CSiBridge V15.0.0+

Verification of Composite Steel Bridge Design per AASHTO LRFD 2007 with 2008 Interims

Program: CSI Bridge
Version: V15.0.0+
Latest Tested Build: V15.1.0_Y

Tested By: Ondrej Kalny
Date: 5/7/2011

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Design Results Section at G2 0.417L.xls	Formatted CSI Bridge Output
Design Results Section at G2 1.000L.xls	Formatted CSI Bridge Output

References

[1] FHWA/NHI: LRFD Design Example for Steel Girder Superstructure Bridge, 2003
(available at https://wiki.csiberkeley.com/x/IYEn)

[2] CSI Wiki - Tracking of Issues For CSI Bridge Builds V15.0.0_xxx
(available at https://wiki.csiberkeley.com/x/sAEw)

Introduction

- This document provides a documentation for the verification of the composite steel bridge design of CSI Bridge.
- The model used in this tutorial was is loosely based on the model described in the "LRFD Design Example for Steel Girder Superstructure Bridge" (FHWA NHI-04-041) report published by FHWA in November 2003.

Overview

- Model A: apply loads to applicable geometry (utilize bridge design actions)
- Model B: apply all loads to fully constructed bridge (for preliminary design; utilize bridge design actions)
- Model C: use staged construction

Notation

- To easily distinguish load cases, load combinations, and design requests for individual model, precede their names by the model letter. For example, for Model A, the names would be as follows:
  - Load case: “a. LC”
  - Load Combination: “a- COMB”
  - Design Request: “a_ DReq”

Model A

Model A

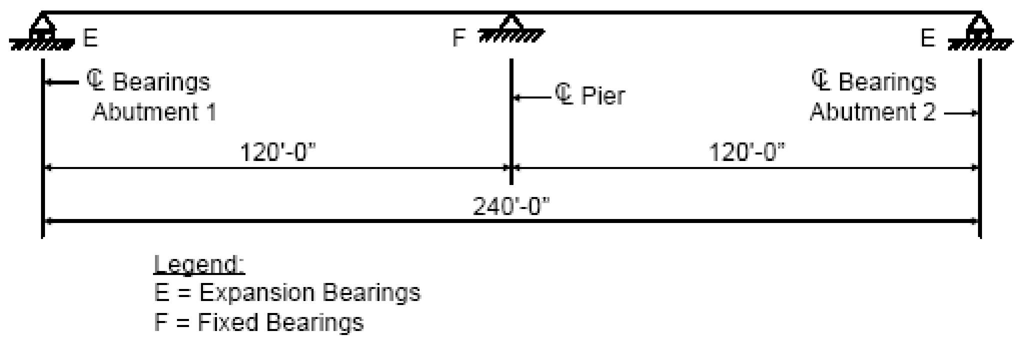
- Model A will not use staged construction analysis.
- Each loading will be applied in a separate (single-staged staged construction) load case that reflects the actual structure (non-composite, long-term composite, short-term composite) that resists the applied loads.

Geometry

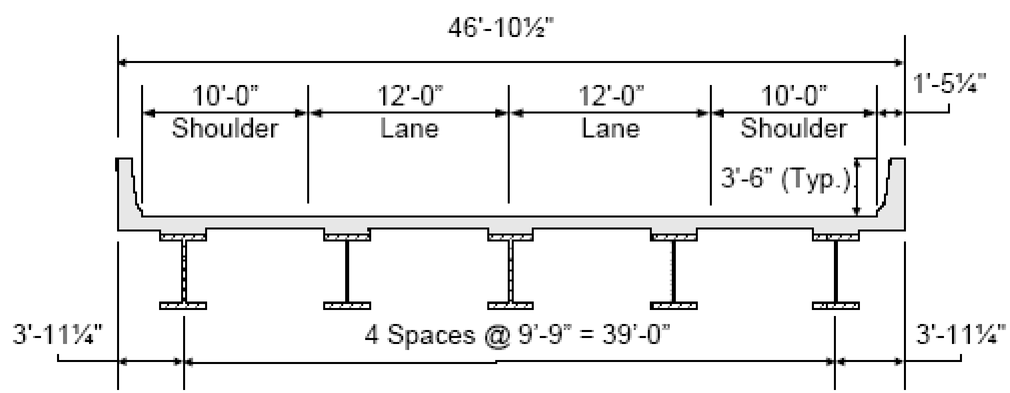
Geometry

- Haunch (distance from top of web to bottom of of slab) is 3.5”. See p. 3-12 of the FHWA Example.
- Effective deck thickness is 8.0” and this was entered into the CSI Bridge model (Note that total deck thickness including the sacrificial overlay is 8.5”.)

Geometry

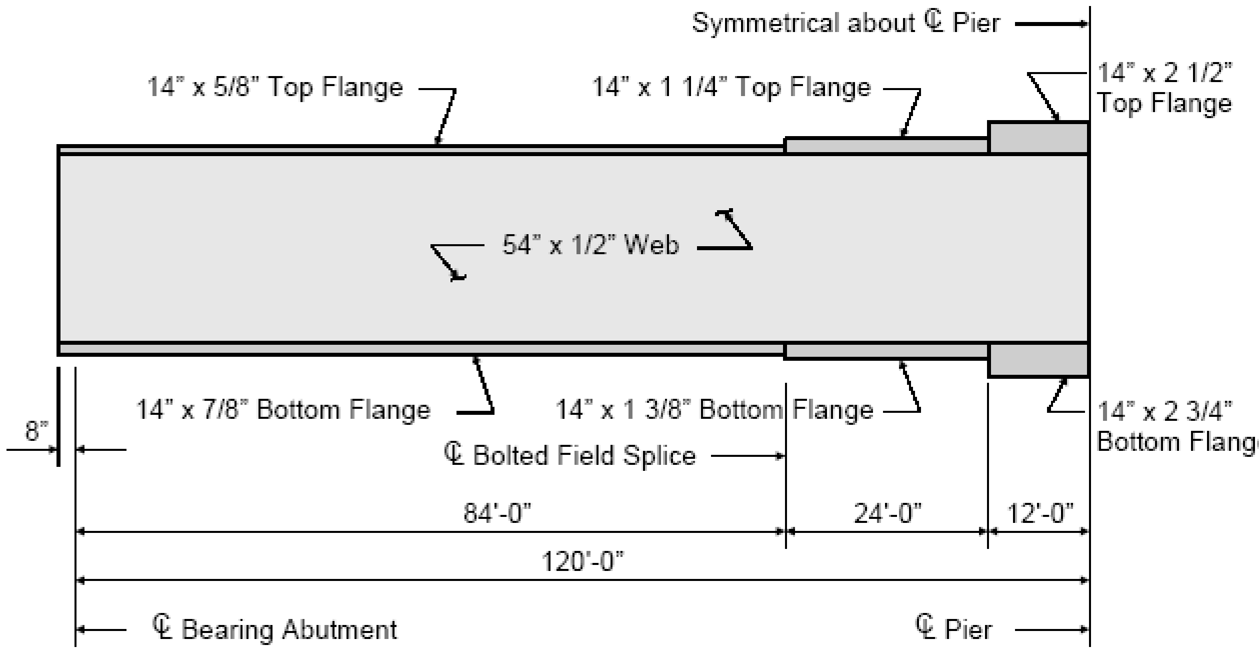


E
F
E
℄ Bearings
Abutment 1
℄ Pier
℄ Bearings
Abutment 2
120'-0"
120'-0"
240'-0"
Legend:
E = Expansion Bearings
F = Fixed Bearings



46'-10½"
10'-0" Shoulder
12'-0" Lane
12'-0" Lane
10'-0" Shoulder
1'-5¼"
3'-6" (Typ.)
3'-11¼"
4 Spaces @ 9'-9" = 39'-0"
3'-11¼"

Geometry – Initial Trial Plate Sizes



okAgenda

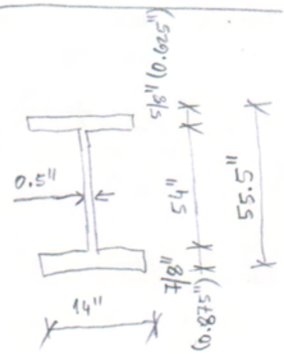
Subject: steel bridge design capabilities of CSI bridge

Subtask: Section Properties

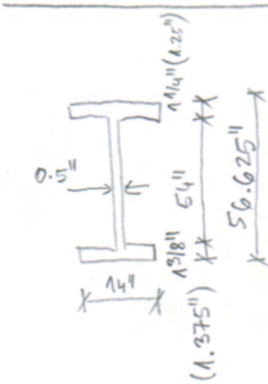
Prepared by: ok   Date: 5/11/2010

Sheet No. 1   of

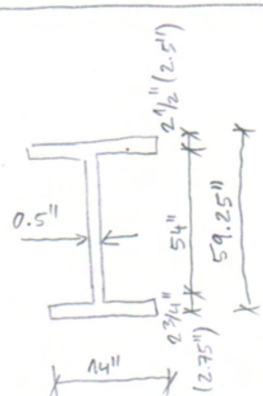
I-Girder 5/8" Top Flange



I-Girder 1 1/4" Top Flange



I-Girder 2 1/2" Top Flange



okAgenda

Subject:

Subtask:

Prepared by: Date:

Sheet No. 2 of

- Effective slab width should be based on "12 times the slab thickness plus ..." which governs
  - AASHTO Art. 4.6.2.6 (p. 4-52)
  - FHWA Example (p. 3-11)

okAgenda

Subject:

Subtask: section G2 0.417L – Geometry checks

Prepared by: Date:

Sheet No. 3 of

---

section G2 0.417L

Bottom flange: A6.3.2-4; p. 6-254

$$\lambda_f = \frac{b}{2t} = 8$$ ... slenderness ratio for ~~compression~~ (bottom) flange

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29000\,\text{ksi}}{50\,\text{ksi}}} = 9.15$$ ... Limiting slenderness ratio for compact flange

$$\lambda_{rf} = 0.95\sqrt{\frac{Ek_c}{F_{yr}}} = 0.95\sqrt{\frac{(29{,}000\,\text{ksi})(0.364)}{35\,\text{ksi}}} = 16.967$$ ... limiting slenderness ratio for noncompact flange

$$k_c = \frac{4}{\sqrt{\frac{D}{t_w}}} = \frac{4}{\sqrt{\frac{54''}{0.5''}}} = 0.384$$

Web:

$$\lambda_w = \frac{2D_c}{t_w} = \frac{(2)(25'')}{(0.5')} = 100$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{50}} = 137.3$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} = \frac{\sqrt{\frac{29000}{50}}}{\left(0.54\frac{46{,}674\,\text{kip-in}}{25{,}918\,\text{kip-in}} - 0.09\right)^2} =$$

$$= \frac{24.083189}{0.778718} = 30.92$$

okAgenda

Subject:

Subtask: section G2 1.000 L – Negative flexure checks

Prepared by: Date:

Sheet No. 4 of

Calculate $F_{nc(LTB)}$

[AASHTO Eqn. 6.10.8.2.3-3]

$$F_{nc(LTB)} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] R_b R_h F_{yc} \leq R_b R_h F_{yc}$$

$$= (1.0) \left[1 - \left(1 - \frac{35 ksi}{(0.99)(50 ksi)} \right) \left(\frac{20' - 7.9'}{29.7' - 7.9'} \right) \right] (1.0)(0.99)(50 ksi) =$$

$$= (1 - (0.293)(0.555))(49.5 ksi) =$$

$$= (0.837)(49.5 ksi) = \underline{\underline{41.45 ksi}}$$

vs. 19.4 ksi calculated by CSI bridge V15.0.0-T

Note that C_b is taken as 1.0 in CSI Bridge. In the FHWA Example, C_b is more accurately calculated as 1.3, resuting in $F_{nc(LTB)} = 50 ksi$.

Geometry

- Define layout line on straight alignment from 0ft to 240ft.
- Define “Span 1 Girder” and “Span 2 Girder” frame section as nonprismatic sections.
- Define “Span 1 Bridge Section” and “Span 2 Bridge Section” bridge deck sections.
- Added weight-less end diaphragms to stabilize the bridge before the deck is constructed.

Geometry - Groups

- Use the bridge object to define the following groups: Deck, Girders, Substructure, All But Deck

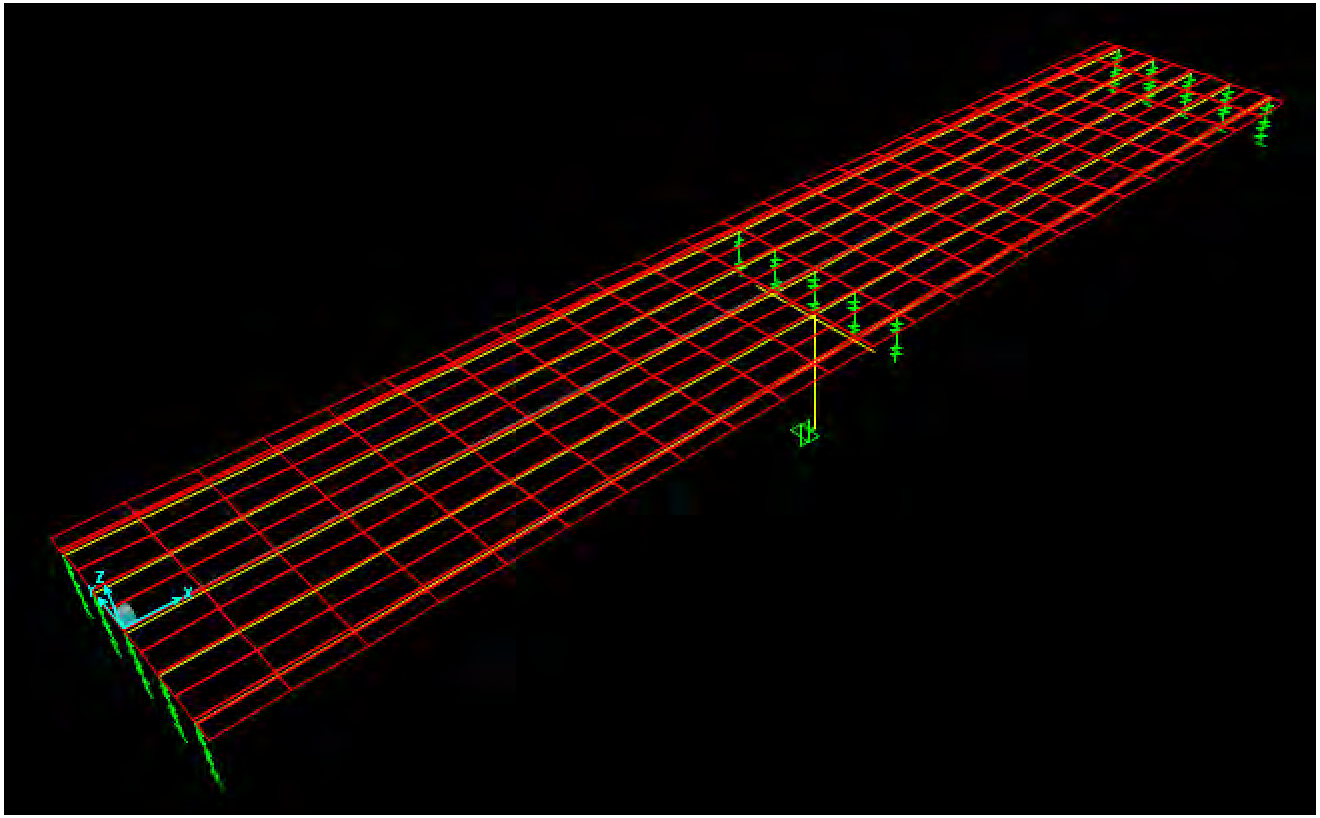
Geometry – Property Modifiers

The following sets of property modifiers were defined for area objects to allow simulating different types of sections during staged construction:

- “W 0, S 0”: no weight, no stiffness
- “W 0, S 1/3”: no weight, long-term stiffness
- “W 1, S 0”: full weight, no stiffness
- “W 1, S 1”: full weight, full (short-term) stiffness
- “W 1, S 1/3”: full weight, long-term stiffness

