0.0816	0.0000	-0.0078	-0.0479	-0.1373
1	9	42.30	-0.0431	0.0000	-0.0041	-0.0253	-0.0726
1	10	47.00	0.0000	0.0000	0.0000	0.0000	0.0000

NOTE: " - " indicates downward deflection

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 2850'x17'-Swanzey\M

TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (FT)	NONCOMPOSITE STEEL DEFL.	NONCOMPOSITE CAMBER (IN) SIZE	DEAD LOADS SLAB DEFL.	DEAD LOADS CAMBER (IN) SIZE	OTHER DL DEFL.	OTHER DL CAMBER (IN) SIZE	TOTAL DEFL.	TOTAL CAMBER (IN) SIZE
1	0	0.0	0.000	--	0.000	--	0.000	--	0.000	--
1	1	4.7	-0.043	0 1/16	0.000	0 0/16	-0.029	0 1/16	-0.073	0 1/ 8
1	2	9.4	-0.082	0 1/ 8	0.000	0 0/16	-0.056	0 1/16	-0.137	0 3/16
1	3	14.1	-0.112	0 1/ 8	0.000	0 0/16	-0.076	0 1/ 8	-0.188	0 1/ 4
1	4	18.8	-0.131	0 3/16	0.000	0 0/16	-0.089	0 1/ 8	-0.220	0 1/ 4
1	5	23.5	-0.137	0 3/16	0.000	0 0/16	-0.094	0 1/ 8	-0.231	0 1/ 4
1	6	28.2	-0.131	0 3/16	0.000	0 0/16	-0.089	0 1/ 8	-0.220	0 1/ 4
1	7	32.9	-0.112	0 1/ 8	0.000	0 0/16	-0.076	0 1/ 8	-0.188	0 1/ 4
1	8	37.6	-0.082	0 1/ 8	0.000	0 0/16	-0.056	0 1/16	-0.137	0 3/16
1	9	42.3	-0.043	0 1/16	0.000	0 0/16	-0.029	0 1/16	-0.073	0 1/ 8
1	10	47.0	0.000	--	0.000	--	0.000	--	0.000	--

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
(UNFACTORED)

SPAN NO.	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	1/800 OF SPAN L AASHTO 2.5.2.6.2	ROTATION [5] Rad.
1	23.50	1 0.125	-0.593 MAX	HL-93	0.70	0.02005
			0.065 MIN			
			-0.144 MAX	LANE		
			0.065 MIN	LANE		
		1 0.125	-0.373 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) ..... AASHTO LRFD Art. 2.5.2.6

where NL= no. of traffic lanes
Ng= no. of girders

No multi-presence factor applied for LL deflection

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

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

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

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

[7] For truck rating the most probable number of lanes is assumed for avereaging.

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN NO	D FROM L SUPT (ft)	N.COMP DEAD LOAD STEEL BEAM TOP (ksi)	N.COMP DEAD LOAD STEEL BEAM BOT (ksi)	OTHER DEAD LOAD STEEL BEAM TOP (ksi)	OTHER DEAD LOAD STEEL BEAM BOT (ksi)	TOTAL DEAD LOAD STEEL BEAM TOP (ksi)	TOTAL DEAD LOAD STEEL BEAM BOT (ksi)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	1	4.70	-0.52	0.52	-0.36	0.36	-0.87	0.87
1	2	9.40	-0.92	0.92	-0.63	0.63	-1.54	1.54
1	3	14.10	-1.21	1.21	-0.83	0.83	-2.03	2.03
1	4	18.80	-1.38	1.38	-0.94	0.94	-2.32	2.32

1	5	23.50	-1.44	1.44	-0.98	0.98	-2.42	2.42
1	6	28.20	-1.38	1.38	-0.94	0.94	-2.32	2.32
1	7	32.90	-1.21	1.21	-0.83	0.83	-2.03	2.03
1	8	37.60	-0.92	0.92	-0.63	0.63	-1.54	1.54
1	9	42.30	-0.52	0.52	-0.36	0.36	-0.87	0.87
1	10	47.00	0.00	0.00	0.00	0.00	0.00	0.00

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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

SP NO	IN NO	D L (ft)	FROM SUPT	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	0.00	0.00	0.00	0.00
1	1		4.70	-8.54	8.54	0.00	0.00
1	2		9.40	-14.74	14.74	0.00	0.00
1	3		14.10	-18.59	18.59	0.00	0.00
1	4		18.80	-20.77	20.77	0.00	0.00
1	5		23.50	-20.94	20.94	0.00	0.00
1	6		28.20	-20.77	20.77	0.00	0.00
1	7		32.90	-18.59	18.59	0.00	0.00
1	8		37.60	-14.74	14.74	0.00	0.00
1	9		42.30	-8.54	8.54	0.00	0.00
1	10		47.00	0.00	0.00	0.00	0.00

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BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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

SP NO	IN NO	D L (ft)	FROM SUPT	TOP OF TOP FLANGE TRUCK	TOP OF TOP FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE	BOT OF BOT FLANGE TRUCK	BOT OF BOT FLANGE ONLY	GOVERNING STRESS RANGE	GOVERNING LOAD TP	GOVERNING LOAD TYPE
1	0		0.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL
1	1		4.70	4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	2		9.40	7.5	0.0	7.5	C	HL	7.5	0.0	7.5	T	HL
1	3		14.10	8.9	0.0	8.9	C	HL	8.9	0.0	8.9	T	HL
1	4		18.80	10.5	0.0	10.5	C	HL	10.5	0.0	10.5	T	HL
1	5		23.50	10.7	0.0	10.7	C	HL	10.7	0.0	10.7	T	HL
1	6		28.20	10.5	0.0	10.5	C	HL	10.5	0.0	10.5	T	HL
1	7		32.90	8.9	0.0	8.9	C	HL	8.9	0.0	8.9	T	HL
1	8		37.60	7.5	0.0	7.5	C	HL	7.5	0.0	7.5	T	HL
1	9		42.30	4.5	0.0	4.5	C	HL	4.5	0.0	4.5	T	HL
1	10		47.00	0.0	0.0	0.0	T	HL	0.0	0.0	0.0	T	HL

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
ANALYSIS

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TABLE 2.2.9.3A=SERVICE I TOTAL (DC+DW+(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-9.42	9.42	-0.87	0.87
1	2		9.40	50.	50.	-16.29	16.29	-1.54	1.54
1	3		14.10	50.	50.	-20.63	20.63	-2.03	2.03
1	4		18.80	50.	50.	-23.09	23.09	-2.32	2.32
1	5		23.50	50.	50.	-23.36	23.36	-2.42	2.42
1	6		28.20	50.	50.	-23.09	23.09	-2.32	2.32
1	7		32.90	50.	50.	-20.63	20.63	-2.03	2.03
1	8		37.60	50.	50.	-16.29	16.29	-1.54	1.54
1	9		42.30	50.	50.	-9.42	9.42	-0.87	0.87
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-11.98	11.98	-0.87	0.87
1	2		9.40	50.	50.	-20.71	20.71	-1.54	1.54
1	3		14.10	50.	50.	-26.20	26.20	-2.03	2.03
1	4		18.80	50.	50.	-29.32	29.32	-2.32	2.32
1	5		23.50	50.	50.	-29.64	29.64	-2.42	2.42
1	6		28.20	50.	50.	-29.32	29.32	-2.32	2.32
1	7		32.90	50.	50.	-26.20	26.20	-2.03	2.03
1	8		37.60	50.	50.	-20.71	20.71	-1.54	1.54
1	9		42.30	50.	50.	-11.98	11.98	-0.87	0.87
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-14.52	14.52	0.44	-0.44
1	2		9.40	50.	50.	-25.03	25.03	0.77	-0.77
1	3		14.10	50.	50.	-31.52	31.52	1.02	-1.02
1	4		18.80	50.	50.	-35.19	35.19	1.16	-1.16
1	5		23.50	50.	50.	-35.43	35.43	1.21	-1.21
1	6		28.20	50.	50.	-35.19	35.19	1.16	-1.16
1	7		32.90	50.	50.	-31.52	31.52	1.02	-1.02
1	8		37.60	50.	50.	-25.03	25.03	0.77	-0.77
1	9		42.30	50.	50.	-14.52	14.52	0.44	-0.44
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-14.21	14.21	0.74	-0.74
1	2		9.40	50.	50.	-24.48	24.48	1.31	-1.31
1	3		14.10	50.	50.	-30.81	30.81	1.73	-1.73
1	4		18.80	50.	50.	-34.38	34.38	1.97	-1.97
1	5		23.50	50.	50.	-34.58	34.58	2.06	-2.06
1	6		28.20	50.	50.	-34.38	34.38	1.97	-1.97
1	7		32.90	50.	50.	-30.81	30.81	1.73	-1.73
1	8		37.60	50.	50.	-24.48	24.48	1.31	-1.31
1	9		42.30	50.	50.	-14.21	14.21	0.74	-0.74
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-12.62	12.62	-1.09	1.09
1	2		9.40	50.	50.	-21.83	21.83	-1.93	1.93
1	3		14.10	50.	50.	-27.64	27.64	-2.54	2.54
1	4		18.80	50.	50.	-30.94	30.94	-2.90	2.90
1	5		23.50	50.	50.	-31.29	31.29	-3.02	3.02
1	6		28.20	50.	50.	-30.94	30.94	-2.90	2.90
1	7		32.90	50.	50.	-27.64	27.64	-2.54	2.54
1	8		37.60	50.	50.	-21.83	21.83	-1.93	1.93
1	9		42.30	50.	50.	-12.62	12.62	-1.09	1.09
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0		0.00	50.	50.	0.00	0.00	0.00	0.00
1	1		4.70	50.	50.	-12.32	12.32	-0.78	0.78
1	2		9.40	50.	50.	-21.29	21.29	-1.39	1.39
1	3		14.10	50.	50.	-26.93	26.93	-1.83	1.83
1	4		18.80	50.	50.	-30.13	30.13	-2.09	2.09
1	5		23.50	50.	50.	-30.44	30.44	-2.18	2.18
1	6		28.20	50.	50.	-30.13	30.13	-2.09	2.09
1	7		32.90	50.	50.	-26.93	26.93	-1.83	1.83
1	8		37.60	50.	50.	-21.29	21.29	-1.39	1.39
1	9		42.30	50.	50.	-12.32	12.32	-0.78	0.78
1	10		47.00	50.	50.	0.00	0.00	0.00	0.00

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

MEMBER NO	LENGTH (ft)	STEEL TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH THICK	COVER PLATE -- (in) BOT WIDTH THICK
1	47.00	W	9.0	0.6800	22.5	0.4390	9.0	0.6800	0.00 0.0000	0.00 0.0000

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 3550'x17'-Swanzey\M

TABLE 2.2.22.4=DEPTH RATIOS

** [1] = Span length (ft) = 47.0

MEMBER NO	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [2]/(12*[1]) <=> [3]= GIRDER D. [3]/(12*[1])	LIMITATION 1/25 <=> 1/30	STATUS
1	47.00	23.89	[2] does not apply		
			[3]= 23.89	0.04236 > 0.03333	OK

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

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

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

[3] -- same as NOTE [2]

** -- cover plates not taken into account

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 3650'x17'-Swanzey\M

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

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp [1]	Web depth	Web thick	D/tw [2]	2Dcp/tw [3]	2Dcp/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0		0.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	1		4.70	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	2		9.40	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	3		14.10	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	4		18.80	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	5		23.50	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	6		28.20	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	7		32.90	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	8		37.60	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	9		42.30	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0
1	10		47.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55		0

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 3750'x17'-Swanzey\M

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

SP NO	IN NO	D L	FROM SUPT	Lo ng [1]	Co mp [1]	Web depth	Web thick	2Dc/tw [2]	2Dc/tw [3]	Web Category
1	0		0.00	0	1	22.53	0.439	51.32	137.27	2
1	1		4.70	0	1	22.53	0.439	51.32	137.27	2
1	2		9.40	0	1	22.53	0.439	51.32	137.27	2
1	3		14.10	0	1	22.53	0.439	51.32	137.27	2
1	4		18.80	0	1	22.53	0.439	51.32	137.27	2
1	5		23.50	0	1	22.53	0.439	51.32	137.27	2
1	6		28.20	0	1	22.53	0.439	51.32	137.27	2
1	7		32.90	0	1	22.53	0.439	51.32	137.27	2
1	8		37.60	0	1	22.53	0.439	51.32	137.27	2
1	9		42.30	0	1	22.53	0.439	51.32	137.27	2
1	10		47.00	0	1	22.53	0.439	51.32	137.27	2

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 3850'x17'-Swanzey\M

TABLE 2.2.22.6=FLANGE PROPORTIONS CHECK

SP NO	NO L	D SUPT	bf/2tf [1] (ft)	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0	0.00	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	1	4.70	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	2	9.40	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	3	14.10	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	4	18.80	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	5	23.50	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	6	28.20	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	7	32.90	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	8	37.60	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	9	42.30	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0
1	10	47.00	6.61	12.	9.0	3.8	0.680	0.483	
			6.61	12.	9.0	3.8	0.680	0.483	1.00 0

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

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

1

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TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING

LATERAL BRACING NO	DIST. FROM LEFT SUPT FROM (FT)	TO (FT)	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	10.0	0.000	28.667	16.788	4.909	1.579
2	10.0	20.0	28.667	39.351	35.876	32.401	1.089
3	20.0	30.0	37.649	39.351	39.530	39.708	1.000
4	30.0	40.0	21.761	37.649	31.794	25.940	1.169

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

1

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES

SP NO	IN NO	D L	FROM SUPT	Co mp [1]	FLB LMDAf (ft)	LMDApf	LMDArf	CAT	LTB Lb (ft)	Lp (ft)	Lr (ft)	GOV CAT
1	0		0.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	1		4.70	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	2		9.40	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	3		14.10	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	4		18.80	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	5		23.50	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	6		28.20	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	7		32.90	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	8		37.60	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	9		42.30	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2
1	10		47.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2 2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

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

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

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

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.58	50.00	50.00	50.00
1	1		4.70	1	1.000	1.000	1.58	50.00	50.00	50.00
1	2		9.40	1	1.000	1.000	1.58	50.00	50.00	50.00
1	3		14.10	1	1.000	1.000	1.09	50.00	47.54	47.54
1	4		18.80	1	1.000	1.000	1.09	50.00	47.54	47.54
1	5		23.50	1	1.000	1.000	1.00	50.00	43.66	43.66
1	6		28.20	1	1.000	1.000	1.00	50.00	43.66	43.66
1	7		32.90	1	1.000	1.000	1.17	50.00	50.00	50.00
1	8		37.60	1	1.000	1.000	1.17	50.00	50.00	50.00
1	9		42.30	1	1.000	1.000	1.00	50.00	43.66	43.66
1	10		47.00	1	1.000	1.000	1.00	50.00	43.66	43.66

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

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

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Category Non-Comp.
1	0		0.00	1	2	2			2
1	1		4.70	1	2	2			2
1	2		9.40	1	2	2			2
1	3		14.10	1	2	2			2
1	4		18.80	1	2	2			2
1	5		23.50	1	2	2			2
1	6		28.20	1	2	2			2
1	7		32.90	1	2	2			2
1	8		37.60	1	2	2			2
1	9		42.30	1	2	2			2
1	10		47.00	1	2	2			2

Please read NOTE on the following page

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

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

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

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

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

SP NO	IN NO	D L	FROM (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0		0.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	1		4.70	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	2		9.40	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	3		14.10	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.5	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	4		18.80	0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.5	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	5		23.50	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	6		28.20	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	7		32.90	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	8		37.60	0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	9		42.30	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	10		47.00	0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0

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

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

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

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

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

TABLE 2.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP NO	IN NO	D L	FROM	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0		0.00	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	1		4.70	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	2		9.40	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	3		14.10	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	4		18.80	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	5		23.50	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	6		28.20	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	7		32.90	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	8		37.60	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	9		42.30	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	10		47.00	0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3

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

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

TABLE 2.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0		0.00	0	-	-	0.0	-	0.0	30.0	0.0	50.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-
1	1		4.70	0	-	-	16.0	-	0.0	30.0	16.0	50.0	-0--
					-	-	16.0	-	0.0	30.0	16.0	50.0	--0-
1	2		9.40	0	-	-	27.7	-	0.0	30.0	27.7	50.0	-0--
					-	-	27.7	-	0.0	30.0	27.7	50.0	--0-
1	3		14.10	0	-	-	35.1	-	0.0	30.0	35.1	47.5	-0--
					-	-	35.1	-	0.0	30.0	35.1	50.0	--0-
1	4		18.80	0	-	-	39.2	-	0.0	30.0	39.2	47.5	-0--
					-	-	39.2	-	0.0	30.0	39.2	50.0	--0-
1	5		23.50	0	-	-	39.7	-	0.0	30.0	39.7	43.7	-0--
					-	-	39.7	-	0.0	30.0	39.7	50.0	--0-
1	6		28.20	0	-	-	39.2	-	0.0	30.0	39.2	43.7	-0--
					-	-	39.2	-	0.0	30.0	39.2	50.0	--0-
1	7		32.90	0	-	-	35.1	-	0.0	30.0	35.1	50.0	-0--
					-	-	35.1	-	0.0	30.0	35.1	50.0	--0-
1	8		37.60	0	-	-	27.7	-	0.0	30.0	27.7	50.0	-0--
					-	-	27.7	-	0.0	30.0	27.7	50.0	--0-
1	9		42.30	0	-	-	16.0	-	0.0	30.0	16.0	43.7	-0--
					-	-	16.0	-	0.0	30.0	16.0	50.0	--0-
1	10		47.00	0	-	-	0.0	-	0.0	30.0	0.0	45.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-

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

         "-" is N.A.

         Under FLAG Column, 0 = OK; 1= NG
         M = Moment; C = Comp. Flange; T = Tension Flange
         D = Ductility; Mu = maximum of Strengh I, II and IV

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

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

         The value of fbu is the actual stress in this table,
         the users can use the maximum value within the
         unbraced length to do their own check
1
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   TABLE 2.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
                  **********************************************

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

    1  0   0.00  0                              0.000      0.000 0.000     4
                                                0.000      0.000 0.000     4
    1  1   4.70  0                              0.000      0.321 0.321     4
                                                0.000      0.321 0.321     4
    1  2   9.40  0                              0.000      0.555 0.555     4
                                                0.000      0.555 0.555     4
    1  3  14.10  0                              0.000      0.738 0.738     4
                                                0.000      0.702 0.702     4
    1  4  18.80  0                              0.000      0.825 0.825     4
                                                0.000      0.785 0.785     4
    1  5  23.50  0                              0.000      0.908 0.908     4
                                                0.000      0.793 0.793     4
    1  6  28.20  0                              0.000      0.899 0.899     4
                                                0.000      0.785 0.785     4
    1  7  32.90  0                              0.000      0.702 0.702     4
                                                0.000      0.702 0.702     4
    1  8  37.60  0                              0.000      0.555 0.555     4
                                                0.000      0.555 0.555     4
    1  9  42.30  0                              0.000      0.367 0.367     4
                                                0.000      0.321 0.321     4
    1 10  47.00  0                              0.000      0.000 0.000     4
                                                0.000      0.000 0.000     4


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

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

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


1
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 TABLE 2.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY
                 ***********************************

 SP IN D FROM  Fy     k     Art.6.10.9.2         C     Vp        SHEAR CAPACITY
 NO NO L SUPT ----- ----- -------------- D/tw  ----- ----- -----  -Eq.6.10.9.2--
        (ft)  (ksi)  [1]    [2]    [3]          [4]  (ksi)        Vn = C x Vp
 ------------------------------------------------------------------------------

  1  0   0.00  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  1   4.70  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  2   9.40  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  3  14.10  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  4  18.80  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  5  23.50  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  6  28.20  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  7  32.90  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  8  37.60  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1  9  42.30  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8
  1 10  47.00  50.0  5.0    60.3   75.4  51.3  1.000   286.8     286.8


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

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

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

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

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

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

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


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

                    Vu = CVp

               where D = clear, unsupported distance between flange components

                    d0 = distance between transverse stiffeners

            ** This Article (6.10.9.2) indicates that a designer cannot
               count on post-buckling shear resistance from tension-
               field action for an unstiffened girder.
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      TABLE 2.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY
                      *********************************

                              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             56.8   <      286.8       2           2
       1  1   4.70       2             48.8   <      286.8       2           2
       1  2   9.40       2             41.1   <      286.8       2           2
       1  3  14.10       2             33.4   <      286.8       2           2
       1  4  18.80       2             26.1   <      286.8       2           2
       1  5  23.50       2             19.8   <      286.8       2           2
       1  6  28.20       2             26.1   <      286.8       2           2
       1  7  32.90       2             33.4   <      286.8       2           2
       1  8  37.60       2             41.1   <      286.8       2           2
       1  9  42.30       2             48.8   <      286.8       2           2
       1 10  47.00       2             56.8   <      286.8       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 <= 150 (Eq. 6.10.2.1.1-1)

                   w / long. stiff.