Geometry

Final Geometry



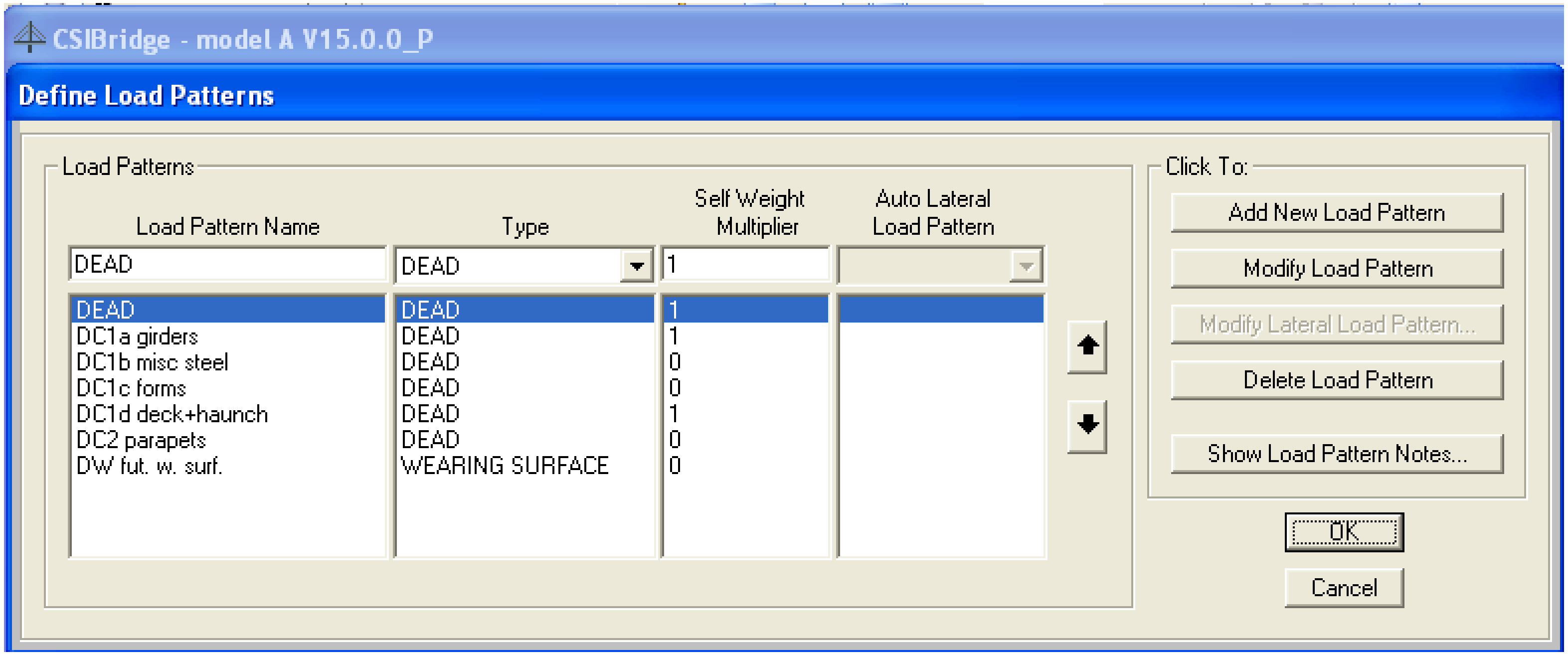
Dead Loads

Dead Loads

The following load cases were considered for dead load analysis

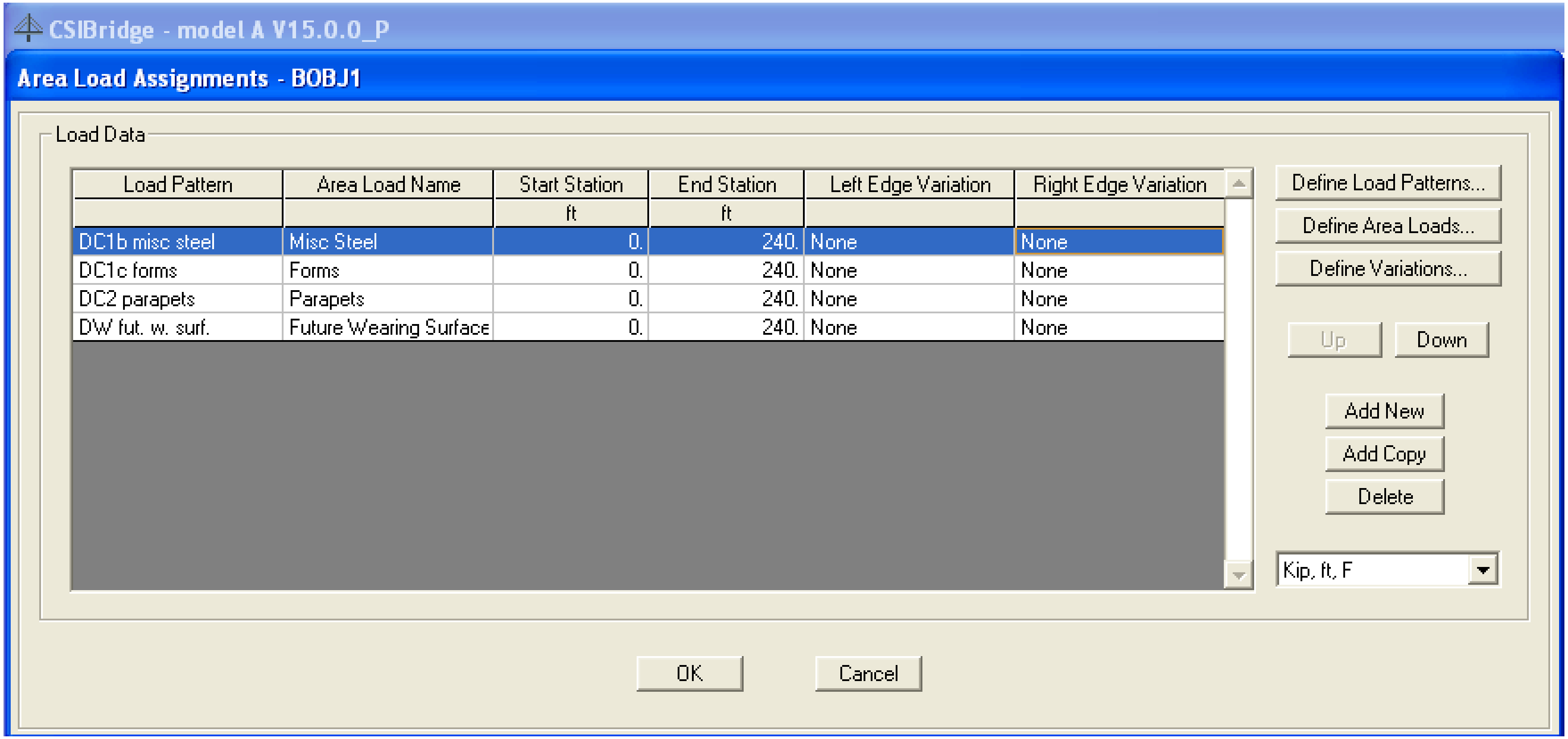
- “DC1a girders”: girder self-weight acting on non-composite section. Calculated directly from the selfweight of the girders.
- “DC1b misc steel”: weight of shear studs, cross-frames and bolts applied to non-composite section using **0.015 kips/ft per girder** (or (0.015 kip/ft)(5 girders)/(46.875 ft width) = **0.0016 ksf**).
- “DC1c forms”: weight of forms. Applied as area load to the bridge object using **0.015ksf**.
- “DC1d deck and haunch”: weight of deck and haunches applied to non-composite section. Calculated directly from the self-weight of the deck, the haunch was neglected.
- “DC2 parapets”: weight of parapets applied to long-term composite section using **0.5 kip/ft per parapet** (or (0.5kip/ft)(2 parapets)/(46.875 ft width) = **0.0213 ksf**).
- “DW fut. w. surface”: weight of future wearing surface applied to long-term composite section using **0.0292 ksf** applied over the roadway of 44ft width.

Load Patterns



Bridge Area Loads

Definition of dead loads using bridge area loads:



Bridge Design Action Types

The table below shows bridge design action types for individual load cases defined in the model:

Load Case	**Bridge Design Action**
DC1a girders	Non-composite
DC1b misc steel	Non-composite
DC1c forms	Non-composite
DC1d deck + haunch	Non-composite
DC2 parapets	Long-Term Composite
DW fut. w. surf.	Long-Term Composite
LL 1 lane	Short-Term Composite

Load Case “DC1a girders”

- This load case is used to apply the self-weight of the girders to non-composite section.
- Single-stage staged construction load case was used to add the entire structure except for the deck (note the the girders are stabilized by diaphragms) and then load the added structure by its self-weight.

Load Case “DC1b misc steel”

- This load case is used to apply the weight of shear studs, cross-frames, bolts, etc. to the non-composite section.
- The load was calculated as 0.0016ksf and is applied directly to the bridge deck via the “Bridge Loads” feature of the program.
- Single-stage staged construction load case was used to add the entire structure and then load deck by the above load. Note that the deck was added with negligible stiffness and zero weight to correctly model the non-composite section.

Load Case “DC1c forms”

- This load case is used to apply the weight of the forms to the non-composite section.
- The load was calculated as 0.015ksf and is applied directly to the bridge deck via the “Bridge Loads” feature of the program.
- Single-stage staged construction load case was used to add the entire structure and then load deck by the above load. Note that the deck was added with negligible stiffness and zero weight to correctly model the non-composite section.

Load Case “DC1d deck+haunch”

- This load case is used to apply the weight of the deck and haunches to the non-composite section.
- Single-stage staged construction load case was used to add the entire structure, make the stiffness of the deck negligible and apply the self-weight of the deck. Note that the deck was added with negligible stiffness to correctly model the non-composite section.
- The weight of the haunches is neglected, but it could be included by by applying bridge load to the deck that would represent the weight of the haunches.

Load Case “DC2 parapets”

- This load case is used to apply the weight of the parapets to the long-term composite section.
- The parapet load was calculated as 0.0213ksf and is applied directly to the bridge deck via the “Bridge Loads” feature of the program.
- Single-stage staged construction load case was used to add the entire structure, reduce the stiffness of the deck by a factor of 1/3 to model the long-term composite section and apply the above parapet load as uniform area load to the entire deck using the bridge load feature of the program.

Load Case “DW fut. w. surf.”

- This load case is used to apply the weight of the future wearing surface to the long-term composite section.
- The future wearing surface load was calculated as 0.0292ksf and is applied directly to the bridge deck via the “Bridge Loads” feature of the program.
- Single-stage staged construction load case was used to add the entire structure, reduce the stiffness of the deck by a factor of 1/3 to model the long-term composite section and apply the above future wearing surface load as uniform area load to the entire deck using the bridge load feature of the program.

Verify Dead Load Results

- The dead load moments were compared against moments published on p. 3-18 of the FHWA design example and reasonable agreement was achieved. See the table on the next page.

Check Dead Load Results

Dead load moments for Interior Girder 1 [kip-ft]

Load Case	Location in Span 1		Location in Span 1	
	0.5L		1.0L	
	FHWA Example	CSI Bridge	FHWA Example	CSI Bridge
DC1a Steel Girder	124.4	114.8	-421.5	-402.4
DC1b Misc Steel (see note 1)	111.7	10.5	-357.1	-34.9
DC1c Forms	-	98.8	-	-326.8
DC1d Deck+Haunches	758.4	658.1	-2418.3	-2176.9
DC2 Parapets	163.8	157.7	-436.1	-420.8
DW Future Wearing Surface	198.4	216.2	-528.2	-576.8

Notes:

(1) FHWA Example includes both Misc Steel and Forms

Live Loads

Live Loads

The following load cases were considered for live load analysis:

- “LL - 1 lane”: single lane is applied to the entire bridge and distribution into individual girders is considered in the design phase by using user-defined user distribution factors

Live load distribution factors

- User-specified live load distribution factors were used to match the factors used in the “FHWA LRFD Steel Bridge Design Example”, p. 3-22

  The following distribution were used:
- Interior girder moment: 0.696
- Exterior girder moment: 0.892
- Interior girder shear: 0.935
- Exterior girder shear: 0.795

Check Live Load Results

Check Live Load Effects for Interior Girder 1

	LLDF	Location in Span 1			Location in Span 1		
		0.5L			1.0L		
		FHWA Example	CSI Bridge		FHWA Example	CSI Bridge	
			Per Lane	Per Girder		Per Lane	Per Girder
Maximum Positive Moment [kip-ft]	0.696	1857	2741.4	**1908.0**	983	6.8	**4.7**
Maximum Negative Moment [kip-ft]	0.696	-968	-938.2	**-653.0**	-2450	-3375.8	**-2349.6**
Maximum Positive Shear [kips]	0.935	42.5	67.9	**63.5**	35.8	137.9	**128.9**
Maximum Negative Shear [kips]	0.935	-62.2	-37.3	**-34.9**	-131.4	-0.1	**-0.1**

Notes:

(1) LLDF stands for Live Load Distribution Factor

(2) The yellow highligthed cells can still be considered to provide reasonable agreement between the FHWA Example and CSI Bridge if the following is considered:

- FHWA example shears seems to follow different sign convention from CSI bridge
- FHWA example maximum positive moment at 1.0L does not seem to be correct (it should be essentially zero)
- FHWA example maximum positive shear at 1.0L does not seem to be correct (it should be essentially zero)

Strength Limit State

ok, 4/16/2011

Strength Limit State Verification Approach

- Detailed verification of CSiBridge results was performed for locations at 0.417L (50') and 1.0L (120') of the first span that correspond to the locations of the maximum positive moment and the maximum negative moment.

Design Request Definition

- User-specified live load distribution factors (LLFD) were used to exactly match the LLDF used in the FHWA example.

Section at 0.417L (50') for G2 – Positive Flexure

- Checking procedure: CSiBridge output tables for girder G2 at station 50ft were printed (each intermediate parameter or result on one row) and the checked against independent hand calculations and spreadsheet calculations
- Reasonable agreement was achieved. See attachment with hand calculations for details.

Demands for G2 Section at 0.417L (50') – Positive Flexure

Load Case	Unfactored Moment [kip-ft]	Factor	Factored Moments [kip-ft]	Cumulative Factored Moments [kip-ft]	Model C Output (Staged Constr.)	Comment
a. DC1a girders	136.5					
a. DC1b misc steel	12.7					
a. DC1c forms	118.7					
a. DC1d deck+haunches	790.9					
a. DC1 total	1058.8	1.25	1323.5	1323.5	1309.9	
a. DC2 parapets	182.4	1.25	228.0	1551.5	1537.3	
a. DW fut. w. surface	250	1.5	375.0	1926.5	1911.4	
LL+I - 1 lane (0.696 distr. Factor)	1963.07	1.75	3435.4	5361.9		
negative LL moment =	-544.2					

Total for “a- Strength 1” load combination = 5361.9

ok, 4/16/2011

Demands for G2 Section at 0.417L (50') – Shear

Load Case	Unfactored Shear [kips]	Factor	Factored Shear [kips]	Comment
a. DC1a girders	1.4			
a. DC1b misc steel	0.1			
a. DC1c forms	0.5			
a. DC1d deck+haunches	8.2			
a. DC1 total	10.2	1.25	12.75	
DC2 parapets	0.4	1.25	0.5	
a. DW fut. w. surface	0.5	1.5	0.75	
a. LL+I (0.935 distr. Factor)	40.39	1.75	70.7	

Total for “Strength 1” load combination = 84.7

Section at 1.000L (120') for G2 – Negative Flexure

- Checking procedure: CSiBridge output tables for girder G2 at station 120ft were printed (each intermediate parameter or result on one row) and the checked against independent hand calculations and spreadsheet calculations
- Reasonable agreement with hand calculations was achieved. See attachments for details.

Demands for G2 Section at 1.000L (120') – Negative Flexure

Load case	Unfactored Moment [kip-ft]	Factor	Factored Moments [kip-ft]	Cumulative Factored Moments [kip-ft]	Model C Output (Stage Constr.)	Comment
a. DC1a girders	-401.8					
a. DC1b misc steel	-34.3					
a. DC1c forms	-321.9					
a. DC1d deck+haunches	-2151.5					
a. DC1 total	-2909.5	1.25	-3636.9	-3636.9		
a. DC2 parapets	-423.5	1.25	-529.4	-4166.3		
a. DW fut. w. surface	-580.5	1.5	-870.8	-5037.0		
LL+I - 1 lane (0.696 distr. Factor)	-2345.32	1.75	-4104.3	-9141.3		

Total for “a- Strength 1” load combination = -9141.3

Notes:

ok, 4/16/2011

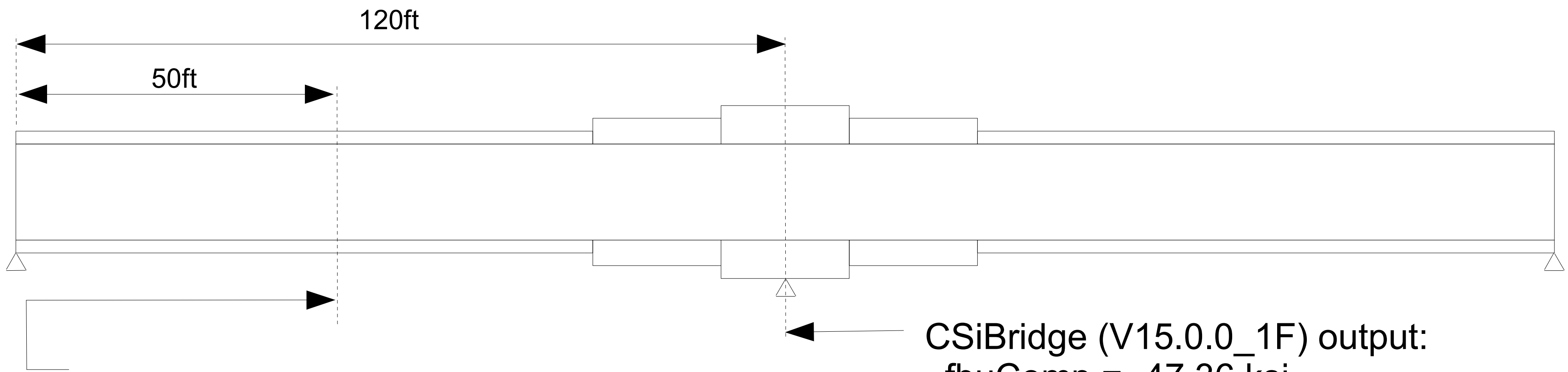
Demands for G2 Section at 1.000L (120') – Shear

Load Case	Unfactored Shear	Factor	Factored Shear		FHWA Example (p. 3-31) Unfactored Shear	FHWA Example (p. 3-31) Factored Shear
	[kips]		[kips]		[kips]	[kips]
a. DC1a girders	16.2					
a. DC1b misc steel	1.1					
a. DC1c forms	10.3					
a. DC1d deck+haunches	75.0					
a. DC1 total	102.6	1.25	128.25		114.7	143.4
a. DC2 parapets	15	1.25	18.75		16.4	20.5
a. DW fut. w. surface	20.6	1.5	30.9		19.8	29.7
LL+I - 1 lane (0.935 distr. Factor)	123.61	1.75	216.31		131.4	230.0

Total for “a- Strength 1” load combination = **394.21**

Sum = 423.5

Strength Limit State Summary - Flexure and Shear



CSiBridge (V15.0.0_1F) output:
MuPos = 5340 kip-ft
MrPos = 6151 kip-ft
D/C ratio = 0.868

Vu = 84.39 kips
Vr = 306.50 kips
D/C ratio = 0.275

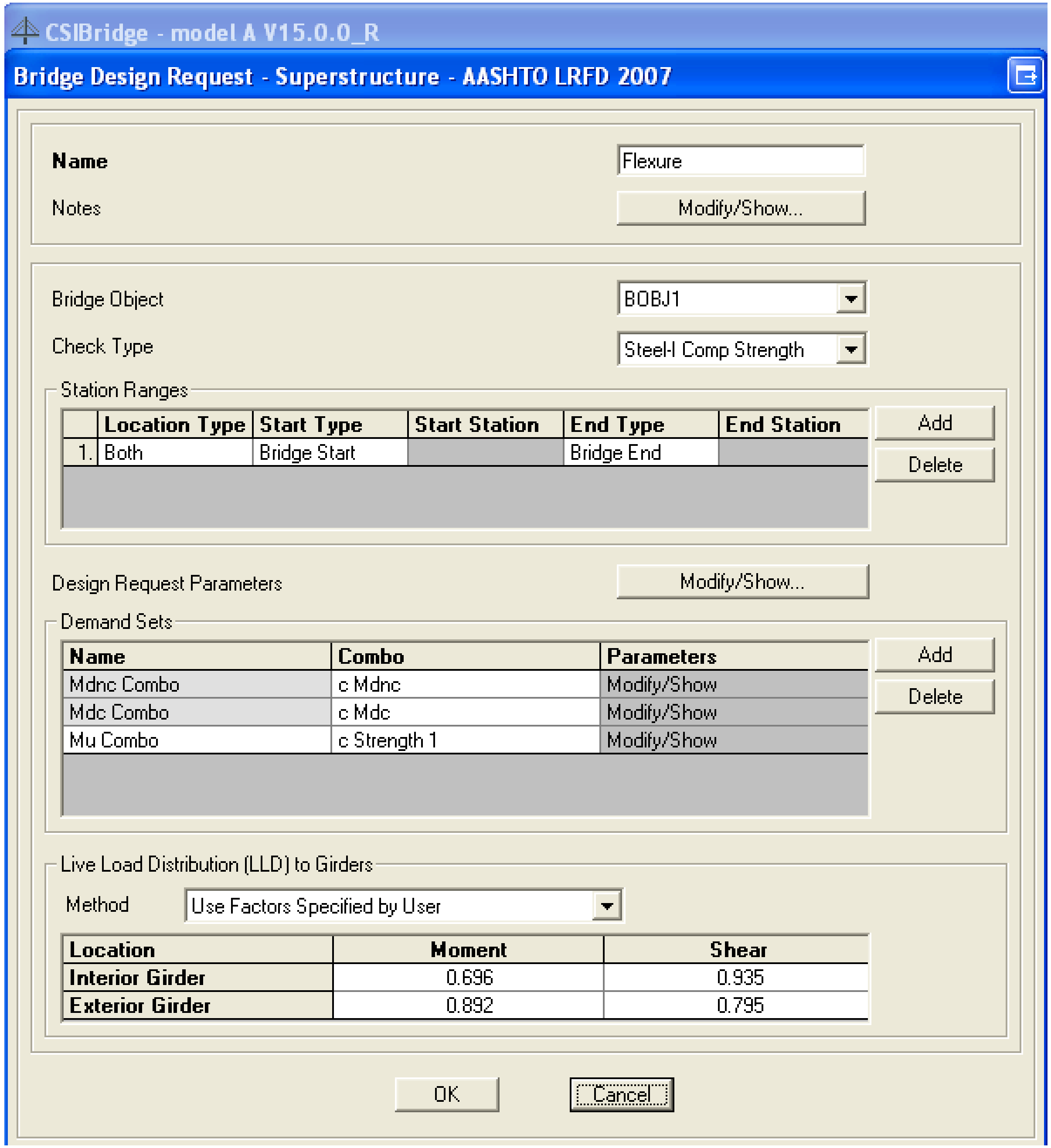
CSiBridge (V15.0.0_1F) output:
fbuComp = -47.36 ksi
FrcNeg = -41.56 ksi
D/C ratio = 1.139

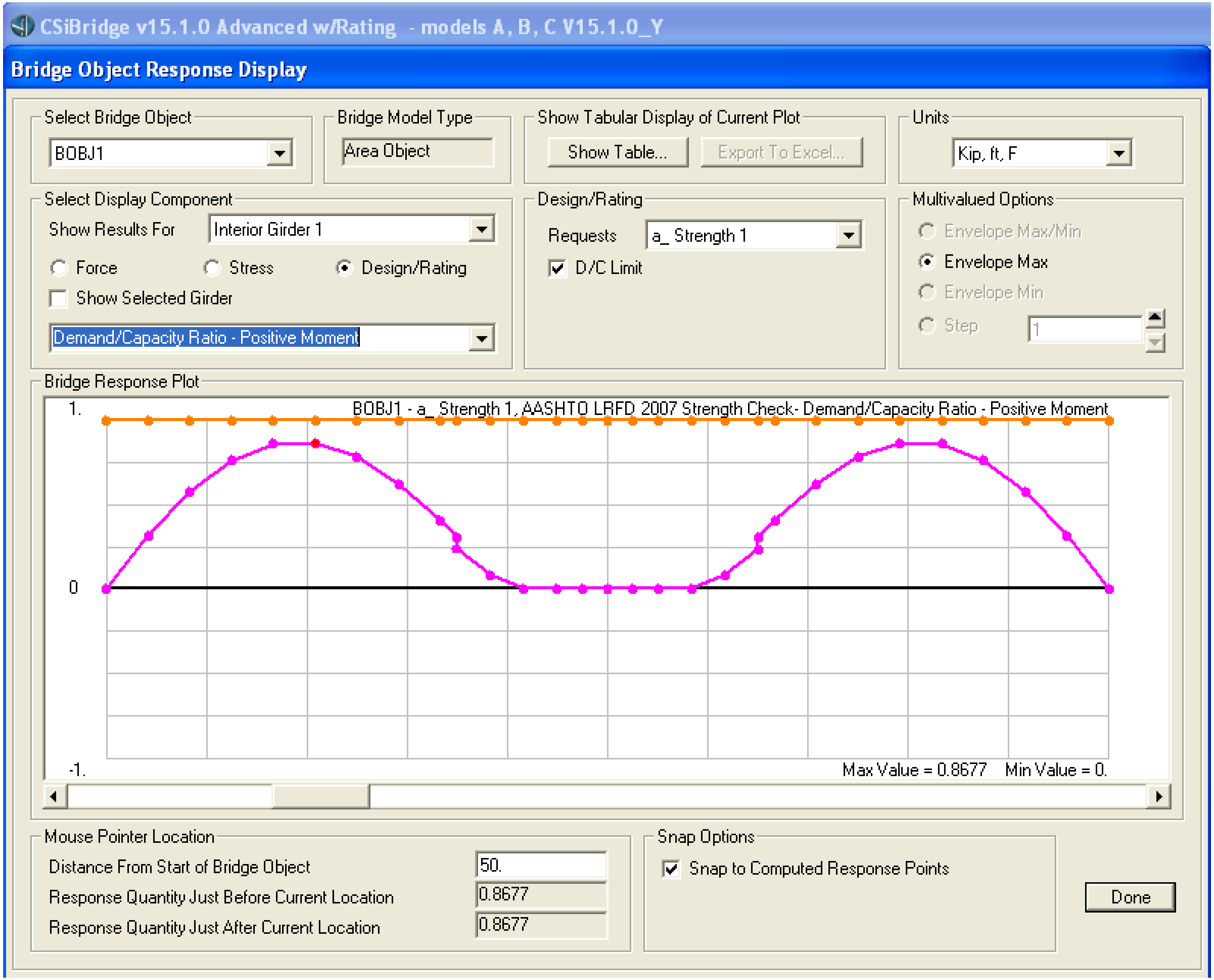
Vu = 391.28 kips
Vr = 306.50 kips
D/C ratio = 1.276

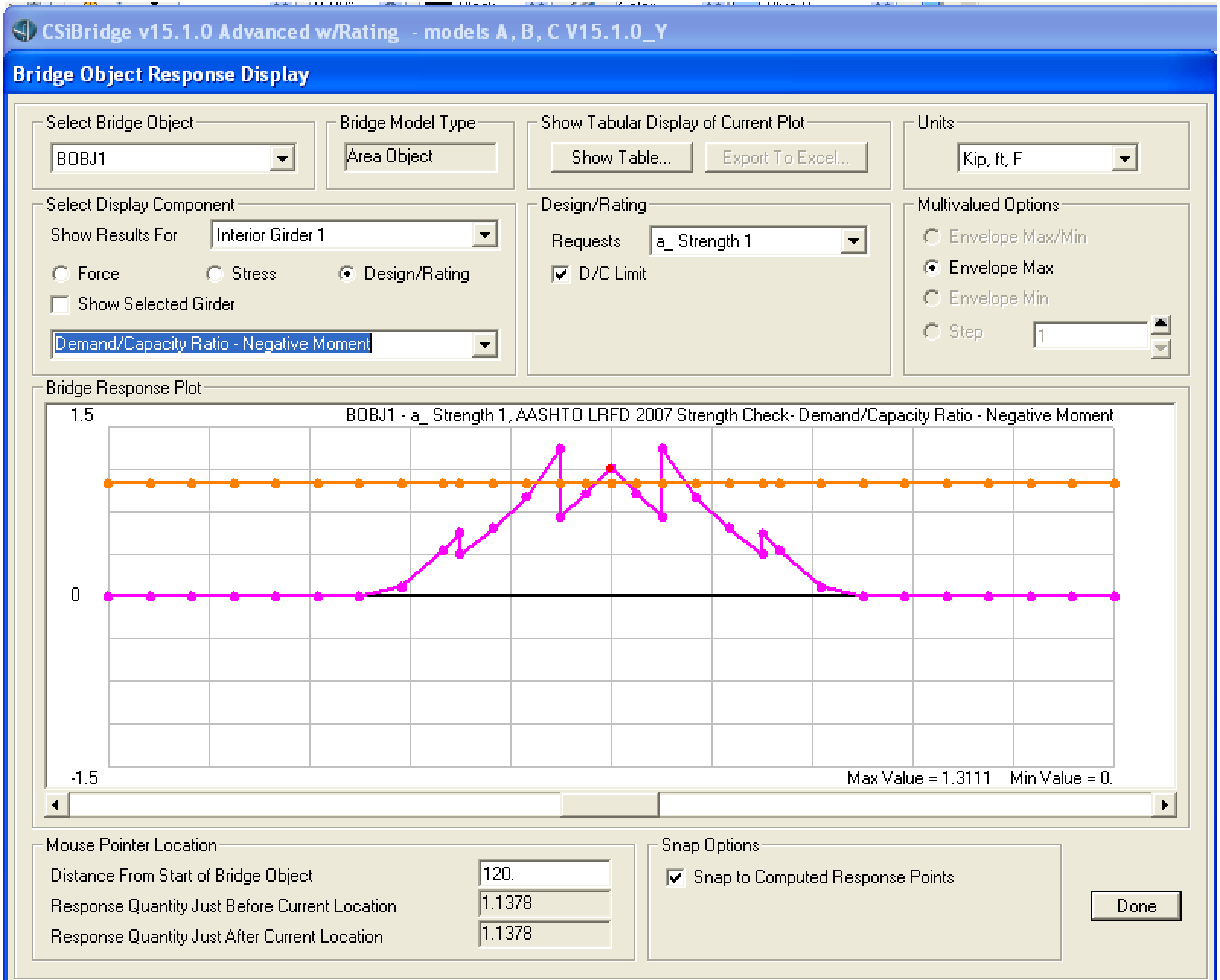
Screenshots of Design Results

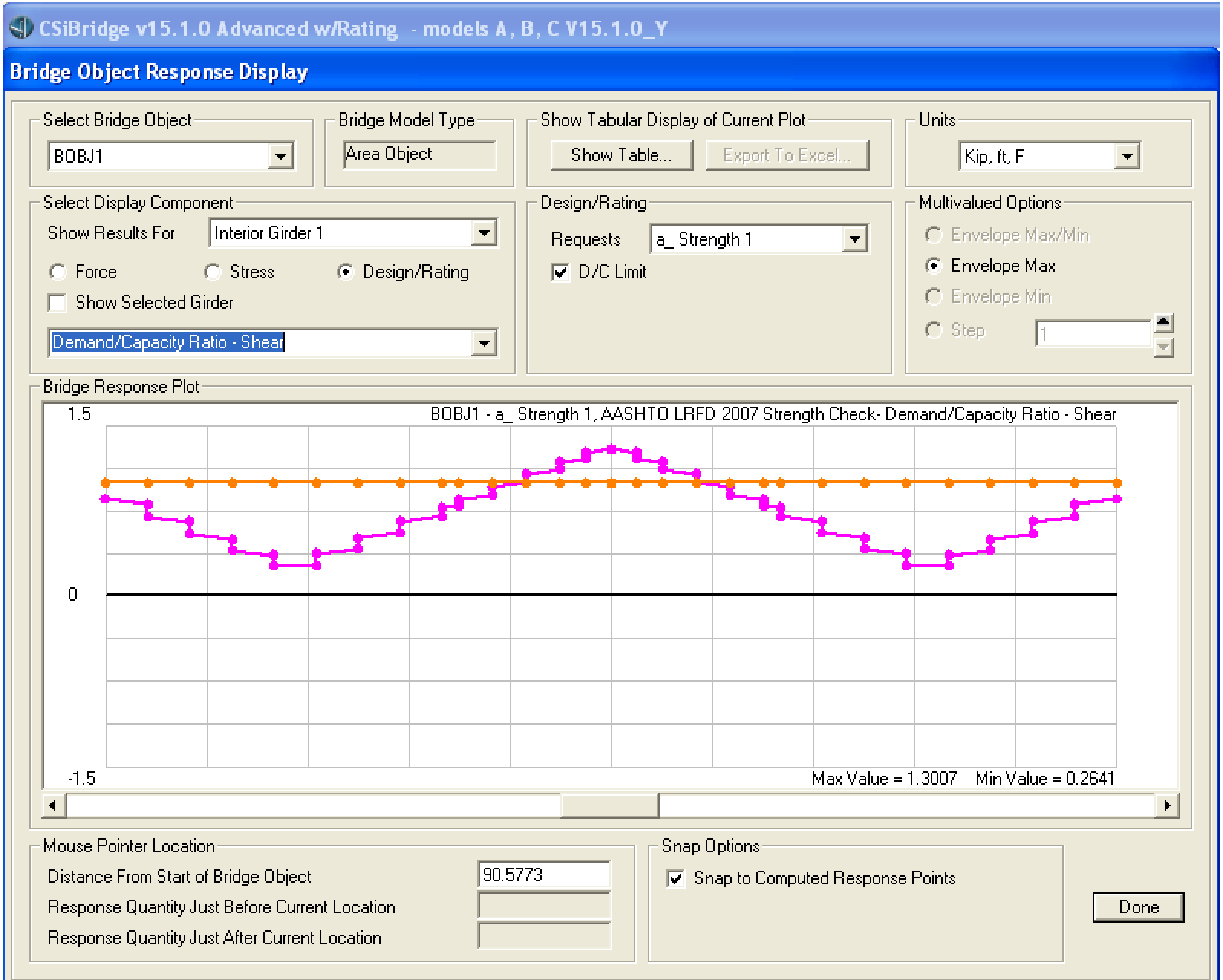
Screenshots of Design Results

- The following pages contain screenshots of selected design output from CSiBridge.


CSIBridge - model A V15.0.0_R
Bridge Design Request - Superstructure - AASHTO LRFD 2007
Name
Flexure
Notes
Modify/Show...
Bridge Object
BOBJ1
Check Type
Steel-I Comp Strength
Station Ranges
Location Type
Start Type
Start Station
End Type
End Station
1.
Both
Bridge Start
Bridge End
Add
Delete
Design Request Parameters
Modify/Show...
Demand Sets
Name
Combo
Parameters
Mdnc Combo
c Mdnc
Modify/Show
Mdc Combo
c Mdc
Modify/Show
Mu Combo
c Strength 1
Modify/Show
Add
Delete
Live Load Distribution (LLD) to Girders
Method
Use Factors Specified by User
Location
Moment
Shear
Interior Girder
0.696
0.935
Exterior Girder
0.892
0.795
OK
Cancel


CSiBridge v15.1.0 Advanced w/Rating - models A, B, C V15.1.0_Y
Bridge Object Response Display
Select Bridge Object
BOBJ1
Bridge Model Type
Area Object
Show Tabular Display of Current Plot
Show Table...
Export To Excel...
Units
Kip, ft, F
Select Display Component
Show Results For
Interior Girder 1
Force
Stress
Design/Rating
Show Selected Girder
Demand/Capacity Ratio - Positive Moment
Design/Rating
Requests
a_ Strength 1
D/C Limit
Multivalued Options
Envelope Max/Min
Envelope Max
Envelope Min
Step
1
Bridge Response Plot
BOBJ1 - a_ Strength 1, AASHTO LRFD 2007 Strength Check- Demand/Capacity Ratio - Positive Moment
1.
0
-1.
Max Value = 0.8677
Min Value = 0.
Mouse Pointer Location
Distance From Start of Bridge Object
50.
Response Quantity Just Before Current Location
0.8677
Response Quantity Just After Current Location
0.8677
Snap Options
Snap to Computed Response Points
Done


CSiBridge v15.1.0 Advanced w/Rating - models A, B, C V15.1.0_Y
Bridge Object Response Display
Select Bridge Object
BOBJ1
Bridge Model Type
Area Object
Show Tabular Display of Current Plot
Show Table...
Export To Excel...
Units
Kip, ft, F
Select Display Component
Show Results For
Interior Girder 1
Force
Stress
Design/Rating
Show Selected Girder
Demand/Capacity Ratio - Negative Moment
Design/Rating
Requests
a_ Strength 1
D/C Limit
Multivalued Options
Envelope Max/Min
Envelope Max
Envelope Min
Step
1
Bridge Response Plot
BOBJ1 - a_ Strength 1, AASHTO LRFD 2007 Strength Check- Demand/Capacity Ratio - Negative Moment
1.5
0
-1.5
Max Value = 1.3111 Min Value = 0.
Mouse Pointer Location
Distance From Start of Bridge Object
120.
Response Quantity Just Before Current Location
1.1378
Response Quantity Just After Current Location
1.1378
Snap Options
Snap to Computed Response Points
Done



CSiBridge v15.1.0 Advanced w/Rating - models A, B, C V15.1.0_Y
Bridge Object Response Display
Select Bridge Object
BOBJ1
Bridge Model Type
Area Object
Show Tabular Display of Current Plot
Show Table...
Export To Excel...
Units
Kip, ft, F
Select Display Component
Show Results For
Interior Girder 1
Force
Stress
Design/Rating
Show Selected Girder
Demand/Capacity Ratio - Shear
Design/Rating
Requests
a_ Strength 1
D/C Limit
Multivalued Options
Envelope Max/Min
Envelope Max
Envelope Min
Step
1
Bridge Response Plot
BOBJ1 - a_ Strength 1, AASHTO LRFD 2007 Strength Check- Demand/Capacity Ratio - Shear
1.5
0
-1.5
Max Value = 1.3007 Min Value = 0.2641
Mouse Pointer Location
Distance From Start of Bridge Object
90.5773
Response Quantity Just Before Current Location
Response Quantity Just After Current Location
Snap Options
Snap to Computed Response Points
Done

Strength Limit State - With Pours

- The analysis is similar to the Strength Limit State without pours, but the deck and hauch load is applied in pours.