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

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

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

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

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


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

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

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

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

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

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

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

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

               * STRENGTH CATEGORY; 0 = compact section
                                    2 = braced non-compact section
                                    3 = slender section
1
```

TABLE 2.2.22.17=TRANSVERSE STIFFENER SPACING

SP NO	IN NO	D FROM L SUPT (ft)	YIELD STRESS Fy (ksi)	LFD/LRFD MAXIMUM SHEAR (k)	UNSTIFFENED SHEAR CAPACITY (k)	REQUIREMENT OF TRANS. STIFFENERS 1=YES , 0=NO	MAX. ALLOWABLE STIFFENERS SPACING (ft-in)
1	0	0.00	50.	56.77	286.83	0	
1	1	4.70	50.	48.79	286.83	0	
1	2	9.40	50.	41.07	286.83	0	
1	3	14.10	50.	33.44	286.83	0	
1	4	18.80	50.	26.07	286.83	0	
1	5	23.50	50.	19.78	286.83	0	
1	6	28.20	50.	26.07	286.83	0	
1	7	32.90	50.	33.44	286.83	0	
1	8	37.60	50.	41.07	286.83	0	
1	9	42.30	50.	48.79	286.83	0	
1	10	47.00	50.	56.77	286.83	0	

TABLE 2.2.22.19A=SPECIAL FATIGUE REQUIREMENTS FOR WEBS

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR DL (k)	SDL (k)	FACTORED FATIGUE PERM. V (k)	<=>	Vcr (k)	<=>	UNF. PERM + FAC FATIG V (k)
1	0	0.00	3.02	0.00	16.72	3.78 <	286.83	>	32.29
1	1	4.70	2.38	0.00	14.10	2.98 <	286.83	>	27.06
1	2	9.40	1.81	0.00	11.67	2.27 <	286.83	>	22.24
1	3	14.10	1.17	0.00	9.24	1.47 <	286.83	>	17.35
1	4	18.80	0.60	0.00	8.20	0.76 <	286.83	>	14.95
1	5	23.50	0.04	0.00	6.68	0.05 <	286.83	>	11.73
1	6	28.20	0.60	0.00	8.20	0.76 <	286.83	>	14.95
1	7	32.90	1.17	0.00	9.24	1.47 <	286.83	>	17.35
1	8	37.60	1.81	0.00	11.67	2.27 <	286.83	>	22.24
1	9	42.30	2.38	0.00	14.10	2.98 <	286.83	>	27.06
1	10	47.00	3.02	0.00	16.72	3.78 <	286.83	>	32.29

NOTE: This table checks Art. 6.10.3.3 and Art. 6.10.5.3.
Vcr is the shear-buckling resistance determined from Eq. 6.10.9.3.3-1.

TABLE 2.2.22.21=SERVICE LIMIT STATE CHECK

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1] (ksi)	RATIO1	BOT. FLANGE ff+fl/2 (ksi)	[2] (ksi)	RATIO2	COMP. FLG fc (ksi)	fcrw [3] (ksi)	FLAG TBC
1	0	0.00	-	0.0	40.0	0.000	0.0	40.0	Infin	0.0	50.0	000
1	1	4.70	-	12.0	40.0	0.299	12.0	40.0	Infin	12.0	50.0	000
1	2	9.40	-	20.7	40.0	0.518	20.7	40.0	Infin	20.7	50.0	000
1	3	14.10	-	26.2	40.0	0.655	26.2	40.0	Infin	26.2	50.0	000
1	4	18.80	-	29.3	40.0	0.733	29.3	40.0	Infin	29.3	50.0	000
1	5	23.50	-	29.6	40.0	0.741	29.6	40.0	Infin	29.6	50.0	000
1	6	28.20	-	29.3	40.0	0.733	29.3	40.0	Infin	29.3	50.0	000
1	7	32.90	-	26.2	40.0	0.655	26.2	40.0	Infin	26.2	50.0	000
1	8	37.60	-	20.7	40.0	0.518	20.7	40.0	Infin	20.7	50.0	000
1	9	42.30	-	12.0	40.0	0.299	12.0	40.0	Infin	12.0	50.0	000
1	10	47.00	-	0.0	40.0	0.000	0.0	40.0	NaN	0.0	50.0	000

NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

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

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

RATIO1 = ff / [1] or (ff + fl/2) / [1] (top flange)
RATIO2 = (ff + fl/2) / [2] (bot. flange)

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING	RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING	RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.70	TR	-7.9	A B B^C^C D E E^	TR	7.9	A B B^C^C
1	2	9.40	TR	-13.1	A B B^C^C D E E^	TR	13.1	A B
1	3	14.10	TR	-15.6	A B B^C^C D E E^	TR	15.6	A B
1	4	18.80	TR	-18.4	A B B^C^C D E E^	TR	18.4	A
1	5	23.50	TR	-18.8	A B B^C^C D E E^	TR	18.8	A
1	6	28.20	TR	-18.4	A B B^C^C D E E^	TR	18.4	A
1	7	32.90	TR	-15.6	A B B^C^C D E E^	TR	15.6	A B
1	8	37.60	TR	-13.1	A B B^C^C D E E^	TR	13.1	A B
1	9	42.30	TR	-7.9	A B B^C^C D E E^	TR	7.9	A B B^C^C
1	10	47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.75 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

TABLE 2.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D FROM L SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING	RANGE (ksi)	ACCEPTABLE STRESS CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING	RANGE (ksi)	ACCEPTABLE STRESS CATEGORY
1	0	0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1	4.70	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C D
1	2	9.40	TR	-6.0	A B B^C^C D E E^	TR	6.0	A B B^
1	3	14.10	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B
1	4	18.80	TR	-8.4	A B B^C^C D E E^	TR	8.4	A
1	5	23.50	TR	-8.6	A B B^C^C D E E^	TR	8.6	A
1	6	28.20	TR	-8.4	A B B^C^C D E E^	TR	8.4	A
1	7	32.90	TR	-7.1	A B B^C^C D E E^	TR	7.1	A B
1	8	37.60	TR	-6.0	A B B^C^C D E E^	TR	6.0	A B B^
1	9	42.30	TR	-3.6	A B B^C^C D E E^	TR	3.6	A B B^C^C D
1	10	47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.80 load factor.
Design for Finite Life w/ ADTT Single Lane= 1000 & No. of Cycles= 27375000
* 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

TABLE 1.2.30.1=AASHTO CODE CHECK STATUS SUMMARY

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

```
  OK     1.2.22.21=SERVICE LIMIT STATE CHECK



         MORE TABLES TO BE INSPECTED ...

         1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

         1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

         1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

         1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS
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 TABLE 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
               ***********************************

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

  1  0   0.00    50.00  TOP          0.00      0.00     0.00 99.99  HL-93 99.99  0
                50.00  BOT          0.00      0.00     0.00 99.99  HL-93 99.99  0
  1  1   4.70    50.00  TOP         -0.87     -8.54     0.00  4.24  HL-93  3.27  0
                50.00  BOT          0.87      8.54     0.00  4.24  HL-93  3.27  0
  1  2   9.40    50.00  TOP         -1.54    -14.74     0.00  2.42  HL-93  1.86  0
                50.00  BOT          1.54     14.74     0.00  2.42  HL-93  1.86  0
  1  3  14.10    47.54  TOP         -2.03    -18.59     0.00  1.79  HL-93  1.38  0
                50.00  BOT          2.03     18.59     0.00  1.89  HL-93  1.46  0
  1  4  18.80    47.54  TOP         -2.32    -20.77     0.00  1.59  HL-93  1.23  0
                50.00  BOT          2.32     20.77     0.00  1.68  HL-93  1.30  0
  1  5  23.50    43.66  TOP         -2.42    -20.94     0.00  1.44  HL-93  1.11  0
                50.00  BOT          2.42     20.94     0.00  1.66  HL-93  1.28  0
  1  6  28.20    43.66  TOP         -2.32    -20.77     0.00  1.45  HL-93  1.12  0
                50.00  BOT          2.32     20.77     0.00  1.68  HL-93  1.30  0
  1  7  32.90    50.00  TOP         -2.03    -18.59     0.00  1.89  HL-93  1.46  0
                50.00  BOT          2.03     18.59     0.00  1.89  HL-93  1.46  0
  1  8  37.60    50.00  TOP         -1.54    -14.74     0.00  2.42  HL-93  1.86  0
                50.00  BOT          1.54     14.74     0.00  2.42  HL-93  1.86  0
  1  9  42.30    43.66  TOP         -0.87     -8.54     0.00  3.69  HL-93  2.85  0
                50.00  BOT          0.87      8.54     0.00  4.24  HL-93  3.27  0
  1 10  47.00    44.99  TOP          0.00      0.00     0.00 99.99  HL-93 99.99  0
                50.00  BOT          0.00      0.00     0.00 99.99  HL-93 99.99  0


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

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

                [2]: AASHTO LRFR 6.4.2 LOAD-RATING EQUATION

                [3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
                     SECTION IS NOT GOVERNING AT ALL.

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

                [4]:  LRFD SECTION IS RATED BY
                        RF=(Mu-DRI1*(GDC*D1+GDW*D2)))/DRI1*GLLST1*(L+I)
                      FOR NON COMPACT SECTIONS, RATING FORMULA IS MODIFIED TO
                        RF=(Mu-DRI1*(GDC*D1*S2/S0+GDW*D2*S2/S1)))/DRI1*GLLST1*(L+I)
                      WHERE S0,S1 & S2 ARE SECTION MODULUS OF D1,D2 & L
                           STRESS CATEGORY SHOWN ON THE LAST COL. (0=COMPACT)
                      DRI: Factor related to Ductility, Redundancy and
                           operational Importance
                           Maximum for Strength and Minimum for Other Limit State
                           (AASHTO LRFD Art. 1.3.2.1)

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

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 TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH
               ********************************

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

  1  0   0.00  40.0   0.00   0.00   0.00  0.00  0.00   0.00 99.99   HL-93  76.92
  1  1   4.70  40.0  -0.87   0.87  -8.54  8.54  0.00   0.00  4.58   HL-93   3.52
  1  2   9.40  40.0  -1.54   1.54 -14.74 14.74  0.00   0.00  2.61   HL-93   2.01
  1  3  14.10  40.0  -2.03   2.03 -18.59 18.59  0.00   0.00  2.04   HL-93   1.57
  1  4  18.80  40.0  -2.32   2.32 -20.77 20.77  0.00   0.00  1.81   HL-93   1.40
  1  5  23.50  40.0  -2.42   2.42 -20.94 20.94  0.00   0.00  1.79   HL-93   1.38
  1  6  28.20  40.0  -2.32   2.32 -20.77 20.77  0.00   0.00  1.81   HL-93   1.40
  1  7  32.90  40.0  -2.03   2.03 -18.59 18.59  0.00   0.00  2.04   HL-93   1.57
  1  8  37.60  40.0  -1.54   1.54 -14.74 14.74  0.00   0.00  2.61   HL-93   2.01
  1  9  42.30  40.0  -0.87   0.87  -8.54  8.54  0.00   0.00  4.58   HL-93   3.52
  1 10  47.00  40.0   0.00   0.00   0.00  0.00  0.00   0.00 99.99   HL-93  76.92
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          NOTE [1]: AASHTO 5.5.2.2 OPERATING RATING OF SECTIONS GOVERNED BY
                    SERVICEABILITY LOAD-STRENGTH RELATIONSHIP, PP.37
                    ---> BASED ON LRFD, SERVICE II LIMIT STATE CRITERIA


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


                [3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
                     SECTION IS NOT GOVERNING AT ALL.

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

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

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                                                                   RATING
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 TABLE 2.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK
                 ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                          GOVERNING
 ------------ ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    RATING     TYPE
         (ft)         (ft)
 ------------------------------------------------------------------------------

   1    47.00        23.50       STRENGTH L.S.  1.44      1.11     HL-93
   1    47.00        23.50       SERVICEBILITY  1.79      1.38     HL-93

 ------------------------------------------------------------------------------
     NOTE: Control Rating Factor for Current Bridge:
           STRENGTH L.S.
                Operating= 1.438
                Inventory= 1.109



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

     SPAN       CRITICAL LOCATION GOVERNING
 ------------ ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    LL MOMENT
         (ft)         (ft)                               (k-ft)
 ------------------------------------------------------------------

   1    47.00        23.50       STRENGTH L.S.  2.52        236.17
   1    47.00        23.50       SERVICEBILITY  2.33

 ------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1000
                Legal Load Rating= 2.331
1
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                                                                   RATING
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 TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR
               ***********************************

                              UNFACTORED   UNFACTORED L+I
                              DEAD LOAD    --------------  OPERAT.        INVENT.
 SP IN D FROM  SHEAR           SHEAR        MAX.  SHEAR    RATING  LIVE  RATING
 NO NO L SUPT  CAP.                                       ---------- FACTOR  LOAD  FACTOR
        (ft)  (kips)          D1; kips  D2     POS.     NEG.  [3]   TYPE
 ------------------------------------------------------------------------------

  1  0   0.00    286.8       0.0      3.0     30.3      0.0  6.91  HL-93  5.33
  1  1   4.70    286.8       0.0      2.4     26.2     -1.5  8.02  HL-93  6.18
  1  2   9.40    286.8       0.0      1.8     22.2     -3.6  9.49  HL-93  7.32
  1  3  14.10    286.8       0.0      1.2     18.3     -6.1 11.56  HL-93  8.92
  1  4  18.80    286.8       0.0      0.6     14.5     -8.6 14.64  HL-93 11.30
  1  5  23.50    286.8       0.0      0.0     11.3    -11.3 18.84  HL-93 14.53
  1  6  28.20    286.8       0.0     -0.6      8.6    -14.5 14.64  HL-93 11.30
  1  7  32.90    286.8       0.0     -1.2      6.1    -18.3 11.56  HL-93  8.92
  1  8  37.60    286.8       0.0     -1.8      3.6    -22.2  9.49  HL-93  7.32
  1  9  42.30    286.8       0.0     -2.4      1.5    -26.2  8.02  HL-93  6.18
  1 10  47.00    286.8       0.0     -3.0      0.0    -30.3  6.91  HL-93  5.33


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                                                                   RATING
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 TABLE 2.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK
                 ************************************************

     SPAN       CRITICAL LOCATION GOVERNING                          GOVERNING
 ------------ ------------------  RATING    OPERATING INVENTORY  LIVE LOAD
  NO.  LENGTH     D FROM L SUPT    CRITERION    RATING    RATING     TYPE
         (ft)         (ft)
 ------------------------------------------------------------------------------

   1    47.00        47.00       STRENGTH L.S.  6.91      5.33     HL-93
```

```
 ---------------------------------------------------------------------------
     NOTE: Control Rating Factor for Current Bridge:
           STRENGTH L.S.
                Operating= 6.906
                Inventory= 5.327


 TABLE 2.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK     FA
                 ******************************************************

     SPAN       CRITICAL LOCATION GOVERNING
 ------------ ------------------  RATING    LEGAL LOAD   MAXIMUM
  NO.  LENGTH    D FROM L SUPT    CRITERION    RATING    LL SHEAR
        (ft)         (ft)                                 (kips)
 ---------------------------------------------------------------------

   1    47.00        47.00       STRENGTH L.S. 11.23         25.14

 ---------------------------------------------------------------
     NOTE: control Rating Factor for Current Bridge:
           STRENGTH L.S. with Live Load Factor =1.300 & ADTT = 1000
                Legal Load Rating=11.227
1
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                                                                    RATING
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 TABLE 2.2.34.1=RATING; LIVE LOAD DEFLECTION
                 ****************************

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

       1    23.50    1   0.125       -0.59    HL-93            1.19

                     1   0.125       -0.59                     1.19

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

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

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

Exterior Beam

1

```
                  ************************************
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     TABLE  0.0.1.1 PROJECT DATA
                    ************

                        DESCRIPTION                                      DATE
                        -----------                                      ----
     42095 Temporary Bridge                                           8/14/2019

     CONTRACT NUMBER     STR NO    STR UNIT  DES  CHK  SPECS. USED
     ---------------     ------    --------  -BY- -BY- ------------
                                           TEM  TPL


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

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

         1       10           2       AASHTO  2019     0     2            3
      ---------------------------------------------------------------------------

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

       * span interval   : maximum = 20
                           default = 10

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

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

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

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

       * EFFECTIVE FLANGE WITH OPTION =    0
          0 - DEFAULT (2008)
          1 - "PRIOR TO 2007" WIDTH IS USED
1
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     TABLE 0.0.3.1 STRUCTURAL DETAILS
                   ******************

                  BEAM     WIDTH             EDGE OF               COMPOSITE  STEEL
RIGHT    R. EDGE OF
                  POSITION  BETWEEN  OVERHANG SLAB TO    HAUNCH    PERCENTAGE  LOAD
OVERHANG   SLAB TO
                  NUMBER --------  CURBS OR   WIDTH   CURB   ------------ AT NEG MOM DETAIL
WIDTH    CURB
                     OF    1=INT.   BARRIER  -------- ------- DEPTH WIDTH.   REGION    FACTOR ----
---- ----------
     GIRDERS 2,3=EXT.  (ft)      (ft)    (ft)   (in)  (in)      (%)     >= 1.0
(ft)     (ft)
     ---------------------------------------------------------------------------------------
---------------

         8     2        16.42     1.27    0.79   0.00   0.00   100.00     1.00
0.00      0.00

     ---------------------------------------------------------------------------------------
---------------
     * 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

     * EXT=2 for the same left and right exterior overhang information or
             only left exterior overhang information

     * EXT=3 for the right exterior overhang information


     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
     ------,------,------,------,------,------,------,------,------,------,

      47.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00



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

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

       2.21   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00   0.00

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                                                                      INPUT
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     TABLE 0.0.4.1 DEFINITION OF SECTIONS
                   **********************

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

        1  W     24      76                 0.00 0.0000 0.00 0.0000

      ---------------------------------------------------------------------------
      NOTE: [1] maximum allowable section number is 70

            [2] For design option (flow 4, 5 or 6) this card need not be input
1
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                                                                      INPUT
 FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE
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     TABLE 0.0.5.1 DEFINITION OF MEMBERS
                   **********************


        MEMBER     MEMB END  MEMBER DESCRIPTN  PARAMETERS FOR     YIELD STRESS
```

NUMBER (IN ORDER)	SECT LEFT	SECT RIGHT	ID	LNGTH (ft)	TYPE 0=PRISMAT	NONPRISMATIC MEMB S(0)	S(1)	(KSI) WEB	TOP	BOT
1	1	1		47.00				50.	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

1

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TABLE 0.0.6.1 AASHTO LIVE LOADING - LOAD TYPE (A)

AASHTO LOADING HL - 93	TANDEM LIVE LOAD 1=YES : 0=NO	AASHTO ROAD TYPE ADTT	ADTTSL	1,2,3 OR 4------ SIDEWALK ---LIVE LOAD--- (k/ft)
HL-93	1	3	1000	0	0.00

HL-93 VEHICLE X FACTOR OF 0.00

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 1000

1

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TABLE 0.0.11.1 DEFINITION OF UNIFORM AND CONCENTRATED LOADS

LOAD NO.	IDENTIFICATION TYPE	DESCRIPTION	UNIFORM LOAD DATA INTENSITY (k/ft)	POSITION FROM	POSITION TO	CONCENTRATED LOAD DATA INTENSITY (Kips)	DISTANCE FROM L SUPT (ft)
1	1	Timber Deck	0.024	0.00	47.00	0.00	0.00
2	1	Railing	0.013	0.00	47.00	0.00	0.00
3	0	Intermediate	0.000	0.00	0.00	0.04	3.50
4	0	Intermediate	0.000	0.00	0.00	0.04	13.50
5	0	Intermediate	0.000	0.00	0.00	0.04	23.50
6	0	Intermediate	0.000	0.00	0.00	0.04	33.50
7	0	Intermediate	0.000	0.00	0.00	0.04	43.50

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

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

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

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

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

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

1

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

LOCATION FROM (ft)	LOCATION TO (ft)	YIELD STRESS Fy (ksi)	YIELD STRESS Fy (WEB) (ksi)	SPACING OF LATERAL BRACING Lb (ft)
0.00	47.00	50.0	50.0	10.00

** WARNING: LATERAL BRACING DISTANCE CAN NOT BE EVENLY DIVIDED **

NOTE: [1] default Fy = 36 ksi

[2] default spacing of lateral bracing = 25 feet

Please refer to AASHTO LRFD Art. 6.7.4 for requirement.