Demands for G2 Section at 0.417L (50') – Positive Flexure

Load Case	Unfactored Moment [kip-ft]	Factor	Factored Moment	Cumulative Factored Moments [kip-ft]	Model C Output (Staged Constr.)	Comment
a. DC1a girders	136.5					
a. DC1b misc steel	12.7					
a. DC1c forms	118.7					
a. DC1d pour 1 (0-80)	1031.11					
a. DC1d pour 2 (160-240)	-366.47					
a. DC1d pour 3 (80-160)	99.15					
a. DC1 total	1031.69	1.25	1289.61	1289.6		
a. DC2 parapets	182.4	1.25	228	1517.6		
a. DW fut. w. surface	250	1.5	375	1892.6		
LL+I - 1 lane (0.696 distr. Factor)	1963.07	1.75	3435.37	5328.0		
negative LL moment =	-544.2					

Total for “a- Strength 1 (pours)” load combination = 5327.98

Notes:

Demands for G2 Section at 1.000L (120') – Negative Flexure

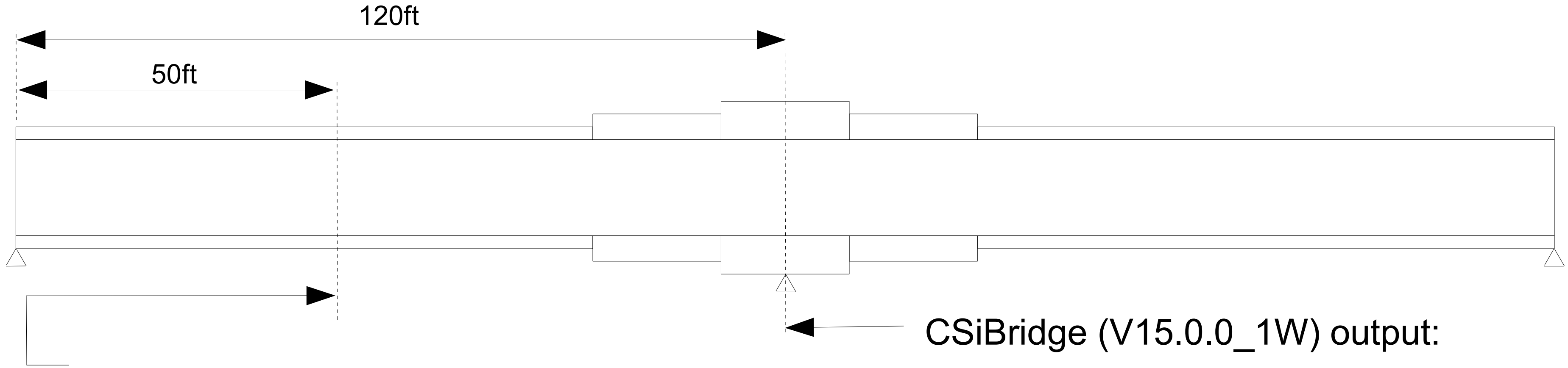
Load Case	Unfactored Moment [kip-ft]	Factor	Factored Moment [kip-ft]	Cumulative Factored Moments [kip-ft]	Model C Output (Staged Constr.)	Comment
a. DC1a girders	-401.79					
a. DC1b misc steel	-33.51					
a. DC1c forms	-314.24					
a. DC1d pour 1 (0-80)	-762.79					
a. DC1d pour 2 (160-240)	-894.49					
a. DC1d pour 3 (80-160)	-552.18					
a. DC1 total	-2959	1.25	-3698.75	-3698.8		
a. DC2 parapets	-417.72	1.25	-522.15	-4220.9		
a. DW fut. w. surface	-572.65	1.5	-858.98	-5079.9		
LL+I - 1 lane (0.696 distr. Factor)	-2336.22	1.75	-4088.39	-9168.3		

Total for “a- Strength 1 (pours)” load combination = -9168.26

Notes:

Fatigue Limit State

Fatigue Limit State Summary - Web Fatigue



CSiBridge (V15.0.0_1W) output:

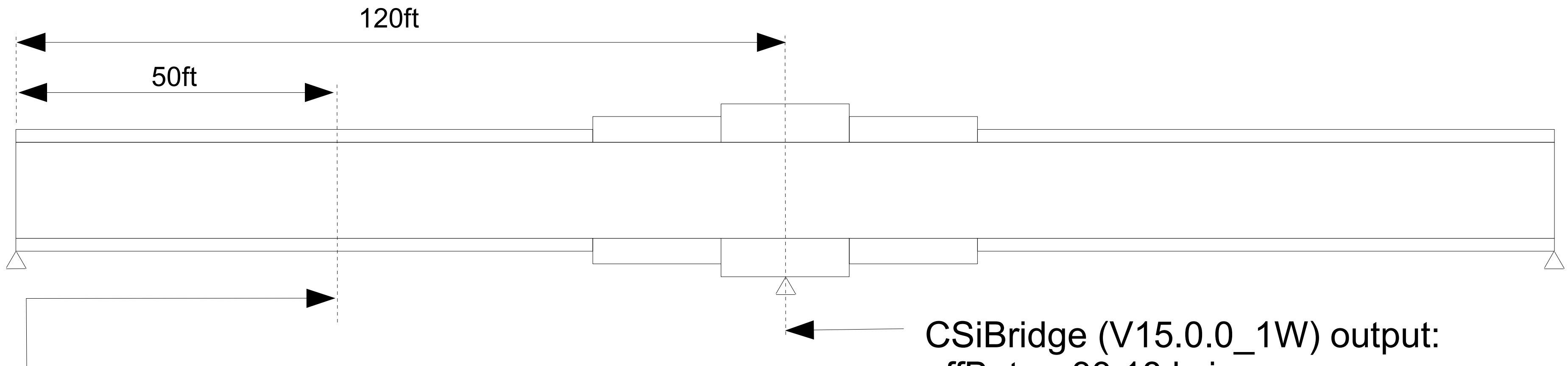
Vu = 12.89 kips
Vcr = 368.56 kips
D/C ratio = 0.034

CSiBridge (V15.0.0_1W) output:

Vu = 32.58 kips
Vcr = 368.56 kips
D/C ratio = 0.088

Service Limit State

Service Limit State Summary - Flexure



CSiBridge (V15.0.0_1W) output:
ffBot = 41.8 ksi
FrBot = 47.5 ksi
D/C ratio = 0.880

CSiBridge (V15.0.0_1W) output:
ffBot = -33.18 ksi
FrBot = 47.28 ksi
D/C ratio = 0.701

Constructability Limit State

General Comments

- Use Strength IV load combination for constructability limit state (high dead load to live load ratio); relevant AASHTO LRFD 2008 References:
  - Art. 3.4.1 (Load Factors and Load Combinations) on p. 3-8
  - Table 3.4.1-1 (Load Combinations and Load Factors) on p. 3-13
  - Art. 3.4.2 (Load Factors for Construction Loads) on p. 3.14-1
- Other References:
  - AASHTO LRFD 2008, Figure C6.4.1 (Flowchart for LRFD Article 6.10.3 - Constructibility)
  - FHWA Example p. 3-48 to 3-56

Demands for G2 Section at 0.417L (50') - Flexure

Load Case	Unfactored Moment [kip-ft]	Factor	Cumulative Factored Moment [kip-ft]	Comment
a. DC1a girders	136.77			
a. DC1b misc steel	12.36			
a. DC1c form	115.92			
a. DC1d deck+haunches	781.58			
a. DC1 total	1046.63	1.5	1569.95	The factored moment represents the "a- Strength 4" load combination

Notes:
(1) "a- Strength 4" load combination is used to obtain demands for the constructability limit state.
(2) The above results were obtained from CSiBridge V15.1.0_S
(3) G2 is interior girder 1

ok, 4/25/2011

Demands for G2 Section at 1.000L (120') - Flexure

Load Case	Unfactored Moment [kip-ft]	Factor	Cumulative Factored Moment [kip-ft]	Comment
a. DC1a girders	-401.79			
a. DC1b misc steel	-33.57			
a. DC1c form	-314.7			
a. DC1d deck+haunches	-2127.89			
a. DC1 total	-2877.95	1.5	-4316.93	The factored moment represents the "a- Strength 4" load combination

Notes:
(1) "a- Strength 4" load combination is used to obtain demands for the constructability limit state.
(2) The above results were obtained from CSiBridge V15.1.0_S.
(3) G2 is interior girder 1

ok, 4/25/2011

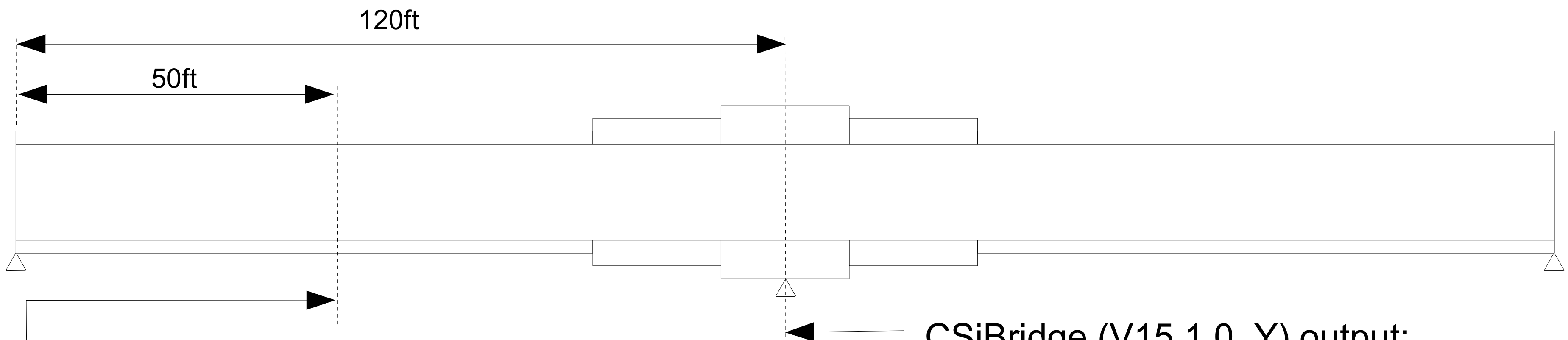
Demands for G2 Section at 1.000L (120') - Shear

Load Case	Unfactored Moment [kip-ft]	Factor	Cumulative Factored Moment [kip-ft]	Comment
a. DC1a girders	16.19			
a. DC1b misc steel	1.13			
a. DC1c form	10.62			
a. DC1d deck+haunches	75.04			
a. DC1 total	102.98	1.5	154.47	The factored moment represents the "a- Strength 4" load combination

Notes:
(1) "a- Strength 4" load combination is used to obtain demands for the constructability limit state.
(2) The above results were obtained from CSiBridge V15.1.0_S.
(3) G2 is interior girder 1

ok, 4/25/2011

Constructability Limit State Summary - Flexure and Shear



CSiBridge (V15.1.0_Y) output:
fbuComp = -25.12 ksi
FrcPos = 38.71 ksi
D/C ratio = 0.649

Vu = 15.02 kips
Vr = 306.50 kips
D/C ratio = 0.049

CSiBridge (V15.1.0_Y) output:
fbuComp = -22.56 ksi
FrcNeg = 41.28 ksi
D/C ratio = 0.547

Vu = 153.50 kips
Vr = 306.50 kips
D/C ratio = 0.501

ok, 5/7/2011

CSiBridge Tabular Output

G2 section at 50' (0.417L)
Strength Limit State

G2 Section at 0.417L (50') Do not use Appendix A

C6.4.4 Flowchart for LRFD Article 6.10.6

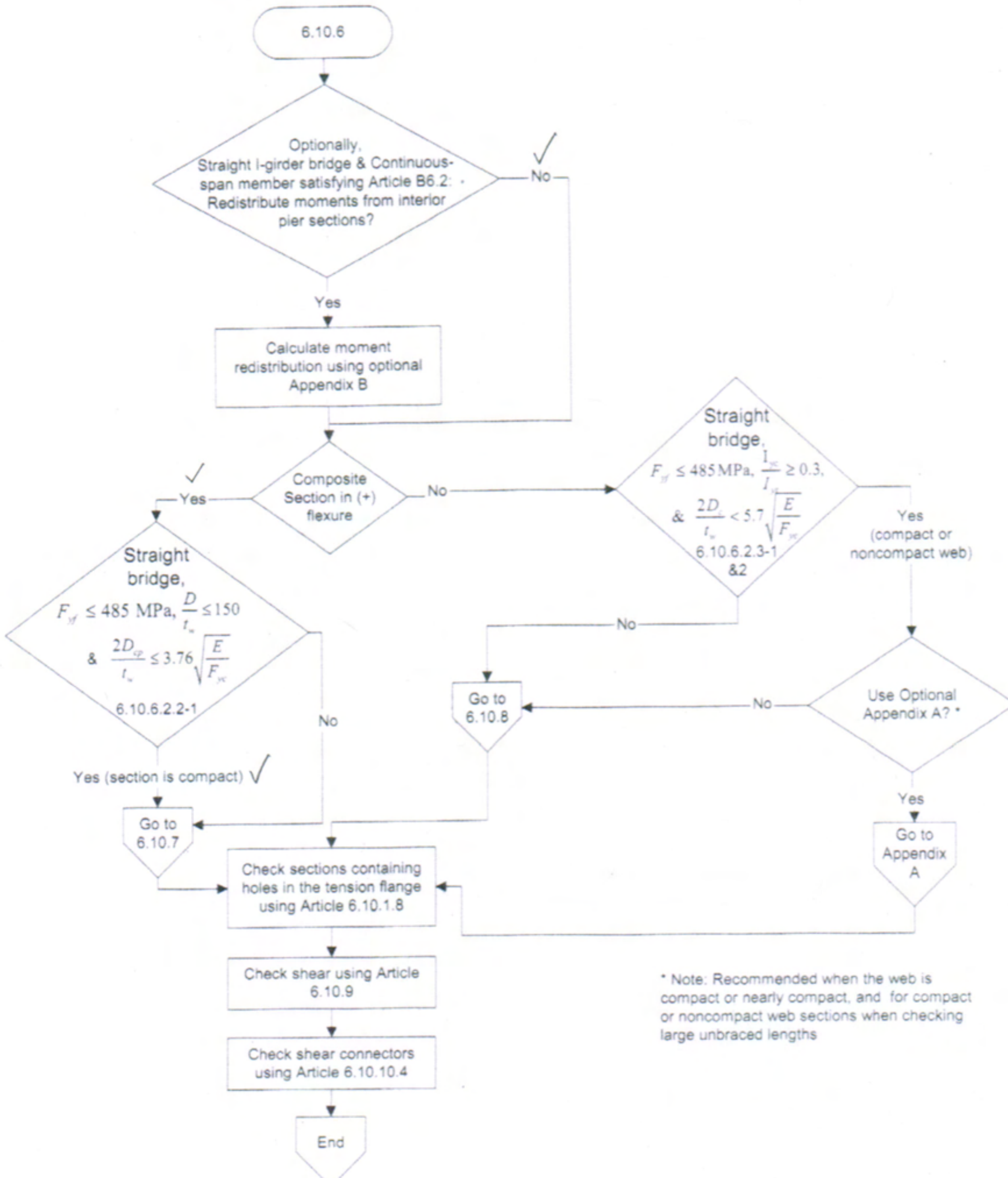


Figure C6.4.4-1 Flowchart for LRFD Article 6.10.6—Strength Limit State.

G2 section at 0.417L

C6.4.5 Flowchart for LRFD Article 6.10.7

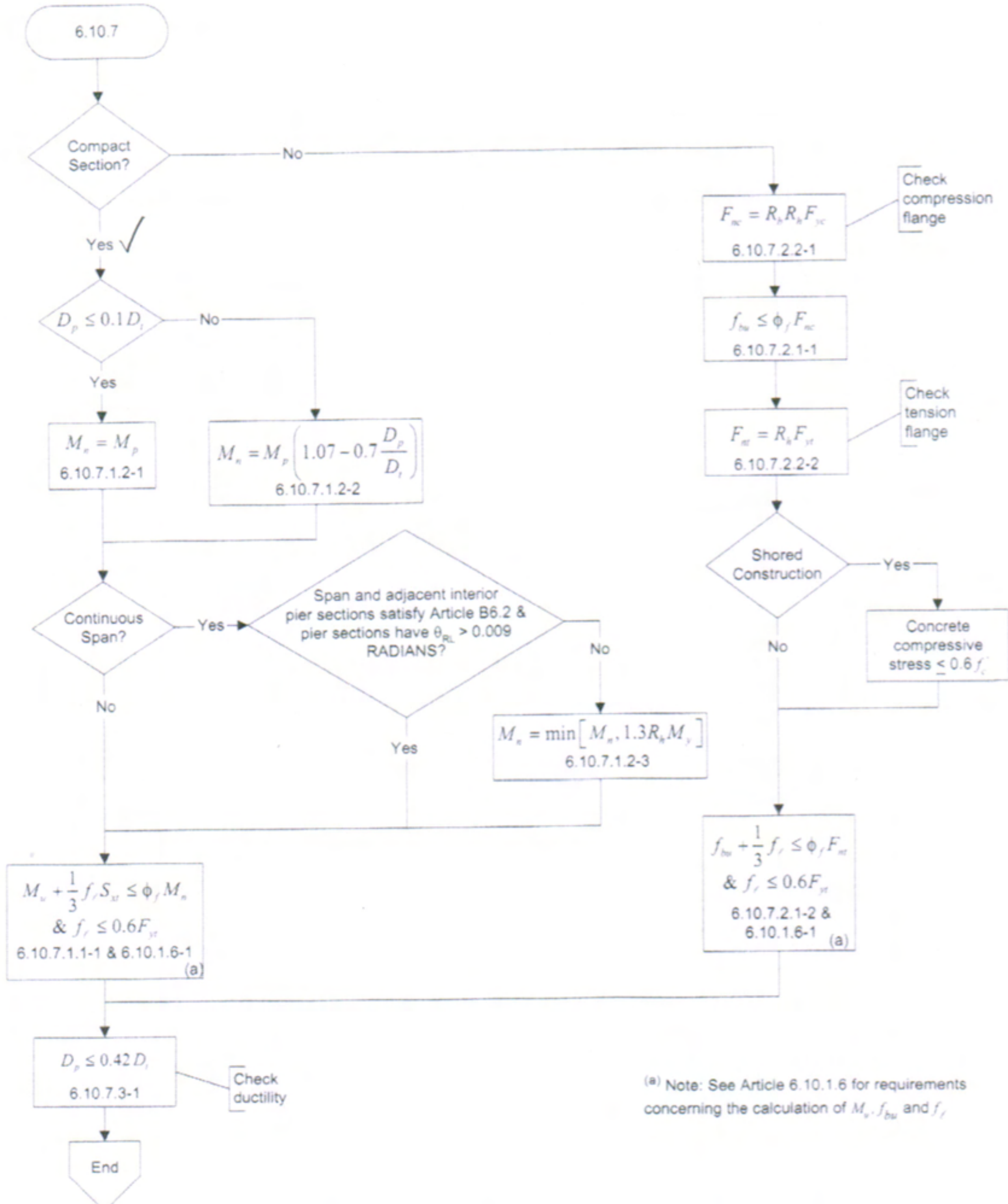


Figure C6.4.5-1 Flowchart for LRFD Article 6.10.7—Composite Sections in Positive Flexure.

TABLE: Bridge Super Design 01 - Design Result Status			
Parameter	Unit	Value Before Station	Value After Station
DesReqName	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

The results on the following 6 pages are for build V15.0.0_1F.

TABLE: Bridge Super Design 29 - AASHTOLRFD07 - SteelICompStrgth-Prop

Parameter	Unit	Value Before Station	Value After Station
Request	Text	Strength 1	Strength 1
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
BeamProp	Text	I-Girder 0.625in T	I-Girder 0.625in T
LLDFactM	Unitless	0.696	0.696
LLDFactV	Unitless	0.935	0.935
ASlabTri	ft2	6.5	6.5
ThSlab	ft	0.66667	0.66667
WSlabEff	ft	9.75	9.75
fcConcSlab	Kip/ft2	576	576
ESlab	Kip/ft2	519119.5	519119.5
nLongTerm	Unitless	3	3
ARebSlabTop	ft2	0	0
ARebSlabBot	ft2	0	0
YRebSlabTop	ft	0	0
YRebSlabBot	ft	0	0
fysLRebar	Kip/ft2	8640	8640
ABeam	ft2	0.3335	0.3335
EBeam	Kip/ft2	4176000	4176000
IxBeam	ft4	1.0668	1.0668
BeamRolled	Yes/No	No	No
ThFlgTop	ft	0.0521	0.0521
WdthFlgTop	ft	1.1667	1.1667
fyFlgTop	Kip/ft2	7200	7200
ThFlgBot	ft	0.0729	0.0729
WdthFlgBot	ft	1.1667	1.1667
fyFlgBot	Kip/ft2	7200	7200
fyrFlgBot	Kip/ft2	5040	5040
LamfBotFlg	Unitless	8.002058	8.002058
LampfBotFlg	Unitless	9.151612	9.151612
LamrfBotFlg	Unitless	16.119553	16.119553
kcBotFlg	Unitless	0.385054	0.385054
CmpctFlgBot	Yes/No	Yes	Yes
DepthWeb	ft	4.5	4.5
ThickWeb	ft	0.0417	0.0417
fyWeb	Kip/ft2	7200	7200
DcpWebPos	ft	0	0
DcWebNeg	ft	2.08172	2.08172
DcpWebNeg	ft	1.95902	1.95902
LamwWeb	Unitless	99.842831	99.842831
LampwDcWeb	Unitless	72.202398	72.202398
LampwDcpWeb	Unitless	67.946726	67.946726
LamrwWeb	Unitless	137.274178	137.274178
CmpctWebNeg	Yes/No	No	No
RpcWeb	Unitless	0	0
RptWeb	Unitless	0	0
rt	ft	0.29393	0.29393
J	ft4	0	0
RbPos	Unitless	1	1
RbNeg	Unitless	1	1
RhPos	Unitless	1	1
RhNeg	Unitless	0.985987	0.985987
CmpctGrdPos	Yes/No	Yes	Yes
CmpctGrdNeg	Yes/No	Yes	Yes

SxSteelBot	ft3	0.495122	0.495122
SxSteelTop	ft3	0.431837	0.431837
SxLTermBPos	ft3	0.699063	0.699063
SxLTermTPos	ft3	2.211966	2.211966
SxSTermBPos	ft3	0.761928	0.761928
SxSTermTPos	ft3	10.383153	10.383153
SxCompBNeg	ft3	0.495122	0.495122
SxCompTNeg	ft3	0.431837	0.431837
MpPos	Kip-ft	7506.8302	7506.8302
MpNeg	Kip-ft	3892.4384	3892.4384
PNADistPos	ft	0.503	0.503
PNADistLmt	ft	2.2225	2.2225
PNADistNeg	ft	3.49931	3.49931
MuDNC	Kip-ft	1304.1114	1304.1114
MuDCLTerm	Kip-ft	596.4576	596.4576
MyPos	Kip-ft	4731.7565	4731.7565
MyNegCtr	Kip-ft	3110.5085	3110.5085
MyNegBot	Kip-ft	3566.3448	3566.3448
MyNegTop	Kip-ft	3110.5085	3110.5085
Lp	ft	7.07873	7.07873
Lr	ft	26.58006	26.58006
Lb	ft	20	20
BeamPBklShr	Yes/No	No	No

TABLE: Bridge Super Design 30 - AASHTOLRFD07 - SteelICompStrgth-FlexPos

Parameter	Unit	Value Before Station	Value After Station
Request	Text	Strength 1	Strength 1
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
Combo	Text	c Strength 1	c Strength 1
StepType	Text	Max	Min
Step	Text	0	0
DSet	Text	Mu Combo	Mu Combo
CodeEqtn	Text	6.10.7.1.1-1 Compact Section Positive Flexure	6.10.7.1.1-1 Compact Section Positive Flexure
MuPos	Kip-ft	5340.5274	5340.5274
fl	Kip/ft2	0	0
MrPos	Kip-ft	6151.2834	6151.2834
Pu	Kip	0	0
MuNonComp	Kip-ft	0	0
MuLTerm	Kip-ft	0	0
MuSTerm	Kip-ft	0	0
fbuComp	Kip/ft2	0	0
fbuTens	Kip/ft2	0	0
FrcPos	Kip/ft2	0	0
FrtPos	Kip/ft2	0	0
DCRatio	Unitless	0.868197	0.868197

TABLE: **Bridge Super Design 31 - AASHTOLRFD07 - SteelICompStrgth-FlexNeg**

Parameter	Unit	Value Before Station	Value After Station
Request	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling	6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling
MuNeg	*Kip-ft*	0	0
fl	*Kip/ft2*	0	0
MncFLB	*Kip-ft*	0	0
MncLTB	*Kip-ft*	0	0
MrcNeg	*Kip-ft*	0	0
MrtNeg	*Kip-ft*	0	0
Pu	*Kip*	2.347E-08	4.145E-09
MuNonComp	*Kip-ft*	0	0
MuLTerm	*Kip-ft*	0	0
MuSTerm	*Kip-ft*	0	0
fbuComp	*Kip/ft2*	0	0
fbuTens	*Kip/ft2*	0	0
FncFLB	*Kip/ft2*	7099.11	7099.11
FncLTB	*Kip/ft2*	5734.78	5734.78
FrcNeg	*Kip/ft2*	5734.78	5734.78
FrtNeg	*Kip/ft2*	7099.11	7099.11
DCRatio	*Unitless*	0	0

TABLE: Bridge Super Design 32 - AASHTOLRFD07 - SteelICompStrgth-Shear			
Parameter	Unit	Value Before Station	Value After Station
Request	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Min
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.9.2-1	6.10.9.2-1
PanelType	*Text*	Unstiffened	Unstiffened
Vu	*Kip*	84.393	114.07
Vr	*Kip*	306.502	306.502
Vcr	*Kip*	306.502	306.502
Vp	*Kip*	783.949	783.949
C	*Unitless*	0.390971	0.390971
k	*Unitless*	5	5
d0	*ft*	0	0
d0req	*ft*	0	0
VrWithD0req	*Kip*	306.502	306.502
DCRatio	*Unitless*	0.275342	0.372168

G2 section at 120' (1.000L)
Strength Limit State

G2 section at 1.000L

Do not use Appendix A

C6.4.4 Flowchart for LRFD Article 6.10.6

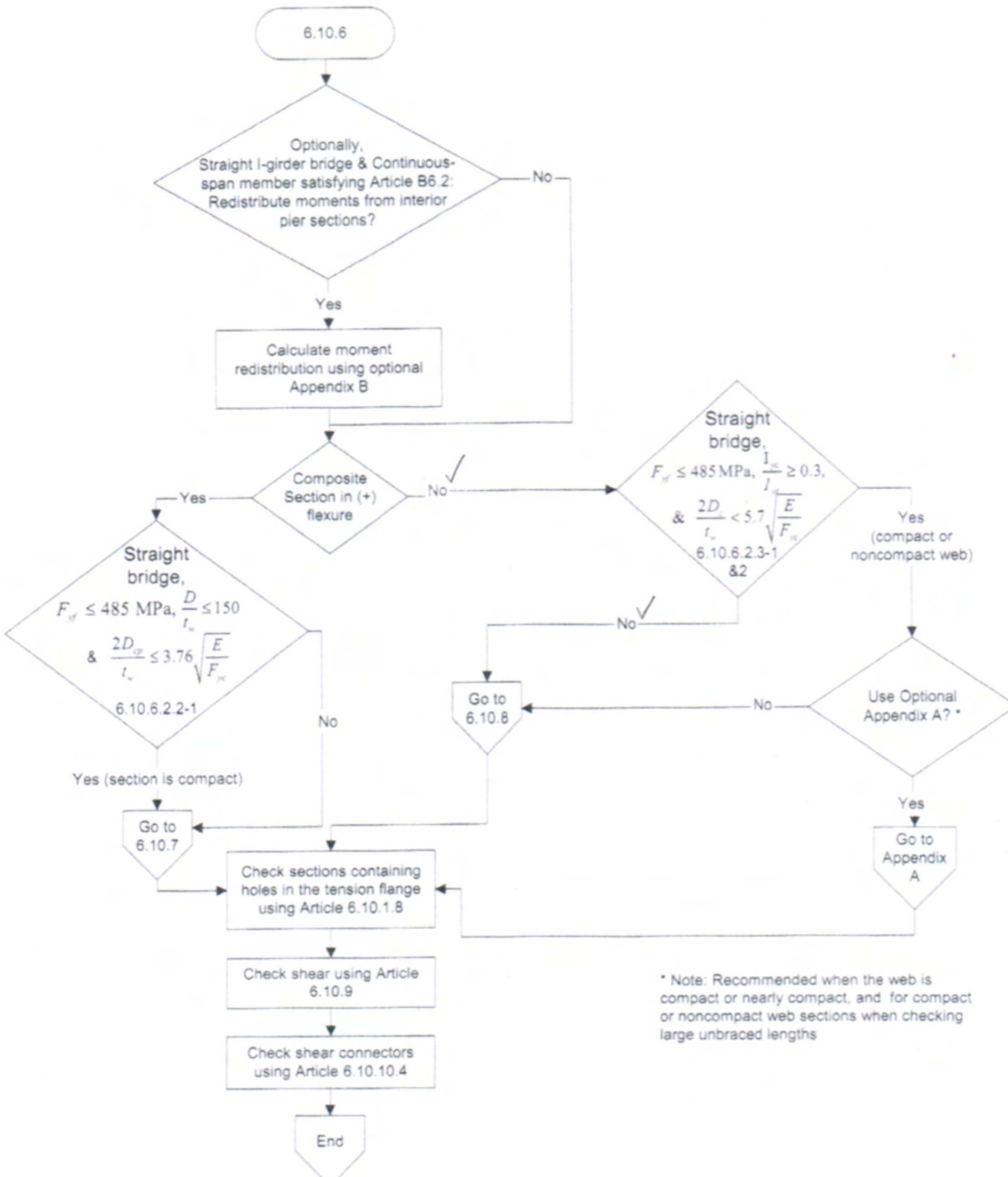
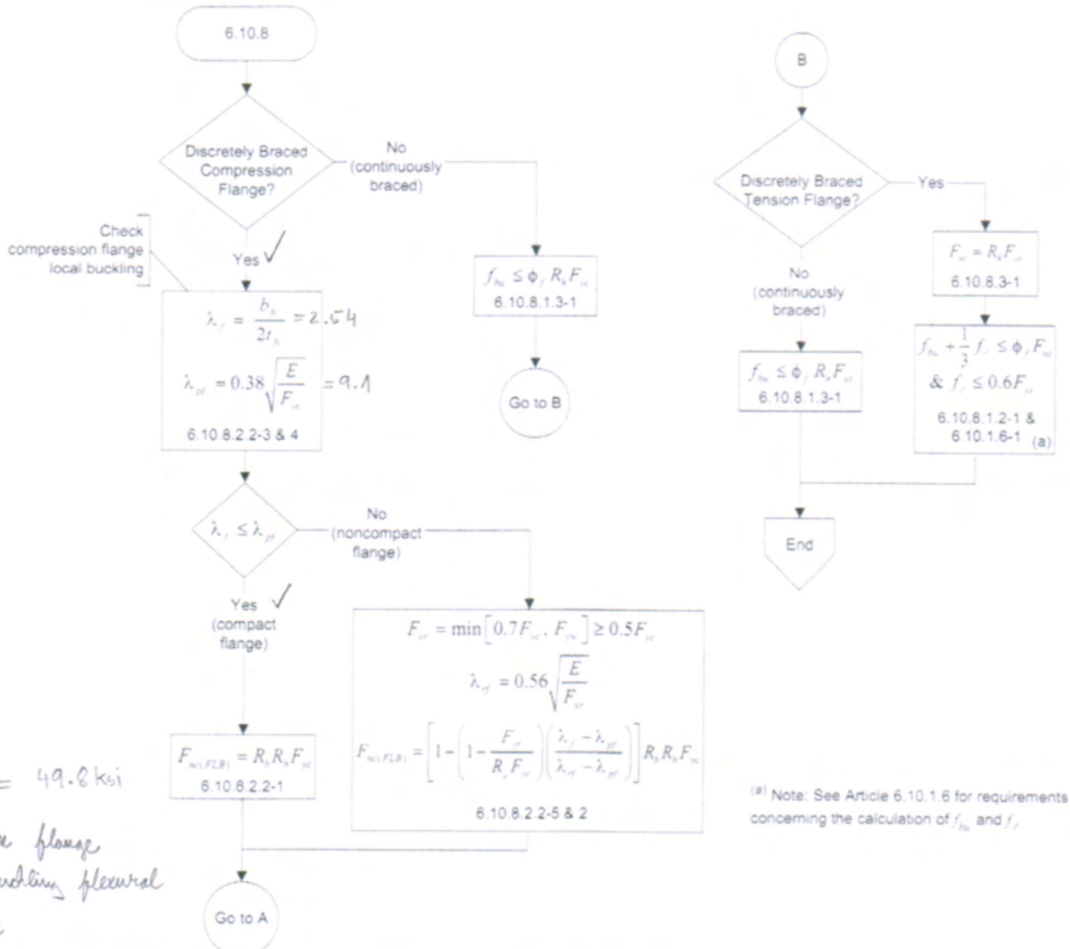


Figure C6.4.4-1 Flowchart for LRFD Article 6.10.6—Strength Limit State.

G2 section at 1.000L

C6.4.6 Flowchart for LRFD Article 6.10.8



$F_{nc(FLB)}$ = 49.8 ksi
compression flange local buckling flexural resistance

Figure C6.4.6-1 Flowchart for LRFD Article 6.10.8—Composite Sections in Negative Flexure and Noncomposite Sections.

G2 section at 1.000L

A

Check lateral-torsional buckling [c]

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \quad \text{(a)}$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = 7.90$$

6.10.8.2.3-9 & 4

limiting unbraced length to achieve nominal flexural resistance of $R_b R_h F_{yc}$

$L_b = 20'$

$L_b \le L_p$ [b]

No

$$F_{yr} = \min\left[0.7F_{yc}, F_{yw}\right] \ge 0.5F_{yc}$$

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = 29.6$$

6.10.8.2.3-5

limiting unbraced length to achieve onset of nominal yielding

$L_p < L_b \le L_r$

No (slender unbraced length)

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

6.10.8.2.3-8

$$F_{nc(LTB)} = F_{cr} \le R_b R_h F_{yc}$$

6.10.8.2.3-3

Yes (noncompact unbraced length)

$$F_{nc(LTB)} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b R_h F_{yc} \le R_b R_h F_{yc}$$

6.10.8.2.3-2

41.45 ksi

Yes (compact unbraced length)

$$F_{nc(LTB)} = R_b R_h F_{yc}$$

6.10.8.2.3-1

$$F_{nc} = \min\left[F_{nc(FLB)}, F_{nc(LTB)}\right]$$

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc}$$
$$\& \; f_\ell \le 0.6F_{yc}$$

6.10.8.1.1-1 & 6.10.1.6-1 [d]

Go to B

[a] Note: exact $r_t = \dfrac{b_{fc}}{\sqrt{12\left(\frac{h}{d}+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\frac{D^2}{hd}\right)}}$

[b] Note: When $C_b > 1$, see the flowchart for Article D6.4.1 for explicit calculation of the larger bracing limit for which the flexural resistance is given by Eq. (6.10.8.2.3-1)

[c] Note: See Articles 6.10.8.2.3 and C6.10.8.2.3 regarding the treatment of nonprismatic sections

[d] Note: See Article 6.10.1.6 for requirements concerning the calculation of f_{bu} and f_ℓ

Figure C6.4.6-1 (continued) Flowchart for LRFD Article 6.10.8—Composite Sections in Negative Flexure and Noncomposite Sections.

TABLE: Bridge Super Design 01 - Design Result Status			
Parameter	Unit	Value Before Station	Value After Station
DesReqName	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

The results on the following 6 pages are for build V15.0.0_1F.