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

1

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TABLE 2.2.1.1=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

1

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TABLE 2.2.2.1=LOADING INFORMATION

AVERAGE DEAD LOAD INTENSITIES

SPAN NO.	SLAB (K/FT)	+ STEEL (K/FT)	= TOTAL (K/FT)
1	0.000	0.0762	0.0762

NONCOMPOSITE SUPERIMPOSED DEAD LOADS

LOAD	INTENSITY (K or K/Ft)	DIST FROM (Ft)	DIST TO (Ft)
1	0.024	0.000	47.000
2	0.013	0.000	47.000

1

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TABLE 2.1.3.1=BRIDGE SUPERSTRUCTURE QUANTITIES

CONCRETE DECK VOLUME UNIT WEIGHT (pcf)	(ft**3)	(yard**3)	TOTAL WEIGHT (kip)	STEEL TOTAL WEIGHT (kip)	SUPERSTRUCTURE TOTAL WEIGHT (kip)
150.00	0.0	0.0	0.0	28.6	28.65

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

[2] Superimposed dead load not included

1 0.036 3.500

2	0.036	13.500
3	0.036	23.500
4	0.036	33.500
5	0.036	43.500

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TABLE 2.1.3.2=DISTRIBUTION OF LRFD LIVE LOADS

SPAN NO.	AASHTO LOADING (A)	DUMP TRUCK (D)	MAXIMUM TRUCK (M)	SPECIAL TRUCK (G,C)	
1	0.187	0.187	0.187	0.187	FOR STRENGTH POSITIVE MOMENT
	0.187				FOR STRENGTH POSITIVE SHEAR
1	0.246				FOR FATIGUE POSITIVE MOMENT
	0.246				FOR FATIGUE POSITIVE SHEAR

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TABLE 2.2.4.1=NONCOMPOSITE SECTION PROPERTIES FOR N=INFINITY

SP NO	IN D FROM NO L SUPT	MOMENT OF INERTIA Ix (in**4)	WEB DEPTH D (in)	LOCATION OF N.A. FROM BOT OF STEEL Y(BS) (in)	ELASTIC SECTION MODULUS OF STEEL BOT. (in**3)	TOP. (in**3)
1	0 0.00	2100.0	23.9	11.94	175.8	175.8
1	1 4.70	2100.0	23.9	11.94	175.8	175.8
1	2 9.40	2100.0	23.9	11.94	175.8	175.8
1	3 14.10	2100.0	23.9	11.94	175.8	175.8
1	4 18.80	2100.0	23.9	11.94	175.8	175.8
1	5 23.50	2100.0	23.9	11.94	175.8	175.8
1	6 28.20	2100.0	23.9	11.94	175.8	175.8
1	7 32.90	2100.0	23.9	11.94	175.8	175.8
1	8 37.60	2100.0	23.9	11.94	175.8	175.8
1	9 42.30	2100.0	23.9	11.94	175.8	175.8
1	10 47.00	2100.0	23.9	11.94	175.8	175.8

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TABLE 2.2.5.1=NONCOMPOSITE DEAD LOAD MOMENTS FOR N=INFINITY (UNFACTORED)

SP NO	IN D FROM NO L SUPT (ft)	DEAD LOAD BEAM (k-ft)	DEAD LOAD SLAB (k-ft)	CONCENTRATED LOADS (k-ft)	UNIFORM LOADS (k-ft)	TOTAL (k-ft) NONCOMPOSITE DEAD	TOTAL (k-ft) NONCOMPOSITE LOAD
1	0 0.00	0.0	0.0	0.0	0.0	0.0	
1	1 4.70	7.6	0.0	0.4	3.6	11.6	
1	2 9.40	13.5	0.0	0.6	6.4	20.5	
1	3 14.10	17.7	0.0	0.9	8.4	26.9	
1	4 18.80	20.2	0.0	1.0	9.6	30.7	
1	5 23.50	21.0	0.0	1.0	10.0	32.1	
1	6 28.20	20.2	0.0	1.0	9.6	30.7	
1	7 32.90	17.7	0.0	0.9	8.4	26.9	
1	8 37.60	13.5	0.0	0.6	6.4	20.5	
1	9 42.30	7.6	0.0	0.4	3.6	11.6	
1	10 47.00	0.0	0.0	0.0	0.0	0.0	

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TABLE 2.2.5.2=LIVE LOAD MOMENTS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D FROM NO L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (k-ft)	SIDEWALK (MAXIMUM) NEGATIVE (k-ft)	LL+I ,(k-ft), LOAD TYPE= HL -93 MAXIMUM POSITIVE	LOAD TYPE	MAXIMUM NEGATIVE	LOAD TYPE
1	0 0.00	0.0	0.0	0.0	HL-93	0.0	HL-93
1	1 4.70	0.0	0.0	70.8	HL-93	0.0	HL-93
1	2 9.40	0.0	0.0	122.2	HL-93	0.0	HL-93
1	3 14.10	0.0	0.0	154.2	HL-93	0.0	HL-93
1	4 18.80	0.0	0.0	172.2	HL-93	0.0	HL-93
1	5 23.50	0.0	0.0	173.6	HL-93	0.0	HL-93
1	6 28.20	0.0	0.0	172.2	HL-93	0.0	HL-93
1	7 32.90	0.0	0.0	154.2	HL-93	0.0	HL-93
1	8 37.60	0.0	0.0	122.2	HL-93	0.0	HL-93
1	9 42.30	0.0	0.0	70.8	HL-93	0.0	HL-93
1	10 47.00	0.0	0.0	0.0	HL-93	0.0	HL-93

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

SP NO	IN D FROM NO L SUPT (ft)	TRUCK POS	TRUCK NEG	TRUCK RANGE
1	0 0.00	0.	0.	0.
1	1 4.70	49.	0.	49.
1	2 9.40	82.	0.	82.
1	3 14.10	97.	0.	97.
1	4 18.80	115.	0.	115.
1	5 23.50	117.	0.	117.
1	6 28.20	115.	0.	115.
1	7 32.90	97.	0.	97.
1	8 37.60	82.	0.	82.
1	9 42.30	49.	0.	49.
1	10 47.00	0.	0.	0.

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TABLE 2.2.5.3=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)

SP NO	IN D FROM NO L SUPP (ft)	TOTAL DL (k-ft)	TOTAL POS LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL NEG LL+I (k-ft)	GOVERN LOAD TYPE	TOTAL MAX POS (k-ft)	TOTAL MAX NEG (k-ft)
1	0 0.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0
1	1 4.70	11.6	70.8	HL-93	0.0	HL-93	82.4	11.6
1	2 9.40	20.5	122.2	HL-93	0.0	HL-93	142.7	20.5
1	3 14.10	26.9	154.2	HL-93	0.0	HL-93	181.1	26.9
1	4 18.80	30.7	172.2	HL-93	0.0	HL-93	203.0	30.7
1	5 23.50	32.1	173.6	HL-93	0.0	HL-93	205.7	32.1
1	6 28.20	30.7	172.2	HL-93	0.0	HL-93	203.0	30.7
1	7 32.90	26.9	154.2	HL-93	0.0	HL-93	181.1	26.9
1	8 37.60	20.5	122.2	HL-93	0.0	HL-93	142.7	20.5
1	9 42.30	11.6	70.8	HL-93	0.0	HL-93	82.4	11.6
1	10 47.00	0.0	0.0	HL-93	0.0	HL-93	0.0	0.0

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TABLE 2.2.5.4=MOMENT SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)

SP NO	IN D FROM NO L SUPT (ft)	SERVICE I (k-ft)	SERVICE II (k-ft)	STRENGTH I (k-ft)	STRENGTH II (k-ft)	STRENGTH IV (k-ft)	FATIGUE RANGE (k-ft)
1	0 0.00	0.0	0.0	0.0	0.0	0.0	0.0
1	1 4.70	82.4	103.7	138.4	110.1	17.3	39.6
1	2 9.40	142.7	179.4	239.5	190.6	30.7	65.5
1	3 14.10	181.1	227.4	303.5	241.8	40.4	77.9
1	4 18.80	203.0	254.6	339.8	270.9	46.1	92.0
1	5 23.50	205.7	257.8	343.9	274.5	48.1	93.7
1	6 28.20	203.0	254.6	339.8	270.9	46.1	92.0
1	7 32.90	181.1	227.4	303.5	241.8	40.4	77.9
1	8 37.60	142.7	179.4	239.5	190.6	30.7	65.5
1	9 42.30	82.4	103.7	138.4	110.1	17.3	39.6
1	10 47.00	0.0	0.0	0.0	0.0	0.0	0.0

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

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

1	0	0.00	1.79	0.00	0.1	0.85	2.73
1	1	4.70	1.43	0.00	0.1	0.68	2.17
1	2	9.40	1.07	0.00	0.1	0.51	1.64
1	3	14.10	0.72	0.00	0.0	0.34	1.07
1	4	18.80	0.36	0.00	0.0	0.17	0.55
1	5	23.50	0.00	0.00	0.0	0.00	0.02
1	6	28.20	-0.36	0.00	0.0	-0.17	-0.55
1	7	32.90	-0.72	0.00	0.0	-0.34	-1.07
1	8	37.60	-1.07	0.00	-0.1	-0.51	-1.64
1	9	42.30	-1.43	0.00	-0.1	-0.68	-2.17
1	10	47.00	-1.79	0.00	-0.1	-0.85	-2.73

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

SP NO	IN NO	D FROM L SUPT (ft)	SIDEWALK (MAXIMUM) POSITIVE (kips)	SIDEWALK (MAXIMUM) NEGATIVE (kips)	LL+I ,(kips), LOAD TYPE= HL -93 MAXIMUM POSITIVE	GOVERN. LOAD TYPE	MAXIMUM NEGATIVE	GOVERN. LOAD TYPE
1	0	0.00	0.0	0.0	17.14	HL-93	0.00	HL-93
1	1	4.70	0.0	0.0	14.82	HL-93	-0.82	HL-93
1	2	9.40	0.0	0.0	12.55	HL-93	-2.07	HL-93
1	3	14.10	0.0	0.0	10.34	HL-93	-3.45	HL-93
1	4	18.80	0.0	0.0	8.19	HL-93	-4.89	HL-93
1	5	23.50	0.0	0.0	6.38	HL-93	-6.38	HL-93
1	6	28.20	0.0	0.0	4.89	HL-93	-8.19	HL-93
1	7	32.90	0.0	0.0	3.45	HL-93	-10.34	HL-93
1	8	37.60	0.0	0.0	2.07	HL-93	-12.55	HL-93
1	9	42.30	0.0	0.0	0.82	HL-93	-14.82	HL-93
1	10	47.00	0.0	0.0	0.00	HL-93	-17.14	HL-93

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

SP NO	IN NO	D FROM L SUPT (ft)	T R U C K POS	NEG	RANGE
1	0	0.00	12.	0.	12.
1	1	4.70	11.	-1.	11.
1	2	9.40	9.	-2.	11.
1	3	14.10	7.	-3.	10.
1	4	18.80	6.	-4.	10.
1	5	23.50	5.	-5.	10.
1	6	28.20	4.	-6.	10.
1	7	32.90	3.	-7.	10.
1	8	37.60	2.	-9.	11.
1	9	42.30	1.	-11.	11.
1	10	47.00	0.	-12.	12.

TABLE 2.2.6.2B=MINIMUM WELD SIZE
****

SP NO	IN NO	D FROM L SUPT (ft)	TOP FLANGE Vr [1] (Kip)	TOP FLANGE Q/I [2] (1/in)	TOP FLANGE SHEAR FLOW [3]	TOP FLANGE WELD SIZE (in)	BOTTOM FLANGE Q/I (1/in)	BOTTOM FLANGE SHEAR FLOW	BOTTOM FLANGE WELD SIZE (in)
1	0	0.00	12.5	.36962E-01	0.23	0.2500*	.36962E-01	0.23	0.2500*
1	1	4.70	11.4	.36962E-01	0.21	0.2500*	.36962E-01	0.21	0.2500*
1	2	9.40	10.5	.36962E-01	0.19	0.2500*	.36962E-01	0.19	0.2500*
1	3	14.10	9.6	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	4	18.80	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	5	23.50	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	6	28.20	10.0	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	7	32.90	9.6	.36962E-01	0.18	0.2500*	.36962E-01	0.18	0.2500*
1	8	37.60	10.5	.36962E-01	0.19	0.2500*	.36962E-01	0.19	0.2500*
1	9	42.30	11.4	.36962E-01	0.21	0.2500*	.36962E-01	0.21	0.2500*
1	10	47.00	12.5	.36962E-01	0.23	0.2500*	.36962E-01	0.23	0.2500*

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

[2] For non-composite construction:

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

For composite construction:

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

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

* -- minimum weld size governs

TABLE 2.2.6.2C=WELD DESIGN BY SHEAR FORCE
****

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR FLOW St [1] (kip/in)	SHEAR FLOW Sb [2] (kip/in)	SHEAR FLOW Sg [3] (kip/in)	MIN SIZE WELD (in)	ALLOWABLE SHEAR FLOW Vw [4] (kip/in)	ALLOWABLE SHEAR FLOW Vc [5] (kip/in)	ALLOWABLE SHEAR FLOW Vg [6] (kip/in)	FL AG
1	0	0.00	1.11	1.11	1.11	0.2500	11.88	12.73	11.88	
1	1	4.70	0.96	0.96	0.96	0.2500	11.88	12.73	11.88	
1	2	9.40	0.81	0.81	0.81	0.2500	11.88	12.73	11.88	
1	3	14.10	0.67	0.67	0.67	0.2500	11.88	12.73	11.88	
1	4	18.80	0.53	0.53	0.53	0.2500	11.88	12.73	11.88	
1	5	23.50	0.41	0.41	0.41	0.2500	11.88	12.73	11.88	
1	6	28.20	0.53	0.53	0.53	0.2500	11.88	12.73	11.88	
1	7	32.90	0.67	0.67	0.67	0.2500	11.88	12.73	11.88	
1	8	37.60	0.81	0.81	0.81	0.2500	11.88	12.73	11.88	
1	9	42.30	0.96	0.96	0.96	0.2500	11.88	12.73	11.88	
1	10	47.00	1.11	1.11	1.11	0.2500	11.88	12.73	11.88	

NOTE: [1] St = top flange shear flow
= VQ/I
where : V = shear force
Q = statical moment of the area about neutral axis
I = moment of inertia

[2] Sb = bottom flange shear flow

[3] Sg = govern. shear flow (max. of St and Sb)

[4] Vw = allowable shear flow for min. size weld
= 0.6 * (phi)e2 * Fexx * 0.707 * min. weld size * 2
where: (Phi)e2 = weld resistance factor (0.8)
Fexx = class. strength of the weld metal (70 ksi)

[5] Vc = allowable shear flow on the connected material
= (Phi)v * (0.58 * Ag * Fy)
where: (Phi)v = resistance factor for shear (1.0)
Ag = gross area of the connection element
Fy = min. yield strength of connection element

[6] Vg = govern. allow. shear flow (min. of Vw and Vc)

If [6] is less than [3], then the flag ** will show up.