TABLE: Bridge Super Design 29 - AASHTOLRFD07 - SteelICompStrgth-Prop			
Parameter	Unit	Value Before Station	Value After Station
Request	Text	Strength 1	Strength 1
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	120	120
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
BeamProp	Text	I-Girder 2.5in T	I-Girder 2.5in T
LLDFactM	Unitless	0.696	0.696
LLDFactV	Unitless	0.935	0.935
ASlabTri	ft2	6.5	6.5
ThSlab	ft	0.66667	0.66667
WSlabEff	ft	9.75	9.75
fcConcSlab	Kip/ft2	576	576
ESlab	Kip/ft2	519119.5	519119.5
nLongTerm	Unitless	3	3
ARebSlabTop	ft2	0	0
ARebSlabBot	ft2	0	0
YRebSlabTop	ft	0	0
YRebSlabBot	ft	0	0
fysLRebar	Kip/ft2	8640	8640
ABeam	ft2	0.6981	0.6981
EBeam	Kip/ft2	4176000	4176000
IxBeam	ft4	3.155525	3.155525
BeamRolled	Yes/No	No	No
ThFlgTop	ft	0.2083	0.2083
WdthFlgTop	ft	1.1667	1.1667
fyFlgTop	Kip/ft2	7200	7200
ThFlgBot	ft	0.2292	0.2292
WdthFlgBot	ft	1.1667	1.1667
fyFlgBot	Kip/ft2	7200	7200
fyrFlgBot	Kip/ft2	5040	5040
LamfBotFlg	Unitless	2.545157	2.545157
LampfBotFlg	Unitless	9.151612	9.151612
LamrfBotFlg	Unitless	16.119553	16.119553
kcBotFlg	Unitless	0.385054	0.385054
CmpctFlgBot	Yes/No	Yes	Yes
DepthWeb	ft	4.5	4.5
ThickWeb	ft	0.0417	0.0417
fyWeb	Kip/ft2	7200	7200
DcpWebPos	ft	0	0
DcWebNeg	ft	2.16377	2.16377
DcpWebNeg	ft	1.95763	1.95763
LamwWeb	Unitless	103.777764	103.777764
LampwDcWeb	Unitless	95.593501	95.593501
LampwDcpWeb	Unitless	86.48636	86.48636
LamrwWeb	Unitless	137.274178	137.274178
CmpctWebNeg	Yes/No	No	No
RpcWeb	Unitless	0	0
RptWeb	Unitless	0	0
rt	ft	0.32814	0.32814
J	ft4	0	0
RbPos	Unitless	1	1
RbNeg	Unitless	1	1
RhPos	Unitless	1	1
RhNeg	Unitless	0.995361	0.995361
CmpctGrdPos	Yes/No	Yes	Yes
CmpctGrdNeg	Yes/No	Yes	Yes

SxSteelBot	ft3	1.318667	1.318667
SxSteelTop	ft3	1.240119	1.240119
SxLTermBPos	ft3	1.513569	1.513569
SxLTermTPos	ft3	2.831093	2.831093
SxSTermBPos	ft3	1.624842	1.624842
SxSTermTPos	ft3	6.768451	6.768451
SxCompBNeg	ft3	1.318667	1.318667
SxCompTNeg	ft3	1.240119	1.240119
MpPos	Kip-ft	14020.0069	14020.0069
MpNeg	Kip-ft	10170.3594	10170.3594
PNADistPos	ft	0.85978	0.85978
PNADistLmt	ft	2.35375	2.35375
PNADistNeg	ft	3.50071	3.50071
MuDNC	Kip-ft	-3567.8914	-3567.8914
MuDCLTerm	Kip-ft	-1341.4861	-1341.4861
MyPos	Kip-ft	12630.7233	12630.7233
MyNegCtr	Kip-ft	8932.5399	8932.5399
MyNegBot	Kip-ft	9498.3148	9498.3148
MyNegTop	Kip-ft	8932.5399	8932.5399
Lp	ft	7.90269	7.90269
Lr	ft	29.674	29.674
Lb	ft	20	20
BeamPBklShr	Yes/No	Yes	Yes

TABLE: Bridge Super Design 30 - AASHTOLRFD07 - SteelICompStrgth-FlexPos

Parameter	Unit	Value Before Station	Value After Station
Request	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.7.1.1-1 Compact Section Positive Flexure	6.10.7.1.1-1 Compact Section Positive Flexure
MuPos	*Kip-ft*	0	0
fl	*Kip/ft2*	0	0
MrPos	*Kip-ft*	14020.0069	14020.0069
Pu	*Kip*	0	0
MuNonComp	*Kip-ft*	0	0
MuLTerm	*Kip-ft*	0	0
MuSTerm	*Kip-ft*	0	0
fbuComp	*Kip/ft2*	0	0
fbuTens	*Kip/ft2*	0	0
FrcPos	*Kip/ft2*	0	0
FrtPos	*Kip/ft2*	0	0
DCRatio	*Unitless*	0	0

TABLE: Bridge Super Design 31 - AASHTOLRFD07 - SteelICompStrgth-FlexNeg

Parameter	Unit	Value Before Station	Value After Station
Request	*Text*	Strength 1	Strength 1
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Min	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling	6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling
MuNeg	*Kip-ft*	0	0
fl	*Kip/ft2*	0	0
MncFLB	*Kip-ft*	0	0
MncLTB	*Kip-ft*	0	0
MrcNeg	*Kip-ft*	0	0
MrtNeg	*Kip-ft*	0	0
Pu	*Kip*	4.093E-09	2.878E-08
MuNonComp	*Kip-ft*	-3567.8914	-3567.8914
MuLTerm	*Kip-ft*	-1341.4861	-1341.4861
MuSTerm	*Kip-ft*	-4088.1451	-4088.1451
fbuComp	*Kip/ft2*	stress -6820.38 (-47.36ksi)	-6820.38
fbuTens	*Kip/ft2*	7252.38 (50.36 ksi)	7252.38
FncFLB	*Kip/ft2*	7166.6 (49.76 ksi)	7166.6
FncLTB	*Kip/ft2*	resistance 5984.95 (41.56ksi)	5984.95
FrcNeg	*Kip/ft2*	5984.95 (41.56 ksi)	5984.95
FrtNeg	*Kip/ft2*	7166.6 (49.76 ksi)	7166.6
DCRatio	*Unitless*	1.13959	1.13959

TABLE: Bridge Super Design 32 - AASHTOLRFD07 - SteelICompStrgth-Shear			
Parameter	Unit	Value Before Station	Value After Station
Request	Text	Strength 1	Strength 1
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	120	120
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
Combo	Text	c Strength 1	c Strength 1
StepType	Text	Max	Max
Step	Text	0	0
DSet	Text	Mu Combo	Mu Combo
CodeEqtn	Text	6.10.9.2-1	6.10.9.2-1
PanelType	Text	Unstiffened	Unstiffened
Vu	Kip	391.283	391.284
Vr	Kip	306.502	306.502
Vcr	Kip	306.502	306.502
Vp	Kip	783.949	783.949
C	Unitless	0.390971	0.390971
k	Unitless	5	5
d0	ft	0	0
d0req	ft	13.5	13.5
VrWithD0req	Kip	462.543	462.543
DCRatio	Unitless	1.276611	1.276612

G2 sections at 0.417L and 1.000L
Fatigue Limit State

C6.4.2 Flowchart for LRFD Article 6.10.4

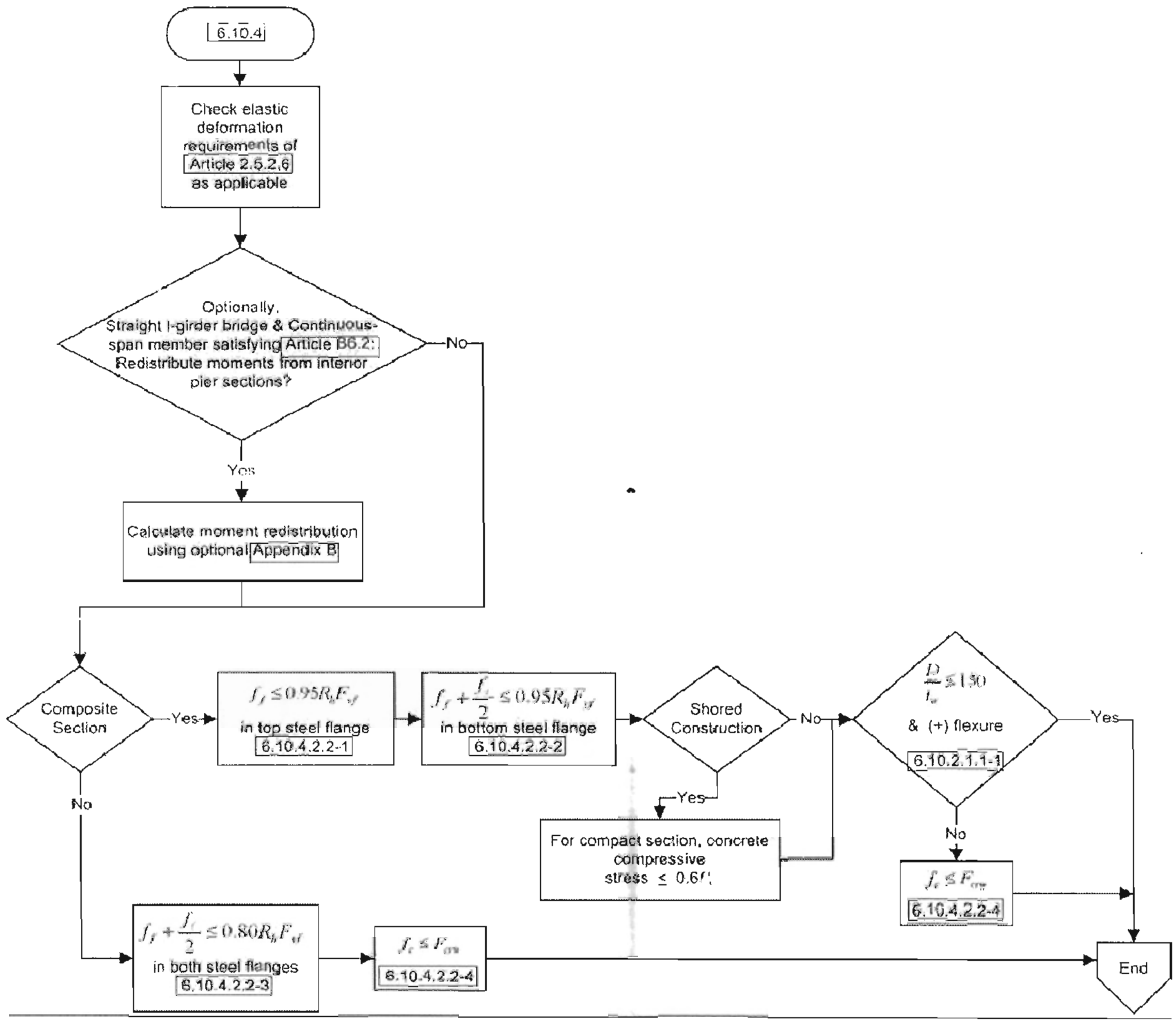


Figure C6.4.2-1 Flowchart for LRFD Article 6.10.4—Service Limit State.

C6.4.3 Flowchart for LRFD Article 6.10.5

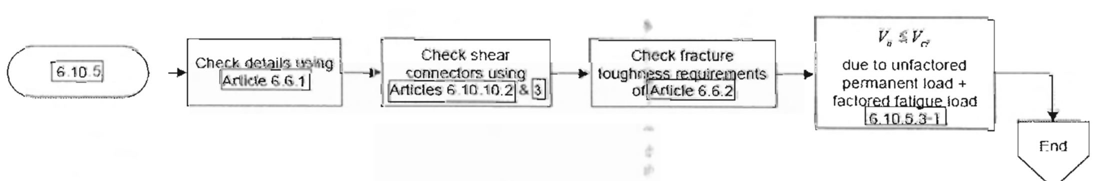


Figure C6.4.3-1 Flowchart for LRFD Article 6.10.5—Fatigue and Fracture Limit State.

Interim 2008

G2 at 50)

TABLE: Bridge Super Design 01 - Design Result Status

DesReqName	Text	Fatique	Fatique
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Status	Unitless	0	0
Message	Text	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

TABLE: Bridge Super Design 36 - AASHTOLRFD07 - SteelICompWebFatigue

Request	Text	Fatique	Fatique
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Girder	Text	Interior Girder 1 ✓	Interior Girder 1 ✓
Combo	Text	c Fatigue ✓	c Fatigue ✓
StepType	Text	Min	Min
Step	Text	0	0
DSet	Text	Fatigue Combo	Fatigue Combo
CodeEqtn	Text	6.10.9.3.3-1	6.10.9.3.3-1
PanelType	Text	Internal Stiffened	Internal Stiffened
LLDFactV	Unitless	0.625	0.625
EBeam	Kip/ft2	4176000	4176000
DepthWeb	ft	4.5	4.5
ThickWeb	ft	0.0417	0.0417
fyWeb	Kip/ft2	7200	7200
C	Unitless	0.470143	0.470143
k	Unitless	6.0125	6.0125
d0	ft	10	10
Vp	Kip	783.949	783.949
Vcr	Kip	368.568	368.568
Vu	Kip	12.892	12.878
DCRatio	Unitless	0.034978	0.034942

see the strength check (calcs similar except for k)

The stiffener spacing was entered as 10' on the optimization form.

G2 at 120'

TABLE: Bridge Super Design 01 - Design Result Status

DesReqName	*Text*	Fatique	Fatique
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	110	120
Location	*Text*	After	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

TABLE: Bridge Super Design 36 - AASHTOLRFD07 - SteelICompWebFatigue

Request	*Text*	Fatique	Fatique
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Fatigue	c Fatigue
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Fatigue Combo	Fatigue Combo
CodeEqtn	*Text*	6.10.9.3.3-1	6.10.9.3.3-1
PanelType	*Text*	Internal Stiffened	Internal Stiffened
LLDFactV	*Unitless*	0.625	0.625
EBeam	*Kip/ft2*	4176000	4176000
DepthWeb	*ft*	4.5	4.5
ThickWeb	*ft*	0.0417	0.0417
fyWeb	*Kip/ft2*	7200	7200
C	*Unitless*	0.470143	0.470143
k	*Unitless*	6.0125	6.0125
d0	*ft*	10	10
Vp	*Kip*	783.949	783.949
Vcr	*Kip*	368.568	368.568
Vu	*Kip*	32.58	32.58
DCRatio	*Unitless*	0.088397	0.088397

G2 sections at 0.417L and 1.000L
Service Limit State

C6.4.2 Flowchart for LRFD Article 6.10.4

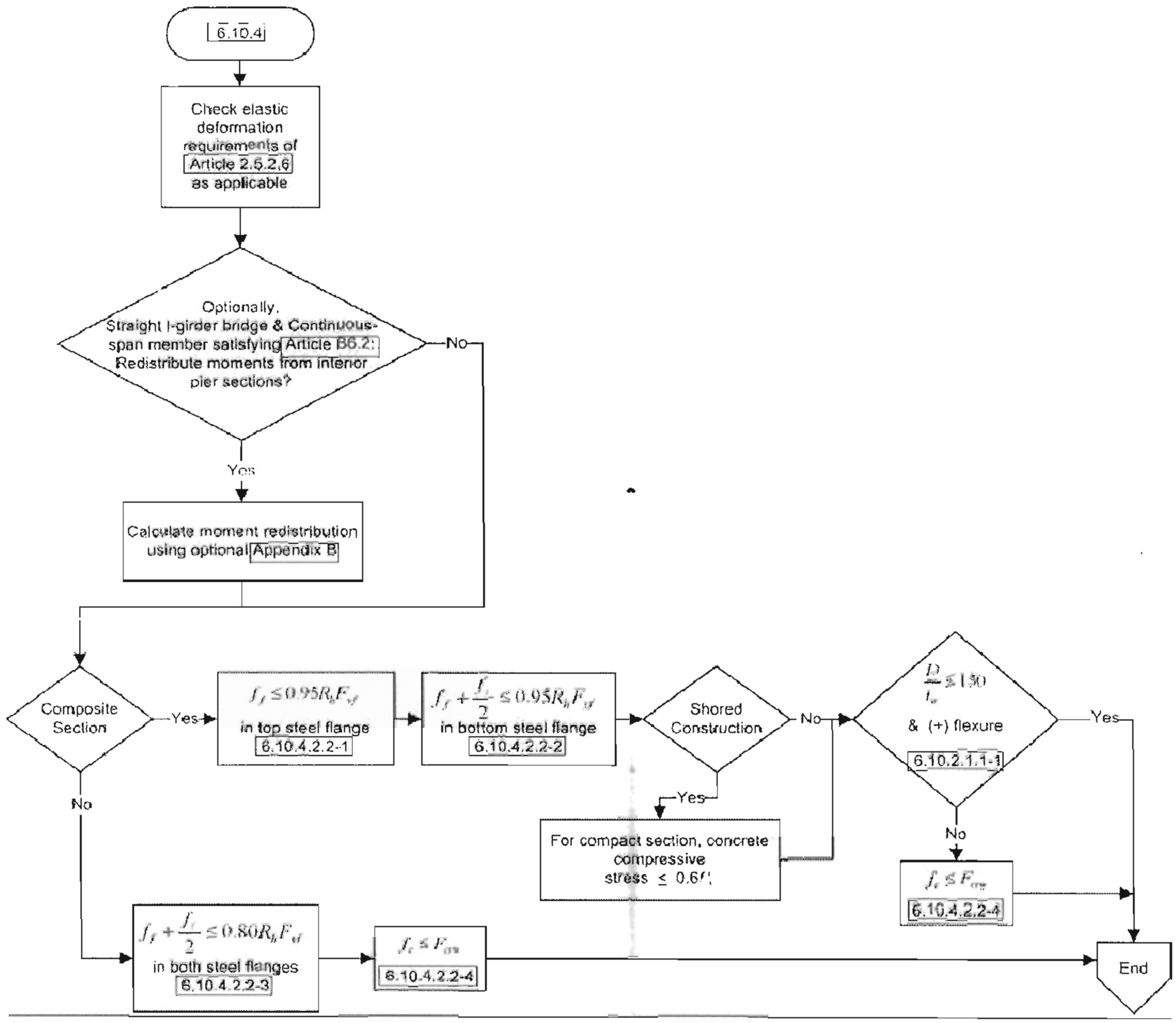


Figure C6.4.2-1 Flowchart for LRFD Article 6.10.4—Service Limit State.

C6.4.3 Flowchart for LRFD Article 6.10.5

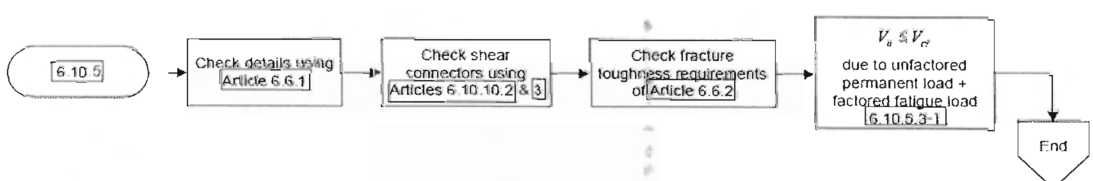


Figure C6.4.3-1 Flowchart for LRFD Article 6.10.5—Fatigue and Fracture Limit State.

Interim 2008

G2 at 501

TABLE: Bridge Super Design 01 - Design Result Status

DesReqName	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

TABLE: Bridge Super Design 33 - AASHTOLRFD07 - SteelICompServ-Prop

Request	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
BeamProp	*Text*	I-Girder 0.625in T	I-Girder 0.625in T
LLDFactM	*Unitless*	0.686017	0.686017
LLDFactV	*Unitless*	0.934898	0.934898
ASlabTri	*ft2*	6.5	6.5
ThSlab	*ft*	0.66667	0.66667
WSlabEff	*ft*	9.75	9.75
fcConcSlab	*Kip/ft2*	576	576
ESlab	*Kip/ft2*	519119.5	519119.5
nLongTerm	*Unitless*	3	3
ARebSlabTop	*ft2*	0	0
ARebSlabBot	*ft2*	0	0
YRebSlabTop	*ft*	0	0
YRebSlabBot	*ft*	0	0
fysLRebar	*Kip/ft2*	8640	8640
ABeam	*ft2*	0.3335	0.3335
EBeam	*Kip/ft2*	4176000	4176000
IxBeam	*ft4*	1.0668	1.0668
BeamRolled	*Yes/No*	No	No
ThFlgTop	*ft*	0.0521	0.0521
WdthFlgTop	*ft*	1.1667	1.1667
fyFlgTop	*Kip/ft2*	7200	7200
ThFlgBot	*ft*	0.0729	0.0729
WdthFlgBot	*ft*	1.1667	1.1667
fyFlgBot	*Kip/ft2*	7200	7200
fyrFlgBot	*Kip/ft2*	5040	5040
LamfBotFlg	*Unitless*	8.002058	8.002058
LampfBotFlg	*Unitless*	9.151612	9.151612
LamrfBotFlg	*Unitless*	16.119553	16.119553
kcBotFlg	*Unitless*	0.385054	0.385054

G2 at 50'

CmpctFlgBot	Yes/No	Yes	Yes
DepthWeb	ft	4.5	4.5
ThickWeb	ft	0.0417	0.0417
fyWeb	Kip/ft2	7200	7200
DcpWebPos	ft	0	0
DcWebNeg	ft	2.08172	2.08172
DcpWebNeg	ft	1.95902	1.95902
LamwWeb	Unitless	99.842831	99.842831
LampwDcWeb	Unitless	72.202398	72.202398
LampwDcpWeb	Unitless	67.946726	67.946726
LamrwWeb	Unitless	137.274178	137.274178
CmpctWebNeg	Yes/No	No	No
RpcWeb	Unitless	0	0
RptWeb	Unitless	0	0
rt	ft	0.29393	0.29393
J	ft4	0	0
RbPos	Unitless	1	1
RbNeg	Unitless	1	1
RhPos	Unitless	1	1
RhNeg	Unitless	0.985987	0.985987
CmpctGrdPos	Yes/No	Yes	Yes
CmpctGrdNeg	Yes/No	Yes	Yes
SxSteelBot	ft3	0.495122	0.495122
SxSteelTop	ft3	0.431837	0.431837
SxLTermBPos	ft3	0.699063	0.699063
SxLTermTPos	ft3	2.211966	2.211966
SxSTermBPos	ft3	0.761928	0.761928
SxSTermTPos	ft3	10.383153	10.383153
SxCompBNeg	ft3	0.495122	0.495122
SxCompTNeg	ft3	0.431837	0.431837
MpPos	Kip-ft	7506.8302	7506.8302
MpNeg	Kip-ft	3892.4384	3892.4384
PNADistPos	ft	0.503	0.503
PNADistLmt	ft	2.2225	2.2225
PNADistNeg	ft	3.49931	3.49931
MuDNC	Kip-ft	1304.1114	1304.1114
MuDCLTerm	Kip-ft	596.4576	596.4576
MyPos	Kip-ft	4731.7565	4731.7565
MyNegCtr	Kip-ft	3110.5085	3110.5085
MyNegBot	Kip-ft	3566.3448	3566.3448
MyNegTop	Kip-ft	3110.5085	3110.5085
Lp	ft	0	0
Lr	ft	0	0
Lb	ft	20	20
BeamPBklShr	Yes/No	No	No

G2 at 50'

TABLE: Bridge Super Design 34 - AASHTOLRFD07 - SteelICompServ-FlexPos				
Request	Text	Service ✓	Service ✓	
BridgeObj	Text	BOBJ1 ✓	BOBJ1 ✓	
Station	ft	50 ✓	50 ✓	
Location	Text	Before ✓	After ✓	
Girder	Text	Interior Girder 1 ✓	Interior Girder 1 ✓	
Combo	Text	c Service 2 ✓	c Service 2 ✓	
StepType	Text	Max	Min	
Step	Text	0	0	
DSet	Text	Ms Combo ✓	Ms Combo ✓	
CodeEqtn	Text	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section	
Pu	Kip	2.822E-08	2.822E-08	
MuNonComp	Kip-ft	1043.2891 ✓	1043.2891	
MuLTerm	Kip-ft	427.7048 ✓	427.7048	
MuSTerm	Kip-ft	2518.7441 ✓	2518.7441	
ffTop	Kip/ft2	-2850.7 (-19.8ksi)	-2850.7	vs. 19.6 in sprd.
ffBot	Kip/ft2	6022.23 (41.8ksi)	6022.23	vs. 42.2 in sprd.
fl	Kip/ft2	0	0	
fDeck	Kip/ft2	-100.19 (-0.7ksi)	-100.19	
FrTop	Kip/ft2	6840 (47.5ksi)	6840	vs. 47.5 in sprd.
FrBot	Kip/ft2	6840 (47.5ksi)	6840	vs. 47.5 in sprd.
DCRatio	Unitless	0.880443	0.880443	

Mu Sterm = (2745.8 k-ft)(0.6860 distr. Factor)(1.3) = (1883.62)(1.3) =
service 2 factor
= 2448.7k-ft

moment demands on noncomposite, long term composite, and short-term composite sections

ffTop: top flange compression stress without lateral bending

ffBot: bottom flange tension stress without lateral bending

fDeck: top slab fiber compression stress

fl: bottom flange lateral bending stress

Fr Top: flexural resistance stress of top compression flange for positive moment.

Fr Bot: flexural resistance stress of bottom tension flange for positive moment.

G2 at 50' – negative flexure not applicable for midspan section

TABLE: Bridge Super Design 35 - AASHTOLRFD07 - SteelCompServ-FlexNeg

Request	Text	Service	Service
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
Combo	Text	c Service 2	c Service 2
StepType	Text	Max	Max
Step	Text	0	0
DSet	Text	Ms Combo	Ms Combo
CodeEqtn	Text	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section
Pu	Kip	2.822E-08	1.453E-08
MuNonComp	Kip-ft	1043.2891	1043.2891
MuLTerm	Kip-ft	427.7048	427.7048
MuSTerm	Kip-ft	2518.7441	-692.9148
ffTop	Kip/ft2	0	0
ffBot	Kip/ft2	0	0
fl	Kip/ft2	0	0
fDeck	Kip/ft2	0	9.02
FrTop	Kip/ft2	6744.15	6744.15
FrBot	Kip/ft2	6744.15	6744.15
Dc	ft	0	0
k	Unitless	0	0
Fcrw	Kip/ft2	7099.11	7099.11
DCRatio	Unitless	0	0

G2 at 120)

TABLE: Bridge Super Design 01 - Design Result Status			
DesReqName	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.
TABLE: Bridge Super Design 33 - AASHTOLRFD07 - SteelICompServ-Prop			
Request	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
BeamProp	*Text*	I-Girder 2.5in T	I-Girder 2.5in T
LLDFactM	*Unitless*	0.737113	0.737113
LLDFactV	*Unitless*	0.934898	0.934898
ASlabTri	*ft2*	6.5	6.5
ThSlab	*ft*	0.66667	0.66667
WSlabEff	*ft*	9.75	9.75
fcConcSlab	*Kip/ft2*	576	576
ESlab	*Kip/ft2*	519119.5	519119.5
nLongTerm	*Unitless*	3	3
ARebSlabTop	*ft2*	0	0
ARebSlabBot	*ft2*	0	0
YRebSlabTop	*ft*	0	0
YRebSlabBot	*ft*	0	0
fysLRebar	*Kip/ft2*	8640	8640
ABeam	*ft2*	0.6981	0.6981
EBeam	*Kip/ft2*	4176000	4176000
IxBeam	*ft4*	3.155525	3.155525
BeamRolled	*Yes/No*	No	No
ThFlgTop	*ft*	0.2083	0.2083
WdthFlgTop	*ft*	1.1667	1.1667
fyFlgTop	*Kip/ft2*	7200	7200
ThFlgBot	*ft*	0.2292	0.2292
WdthFlgBot	*ft*	1.1667	1.1667
fyFlgBot	*Kip/ft2*	7200	7200
fyrFlgBot	*Kip/ft2*	5040	5040
LamfBotFlg	*Unitless*	2.545157	2.545157
LampfBotFlg	*Unitless*	9.151612	9.151612
LamrfBotFlg	*Unitless*	16.119553	16.119553
kcBotFlg	*Unitless*	0.385054	0.385054

G2 at 120)

CmpctFlgBot	Yes/No	Yes	Yes
DepthWeb	ft	4.5	4.5
ThickWeb	ft	0.0417	0.0417
fyWeb	Kip/ft2	7200	7200
DcpWebPos	ft	0	0
DcWebNeg	ft	2.16377	2.16377
DcpWebNeg	ft	1.95763	1.95763
LamwWeb	Unitless	103.777764	103.777764
LampwDcWeb	Unitless	95.593501	95.593501
LampwDcpWeb	Unitless	86.48636	86.48636
LamrwWeb	Unitless	137.274178	137.274178
CmpctWebNeg	Yes/No	No	No
RpcWeb	Unitless	0	0
RptWeb	Unitless	0	0
rt	ft	0.32814	0.32814
J	ft4	0	0
RbPos	Unitless	1	1
RbNeg	Unitless	1	1
RhPos	Unitless	1	1
RhNeg	Unitless	0.995361	0.995361
CmpctGrdPos	Yes/No	Yes	Yes
CmpctGrdNeg	Yes/No	Yes	Yes
SxSteelBot	ft3	1.318667	1.318667
SxSteelTop	ft3	1.240119	1.240119
SxLTermBPos	ft3	1.513569	1.513569
SxLTermTPos	ft3	2.831093	2.831093
SxSTermBPos	ft3	1.624842	1.624842
SxSTermTPos	ft3	6.768451	6.768451
SxCompBNeg	ft3	1.318667	1.318667
SxCompTNeg	ft3	1.240119	1.240119
MpPos	Kip-ft	14020.0069	14020.0069
MpNeg	Kip-ft	10170.3594	10170.3594
PNADistPos	ft	0.85978	0.85978
PNADistLmt	ft	2.35375	2.35375
PNADistNeg	ft	3.50071	3.50071
MuDNC	Kip-ft	-3567.8914	-3567.8914
MuDCLTerm	Kip-ft	-1341.4861	-1341.4861
MyPos	Kip-ft	12630.7233	12630.7233
MyNegCtr	Kip-ft	8932.5399	8932.5399
MyNegBot	Kip-ft	9498.3148	9498.3148
MyNegTop	Kip-ft	8932.5399	8932.5399
Lp	ft	0	0
Lr	ft	0	0
Lb	ft	20	20
BeamPBklShr	Yes/No	Yes	Yes

G2 at 120' - positive moment not applicable at pier section

TABLE: Bridge Super Design 34 - AASHTOLRFD07 - SteelICompServ-FlexPos			
Request	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Service 2	c Service 2
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Ms Combo	Ms Combo
CodeEqtn	*Text*	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section
Pu	*Kip*	2.826E-08	1.017E-09
MuNonComp	*Kip-ft*	-2854.3132	-2854.3132
MuLTerm	*Kip-ft*	-961.946	-961.946
MuSTerm	*Kip-ft*	8.536	-3216.3003
ffTop	*Kip/ft2*	0	0
ffBot	*Kip/ft2*	0	0
fl	*Kip/ft2*	0	0
fDeck	*Kip/ft2*	0	0
FrTop	*Kip/ft2*	6840	6840
FrBot	*Kip/ft2*	6840	6840
DCRatio	*Unitless*	0	0

G2 at 120'

TABLE: Bridge Super Design 35 - AASHTOLRFD07 - SteelICompServ-FlexNeg			
Request	*Text*	Service	Service
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Service 2	c Service 2
StepType	*Text*	Min	Max
Step	*Text*	0	0
DSet	*Text*	Ms Combo	Ms Combo
CodeEqtn	*Text*	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section	6.10.4.2.2.-2 Bottom Steel Flange of Composite Section
Pu	*Kip*	1.452E-08	1.017E-09
MuNonComp	*Kip-ft*	-2854.3132 ✓	-2854.3132
MuLTerm	*Kip-ft*	-961.946 ✓	-961.946
MuSTerm	*Kip-ft*	-3216.3003 ✓	-3216.3003
ffTop	*Kip/ft2*	3115.33 (21.63ksi)	3115.33
ffBot	*Kip/ft2*	-4777.58 (-33.18ksi)	-4777.58
fl	*Kip/ft2*	0	0
fDeck	*Kip/ft2*	87.78 (0.61 ksi)	87.78
FrTop	*Kip/ft2*	6808.27 (47.28ksi)	6808.27
FrBot	*Kip/ft2*	6808.27 (47.28ksi)	6808.27
Dc	*ft*	2.75947 (33.11")	2.75947
k	*Unitless*	23.934057	23.934057
Fcrw	*Kip/ft2*	7166.6 (49.77ksi)	7166.6
DCRatio	*Unitless*	0.701731	0.701731

$= \frac{33.18 ksi}{47.28 ksi} = 0.7018$ ok✓

moment demands on noncomposite, long term composite, and short-term composite sections

Note that
These stresses are calculated using section properties for composite section, because the concrete slab was specified to resist tension. When slab does not resist tension, the stresses are
ffTop = 35.1 ksi
ffBot = -33.1 ksi } ok

Dc
k
Fcrw
} verified against spreadsheet → ok

G2 sections at 0.417L and 1.000L
Constructibility Limit State

C6.4 FLOWCHARTS FOR FLEXURAL DESIGN OF I-SECTIONS

C6.4.1 Flowchart for LRFD Article 6.10.3

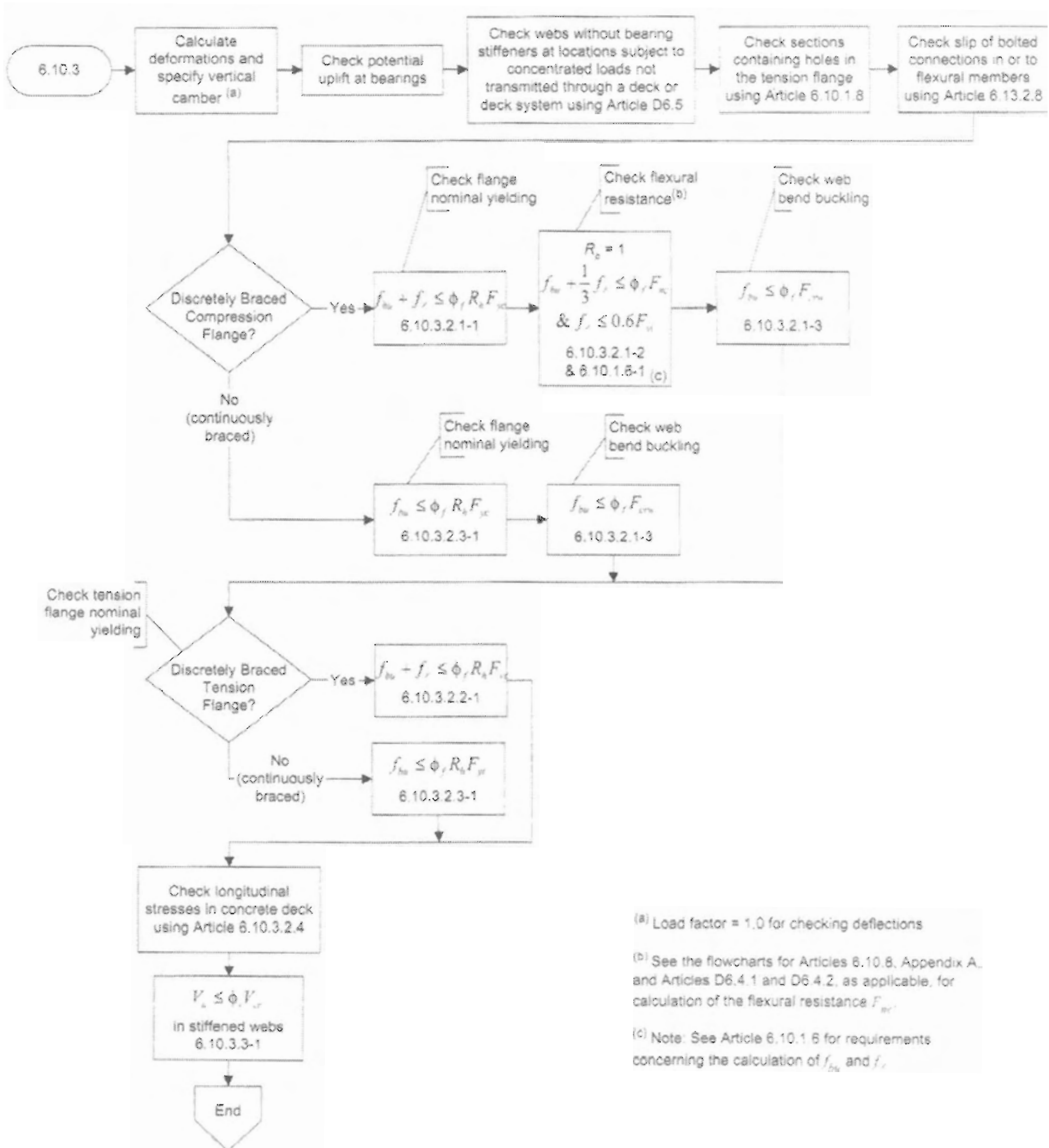


Figure C6.4.1-1 Flowchart for LRFD Article 6.10.3—Constructibility.

(p. 6-101)

CONSTRUCTABILITY V15.1.0_Y - G2 at 50'

TABLE: Bridge Super Design 01 - Design Result Status				
DesReqName	*Text*	a_ Constr	*Text*	a_ Constr
BridgeObj	*Text*	BOBJ1	*Text*	BOBJ1
Station	*ft*	50	*in ...*	600
Location	*Text*	Before	*Text*	Before
Status	*Unitless*	0	*Unitless*	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	*Text*	Design was performed and results are available, whether or not the design passed or failed.