TABLE 2.2.6.3=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
****

SP NO	IN NO	D FROM L SUPP (ft)	TOTAL DL (kips)	TOTAL POS LL+I (kips)	GOVERN LOAD TYPE	TOTAL NEG LL+I (kips)	GOVERN LOAD TYPE	TOTAL MAX ABS (kips)	TOTAL MIN ABS (kips)
1	0	0.00	2.7	17.1	HL-93	0.0	HL-93	19.9	2.7
1	1	4.70	2.2	14.8	HL-93	-0.8	HL-93	17.0	1.3
1	2	9.40	1.6	12.6	HL-93	-2.1	HL-93	14.2	-0.4
1	3	14.10	1.1	10.3	HL-93	-3.4	HL-93	11.4	-2.4
1	4	18.80	0.5	8.2	HL-93	-4.9	HL-93	8.7	-4.3
1	5	23.50	0.0	6.4	HL-93	-6.4	HL-93	6.4	-6.4
1	6	28.20	-0.5	4.9	HL-93	-8.2	HL-93	-8.7	4.3
1	7	32.90	-1.1	3.4	HL-93	-10.3	HL-93	-11.4	2.4
1	8	37.60	-1.6	2.1	HL-93	-12.6	HL-93	-14.2	0.4
1	9	42.30	-2.2	0.8	HL-93	-14.8	HL-93	-17.0	-1.3
1	10	47.00	-2.7	0.0	HL-93	-17.1	HL-93	-19.9	-2.7

TABLE 1.2.6.4=SHEAR SUMMARY FOR NONCOMPOSITE CONSTRUCTION (LRFD)
****

SP NO	IN NO	D FROM L SUPT (ft)	SERVICE I (kips)	SERVICE II (kips)	STRENGTH I (kips)	STRENGTH II (kips)	STRENGTH IV (kips)	FATIGUE RANGE (kips)
1	0	0.00	19.9	25.0	33.4	26.6	4.1	10.0
1	1	4.70	17.0	21.4	28.6	22.7	3.3	9.1
1	2	9.40	14.2	18.0	24.0	19.0	2.5	8.4
1	3	14.10	11.4	14.5	19.4	15.3	1.6	7.7
1	4	18.80	8.7	11.2	15.0	11.7	0.8	8.0
1	5	23.50	6.4	8.3	11.2	8.6	0.0	8.0
1	6	28.20	-8.7	-11.2	-15.0	-11.7	-0.8	8.0
1	7	32.90	-11.4	-14.5	-19.4	-15.3	-1.6	7.7
1	8	37.60	-14.2	-18.0	-24.0	-19.0	-2.5	8.4

```
  1  9  42.30     -17.0       -21.4      -28.6       -22.7        -3.3       9.1
  1 10  47.00     -19.9       -25.0      -33.4       -26.6        -4.1      10.0

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 TABLE 2.2.7.2=SUMMARY OF REACTIONS (UNFACTORED)
               ********************

       TOTAL     LL+I ,(K), LOAD TYPE =  HL - 93     TOTAL DL+LL+I (L R F D)
 SUPT ---------- ------------------------------------ ------------------------
  NO. DEAD LOADS  MINIMUM  GOVERN.  MAXIMUM  GOVERN.    MINIMUM      MAXIMUM
        (K)            LOAD TYPE         LOAD TYPE
 ---------------------------------------------------------------------------

   1      2.73       0.00  HL-93      17.14  HL-93 ST1     2.46         33.41
                                                  ST2     2.46         26.55
                                                  ST4     4.10          4.10
                                                  SE1     2.73         19.87
                                                  SE2     2.73         25.01

   2      2.73       0.00  HL-93      17.14  HL-93 ST1     2.46         33.41
                                                  ST2     2.46         26.55
                                                  ST4     4.10          4.10
                                                  SE1     2.73         19.87
                                                  SE2     2.73         25.01

 ---------------------------------------------------------------------------
 NOTE: [1] " - " Indicates Uplift

          ST1 = STRENGTH I; ST2 = STRENGTH II; SE1 = SERVICE I;SE2 = SERVICE II.
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 TABLE 2.2.8.1=DEAD LOAD DEFLECTIONS FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
               ***************************************************

                    DEAD  LOAD      CONCENTRATED    UNIFORM      T O T A L
 SP IN D FROM  ----------------       LOADS         LOADS     ---------------
 NO NO L SUPT    BEAM    SLAB     -------------  ---------   ---------------
        (ft)     (in)    (in)         (in)         (in)      +=UP , -=DOWN
 ---------------------------------------------------------------------------

  1  0   0.00   0.0000   0.0000       0.0000       0.0000       0.0000
  1  1   4.70  -0.0431   0.0000      -0.0021      -0.0205      -0.0657
  1  2   9.40  -0.0816   0.0000      -0.0039      -0.0388      -0.1242
  1  3  14.10  -0.1117   0.0000      -0.0054      -0.0531      -0.1701
  1  4  18.80  -0.1308   0.0000      -0.0063      -0.0622      -0.1992
  1  5  23.50  -0.1374   0.0000      -0.0066      -0.0653      -0.2092
  1  6  28.20  -0.1308   0.0000      -0.0063      -0.0622      -0.1992
  1  7  32.90  -0.1117   0.0000      -0.0054      -0.0531      -0.1701
  1  8  37.60  -0.0816   0.0000      -0.0039      -0.0388      -0.1242
  1  9  42.30  -0.0431   0.0000      -0.0021      -0.0205      -0.0657
  1 10  47.00   0.0000   0.0000       0.0000       0.0000       0.0000


 ---------------------------------------------------------------------------
 NOTE: " - " indicates downward deflection
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 TABLE 2.0.8.1A=CAMBER INFORMATION (UNFACTORED)
                ******************

                  NONCOMPOSITE  DEAD  LOADS        OTHER  DL        T O T A L
 SP IN D FROM ---------------(IN)--------------- ------(IN)------ ------(IN)------
 NO NO L SUPT STEEL  CAMBER     SLAB    CAMBER     DEFL.  CAMBER     DEFL.  CAMBER
        (FT)  DEFL.   SIZE      DEFL.    SIZE              SIZE               SIZE
 ---------------------------------------------------------------------------------
  1  0   0.0  0.000    --       0.000     --       0.000     --       0.000    --
  1  1   4.7 -0.043  0   1/16  0.000  0   0/16 -0.023  0   1/16 -0.066  0   1/ 8
  1  2   9.4 -0.082  0   1/ 8  0.000  0   0/16 -0.043  0   1/16 -0.124  0   1/ 8
  1  3  14.1 -0.112  0   1/ 8  0.000  0   0/16 -0.058  0   1/16 -0.170  0   3/16
  1  4  18.8 -0.131  0   3/16  0.000  0   0/16 -0.068  0   1/ 8 -0.199  0   1/ 4
  1  5  23.5 -0.137  0   3/16  0.000  0   0/16 -0.072  0   1/ 8 -0.209  0   1/ 4
  1  6  28.2 -0.131  0   3/16  0.000  0   0/16 -0.068  0   1/ 8 -0.199  0   1/ 4
  1  7  32.9 -0.112  0   1/ 8  0.000  0   0/16 -0.058  0   1/16 -0.170  0   3/16
  1  8  37.6 -0.082  0   1/ 8  0.000  0   0/16 -0.043  0   1/16 -0.124  0   1/ 8
  1  9  42.3 -0.043  0   1/16  0.000  0   0/16 -0.023  0   1/16 -0.066  0   1/ 8
  1 10  47.0  0.000    --       0.000     --       0.000     --       0.000    --


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 NOTE: for camber, please refer to AASHTO' Art.10.14 or LRFD Art. 6.7.2
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     TABLE 2.2.8.2=MAX. LIVE LOAD DEFLECTIONS FOR NONCOMP. CONSTRUCTIONS
                   ***************************************************
                        (UNFACTORED)

                   NUMBER OF LANE   LL + I.
          D FROM -----AND------ DEFLECTION  GOVERN.  1/800 OF SPAN L  ROTATION
     SPAN  L SUPT  DIST. FACTOR  ------------  LOAD   --------------------
      NO.   (ft)   FOR LL DEFL.     (inch)     TYPE   AASHTO 2.5.2.6.2  [5] Rad.
     ---------------------------------------------------------------------------

       1    23.50   1    0.125     -0.593 MAX HL-93     0.70            0.02005
                                     0.065 MIN
                                    -0.144 MAX LANE
                                     0.065 MIN LANE
                                     1    0.125     -0.373 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) ..... AASHTO LRFD Art. 2.5.2.6

               where NL= no. of traffic lanes
                     Ng= no. of girders

                     No multi-presence factor applied for LL deflection

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

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

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

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

           [7] For truck rating the most probable number of lanes is assumed for
 avereaging.
 1
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  TABLE 2.2.9.1=DEAD LOAD STRESSES FOR NONCOMPOSITE CONSTRUCTION (UNFACTORED)
                ***************************************************

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

   1  0   0.00    0.00    0.00      0.00    0.00      0.00    0.00
   1  1   4.70   -0.52    0.52     -0.27    0.27     -0.79    0.79
   1  2   9.40   -0.92    0.92     -0.48    0.48     -1.40    1.40
   1  3  14.10   -1.21    1.21     -0.63    0.63     -1.84    1.84
   1  4  18.80   -1.38    1.38     -0.72    0.72     -2.10    2.10
   1  5  23.50   -1.44    1.44     -0.75    0.75     -2.19    2.19
   1  6  28.20   -1.38    1.38     -0.72    0.72     -2.10    2.10
   1  7  32.90   -1.21    1.21     -0.63    0.63     -1.84    1.84
   1  8  37.60   -0.92    0.92     -0.48    0.48     -1.40    1.40
   1  9  42.30   -0.52    0.52     -0.27    0.27     -0.79    0.79
   1 10  47.00    0.00    0.00      0.00    0.00      0.00    0.00


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

                     MAXIMUM POSITIVE,(ksi)  MAXIMUM NEGATIVE,(ksi)
  SP IN D FROM ----------------------  ----------------------
  NO NO L SUPT     ------STEEL-----         -----STEEL------
         (ft)      TOP          BOT        TOP          BOT
  ---------------------------------------------------------------

   1  0   0.00     0.00         0.00       0.00         0.00
   1  1   4.70    -4.84         4.84       0.00         0.00
   1  2   9.40    -8.34         8.34       0.00         0.00
   1  3  14.10   -10.52        10.52       0.00         0.00
   1  4  18.80   -11.76        11.76       0.00         0.00
   1  5  23.50   -11.85        11.85       0.00         0.00
   1  6  28.20   -11.76        11.76       0.00         0.00
   1  7  32.90   -10.52        10.52       0.00         0.00
   1  8  37.60    -8.34         8.34       0.00         0.00
   1  9  42.30    -4.84         4.84       0.00         0.00
   1 10  47.00     0.00         0.00       0.00         0.00


  ---------------------------------------------------------------------
 1
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  TABLE 2.2.9.2A=FATIGUE LIVE LOAD STRESS RANGE (UNFACTORED)
                ******************************

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

   1  0   0.00     0.0     0.0      0.0 T   HL      0.0     0.0      0.0 T   HL
   1  1   4.70     3.4     0.0      3.4 C   HL      3.4     0.0      3.4 T   HL
   1  2   9.40     5.6     0.0      5.6 C   HL      5.6     0.0      5.6 T   HL
   1  3  14.10     6.6     0.0      6.6 C   HL      6.6     0.0      6.6 T   HL
   1  4  18.80     7.8     0.0      7.8 C   HL      7.8     0.0      7.8 T   HL
   1  5  23.50     8.0     0.0      8.0 C   HL      8.0     0.0      8.0 T   HL
   1  6  28.20     7.8     0.0      7.8 C   HL      7.8     0.0      7.8 T   HL
   1  7  32.90     6.6     0.0      6.6 C   HL      6.6     0.0      6.6 T   HL
   1  8  37.60     5.6     0.0      5.6 C   HL      5.6     0.0      5.6 T   HL
   1  9  42.30     3.4     0.0      3.4 C   HL      3.4     0.0      3.4 T   HL
   1 10  47.00     0.0     0.0      0.0 T   HL      0.0     0.0      0.0 T   HL
```

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

1

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TABLE 2.2.9.3A=SERVICE I TOTAL (DC+DW+(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-5.62	5.62	-0.79	0.79
1	2	9.40	50.	50.	-9.74	9.74	-1.40	1.40
1	3	14.10	50.	50.	-12.36	12.36	-1.84	1.84
1	4	18.80	50.	50.	-13.85	13.85	-2.10	2.10
1	5	23.50	50.	50.	-14.04	14.04	-2.19	2.19
1	6	28.20	50.	50.	-13.85	13.85	-2.10	2.10
1	7	32.90	50.	50.	-12.36	12.36	-1.84	1.84
1	8	37.60	50.	50.	-9.74	9.74	-1.40	1.40
1	9	42.30	50.	50.	-5.62	5.62	-0.79	0.79
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3B=SERVICE II TOTAL (DC+DW+1.3(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.08	7.08	-0.79	0.79
1	2	9.40	50.	50.	-12.25	12.25	-1.40	1.40
1	3	14.10	50.	50.	-15.52	15.52	-1.84	1.84
1	4	18.80	50.	50.	-17.38	17.38	-2.10	2.10
1	5	23.50	50.	50.	-17.59	17.59	-2.19	2.19
1	6	28.20	50.	50.	-17.38	17.38	-2.10	2.10
1	7	32.90	50.	50.	-15.52	15.52	-1.84	1.84
1	8	37.60	50.	50.	-12.25	12.25	-1.40	1.40
1	9	42.30	50.	50.	-7.08	7.08	-0.79	0.79
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3C=STRENGTH I TOTAL (1.25DC+1.50DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-8.07	8.07	0.39	-0.39
1	2	9.40	50.	50.	-13.90	13.90	0.70	-0.70
1	3	14.10	50.	50.	-17.50	17.50	0.92	-0.92
1	4	18.80	50.	50.	-19.52	19.52	1.05	-1.05
1	5	23.50	50.	50.	-19.64	19.64	1.09	-1.09
1	6	28.20	50.	50.	-19.52	19.52	1.05	-1.05
1	7	32.90	50.	50.	-17.50	17.50	0.92	-0.92
1	8	37.60	50.	50.	-13.90	13.90	0.70	-0.70
1	9	42.30	50.	50.	-8.07	8.07	0.39	-0.39
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3D=STRENGTH I TOTAL (0.90DC+0.65DW+1.75(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.79	7.79	0.67	-0.67
1	2	9.40	50.	50.	-13.41	13.41	1.19	-1.19
1	3	14.10	50.	50.	-16.85	16.85	1.56	-1.56
1	4	18.80	50.	50.	-18.79	18.79	1.78	-1.78
1	5	23.50	50.	50.	-18.88	18.88	1.86	-1.86
1	6	28.20	50.	50.	-18.79	18.79	1.78	-1.78
1	7	32.90	50.	50.	-16.85	16.85	1.56	-1.56
1	8	37.60	50.	50.	-13.41	13.41	1.19	-1.19
1	9	42.30	50.	50.	-7.79	7.79	0.67	-0.67
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3E=STRENGTH II TOTAL (1.25DC+1.50DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.51	7.51	-0.99	0.99
1	2	9.40	50.	50.	-13.01	13.01	-1.75	1.75
1	3	14.10	50.	50.	-16.51	16.51	-2.30	2.30
1	4	18.80	50.	50.	-18.49	18.49	-2.62	2.62
1	5	23.50	50.	50.	-18.73	18.73	-2.74	2.74
1	6	28.20	50.	50.	-18.49	18.49	-2.62	2.62
1	7	32.90	50.	50.	-16.51	16.51	-2.30	2.30
1	8	37.60	50.	50.	-13.01	13.01	-1.75	1.75
1	9	42.30	50.	50.	-7.51	7.51	-0.99	0.99
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

TABLE 2.2.9.3F=STRENGTH II TOTAL (0.90DC+0.65DW+1.35(LL+I)) STRESS SUMMARY

SP NO	IN D NO L	FROM (ft)	YIELD STRESS Fy (ksi)	MAXIMUM POSITIVE,(ksi) TOP	MAXIMUM POSITIVE,(ksi) BOT	MAXIMUM NEGATIVE,(ksi) TOP	MAXIMUM NEGATIVE,(ksi) BOT
1	0	0.00	50.	50.	0.00	0.00	0.00	0.00
1	1	4.70	50.	50.	-7.24	7.24	-0.71	0.71
1	2	9.40	50.	50.	-12.52	12.52	-1.26	1.26
1	3	14.10	50.	50.	-15.86	15.86	-1.65	1.65
1	4	18.80	50.	50.	-17.76	17.76	-1.89	1.89
1	5	23.50	50.	50.	-17.97	17.97	-1.97	1.97
1	6	28.20	50.	50.	-17.76	17.76	-1.89	1.89
1	7	32.90	50.	50.	-15.86	15.86	-1.65	1.65
1	8	37.60	50.	50.	-12.52	12.52	-1.26	1.26
1	9	42.30	50.	50.	-7.24	7.24	-0.71	0.71
1	10	47.00	50.	50.	0.00	0.00	0.00	0.00

1

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TABLE 2.2.22.3=MEMBER LENGTH AND SECTION GEOMETRY

MEMBER NO	LENGTH (ft)	TOP FLANGE TYPE	TOP FLANGE WIDTH (in)	TOP FLANGE THICK (in)	WEB DEPTH (in)	WEB THICK (in)	BOT FLANGE WIDTH (in)	BOT FLANGE THICK (in)	COVER PLATE -- (in) TOP WIDTH	TOP THICK	BOT WIDTH	BOT THICK
1	47.00	W	9.0	0.6800	22.5	0.4390	9.0	0.6800	0.00	0.0000	0.00	0.0000

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

1

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TABLE 2.2.22.4=DEPTH RATIOS

** [1] = Span length (ft) = 47.0

MEMBER NO	SLAB THICK. (in)	GIRDER DEPTH (in)	[2]=OVERALL D. [3]= GIRDER D.	[2]/(12*[1]) [3]/(12*[1])	LIMITATION 1/25 1/30	<=>	STATUS
1	47.00	23.89	[2] does not apply				
			[3]=	23.89	0.04236	>	0.03333 OK

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

[2] -- AASHTO LRFD TABLE 2.5.2.6.3-1

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

[3] -- same as NOTE [2]

** -- cover plates not taken into account

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

1

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

SP NO	IN D NO L	FROM (ft)	Lo ng [1]	Co mp	Web depth	Web thick	D/tw	2Dcp/tw [2]	2Dcp/tw [3]	2Dc/tw [4]	2Dc/tw [5]	2Dc/tw [6]	Web Cat
1	0	0.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	1	4.70	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	2	9.40	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	3	14.10	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	4	18.80	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	5	23.50	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	6	28.20	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	7	32.90	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	8	37.60	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	9	42.30	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0
1	10	47.00	0	1	22.53	0.439	51.3	150.0	0.00	90.55			0

NOTE:

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

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

1

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

SP NO	IN NO	D L	FROM SUPT	Lo [1]	Co ng	mp	Web depth	thick	2Dc/tw [2]	Web [3]	Category
1	0		0.00	0	1		22.53	0.439	51.32	137.27	2
1	1		4.70	0	1		22.53	0.439	51.32	137.27	2
1	2		9.40	0	1		22.53	0.439	51.32	137.27	2
1	3		14.10	0	1		22.53	0.439	51.32	137.27	2
1	4		18.80	0	1		22.53	0.439	51.32	137.27	2
1	5		23.50	0	1		22.53	0.439	51.32	137.27	2
1	6		28.20	0	1		22.53	0.439	51.32	137.27	2
1	7		32.90	0	1		22.53	0.439	51.32	137.27	2
1	8		37.60	0	1		22.53	0.439	51.32	137.27	2
1	9		42.30	0	1		22.53	0.439	51.32	137.27	2
1	10		47.00	0	1		22.53	0.439	51.32	137.27	2

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

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

1

TABLE 2.2.22.6=FLANGE PROPORTIONS CHECK

SP NO	IN NO	D L	FROM SUPT (ft)	bf/2tf [1]	bf (in)	[2] (in)	tf (in)	[3] (in)	Iyc/Iyt	FLAG
1	0		0.00	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	1		4.70	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	2		9.40	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	3		14.10	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	4		18.80	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	5		23.50	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	6		28.20	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	7		32.90	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	8		37.60	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	9		42.30	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0
1	10		47.00	6.61	12.	9.0	3.8	0.680	0.483	
				6.61	12.	9.0	3.8	0.680	0.483	1.00	0

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

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

1

TABLE 1.2.22.7.0=CB VALUES FOR LATERAL BRACING

LATERAL BRACING NO	DIST. LEFT ----(FT)---- FROM	TO	fo (ksi)	f2 (ksi)	fmid (ksi)	f1 (ksi)	Cb
1	0.0	10.0	0.000	16.908	9.889	2.870	1.580
2	10.0	20.0	16.908	23.267	21.190	19.113	1.090
3	20.0	30.0	22.246	23.267	23.386	23.504	1.000
4	30.0	40.0	12.826	22.246	18.765	15.284	1.170

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

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TABLE 1.2.22.7A=FLB AND LTB CATEGORIES

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	FLB LMDAf	FLB LMDApf	FLB LMDArf	LTB CAT	Lb (ft)	Lp (ft)	Lr (ft)	GOV CAT	GOV CAT
1	0		0.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	1		4.70	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	2		9.40	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	3		14.10	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	4		18.80	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	5		23.50	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	6		28.20	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	7		32.90	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	8		37.60	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	9		42.30	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2
1	10		47.00	1	6.61	9.15	16.12	0	10.00	4.62	17.35	2	2

NOTE: LMDAf = slenderness ratio for the comp. flange
= bfc / 2tfc (Eq. 6.10.8.2.2-3)
LMDApf = limiting slenderness ratio for a compact flange
= 0.38*SQRT(E/Fyc) (Eq. 6.10.8.2.2-4)
LMDArf = limiting slenderness ration for a non-compct flange
= 0.56*SQRT(E/Fyr) (Eq. 6.10.8.2.2-5)
Lb = unbraced length
Lp = limiting unbraced length to achieve the nominal flexural of RbRhFyc under uniform bending
= 1.0rt*SQRT(E/Fyc) (Eq. 6.10.8.2.3-4)
Lr = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of comp. flange residual stress effects
= pi * rt * SQRT(E/Fyr) (Eq. 6.10.8.2.3-5)

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

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

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TABLE 1.2.22.7B=FLB AND LTB RESISTANCE

SP NO	IN NO	D L	FROM SUPT (ft)	Co mp	Rh	Rb	Cb	FLB Fnc (Ksi)	LTB Fnc (Ksi)	GOV Fnc (Ksi)
1	0		0.00	1	1.000	1.000	1.58	50.00	50.00	50.00
1	1		4.70	1	1.000	1.000	1.58	50.00	50.00	50.00
1	2		9.40	1	1.000	1.000	1.58	50.00	50.00	50.00
1	3		14.10	1	1.000	1.000	1.09	50.00	47.59	47.59
1	4		18.80	1	1.000	1.000	1.09	50.00	47.59	47.59
1	5		23.50	1	1.000	1.000	1.00	50.00	43.66	43.66
1	6		28.20	1	1.000	1.000	1.00	50.00	43.66	43.66
1	7		32.90	1	1.000	1.000	1.17	50.00	50.00	50.00
1	8		37.60	1	1.000	1.000	1.17	50.00	50.00	50.00
1	9		42.30	1	1.000	1.000	1.00	50.00	43.66	43.66
1	10		47.00	1	1.000	1.000	1.00	50.00	43.66	43.66

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

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TABLE 2.2.22.9=SUMMARY OF STRENGTH CATEGORY OF CROSS SECTION

SP NO	IN NO	D L	FROM SUPT (ft)	Section Region	STRENGTH CATEGORY, Noncomposite Web [1]	Noncomposite Flange [2]	Composite Web [3]	Composite Flange [4]	Non-Comp.
1	0		0.00	1	2	2			2
1	1		4.70	1	2	2			2
1	2		9.40	1	2	2			2
1	3		14.10	1	2	2			2
1	4		18.80	1	2	2			2
1	5		23.50	1	2	2			2
1	6		28.20	1	2	2			2
1	7		32.90	1	2	2			2
1	8		37.60	1	2	2			2
1	9		42.30	1	2	2			2
1	10		47.00	1	2	2			2

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1

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

* Strength Category of Cross Section

0 = compact section

2 = non-compact section

3 = slender section

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

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

1
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TABLE 2.2.22.10=CONSTRUCTIBILITY CHECK (STRENGTH IV)
****

SP NO	IN NO	D (ft)	FROM SUPT (ft)	fl (ksi)	0.6Fyt (ksi)	fbu (ksi)	[1] (ksi)	fbu+fl (ksi)	[2] (ksi)	fbu+1/3fl (ksi)	[3] (ksi)	FLAG
1	0	0.00		0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	1	4.70		0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	2	9.40		0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	3	14.10		0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.6	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	4	18.80		0.0	30.0	0.0	50.0	0.0	50.0	0.0	47.6	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	5	23.50		0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	6	28.20		0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	7	32.90		0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	8	37.60		0.0	30.0	0.0	50.0	0.0	50.0	0.0	50.0	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	9	42.30		0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0
1	10	47.00		0.0	30.0	0.0	50.0	0.0	50.0	0.0	43.7	0
				0.0	-	0.0	-	0.0	50.0	-	-	0

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

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

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

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

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

1
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TABLE 2.2.22.10A=RATIO OF APPLIED STRESS AND THE CAPACITY
****

SP NO	IN NO	D (ft)	FROM SUPT	fl/ 0.6Fyt	fbu/ [1]	fbu+fl/ [2]	fbu+1/3fl/ [3]	MAX. RAT.	GOVN.
1	0	0.00		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	1	4.70		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	2	9.40		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	3	14.10		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	4	18.80		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	5	23.50		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	6	28.20		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	7	32.90		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	8	37.60		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	9	42.30		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3
1	10	47.00		0.000	0.000	0.000	0.000	0.000	1
						0.000		0.000	3

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

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

1
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TABLE 2.2.22.14=STRENGTH LIMIT STATE CHECK
****

SP NO	IN NO	D (ft)	FROM SUPT	ID	Mu+1/3flSxt (k-ft)	[1] (k-ft)	fbu (ksi)	[2] (ksi)	fl (ksi)	[3] (ksi)	fbu+ 1/3fl (ksi)	[4] (ksi)	FLAG MCTD
1	0	0.00		0	-	-	0.0	-	0.0	30.0	0.0	50.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-
1	1	4.70		0	-	-	9.4	-	0.0	30.0	9.4	50.0	-0--
					-	-	9.4	-	0.0	30.0	9.4	50.0	--0-
1	2	9.40		0	-	-	16.4	-	0.0	30.0	16.4	50.0	-0--
					-	-	16.4	-	0.0	30.0	16.4	50.0	--0-
1	3	14.10		0	-	-	20.7	-	0.0	30.0	20.7	47.6	-0--
					-	-	20.7	-	0.0	30.0	20.7	50.0	--0-
1	4	18.80		0	-	-	23.2	-	0.0	30.0	23.2	47.6	-0--
					-	-	23.2	-	0.0	30.0	23.2	50.0	--0-
1	5	23.50		0	-	-	23.5	-	0.0	30.0	23.5	43.7	-0--
					-	-	23.5	-	0.0	30.0	23.5	50.0	--0-
1	6	28.20		0	-	-	23.2	-	0.0	30.0	23.2	43.7	-0--
					-	-	23.2	-	0.0	30.0	23.2	50.0	--0-
1	7	32.90		0	-	-	20.7	-	0.0	30.0	20.7	50.0	-0--
					-	-	20.7	-	0.0	30.0	20.7	50.0	--0-
1	8	37.60		0	-	-	16.4	-	0.0	30.0	16.4	50.0	-0--
					-	-	16.4	-	0.0	30.0	16.4	50.0	--0-
1	9	42.30		0	-	-	9.4	-	0.0	30.0	9.4	43.7	-0--
					-	-	9.4	-	0.0	30.0	9.4	50.0	--0-
1	10	47.00		0	-	-	0.0	-	0.0	30.0	0.0	45.0	-0--
					-	-	0.0	-	0.0	30.0	0.0	50.0	--0-

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

"-" is N.A.

Under FLAG Column, 0 = OK; 1= NG
M = Moment; C = Comp. Flange; T = Tension Flange
D = Ductility; Mu = maximum of Strengh I, II and IV

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

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

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

1
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TABLE 2.2.22.14A=RATIO OF APPLIED STRESS/MOMENT AND THE CAPACITY
****

SP NO	IN NO	D (ft)	FROM SUPT	ID	Mu+1/3flSxt/ [1]	fbu/ [2]	fl/ [3]	fbu+1/3fl/ [4]	MAX. RAT.	GOVN.
1	0	0.00		0			0.000	0.000	0.000	4
							0.000	0.000	0.000	4
1	1	4.70		0			0.000	0.189	0.189	4
							0.000	0.189	0.189	4
1	2	9.40		0			0.000	0.327	0.327	4
							0.000	0.327	0.327	4
1	3	14.10		0			0.000	0.435	0.435	4
							0.000	0.414	0.414	4
1	4	18.80		0			0.000	0.487	0.487	4
							0.000	0.464	0.464	4
1	5	23.50		0			0.000	0.538	0.538	4
							0.000	0.469	0.469	4
1	6	28.20		0			0.000	0.531	0.531	4
							0.000	0.464	0.464	4
1	7	32.90		0			0.000	0.414	0.414	4
							0.000	0.414	0.414	4
1	8	37.60		0			0.000	0.327	0.327	4
							0.000	0.327	0.327	4
1	9	42.30		0			0.000	0.216	0.216	4
							0.000	0.189	0.189	4
1	10	47.00		0			0.000	0.000	0.000	4
							0.000	0.000	0.000	4

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

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

1

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TABLE 2.2.22.15=UNSTIFFENED SECTION SHEAR CAPACITY

SP NO	IN NO	D L (ft)	FROM SUPT	Fy (ksi)	k [1]	Art.6.10.9.2 [2]	Art.6.10.9.2 [3]	D/tw [4]	C	Vp (ksi)	SHEAR CAPACITY -Eq.6.10.9.2-- Vn = C x Vp
1	0	0.00	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	1	4.70	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	2	9.40	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	3	14.10	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	4	18.80	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	5	23.50	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	6	28.20	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	7	32.90	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	8	37.60	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	9	42.30	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8
1	10	47.00	50.0	5.0	60.3	75.4	51.3	1.000	286.8		286.8

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1

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

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

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

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

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

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

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

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

Vu = CVp

where D = clear, unsupported distance between flange components

d0 = distance between transverse stiffeners

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
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TABLE 2.2.22.16=SUMMARY OF WEB STRENGTH CATEGORY

SP NO	IN NO	D L (ft)	FROM SUPT	WEB STABILITY UNSTIFFENED,[1] D/tw		LFD/LRFD MAX. SHEAR (kip)	<=>	UNSTIFFENED SHEAR CAPA- CITY, [2]	CATEGORY	OVERALL WEB
1	0	0.00		2		33.4	<	286.8	2	2
1	1	4.70		2		28.6	<	286.8	2	2
1	2	9.40		2		24.0	<	286.8	2	2
1	3	14.10		2		19.4	<	286.8	2	2
1	4	18.80		2		15.0	<	286.8	2	2
1	5	23.50		2		11.2	<	286.8	2	2
1	6	28.20		2		15.0	<	286.8	2	2
1	7	32.90		2		19.4	<	286.8	2	2
1	8	37.60		2		24.0	<	286.8	2	2
1	9	42.30		2		28.6	<	286.8	2	2
1	10	47.00		2		33.4	<	286.8	2	2

Please read NOTE on the following page

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

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

For compact section (composite sections in positive flexure)

w/o long. stiff.

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

w / long. stiff.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

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TABLE 2.2.22.17=TRANSVERSE STIFFENER SPACING

SP NO	IN NO	D L (ft)	FROM SUPT	YIELD STRESS Fy (ksi)	LFD/LRFD UNSTIFFENED MAXIMUM SHEAR (k)	REQUIREMENT OF SHEAR CAPACITY (k)	MAX. ALLOWABLE TRANS. STIFFENERS 1=YES , 0=NO	SPACING (ft-in)
1	0	0.00	50.	33.41	286.83		0	
1	1	4.70	50.	28.64	286.83		0	
1	2	9.40	50.	24.01	286.83		0	
1	3	14.10	50.	19.44	286.83		0	
1	4	18.80	50.	15.01	286.83		0	
1	5	23.50	50.	11.19	286.83		0	
1	6	28.20	50.	15.01	286.83		0	
1	7	32.90	50.	19.44	286.83		0	
1	8	37.60	50.	24.01	286.83		0	
1	9	42.30	50.	28.64	286.83		0	
1	10	47.00	50.	33.41	286.83		0	

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

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TABLE 2.2.22.19A=SPECIAL FATIGUE REQUIREMENTS FOR WEBS

SP NO	IN NO	D L (ft)	FROM SUPT	SHEAR DL (k)	SDL (k)	FACTORED FATIGUE PERM. V (k)	<=>	Vcr (k)	<=>	UNF. PERM + FAC FATIG V (k)
1	0	0.00		2.73	0.00	12.47	3.41 <	286.83	>	24.55
1	1	4.70		2.17	0.00	10.52	2.71 <	286.83	>	20.57
1	2	9.40		1.64	0.00	8.70	2.05 <	286.83	>	16.87
1	3	14.10		1.07	0.00	6.89	1.34 <	286.83	>	13.14
1	4	18.80		0.55	0.00	6.11	0.68 <	286.83	>	11.24
1	5	23.50		0.02	0.00	4.98	0.02 <	286.83	>	8.74
1	6	28.20		0.55	0.00	6.11	0.68 <	286.83	>	11.24
1	7	32.90		1.07	0.00	6.89	1.34 <	286.83	>	13.14
1	8	37.60		1.64	0.00	8.70	2.05 <	286.83	>	16.87
1	9	42.30		2.17	0.00	10.52	2.71 <	286.83	>	20.57
1	10	47.00		2.73	0.00	12.47	3.41 <	286.83	>	24.55

NOTE: This table checks Art. 6.10.3.3 and Art. 6.10.5.3.
Vcr is the shear-buckling resistance determined from Eq. 6.10.9.3.3-1.

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5350'x17'\Merlin-Da

TABLE 2.2.22.21=SERVICE LIMIT STATE CHECK
****

SP NO	IN NO	D L	FROM SUPT (ft)	TOP FLANGE ff (ksi)	ff+fl/2 (ksi)	[1]	RATIO1	BOT. FLANGE ff+fl/2 (ksi)	[2]	RATIO2	COMP. fc (ksi)	FLG fcrw (ksi)	[3]	FLAG TBC
1	0		0.00	-	0.0	40.0	0.000	0.0	40.0	Infin	0.0	50.0	000	
1	1		4.70	-	7.1	40.0	0.177	7.1	40.0	Infin	7.1	50.0	000	
1	2		9.40	-	12.2	40.0	0.306	12.2	40.0	Infin	12.2	50.0	000	
1	3		14.10	-	15.5	40.0	0.388	15.5	40.0	Infin	15.5	50.0	000	
1	4		18.80	-	17.4	40.0	0.435	17.4	40.0	Infin	17.4	50.0	000	
1	5		23.50	-	17.6	40.0	0.440	17.6	40.0	Infin	17.6	50.0	000	
1	6		28.20	-	17.4	40.0	0.435	17.4	40.0	Infin	17.4	50.0	000	
1	7		32.90	-	15.5	40.0	0.388	15.5	40.0	Infin	15.5	50.0	000	
1	8		37.60	-	12.2	40.0	0.306	12.2	40.0	Infin	12.2	50.0	000	
1	9		42.30	-	7.1	40.0	0.177	7.1	40.0	Infin	7.1	50.0	000	
1	10		47.00	-	0.0	40.0	0.000	0.0	40.0	NaN	0.0	50.0	000	

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5450'x17'\Merlin-Da

NOTE: [1] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-1)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[2] = 0.95RhFyf for composite sections (Eq. 6.10.4.2.2-2)
= 0.80RhFyf for non-composite sections (Eq. 6.10.4.2.2-3)
[3] = 0.9Ek / (D/tw)^2 (Eq. 6.10.1.9.1-1)
but not to exceed the smaller of RhFyc and Fyw/0.7
k = bending-buckling coefficient
= 9 / (Dc/D)^2 (Eq. 6.10.1.9.1-2)
where:
Dc = depth of the web in compression in the elastic range. For composite sections, Dc shall be determined as specified in Article D6.3.1

"-" is N.A.

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

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

RATIO1 = ff / [1] or (ff + fl/2) / [1] (top flange)
RATIO2 = (ff + fl/2) / [2] (bot. flange)

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5550'x17'\Merlin-Da

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

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY
1	0		0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1		4.70	TR	-5.9	A B B^C^C D E E^	TR	5.9	A B B^C^C D
1	2		9.40	TR	-9.8	A B B^C^C D E E^	TR	9.8	A B B^C^C
1	3		14.10	TR	-11.6	A B B^C^C D E E^	TR	11.6	A B B^C^
1	4		18.80	TR	-13.7	A B B^C^C D E E^	TR	13.7	A B
1	5		23.50	TR	-14.0	A B B^C^C D E E^	TR	14.0	A B
1	6		28.20	TR	-13.7	A B B^C^C D E E^	TR	13.7	A B
1	7		32.90	TR	-11.6	A B B^C^C D E E^	TR	11.6	A B B^C^
1	8		37.60	TR	-9.8	A B B^C^C D E E^	TR	9.8	A B B^C^C
1	9		42.30	TR	-5.9	A B B^C^C D E E^	TR	5.9	A B B^C^C D
1	10		47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue I Limit State with 1.75 load factor. Design for Infinite Life

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

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5650'x17'\Merlin-Da

TABLE 2.2.22.23.2=FATIGUE II STRESS RANGE FOR FINITE LIFE (FACTORED)
****

(1) Main (Longitudinal) Load Carrying Members

(2) Road Type = III --- Other Rural

SP NO	IN NO	D L	FROM SUPT (ft)	TOP OF TOP FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY	BOTTOM OF BOTTOM FLANGE GOVERN. STRESS LOADING RANGE	ACCEPTABLE STRESS (ksi)	CATEGORY
1	0		0.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^
1	1		4.70	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	2		9.40	TR	-4.5	A B B^C^C D E E^	TR	4.5	A B B^C^C
1	3		14.10	TR	-5.3	A B B^C^C D E E^	TR	5.3	A B B^C^C
1	4		18.80	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B
1	5		23.50	TR	-6.4	A B B^C^C D E E^	TR	6.4	A B
1	6		28.20	TR	-6.3	A B B^C^C D E E^	TR	6.3	A B
1	7		32.90	TR	-5.3	A B B^C^C D E E^	TR	5.3	A B B^C^C
1	8		37.60	TR	-4.5	A B B^C^C D E E^	TR	4.5	A B B^C^C
1	9		42.30	TR	-2.7	A B B^C^C D E E^	TR	2.7	A B B^C^C D E
1	10		47.00	TR	0.0	A B B^C^C D E E^	TR	0.0	A B B^C^C D E E^

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

NOTE: TR = Truck loading; LRFD Fatigue II Limit State with 0.80 load factor. Design for Finite Life w/ ADTT Single Lane= 1000 & No. of Cycles= 27375000
* If ADTT Single Lane greater than or equal to 960, refer to Fatigue I Table.

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

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
CODE CHECK

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5750'x17'\Merlin-Da

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

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

MORE TABLES TO BE INSPECTED ...

1.2.22.23A=FATIGUE STRESS RANGE FOR TRUCK (UNFACTORED)

1.2.22.24=SHEAR CONNECTOR (FATIGUE CRITERIA) (UNFACTORED)

1.2.22.24A=SHEAR CONNECTOR (ULTIMATE STRENGTH CRITERIA)

1.2.22.29=SPLICE DESIGN AT SECTION CHANGE POINTS

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5850'x17'\Merlin-Da

TABLE 2.2.32.1=RATING; MAXIMUM STRENGTH FOR MOMENT
****

SP NO	IN NO	D L	FROM SUPT (ft)	MOMENT CAP. (k-ft) or STRESS	UNFACTORED DEAD LOAD MOMENT D1; k-ft	ksi	D2	UNFACTORED L+I MAX. MOMENT POS.	NEG.	OPERAT. RATING [3]	LIVE FACTOR	INVENT. RATING TYPE	LIVE FACTOR	C A T
1	0		0.00	50.00	TOP	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
				50.00	BOT	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
1	1		4.70	50.00	TOP	-0.79		-4.84	0.00	0.00	7.51	HL-93	5.79	0
				50.00	BOT	0.79		4.84	0.00	0.00	7.51	HL-93	5.79	0
1	2		9.40	50.00	TOP	-1.40		-8.34	0.00	0.00	4.28	HL-93	3.30	0
				50.00	BOT	1.40		8.34	0.00	0.00	4.28	HL-93	3.30	0
1	3		14.10	47.59	TOP	-1.84		-10.52	0.00	0.00	3.19	HL-93	2.46	0
				50.00	BOT	1.84		10.52	0.00	0.00	3.36	HL-93	2.59	0
1	4		18.80	47.59	TOP	-2.10		-11.76	0.00	0.00	2.83	HL-93	2.19	0
				50.00	BOT	2.10		11.76	0.00	0.00	2.99	HL-93	2.30	0
1	5		23.50	43.66	TOP	-2.19		-11.85	0.00	0.00	2.56	HL-93	1.97	0
				50.00	BOT	2.19		11.85	0.00	0.00	2.95	HL-93	2.28	0
1	6		28.20	43.66	TOP	-2.10		-11.76	0.00	0.00	2.59	HL-93	1.99	0
				50.00	BOT	2.10		11.76	0.00	0.00	2.99	HL-93	2.30	0
1	7		32.90	50.00	TOP	-1.84		-10.52	0.00	0.00	3.36	HL-93	2.59	0
				50.00	BOT	1.84		10.52	0.00	0.00	3.36	HL-93	2.59	0
1	8		37.60	50.00	TOP	-1.40		-8.34	0.00	0.00	4.28	HL-93	3.30	0
				50.00	BOT	1.40		8.34	0.00	0.00	4.28	HL-93	3.30	0
1	9		42.30	43.66	TOP	-0.79		-4.84	0.00	0.00	6.54	HL-93	5.04	0
				50.00	BOT	0.79		4.84	0.00	0.00	7.51	HL-93	5.79	0
1	10		47.00	44.99	TOP	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0
				50.00	BOT	0.00		0.00	0.00	0.00	99.99	HL-93	99.99	0

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1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED
RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 5950'x17'\Merlin-Da

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

1

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DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6050'x17'\Merlin-Da

TABLE 2.2.32.2=RATING; SERVICEABILITY STRENGTH

SP NO	IN NO	D FROM L SUPT (ft)	0.95Fy OR 0.80Fy (ksi)	UNFACTORED DEAD LOAD ---STRESS--- STEEL TOP;ksi;BOT	UNFACTORED L+I STRESS MAX. POS. ---STEEL--- TOP;ksi;BOT	MAX. NEG. --- STEEL--- TOP;ksi;BOT	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	40.0	0.00	0.00	0.00	99.99	HL-93	76.92
1	1	4.70	40.0	-0.79	0.79	0.00	8.11	HL-93	6.24
1	2	9.40	40.0	-1.40	1.40	0.00	4.63	HL-93	3.56
1	3	14.10	40.0	-1.84	1.84	0.00	3.63	HL-93	2.79
1	4	18.80	40.0	-2.10	2.10	0.00	3.22	HL-93	2.48
1	5	23.50	40.0	-2.19	2.19	0.00	3.19	HL-93	2.45
1	6	28.20	40.0	-2.10	2.10	0.00	3.22	HL-93	2.48
1	7	32.90	40.0	-1.84	1.84	0.00	3.63	HL-93	2.79
1	8	37.60	40.0	-1.40	1.40	0.00	4.63	HL-93	3.56
1	9	42.30	40.0	-0.79	0.79	0.00	8.11	HL-93	6.24
1	10	47.00	40.0	0.00	0.00	0.00	99.99	HL-93	76.92

Please read NOTES on the following page

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6150'x17'\Merlin-Da

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

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

[3]: RATING FACTOR 99.99 INDICATES THAT THE CURRENT
SECTION IS NOT GOVERNING AT ALL.

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

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

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6250'x17'\Merlin-Da

TABLE 2.2.32.3A=BRIDGE MOMENT RATING INFORMATION FOR AASHTO TRUCK

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	CRITERION	RATING	OPERATING INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	47.00	23.50	STRENGTH L.S.	2.56	1.97	HL-93
1	47.00	23.50	SERVICEBILITY	3.19	2.45	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating= 2.558
Inventory= 1.974

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

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	CRITERION	RATING	LEGAL LOAD MAXIMUM LL MOMENT (k-ft)
1	47.00	23.50	STRENGTH L.S.	4.49	133.68
1	47.00	23.50	SERVICEBILITY	4.14	

NOTE: control Rating Factor for Current Bridge:
SERVICEBILITY with Live Load Factor =1.300 & ADTT = 1000
Legal Load Rating= 4.144

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6350'x17'\Merlin-Da

TABLE 2.2.33.1=RATING; MAXIMUM STRENGTH FOR SHEAR

SP NO	IN NO	D FROM L SUPT (ft)	SHEAR CAP. (kips)	UNFACTORED DEAD LOAD SHEAR D1; kips	D2	UNFACTORED L+I MAX. SHEAR POS.	NEG.	OPERAT. RATING FACTOR [3]	LIVE LOAD TYPE	INVENT. RATING FACTOR
1	0	0.00	286.8	0.0	2.7	17.1	0.0	12.22	HL-93	9.43
1	1	4.70	286.8	0.0	2.2	14.8	-0.8	14.18	HL-93	10.94
1	2	9.40	286.8	0.0	1.6	12.6	-2.1	16.78	HL-93	12.95
1	3	14.10	286.8	0.0	1.1	10.3	-3.4	20.43	HL-93	15.76
1	4	18.80	286.8	0.0	0.5	8.2	-4.9	25.88	HL-93	19.96
1	5	23.50	286.8	0.0	0.0	6.4	-6.4	33.29	HL-93	25.68
1	6	28.20	286.8	0.0	-0.5	4.9	-8.2	25.88	HL-93	19.96
1	7	32.90	286.8	0.0	-1.1	3.4	-10.3	20.43	HL-93	15.76
1	8	37.60	286.8	0.0	-1.6	2.1	-12.6	16.78	HL-93	12.95
1	9	42.30	286.8	0.0	-2.2	0.8	-14.8	14.18	HL-93	10.94
1	10	47.00	286.8	0.0	-2.7	0.0	-17.1	12.22	HL-93	9.43

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6450'x17'\Merlin-Da

TABLE 2.2.33.3A=BRIDGE SHEAR RATING INFORMATION FOR AASHTO TRUCK

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	CRITERION	RATING	OPERATING INVENTORY RATING	GOVERNING LIVE LOAD TYPE
1	47.00	47.00	STRENGTH L.S.	12.22	9.43	HL-93

NOTE: Control Rating Factor for Current Bridge:
STRENGTH L.S.
Operating=12.219
Inventory= 9.426

TABLE 2.2.33.3B=BRIDGE SHEAR RATING INFORMATION FOR DUMP TRUCK FA

NO.	SPAN LENGTH (ft)	CRITICAL LOCATION GOVERNING D FROM L SUPT (ft)	CRITERION	RATING	LEGAL LOAD MAXIMUM LL SHEAR (kips)
1	47.00	47.00	STRENGTH L.S.	19.87	14.23

NOTE: control Rating Factor for Current Bridge:
STRENGTH L.S. with Live Load Factor =1.300 & ADTT = 1000
Legal Load Rating=19.866

1

BRIDGE ENGINEERING SOFTWARE & TECHNOLOGY CENTER — MERLIN V10.11
DEPARTMENT OF CIVIL & ENVIRON. ENGINEERING — NONCOMPOSITE
UNIVERSITY OF MARYLAND — LRFD- 8TH ED RATING

FILE NAME = T:\Projects\New Hampshire\Hansen Construction\TemporaryPAGE 6550'x17'\Merlin-Da

TABLE 2.2.34.1=RATING; LIVE LOAD DEFLECTION

SPAN NO	D FROM L SUPT (ft)	NUMBER OF LANE -----AND------ I. DIST. FACTOR FOR LL DEFL.	LL + I. DEFLECTION (inch)	GOVERN. LOAD TYPE	RATING FACTOR FOR LL DEFL. RF = L/(800 * LL DEFL.)
1	23.50	1	0.125	-0.59	HL-93 1.19
		1	0.125	-0.59	1.19

NOTE: [1] "-" indicates downward deflection

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

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

TEMPORARY BRIDGE SUBSTRUCTURE DESIGN

Swanzey, NH (Rabbit Hollow Road over Perry Brook - Hansen Construction)

Notes
No borings were taken at temporary bridge abutment locations
The temporary bridge abutments will be set at the top of the embankments and will not impact the brook.
Use boring information supplied in plans prepared for proposed bridge design.

Boring B-1: Avg. blow count for 0' - 7' depth

$BC_{B1} := 4$

Boring B-2: Avg. blow count for 0' - 7' depth

$BC_{B2} := \frac{(3+4+4+3+4+4+3+3)}{8} = 3.5$

Controlling blow count for design:

$BC := min(BC_{B1}, BC_{B2}) = 3.5$

Conservatively use:

$\phi_f := 28 \cdot deg$ Friction Angle

Allowable Bearing Capacity (Strip footing):
Using Terzaghi Bearing Capcacity:

$\gamma_c := 100\ pcf$ $D_f := 2\ ft$ $q_{bar} := \gamma_c \cdot D_f = 200\ psf$

$N_q := 14.7$ $B := 6\ ft$ $L := 20\ ft$

$N_\gamma := 16.7$ $S_\gamma := 1 - 0.4 \cdot \frac{B}{L} = 0.88$ LRFD Table 10.6.3.1.2a-3

$S_q := 1 + \frac{B}{L} \cdot \tan(\phi_f) = 1.16$

$c_{wq} := 1$ $c_{w\gamma} := 1$ LRFD Table 10.6.3.1.2a-2

for cohesionless soil:
ultimate bearing capacity $q_{ult} := q_{bar} \cdot N_q \cdot S_q \cdot c_{wq} + 0.5 \cdot \gamma_c \cdot B \cdot N_\gamma \cdot S_\gamma \cdot c_{w\gamma}$ LRFD 10.6.3.1.2a-1

safety factor for cohesionless soils = 3 $q_{ult} = 7.82\ ksf$

Use $SF := 3.5$ since no actual boring data at temporary abutment locations

allowable bearing capacity $q_{all} := \frac{q_{ult}}{SF} = 2.23\ ksf$

Superstructure Dead Load (from Merlin-Dash):

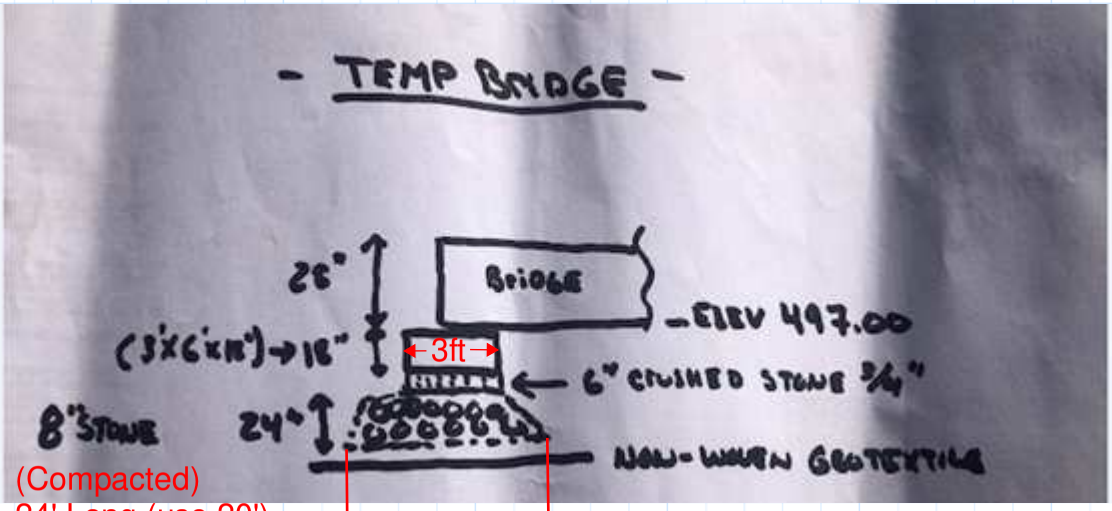
$$DL_{super} := 3.02\ kip \cdot 6 + 2.73\ kip \cdot 2 = 23.58\ kip$$

Live Load (from Appendix A AASHTO 17th Edition: HS20-44), governing reaction for L = 47' plus HL-93 Lane Load

$$LL_{super} := 72.7\ kip$$

Total Factored Superstructure Load on Abutment:

$$TL_{super} := 1.25 \cdot DL_{super} + 1.75 \cdot LL_{super} = 156.7\ kip$$



Abutment DL (factored)

$$DL_{abutment} := (18\ ft \cdot 3\ ft \cdot 1.5\ ft \cdot 150\ pcf + 20\ ft \cdot 3\ ft \cdot 0.5\ ft \cdot 170\ pcf + 7\ ft \cdot 2\ ft \cdot 22\ ft \cdot 170\ pcf) \cdot 1.25 = 87.01\ kip$$

$$DL_{abutment} = 87.01\ kip$$

Actual bearing pressure:

$$q_{act} := \frac{TL_{super} + DL_{abutment}}{B \cdot L} = 2.03\ ksf$$

Is actual bearing pressure less than allowable bearing pressure?

$$\text{if}\left(q_{all} > q_{act}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

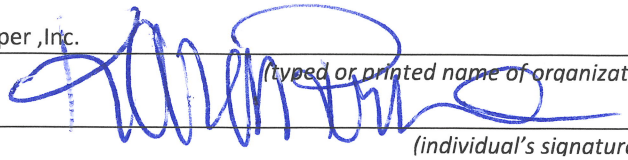
BID SUBMITTAL CERTIFICATION

BIDDER hereby submits this Bid as set forth above:

Bidder:

R.M. Piper ,Inc.

(typed or printed name of organization)

By: 

(individual's signature)

Name: Karen Prince

(typed or printed)

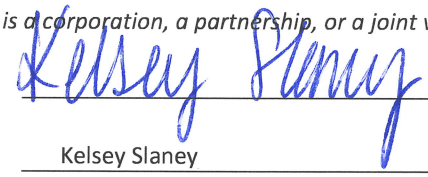
Title: President

(typed or printed)

Date: November 22, 2022

(typed or printed)

If Bidder is a corporation, a partnership, or a joint venture, attach evidence of authority to sign.

Attest: 

(individual's signature)

Name: Kelsey Slaney

(typed or printed)

Title: Secretary

(typed or printed)

Date: November 22, 2022

(typed or printed)

Address for giving notices:

P.O. Box 490 Plymouth, NH 03264-0490

Bidder's Contact:

Name: John Bousquet

(typed or printed)

Title: Vice President

(typed or printed)

Phone: (603) 536-4154

Email: bids@rmpiper.com

Address:

P.O. Box 490 Plymouth, NH 03264-0490

Bidder's Contractor License No.: (if applicable) N/A

END OF SECTION

R.M. PIPER INC.
GENERAL CONTRACTORS
141 Smith Bridge Road
P.O. Box 490, Plymouth, NH 03264-0490
(603)536-4154 • (603)536-3429 Fax

CERTIFICATE OF VOTE

I, Kelsey Slaney, hereby certify that I am duly elected Secretary of R.M. Piper, Inc.

I hereby certify the following is a true copy of a vote taken at a meeting of the Board of Directors of the Corporation, duly called, and held on November 22, 2022, at which a quorum of the Board was present and voting.

VOTED:
That Karen Prince is duly authorized to enter into a specific contract, namely Temporary Bridge Installation – Thornton Ferry Road over Beaver Brook, with the Town of Amherst New Hampshire, and is further authorized to execute any documents that may in her judgment be desirable or necessary to affect the purpose of this vote.

I hereby certify that said vote has not been amended or repealed and remains in full force and effect as of November 22, 2022, and that Karen Prince is duly elected President of this Corporation.

Attest:

November 22, 2022

Kelsey Slaney
Secretary

BID BOND (PENAL SUM FORM)

Bidder	Surety
Name: **R.M. PIPER, INC.** Address *(principal place of business)*: **P.O. Box 490** **Plymouth, NH 03264-0490**	Name: **Swiss Re Corporate Solutions America Insurance Corporation** Address *(principal place of business)*: **AGENCY: P.O. Box 511** **Concord, NH 03302-0511**
Owner Name: **TOWN OF AMHERST** Address *(principal place of business)*: **2 Main Street** **Amherst, NH 03031**	**Bid** Project *(name and location)*: **Temporary Bridge Installation, Thornton Ferry Road over Beaver Brook, Amherst NH 03031** Bid Due Date: **November 22nd, 2022**

Bond

Penal Sum: **FIVE PERCENT OF AMOUNT BID**

Date of Bond: **November 22nd, 2022**

Surety and Bidder, intending to be legally bound hereby, subject to the terms set forth in this Bid Bond, do each cause this Bid Bond to be duly executed by an authorized officer, agent, or representative.

Bidder	Surety
R.M. PIPER, INC. *(Full formal name of Bidder)*	**Swiss Re Corporate Solutions America Insurance Corporation** *(Full formal name of Surety) (corporate seal)*
By: _____ *(Signature)*	By: _____ *(Signature) (Attach Power of Attorney)*
Name: Karen L. Prince *(Printed or typed)*	Name: Matthew R Blaisdell *(Printed or typed)*
Title: President	Title: Attorney-In-Fact
Attest: _____ *(Signature)*	Attest: _____ *(Signature)*
Name: Kelsey Slaney *(Printed or typed)*	Name: Paula J. Cantara *(Printed or typed)*
Title: Secretary	Title: Senior Account Manager

Notes: (1) Note: Addresses are to be used for giving any required notice. (2) Provide execution by any additional parties, such as joint venturers, if necessary.

EJCDC® C-430, Bid Bond (Penal Sum Form).
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1. Bidder and Surety, jointly and severally, bind themselves, their heirs, executors, administrators, successors, and assigns to pay to Owner upon default of Bidder the penal sum set forth on the face of this Bond. Payment of the penal sum is the extent of Bidder's and Surety's liability. Recovery of such penal sum under the terms of this Bond will be Owner's sole and exclusive remedy upon default of Bidder.

2. Default of Bidder occurs upon the failure of Bidder to deliver within the time required by the Bidding Documents (or any extension thereof agreed to in writing by Owner) the executed Agreement required by the Bidding Documents and any performance and payment bonds required by the Bidding Documents.

3. This obligation will be null and void if:

   3.1. Owner accepts Bidder's Bid and Bidder delivers within the time required by the Bidding Documents (or any extension thereof agreed to in writing by Owner) the executed Agreement required by the Bidding Documents and any performance and payment bonds required by the Bidding Documents, or

   3.2. All Bids are rejected by Owner, or

   3.3. Owner fails to issue a Notice of Award to Bidder within the time specified in the Bidding Documents (or any extension thereof agreed to in writing by Bidder and, if applicable, consented to by Surety when required by Paragraph 5 hereof).

4. Payment under this Bond will be due and payable upon default of Bidder and within 30 calendar days after receipt by Bidder and Surety of written notice of default from Owner, which notice will be given with reasonable promptness, identifying this Bond and the Project and including a statement of the amount due.

5. Surety waives notice of any and all defenses based on or arising out of any time extension to issue Notice of Award agreed to in writing by Owner and Bidder, provided that the total time for issuing Notice of Award including extensions does not in the aggregate exceed 120 days from the Bid due date without Surety's written consent.

6. No suit or action will be commenced under this Bond prior to 30 calendar days after the notice of default required in Paragraph 4 above is received by Bidder and Surety, and in no case later than one year after the Bid due date.

7. Any suit or action under this Bond will be commenced only in a court of competent jurisdiction located in the state in which the Project is located.

8. Notices required hereunder must be in writing and sent to Bidder and Surety at their respective addresses shown on the face of this Bond. Such notices may be sent by personal delivery, commercial courier, or by United States Postal Service registered or certified mail, return receipt requested, postage pre-paid, and will be deemed to be effective upon receipt by the party concerned.

9. Surety shall cause to be attached to this Bond a current and effective Power of Attorney evidencing the authority of the officer, agent, or representative who executed this Bond on behalf of Surety to execute, seal, and deliver such Bond and bind the Surety thereby.

10. This Bond is intended to conform to all applicable statutory requirements. Any applicable requirement of any applicable statute that has been omitted from this Bond will be deemed to be included herein as if set forth at length. If any provision of this Bond conflicts with any applicable statute, then the provision of said statute governs and the remainder of this Bond that is not in conflict therewith continues in full force and effect.

11. The term "Bid" as used herein includes a Bid, offer, or proposal as applicable.

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SWISS RE CORPORATE SOLUTIONS

SWISS RE CORPORATE SOLUTIONS AMERICA INSURANCE CORPORATION ("SRCSAIC")
SWISS RE CORPORATE SOLUTIONS PREMIER INSURANCE CORPORATION ("SRCSPIC")
WESTPORT INSURANCE CORPORATION ("WIC")

GENERAL POWER OF ATTORNEY

KNOW ALL MEN BY THESE PRESENTS, THAT SRCSAIC, a corporation duly organized and existing under laws of the State of Missouri, and having its principal office in the City of Kansas City, Missouri, and SRCSPIC, a corporation organized and existing under the laws of the State of Missouri and having its principal office in the City of Kansas City, Missouri, and WIC, organized under the laws of the State of Missouri, and having its principal office in the City of Kansas City, Missouri, each does hereby make, constitute and appoint:

DANIEL E. CHURCH, PAULA J. CANTARA, CHRISTINE M. HOSMER, GARY P. LAPIERRE, RYAN M. STEVENS, MICHAEL P. O'BRIEN, MATTHEW R. BLAISDELL, AND MARK J. STEVENS

JOINTLY OR SEVERALLY

Its true and lawful Attorney(s)-in-Fact, to make, execute, seal and deliver, for and on its behalf and as its act and deed, bonds or other writings obligatory in the nature of a bond on behalf of each of said Companies, as surety, on contracts of suretyship as are or may be required or permitted by law, regulation, contract or otherwise, provided that no bond or undertaking or contract or suretyship executed under this authority shall exceed the amount of:

TWO HUNDRED MILLION ($200,000,000.00) DOLLARS

This Power of Attorney is granted and is signed by facsimile under and by the authority of the following Resolutions adopted by the Boards of Directors of both SRCSAIC and SRCSPIC at meetings duly called and held on the 18th of November 2021 and WIC by written consent of its Executive Committee dated July 18, 2011.

"RESOLVED, that any two of the President, any Managing Director, any Senior Vice President, any Vice President, the Secretary or any Assistant Secretary be, and each or any of them hereby is, authorized to execute a Power of Attorney qualifying the attorney named in the given Power of Attorney to execute on behalf of the Corporation bonds, undertakings and all contracts of surety, and that each or any of them hereby is authorized to attest to the execution of any such Power of Attorney and to attach therein the seal of the Corporation; and it is

FURTHER RESOLVED, that the signature of such officers and the seal of the Corporation may be affixed to any such Power of Attorney or to any certificate relating thereto by facsimile, and any such Power of Attorney or certificate bearing such facsimile signatures or facsimile seal shall be binding upon the Corporation when so affixed and in the future with regard to any bond, undertaking or contract of surety to which it is attached."

SWISS RE CORPORATE SOLUTIONS AMERICA INSURANCE CORPORATION SEAL 1973 MISSOURI

SWISS RE CORPORATE SOLUTIONS PREMIER INSURANCE CORPORATION SEAL MISSOURI

By_____
Erik Janssens, Senior Vice President of SRCSAIC & Senior Vice President of SRCSPIC & Senior Vice President of WIC

By_____
Gerald Jagrowski, Vice President of SRCSAIC & Vice President of SRCSPIC & Vice President of WIC

WESTPORT INSURANCE CORPORATION CORPORATE SEAL MISSOURI

IN WITNESS WHEREOF, SRCSAIC, SRCSPIC, and WIC have caused their official seals to be hereunto affixed, and these presents to be signed by their authorized officers

this 10 day of NOVEMBER, 20 22

State of Illinois
County of Cook SS

Swiss Re Corporate Solutions America Insurance Corporation
Swiss Re Corporate Solutions Premier Insurance Corporation
Westport Insurance Corporation

On this 10 day of NOVEMBER, 20 22, before me, a Notary Public personally appeared Erik Janssens, Senior Vice President of SRCSAIC and Senior Vice President of SRCSPIC and Senior Vice President of WIC and Gerald Jagrowski, Vice President of SRCSAIC and Vice President of SPCSPIC and Vice President of WIC, personally known to me, who being by me duly sworn, acknowledged that they signed the above Power of Attorney as officers of and acknowledged said instrument to be the voluntary act and deed of their respective companies.

OFFICIAL SEAL
CHRISTINA MANISCO
NOTARY PUBLIC, STATE OF ILLINOIS
My Commission Expires March 28, 2026

Christina Manisco, Notary

I, Jeffrey Goldberg, the duly elected Senior Vice President and Assistant Secretary of SRCSAIC and SRCSPIC and WIC, do hereby certify that the above and foregoing is a true and correct copy of a Power of Attorney given by said SRCSAIC and SRCSPIC and WIC, which is still in full force and effect.
IN WITNESS WHEREOF, I have set my hand and affixed the seals of the Companies this 22nd day of November 20 22.

Jeffrey Goldberg, Senior Vice President & Assistant Secretary of SRCSAIC and SRCSPIC and WIC

ADM Welding & Fabrication, LLC
37 Broadhead Street
Warren, PA 16365
O: 814.723.7227
E: admwelding@verizon.net

November 21, 2022

Temporary Bridge
Thornton Ferry Road
Over Beaver Brook
Town of Amherst, NH

Technical Proposal

Bridge Specifications [clarifications to and/or deviations from project specifications in bold]:

- Either 12' or 22' clear travel width [per the Town's selection].
- Overall width no more than 24" wider than clear width.
- Maximum structure depth of 32" [bottom of stringer to top of running plank].
- Nominal 50' overall length.
- Capacity for the following:
  - HL93 loading with no speed restriction [full impact loading].
  - **Live load deflection limited to L/800.**
- New Hampshire PE stamped drawings and calculations for the following:
  - Stringers
  - Timber deck
  - Guardrail
- Tapered beam ends, pipe towbars, and additional reinforcing for ease of handling.
  - Designed to withstand the rigors of relocation between multiple sites.
  - Bridges can be "skidded" utilizing typical excavating/forestry equipment.
- Full-dimension steel end plates [backwalls], above towbars.
- "Strong-post" guardrail system consisting of posts spaced @ 3' – 1.5" c-c and standard galvanized w-rail.
  - Guardrail system will be shown to have TL-1 equivalency by calculation.
- **Two coats epoxy primer/urethane topcoat over SP3 prep.***
- 56-mil waterproofing membrane on 4-mil carrier film, slit to overhang top flanges of stringers by 1/4" – ½" on either side, pre-installed on top flanges between steel and timber deck.
- Timber deck system:
  - Treated, nail-laminated, 2"x4" SYP transverse deck system.
  - Treated, nominal 2" thick, SYP longitudinal timber "running plank" installed over top of transverse deck.
  - Transverse deck boards to be #1 grade. **Running plank to be #2 grade**.
  - All lumber to be MCA treated to UC4A "ground contact". **No additional waterproofing is included as the lumber is already treated.**

***Both a DOT-type paint system and/or a galvanized finish are available as well, at additional cost and lead time.**

Certification of Design

The bridge will be designed/fabricated specifically for this project based on the unique project characteristics/specs however, as noted, PE stamped drawings and calculations for the stringers/decking/guardrail will be provided showing adequacy for the above criteria. ADM has previously fabricated bridges to these same criteria and has no concerns about our ability to meet them.

Description of Bridge System

Single-Lane Bridge

The bridge would consist of two modular steel sections each nominally 8' wide x 50' long x 27,000# [each modular section would consist of three stringer beams with pre-welded bracing]. Each modular section has the guardrail system and timber deck pre-installed. Each section would transport as a legal-dimension/weight load on a standard flatbed trailer.

The sections require a level/uniform bearing surface on each end, preferably a minimum of 2' wide. No special bearing plates are required, simplifying the installation process. Assuming a 2' wide bearing surface at each end, and taking into account the 6" tapered beam ends, maximum clear span is approximately 45'.

The tapered beam ends, towbars, and additional reinforcing facilitate handling with excavating/forestry equipment – the sections can be "skidded" into sites that are not accessible to the delivery vehicle and can often be slid into place with excavators in lieu of a crane. Integrated steel end plates [backwalls] eliminate the need to install a backwall on the temporary abutment.

The sections are set in place over the crossing one at a time, aligned, and bolted together. Lastly a series of steel "center panels" are set in place to cover the bolt-splice location. Typical installation requires only a few hours with two laborers/ironworkers plus equipment/operators. The process is intuitive and requires only basic rigging and hand tools. Dis-assembly is essentially the reverse process, and requires similar resources/timeframe.

Although designed for temporary use in different locations, the bridge is also well-suited for installation as a permanent structure.

Two-Lane Bridge

The two-lane bridge would be conceptually similar to the single-lane structure, except that it would consist of four modular units.

Proposed Project Timeline

- Design Generation [ADM] – 4 weeks
- Design Review/Seal [NH PE] – 2 weeks
- Material Procurement – 2-6 weeks [depending on whether beams can be found in stock or have to be rolled to order]
- Fabrication
    - Single-Lane Bridge – 3 weeks
    - Two-Lane Bridge – 4 weeks

*The above timeline is based on the assumption that a single/complete NH PE sealed submittal package would be provided to HoyleTanner. It may be possible to expedite if there was a willingness to do either/both of the following:

- Review the stringer calculations in advance of the other bridge submittals so that beam procurement could start earlier.
- Review submittals without the NH PE stamp and "approve as noted" subject to final submission including NH seal.

“Hybrid”/Portable Bridge Section being “Skidded” along utility ROW



Single-Lane “Hybrid”/Portable Bridge Section set in place over existing/deteriorated structure [awaiting second section]



Single-Lane “Hybrid”/Portable Bridge being aligned for Bolt-Up



Single-Lane “Hybrid”/Portable Bridge with Guardrail System similar to Proposed Guardrail



ADM Welding & Fabrication, LLC
37 Broadhead Street
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November 21, 2022

Temporary Bridge
Thornton Ferry Road
Over Beaver Brook
Town of Amherst, NH

Technical Proposal

Bridge Specifications [clarifications to and/or deviations from project specifications in bold]:

- Either 12' or 22' clear travel width [per the Town's selection].
- Overall width no more than 6' wider than clear width.
- Maximum structure depth of 32" [bottom of stringer to top of running plank].
- Nominal 50' overall length.
- Capacity for the following:
  - HL93 loading with no speed restriction [full impact loading].
  - **Live load deflection limited to L/800.**
- New Hampshire PE stamped drawings and calculations for the following:
  - Stringers
  - Timber deck
- Tapered beam ends, pipe towbars, and additional reinforcing for ease of handling.
  - Designed to withstand the rigors of relocation between multiple sites.
  - Bridges can be "skidded" utilizing typical excavating/forestry equipment.
- Full-dimension steel end plates [backwalls], above towbars.
- **Two coats epoxy primer/urethane topcoat over SP3 prep.***
- 56-mil waterproofing membrane on 4-mil carrier film, slit to overhang top flanges of stringers by 1/4" – ½" on either side, pre-installed on top flanges between steel and timber deck.
- Timber deck system:
  - Treated, nail-laminated, 2"x4" SYP transverse deck system.
  - Treated, nominal 2" thick, SYP longitudinal timber "running plank" installed over top of transverse deck.
  - Transverse deck boards to be #1 grade. **Running plank to be #2 grade.**
  - All lumber to be MCA treated to UC4A "ground contact". **No additional waterproofing is included as the lumber is already treated.**

***Both a DOT-type paint system and/or a galvanized finish are available as well, at additional cost and lead time.**

Certification of Design

The bridge will be designed/fabricated specifically for this project based on the unique project characteristics/specs however, as noted, PE stamped drawings and calculations for the stringers and decking will be provided showing adequacy for the above criteria. ADM has previously fabricated bridges to these same criteria and has no concerns about our ability to meet them.

Description of Bridge System

Single-Lane Bridge

The bridge would consist of two modular steel sections each nominally 8.5' wide x 50' long x 31,000# [each modular section would consist of four stringer beams with pre-welded bracing]. Each modular section has the timber deck pre-installed. Each section would transport as a legal-dimension/weight load on a standard flatbed trailer.

The sections require a level/uniform bearing surface on each end, preferably a minimum of 2' wide. No special bearing plates are required, simplifying the installation process. Assuming a 2' wide bearing surface at each end, and taking into account the 6" tapered beam ends, maximum clear span is approximately 45'.

The tapered beam ends, towbars, and additional reinforcing facilitate handling with excavating/forestry equipment – the sections can be "skidded" into sites that are not accessible to the delivery vehicle and can often be slid into place with excavators in lieu of a crane. Integrated steel end plates [backwalls] eliminate the need to install a backwall on the temporary abutment.

The sections are set in place over the crossing one at a time, aligned, and bolted together. Lastly a series of steel "center panels" are set in place to cover the bolt-splice location. Typical installation requires only a few hours with two laborers/ironworkers plus equipment/operators. The process is intuitive and requires only basic rigging and hand tools. Dis-assembly is essentially the reverse process, and requires similar resources/timeframe.

Pre-cast concrete jersey barriers will be installed on the modular steel sections after they are set in place to serve as the guardrail system.

Although designed for temporary use in different locations, the bridge is also well-suited for installation as a permanent structure.

Two-Lane Bridge

The two-lane bridge would be conceptually similar to the single-lane structure, except that it would consist of four modular units each nominally 7' wide x 50' long x 27,000#.

Proposed Project Timeline

- Design Generation [ADM] – 4 weeks
- Design Review/Seal [NH PE] – 2 weeks
- Material Procurement – 2-6 weeks [depending on whether beams can be found in stock or have to be rolled to order]
- Fabrication
    - Single-Lane Bridge – 3 weeks
    - Two-Lane Bridge – 4 weeks

*The above timeline is based on the assumption that a single/complete NH PE sealed submittal package would be provided to HoyleTanner. It may be possible to expedite if there was a willingness to do either/both of the following:

- Review the stringer calculations in advance of the other bridge submittals so that beam procurement could start earlier.
- Review submittals without the NH PE stamp and "approve as noted" subject to final submission including NH seal.

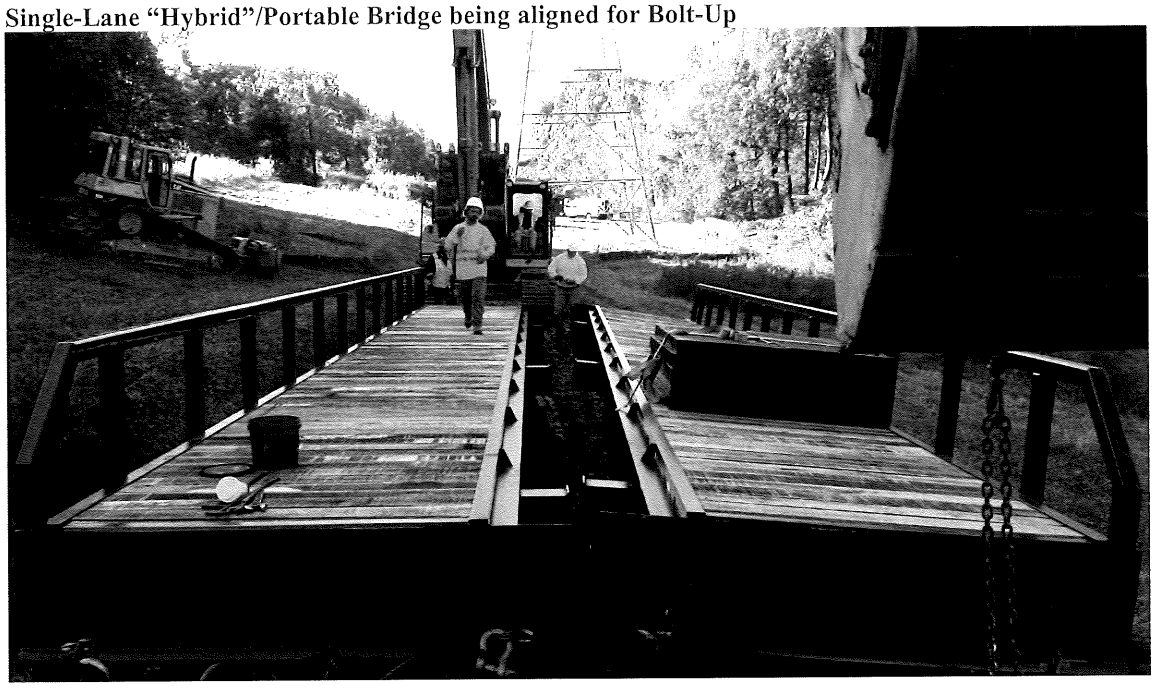
"Hybrid"/Portable Bridge Section being "Skidded" along utility ROW



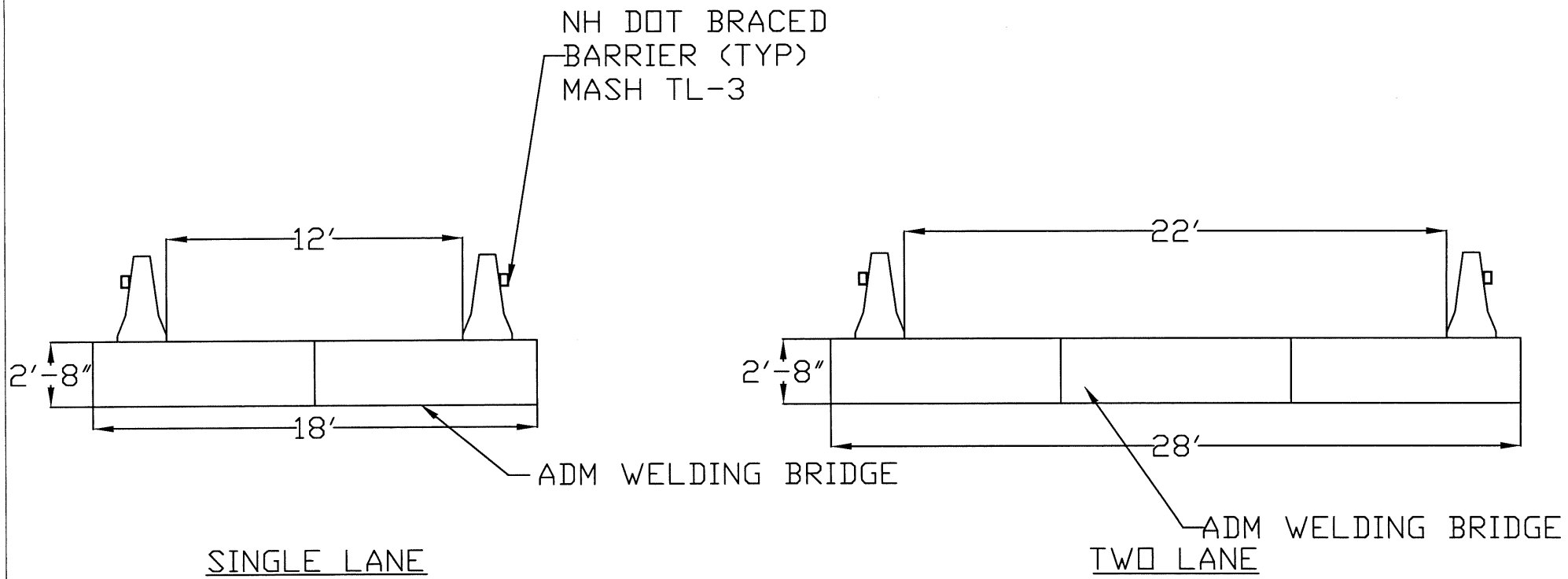
Single-Lane "Hybrid"/Portable Bridge Section set in place over existing/deteriorated structure [awaiting second section]



Single-Lane "Hybrid"/Portable Bridge being aligned for Bolt-Up



Note: Photos depict a pre-installed steel guardrail different from the proposed pre-cast concrete jersey barrier guardrail [to be installed on top of the bridge deck].



SINGLE LANE

TWO LANE

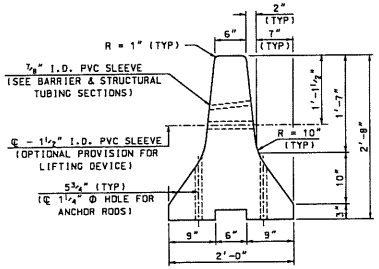
COLD RIVER BRIDGES, LLC
10 LANBRO LANE
TEL 603-756-9300 WALPOLE, NH FAX 603-756-9303

TOWN OF AMHERST NH

TYPICAL SECTION

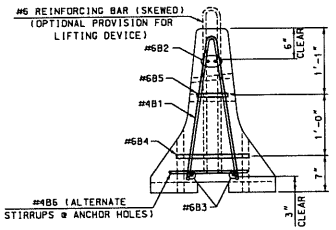
SHEET NUMBER
TB

11/22/22



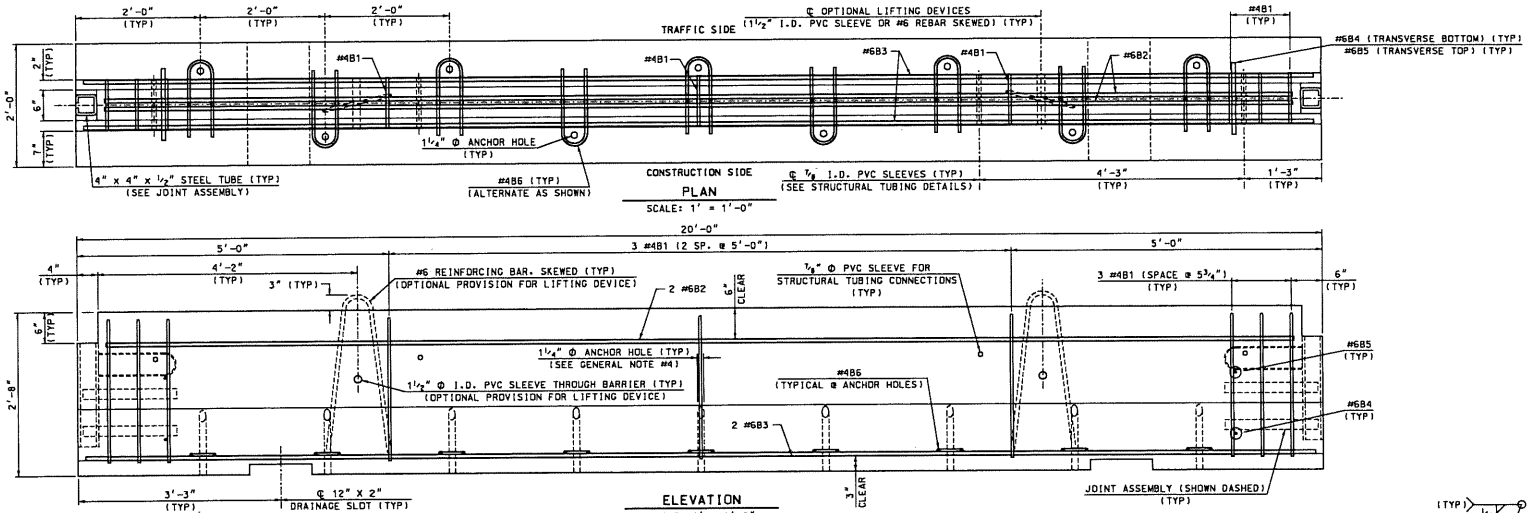
TYPICAL SECTION

SCALE: 1" = 1'-0"



TYPICAL SECTION

SCALE: 1" = 1'-0"



PLAN

SCALE: 1" = 1'-0"

ELEVATION

SCALE: 1" = 1'-0"

GENERAL NOTES

(1) PORTABLE CONCRETE BARRIER SHALL BE FURNISHED BY THE CONTRACTOR AND PAID FOR AS ITEM 606.41741 PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL (BRIDGE). CONCRETE BARRIER AND ALL ATTACHMENTS SHALL BE FABRICATED IN ACCORDANCE WITH SPECIAL PROVISIONS. ALL BARRIER UNITS FOR BRACED SYSTEMS SHALL BE 20' LONG.

(2) PORTABLE CONCRETE BARRIER DETAILS, AS SHOWN IN THESE PLANS, ARE IN COMPLIANCE WITH REQUIREMENTS PER UPDATED NCHRP REPORT 350 FOR TEST NO. 3-11 (MASH TEST LEVEL 3). CRASH TESTED BY MIDWEST ROADSIDE SAFETY; NY BOX BEAM STIFFENING OF UNANCHORED TCB, MARCH 2008, AND ACCEPTED PER FHWA LETTER B-239 (11/1/2012). THE BARRIER SYSTEM HAS BEEN CRASH TESTED WITH A 27.6" DYNAMIC DEFLECTION WHICH WILL ALLOW BRACED BARRIER TO BE PLACED A MINIMUM 12" FROM THE EDGE OF BRIDGE DECK.

(3) A MINIMUM OF TWO BARRIER UNITS, WITH BRACED JOINTS ARE REQUIRED TO BE PLACED BEYOND BOTH ENDS OF THE BRIDGE WORK AREA. FOR SPEEDS GREATER THAN 45 MPH. FOR SPEEDS ≤ 45 MPH, A MINIMUM OF ONE BRACED BARRIER IS REQUIRED TO BE FULLY SET BEYOND EACH END OF BRIDGE WORK AREA.

(4) THE LAST CONCRETE BARRIER UNIT, AT EACH END OF BRACED BARRIER LAYOUT, SHALL BE ANCHORED A MINIMUM 18" BELOW THE ROADWAY SURFACE. REQUIRED 1" Φ ANCHOR RODS (A36 STEEL) SHALL BE INSTALLED WITH 5 ANCHORS ON THE TRAFFIC SIDE OF BARRIER AND 4 ON THE CONSTRUCTION SIDE. IF THE END(S) OF THE BRACED CONCRETE BARRIER SYSTEM EXTENDS 50' OR MORE BEYOND LIMITS OF BRIDGE WORK THE LAST BARRIER UNIT DOES NOT REQUIRE ANCHORAGE.

(5) PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL (BRIDGE), ITEM 606.41741, MAY BE INSTALLED WITH A 230' MINIMUM RADIUS. GAPS CREATED BETWEEN STRUCTURAL TUBES AND CONCRETE BARRIER, DURING A RADIAL LAYOUT, SHALL BE SHIMMED WITH 8" x 8" x 1/2" PLATES & FENDER WASHERS TO FIRMLY ATTACH STRUCTURAL TUBING TO BARRIER.

(6) THE CONTRACTOR SHALL FURNISH AND INSTALL APPROVED RETROREFLECTIVE DELINEATORS AT 25-FOOT INTERVALS ALONG TOP AND/OR ONE FOOT DOWN THE SIDE OF PORTABLE CONCRETE BARRIER. SUBSIDIARY TO ITEM 606.41741 (SEE STANDARD NO. DL-1 OF NHDOT STANDARD PLANS FOR ROAD CONSTRUCTION). THE COLOR OF DELINEATORS SHALL, IN ALL INSTANCES, CONFORM TO THE COLOR OF EDGE LINE MARKINGS. DELINEATORS SUPPLEMENT, BUT DO NOT REPLACE, THE NEED FOR RETROREFLECTIVE SOLID EDGE LINE MARKINGS.

MATERIAL NOTES

(1) BARRIERS SHALL BE LIGHT COLORED CLASS AA CONCRETE, WITH MINIMUM COMPRESSIVE STRENGTH OF 4000 psi, AND SHALL HAVE A SMOOTH UNIFORM SURFACE FREE OF DEFECTS AND IRREGULARITIES. CASTING DATE SHALL BE SHOWN ON BARRIER. ALL EXPOSED EDGES OF CONCRETE SHALL BE CHAMFERED 3/4", UNLESS OTHERWISE NOTED.

(2) ALL REINFORCING STEEL SHALL BE AASHTO M31 (ASTM A615) GRADE 60. ALL REINFORCEMENT SHALL HAVE 1 1/2" MINIMUM CLEAR COVER, UNLESS OTHERWISE NOTED.

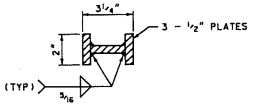
(3) STRUCTURAL STEEL, EXCEPT THE STEEL TUBES, SHALL BE ASTM A36 OR A572. ALL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH SECTION 550.

(4) STEEL TUBES, 6" x 6" x 3/16" & 4" x 4" x 1/2", SHALL BE ASTM A 500 GRADE B OR C. ALL TUBES SHALL BE GALVANIZED IN ACCORDANCE WITH SECTION 550.

(5) ALL STEEL FOR CONNECTION KEY AND TRANSITION KEY ASSEMBLIES SHALL BE GALVANIZED IN ACCORDANCE WITH SECTION 550.

(6) A MINIMUM OF 2 LIFTING DEVICES, EACH WITH THE CAPACITY TO LIFT A MASS OF 6 TONS (MINIMUM), SHALL BE INSTALLED TO EACH BARRIER UNIT. TWENTY FOOT LONG CONCRETE BARRIER UNITS ARE APPROXIMATELY 400 LBS./FT.

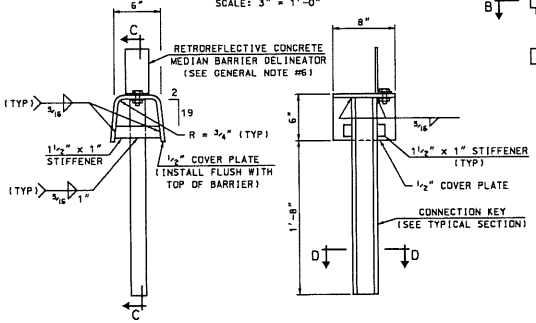
(7) DELINEATORS SHALL BE ATTACHED TO BARRIER USING AN APPROVED ADHESIVE MATERIAL OR AS SHOWN ON THIS SHEET.



SECTION D-D

(CONNECTION KEY)

SCALE: 3" = 1'-0"

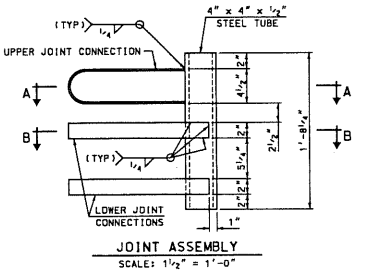


END VIEW

SECTION C-C

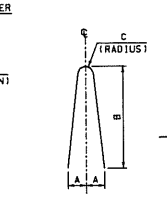
CONNECTION KEY ASSEMBLY DETAILS

SCALE: 1 1/2" = 1'-0"



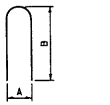
JOINT ASSEMBLY

SCALE: 1 1/2" = 1'-0"



TYPE "1" REBAR

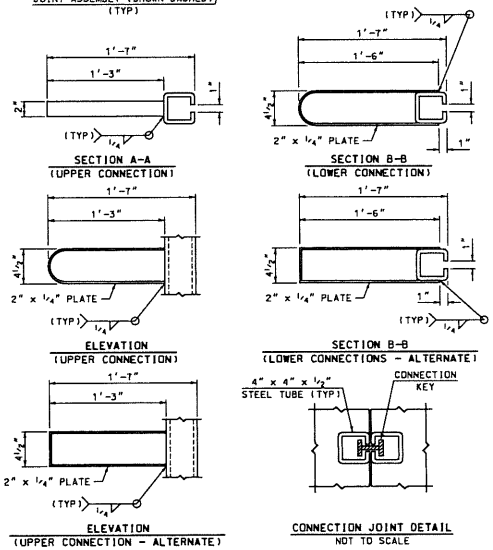
NOT TO SCALE



TYPE "11" REBAR

NOT TO SCALE

NO MODIFICATIONS SHALL BE MADE TO THIS SHEET



JOINT CONNECTION DETAILS

SCALE: 1 1/2" = 1'-0" (EXCEPT AS NOTED)

REINFORCING SCHEDULE (PER 20' BARRIER UNIT)

MARK	SIZE	LENGTH	# PIECES	TYPE	A	B	C	LOCATION
B1	#4	4'-10"	9	I	5"	2'-4"	1"	STIRRUPS
B2	#6	19'-1"	2	—				LONGITUDINAL (TOP)
B3	#6	19'-9"	2	—				LONGITUDINAL (BOTTOM)
B4	#6	1'-2"	2	—				TRANSVERSE (BOTTOM)
B5	#6	6"	2	—				TRANSVERSE (TOP)
B6	#4	2'-9"	9	II	5"	1'-3"		STIRRUPS

SUBDIRECTORY	DGN LOCATOR	SHEET SCALE
English/BARRIER	PCB-BRACED	AS NOTED

STATE OF NEW HAMPSHIRE

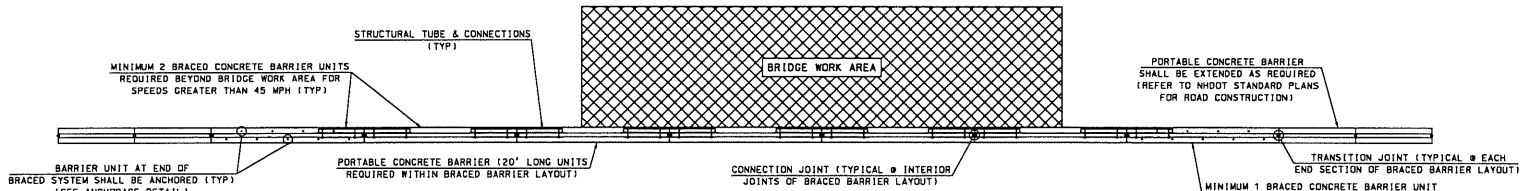
DEPARTMENT OF TRANSPORTATION * BUREAU OF BRIDGE DESIGN

TOWN	BRIDGE NO.	STATE PROJECT
LOCATION		

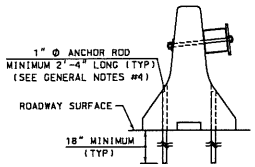
PORTABLE CONCRETE BARRIER - BRACED (1 OF 2)

BRIDGE SHEET OF THIS SHEET

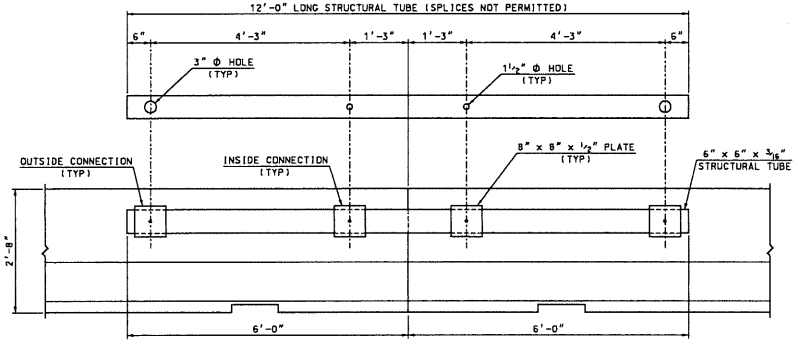
REVISIONS AFTER PROPOSAL	BY	DATE		BY	DATE
	DESIGNED	NHDOT	7/12	CHECKED	ABH 8/12
	DRAWN	PJP	8/12	CHECKED	ABH 8/12
	QUANTITIES			CHECKED	
	ISSUE DATE	8/15/12	FEDERAL PROJECT NO.		SHEET NO.
	REV. DATE	5/15/18	----------		TOTAL SHEETS



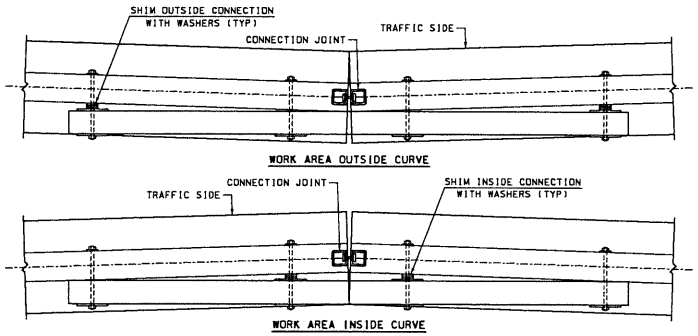
PLAN – BARRIER LAYOUT
NOT TO SCALE



ANCHORAGE DETAIL
SCALE: 3/4" = 1'-0"



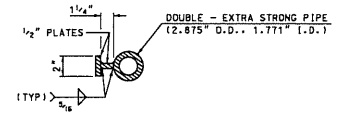
STRUCTURAL TUBING DETAILS
SCALE: 3/4" = 1'-0"



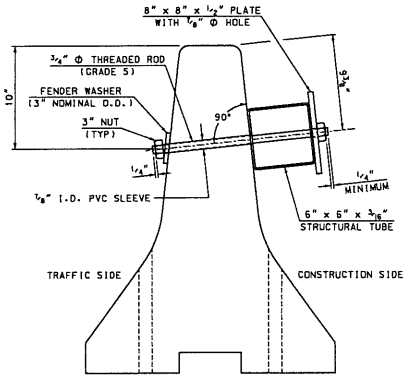
TUBE CONNECTIONS (CURVED ALIGNMENT)
SCALE: 3/4" = 1'-0"

PVC SLEEVE OPENINGS SHALL BE MODIFIED/DRILLED AS REQUIRED TO PROPERLY ALIGN STRUCTURAL TUBE BRACING UNITS FOR CURVED ALIGNMENTS

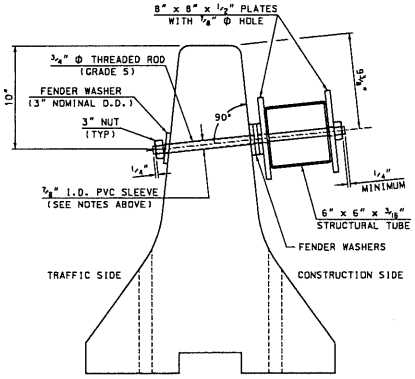
THE PRESENCE OF NORMAL HOLES WHICH HAVE BEEN MODIFIED/DRILLED WILL NOT AFFECT THE REUSE OF CONCRETE BARRIER UNITS



TYPICAL SECTION
(TRANSITION KEY)
SCALE: 2" = 1'-0"

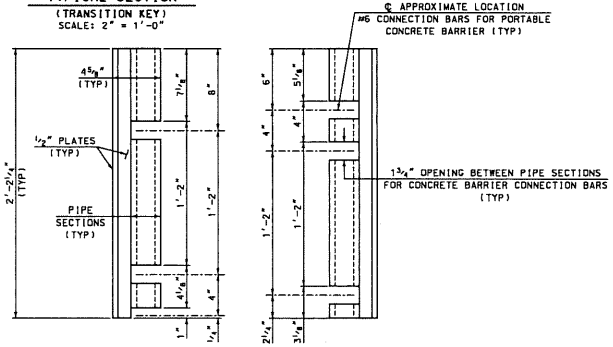


SECTION WITHOUT SHIMS

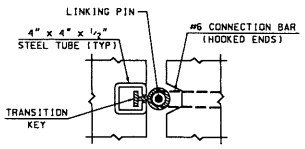


SECTION WITH SHIMS

BARRIER & STRUCTURAL TUBING SECTIONS
SCALE: 2" = 1'-0"



TRANSITION KEY ELEVATIONS
SCALE: 2" = 1'-0"



TRANSITION JOINT DETAILS
NOT TO SCALE

NO MODIFICATIONS SHALL BE MADE TO THIS SHEET

SUBDIRECTORY	.DGN LOCATOR	SHEET SCALE
English/BARRIER	PCB-BRACED	AS NOTED

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION * BUREAU OF BRIDGE DESIGN

TOWN | BRIDGE NO. | STATE PROJECT
LOCATION

PORTABLE CONCRETE BARRIER - BRACED (2 OF 2)

BRIDGE SHEET OF

REVISIONS AFTER PROPOSAL	BY	DATE

DESIGNED	NHDOT	7/12	CHECKED	ABH	8/12	
DRAWN	PJP	8/12	CHECKED	ABH	8/12	
QUANTITIES			CHECKED			
ISSUE DATE	8/15/12	FEDERAL PROJECT NO.		SHEET NO.		TOTAL SHEETS
REV. DATE	5/15/18	----------				

FILE NUMBER