CONSTRUCTABILITY V15.1.0_Y - G2 at 50'

TABLE: Bridge Super Design 41 - AASHTOLRFD07 - SteelICompCstrNStg-Prop

Request	Text	a_ Constr	Text	a_ Constr	
BridgeObj	Text	BOBJ1	Text	BOBJ1	
Station	ft	50	in ...	600	
Location	Text	Before	Text	Before	
Girder	Text	Interior Girder 1	Text	Interior Girder 1	
BeamProp	Text	I-Girder 0.625in T	Text	I-Girder 0.625in T	
ASlabTri	ft2	6.5	in2 ...	936	
ThSlab	ft	0.66667	in ...	8.00004	
WSlabEff	ft	9.75	in ...	117	
fcConcSlab	Kip/ft2	576	kip/in2 (ksi) ...	4	
ESlab	Kip/ft2	519119.5	kip/in2 (ksi) ...	3604.996528	
ABeam	ft2	0.3335	in2 ...	48.024	
EBeam	Kip/ft2	4176000	kip/in2 (ksi) ...	29000	
IxBeam	ft4	1.0668	ft4	1.0668	
BeamRolled	Yes/No	No	Yes/No	No	
ThFlgTop	ft	0.0521	in ...	0.6252	
WdthFlgTop	ft	1.1667	in ...	14.0004	
fyFlgTop	Kip/ft2	7200	kip/in2 (ksi) ...	50	
fyrFlgTop	Kip/ft2	5040	kip/in2 (ksi) ...	35	
LamfFlgTop	Unitless	11.196737	Unitless	11.196737	
LampfFlgTop	Unitless	9.151612	Unitless	9.151612	
LamrfFlgTop	Unitless	16.119553	Unitless	16.119553	
CmpctFlgTop	Yes/No	No	Yes/No	No	
ThFlgBot	ft	0.0729	in ...	0.8748	
WdthFlgBot	ft	1.1667	in ...	14.0004	
fyFlgBot	Kip/ft2	7200	kip/in2 (ksi) ...	50	
fyrFlgBot	Kip/ft2	5040	kip/in2 (ksi) ...	35	
LamfFlgBot	Unitless	8.002058	Unitless	8.002058	
LampfFlgBot	Unitless	9.151612	Unitless	9.151612	
LamrfFlgBot	Unitless	16.119553	Unitless	16.119553	
CmpctFlgBot	Yes/No	Yes	Yes/No	Yes	
DepthWeb	ft	4.5	in ...	54	
ThickWeb	ft	0.0417	in ...	0.5004	
fyWeb	Kip/ft2	7200	kip/in2 (ksi) ...	50	
RbPos	Unitless	1	Unitless	1	
RbNeg	Unitless	1	Unitless	1	
Lb	ft	20	in ...	240 ✓	vs. 240" in sprd

CONSTRUCTABILITY V15.1.0_Y - G2 at 50'

TABLE: Bridge Super Design 42 - AASHTOLRFD07 - SteelICompCstrNStg-FlxPs

Request	Text	a_ Constr	Text	a_ Constr	
BridgeObj	Text	BOBJ1	Text	BOBJ1	
Station	ft	50	in ...	600	
Location	Text	Before	Text	Before	
Girder	Text	Interior Girder 1	Text	Interior Girder 1	
Combo	Text	a- Strength 4	Text	a- Strength 4	
Label	Text		Text	0	
Step	Text	0	Text	0	
DSet	Text	DSet1	Text	DSet1	
CodeEqtn	Text	6.10.3.2.1-2 Discretely Braced Top Flange in Compression 6.10.8.2.3-2 Top Flange Lateral Torsional Buckling	Text	6.10.3.2.1-2 Discretely Braced Top Flange in Compression 6.10.8.2.3-2 Top Flange Lateral Torsional Buckling	
SlabStatus	Text	Non-Composite	Text	Non-Composite	
fbuComp	Kip/ft2	-3617.88	kip/in2 (ksi) ...	-25.12416667 ✓	vs. 25.4ksi in sprd.
fbuTens	Kip/ft2	3155.45	kip/in2 (ksi) ...	21.91284722 ✓	vs. 22.1ksi in sprd.
flTop	Kip/ft2	0	kip/in2 (ksi) ...	0	
flBot	Kip/ft2	0	kip/in2 (ksi) ...	0	
fDeck	Kip/ft2	0	kip/in2 (ksi) ...	0	
Fcrw	Kip/ft2	7200	kip/in2 (ksi) ...	50	
Dc	ft	2.4183	in ...	29.0196	
RhPos	Unitless	1	Unitless	1	
rt	ft	0.2703	in ...	3.2436 ✓	vs. 3.243" in sprd.
Lp	ft	6.5087	in ...	78.1044 ✓	vs. 78.1" in sprd.
Lr	ft	24.4398	in ...	293.2776 ✓	vs. 293.3" in sprd.
FncFLB	Kip/ft2	6566.03	kip/in2 (ksi) ...	45.59743056	
FncLTB	Kip/ft2	5574.83	kip/in2 (ksi) ...	38.71409722	
FrcPos	Kip/ft2	5574.83	kip/in2 (ksi) ...	38.71409722 ✓	vs. 38.7" in sprd.
FrtPos	Kip/ft2	7200	kip/in2 (ksi) ...	50	
DCRatio	Unitless	0.648967	Unitless	0.648967	

CONSTRUCTABILITY V15.1.0_Y - G2 at 50'

TABLE: Bridge Super Design 43 - AASHTOLRFD07 - SteelICompCstrNStg-FlxNg				
Request	Text	a_ Constr	Text	a_ Constr
BridgeObj	Text	BOBJ1	Text	BOBJ1
Station	ft	50	in ...	600
Location	Text	Before	Text	Before
Girder	Text	Interior Girder 1	Text	Interior Girder 1
Combo	Text	a- Strength 4	Text	a- Strength 4
Label	Text		Text	0
Step	Text	0	Text	0
DSet	Text	DSet1	Text	DSet1
CodeEqtn	Text	6.10.3.2.2-1 Discretely Braced Top Flange in Tension	Text	6.10.3.2.2-1 Discretely Braced Top Flange in Tension
SlabStatus	Text	Non-Composite	Text	Non-Composite
fbuComp	Kip/ft2	0	kip/in2 (ksi) ...	0
fbuTens	Kip/ft2	0	kip/in2 (ksi) ...	0
flTop	Kip/ft2	0	kip/in2 (ksi) ...	0
flBot	Kip/ft2	0	kip/in2 (ksi) ...	0
fDeck	Kip/ft2	0	kip/in2 (ksi) ...	0
Fcrw	Kip/ft2	7200	kip/in2 (ksi) ...	50
Dc	ft	2.4183	in ...	29.0196
RhNeg	Unitless	1	Unitless	1
rt	ft	0.2851	in ...	3.4212
Lp	ft	6.8669	in ...	82.4028
Lr	ft	25.7847	in ...	309.4164
FncFLB	Kip/ft2	7200	kip/in2 (ksi) ...	50
FncLTB	Kip/ft2	5700.49	kip/in2 (ksi) ...	39.58673611
FrcNeg	Kip/ft2	5700.49	kip/in2 (ksi) ...	39.58673611
FrtNeg	Kip/ft2	7200	kip/in2 (ksi) ...	50
DCRatio	Unitless	0	Unitless	0

Not Used because the section is in positive flexure

CONSTRUCTABILITY V15.1.0_Y - G2 AT 50'

TABLE: Bridge Super Design 44 - AASHTOLRFD07 - SteelICompCstrNStg-Shear					
Request	Text	a_ Constr		Text	a_ Constr
BridgeObj	Text	BOBJ1		Text	BOBJ1
Station	ft	50		in ...	600
Location	Text	Before		Text	Before
Girder	Text	Interior Girder 1		Text	Interior Girder 1
DSet	Text	DSet1		Text	DSet1
Combo	Text	a- Strength 4		Text	a- Strength 4
Label	Text			Text	0
Step	Text	0		Text	0
CodeEqtn	Text	6.10.9.2-1		Text	6.10.9.2-1
PanelType	Text	Unstiffened		Text	Unstiffened
SlabStatus	Text	Non-Composite		Text	Non-Composite
Vu	Kip	15.024		Kip	15.024
Vr	Kip	306.502		Kip	306.502
Vcr	Kip	306.502		Kip	306.502
Vp	Kip	783.949		Kip	783.949
C	Unitless	0.390971		Unitless	0.390971
k	Unitless	5		Unitless	5
d0	ft	0		in ...	0
d0req	ft	0		in ...	0
VrWithD0req	Kip	0		Kip	0
DCRatio	Unitless	0.049017		Unitless	0.049017

CONSTRUCTABILITY V15.1.0_Y - G2 at 120)

TABLE: Bridge Super Design 01 - Design Result Status

DesReqName	*Text*	a_ Constr √	*Text*	a_ Constr
BridgeObj	*Text*	BOBJ1 √	*Text*	BOBJ1
Station	*ft*	120 √	*in ...*	1440
Location	*Text*	Before √	*Text*	Before
Status	*Unitless*	0	*Unitless*	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed. √	*Text*	Design was performed and results are available, whether or not the design passed or failed.

TABLE: Bridge Super Design 41 - AASHTOLRFD07 - SteelICompCstrNStg-Prop				
Request	Text	a_ Constr	Text	a_ Constr
BridgeObj	Text	BOBJ1	Text	BOBJ1
Station	ft	120	in ...	1440
Location	Text	Before	Text	Before
Girder	Text	Interior Girder 1	Text	Interior Girder 1
BeamProp	Text	I-Girder 2.5in T	Text	I-Girder 2.5in T
ASlabTri	ft2	6.5	in2 ...	936
ThSlab	ft	0.66667	in ...	8.00004
WSlabEff	ft	9.75	in ...	117
fcConcSlab	Kip/ft2	576	kip/in2 (ksi) ...	4
ESlab	Kip/ft2	519119.5	kip/in2 (ksi) ...	3604.996528
ABeam	ft2	0.6981	in2 ...	100.5264
EBeam	Kip/ft2	4176000	kip/in2 (ksi) ...	29000
IxBeam	ft4	3.155525	ft4	3.155525
BeamRolled	Yes/No	No	Yes/No	No
ThFlgTop	ft	0.2083	in ...	2.4996
WdthFlgTop	ft	1.1667	in ...	14.0004
fyFlgTop	Kip/ft2	7200	kip/in2 (ksi) ...	50
fyrFlgTop	Kip/ft2	5040	kip/in2 (ksi) ...	35
LamfFlgTop	Unitless	2.800528	Unitless	2.800528
LampfFlgTop	Unitless	9.151612	Unitless	9.151612
LamrfFlgTop	Unitless	16.119553	Unitless	16.119553
CmpctFlgTop	Yes/No	Yes	Yes/No	Yes
ThFlgBot	ft	0.2292	in ...	2.7504
WdthFlgBot	ft	1.1667	in ...	14.0004
fyFlgBot	Kip/ft2	7200	kip/in2 (ksi) ...	50
fyrFlgBot	Kip/ft2	5040	kip/in2 (ksi) ...	35
LamfFlgBot	Unitless	2.545157	Unitless	2.545157
LampfFlgBot	Unitless	9.151612	Unitless	9.151612
LamrfFlgBot	Unitless	16.119553	Unitless	16.119553
CmpctFlgBot	Yes/No	Yes	Yes/No	Yes
DepthWeb	ft	4.5	in ...	54
ThickWeb	ft	0.0417	in ...	0.5004
fyWeb	Kip/ft2	7200	kip/in2 (ksi) ...	50
RbPos	Unitless	1	Unitless	1
RbNeg	Unitless	1	Unitless	1
Lb	ft	20	in ...	240

CONSTRUCTABILITY V15.1.0_Y - G2 at 120'

TABLE: Bridge Super Design 42 - AASHTOLRFD07 - SteelICompCstrNStg-FlxPs				
Request	Text	a_ Constr	Text	a_ Constr
BridgeObj	Text	BOBJ1	Text	BOBJ1
Station	ft	120	in ...	1440
Location	Text	Before	Text	Before
Girder	Text	Interior Girder 1	Text	Interior Girder 1
Combo	Text	a- Strength 4	Text	a- Strength 4
Label	Text		Text	0
Step	Text	0	Text	0
DSet	Text	DSet1	Text	DSet1
CodeEqtn	Text	6.10.3.2.2-1 Discretely Braced Bottom Flange in Tension	Text	6.10.3.2.2-1 Discretely Braced Bottom Flange in Tension
SlabStatus	Text	Non-Composite	Text	Non-Composite
fbuComp	Kip/ft2	0	kip/in2 (ksi) ...	0
fbuTens	Kip/ft2	0	kip/in2 (ksi) ...	0
flTop	Kip/ft2	0	kip/in2 (ksi) ...	0
flBot	Kip/ft2	0	kip/in2 (ksi) ...	0
fDeck	Kip/ft2	0	kip/in2 (ksi) ...	0
Fcrw	Kip/ft2	7200	kip/in2 (ksi) ...	50
Dc	ft	2.1638	in ...	25.9656
RhPos	Unitless	1	Unitless	1
rt	ft	0.3177	in ...	3.8124
Lp	ft	7.6515	in ...	91.818
Lr	ft	28.7307	in ...	344.7684
FncFLB	Kip/ft2	7200	kip/in2 (ksi) ...	50
FncLTB	Kip/ft2	5934.64	kip/in2 (ksi) ...	41.21277778
FrcPos	Kip/ft2	5934.64	kip/in2 (ksi) ...	41.21277778
FrtPos	Kip/ft2	7200	kip/in2 (ksi) ...	50
DCRatio	Unitless	0	Unitless	0

Not used, because the section is in negative flexure.

CONSTRUCTABILITY V15.1.0_Y - G2 at 120'

TABLE: Bridge Super Design 43 - AASHTOLRFD07 - SteelICompCstrNStg-FlxNg						
Request	Text	a_ Constr		Text	a_ Constr	
BridgeObj	Text	BOBJ1		Text	BOBJ1	
Station	ft	120		in ...	1440	
Location	Text	Before		Text	Before	
Girder	Text	Interior Girder 1		Text	Interior Girder 1	
Combo	Text	a- Strength 4		Text	a- Strength 4	
Label	Text			Text	0	
Step	Text	0		Text	0	
DSet	Text	DSet1		Text	DSet1	
CodeEqtn	Text	6.10.3.2.1-2 Discretely Braced Bottom Flange in Compression 6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling		Text	6.10.3.2.1-2 Discretely Braced Bottom Flange in Compression 6.10.8.2.3-2 Bottom Flange Lateral Torsional Buckling	
SlabStatus	Text	Non-Composite		Text	Non-Composite	
fbuComp	Kip/ft2	-3249.04		kip/in2 (ksi) ...	-22.56277778	vs. 23ksi in sprd.
fbuTens	Kip/ft2	3454.83		kip/in2 (ksi) ...	23.991875	vs 24.4ksi in sprd.
flTop	Kip/ft2	0		kip/in2 (ksi) ...	0	
flBot	Kip/ft2	0		kip/in2 (ksi) ...	0	
fDeck	Kip/ft2	0		kip/in2 (ksi) ...	0	
Fcrw	Kip/ft2	7200		kip/in2 (ksi) ...	50	
Dc	ft	2.1638		in ...	25.9656	
RhNeg	Unitless	1		Unitless	1	
rt	ft	0.3193		in ...	3.8316	
Lp	ft	7.6902		in ...	92.2824	
Lr	ft	28.8761		in ...	346.5132	
FncFLB	Kip/ft2	7200		kip/in2 (ksi) ...	50	vs. 50ksi in sprd.
FncLTB	Kip/ft2	5944.96		kip/in2 (ksi) ...	41.28444444	vs. 41.3ksi in sprd.
FrcNeg	Kip/ft2	5944.96		kip/in2 (ksi) ...	41.28444444	
FrtNeg	Kip/ft2	7200		kip/in2 (ksi) ...	50	
DCRatio	Unitless	0.54652		Unitless	0.54652	

CONSTRUCTABILITY V15.1.0_Y - G2 at 120'

TABLE: Bridge Super Design 44 - AASHTOLRFD07 - SteelICompCstrNStg-Shear					
Request	Text	a_Constr	Text	a_Constr	
BridgeObj	Text	BOBJ1	Text	BOBJ1	
Station	ft	120	in ...	1440	
Location	Text	Before	Text	Before	
Girder	Text	Interior Girder 1	Text	Interior Girder 1	
DSet	Text	DSet1	Text	DSet1	
Combo	Text	a- Strength 4	Text	a- Strength 4	
Label	Text		Text	0	
Step	Text	0	Text	0	
CodeEqtn	Text	6.10.9.2-1	Text	6.10.9.2-1	
PanelType	Text	Unstiffened	Text	Unstiffened	
SlabStatus	Text	Non-Composite	Text	Non-Composite	
Vu	Kip	153.498	Kip	153.498 ✓	vs. 154.47 in stvd.
Vr	Kip	306.502	Kip	306.502	
Vcr	Kip	306.502	Kip	306.502	
Vp	Kip	783.949	Kip	783.949	
C	Unitless	0.390971	Unitless	0.390971	
k	Unitless	5	Unitless	5	
d0	ft	0	in ...	0	
d0req	ft	0	in ...	0	
VrWithD0req	Kip	0	Kip	0	
DCRatio	Unitless	0.500808	Unitless	0.500808	

Spreadsheet Calculations

G2 Girder at 50'

I-Girder Section Analysis, AASHTO LRFD
3rd Ed. (2004), Art. 6.10; ver. 0.15-dev
((c) DesignSpreadsheets.com 2005-2008)

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

G2 @ 0.417L - pos. moment (Model A)

INPUT

Girder

b_{bf}	14	*in*
b_{tf}	14	*in*
t_{bf}	0.875	*in*
t_{tf}	0.625	*in*
h_w	54	*in*
t_w	0.5	*in*

Deck

b_d	117	*in*
t_d	8	*in*
A_{s1}	0.0001	*in^2*
A_{s2}	0.0001	*in^2*
d_1	2	*in*
d_2	2	*in*

Hauch

b_h	14	*in*
t_h	3	*in*

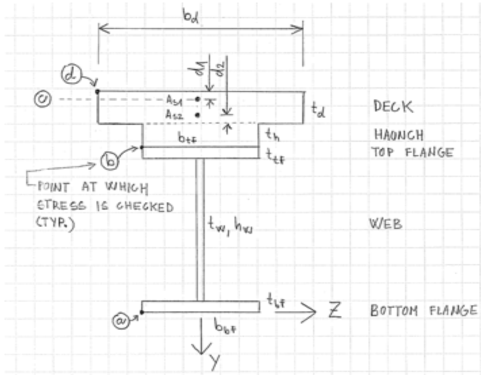
Modular ratio

n	7.9	*[-]*

OUTPUT - Sectional Properties
(section transformed into steel)

	Girder Only	Composite (3n)	Composite (n)	Composite (rebar)
Girder A [in^2]	48.00	48.00	48.00	48.00
Girder y_{cg} [in]	25.852	25.852	25.852	25.852
Girder I_z [in^4]	22114.8	22114.8	22114.8	22114.8
Haunch A [in^2]	-	1.70	5.09	-
Haunch y_{cg} [in]	-	56.9375	56.9375	-
Haunch I_z [in^4]	-	1.2	3.5	-
Deck A [in^2]	-	39.49	118.48	-
Deck y_{cg} [in]	-	62.375	62.375	-
Deck I_z [in^4]	-	210.6	631.9	-
Rebar A	-	-	-	0.0002
Rebar y_{cg}	-	-	-	62.375
Rebar I_z	-	-	-	0.0
Total A	48.00	89.19	171.58	48.00
Total y_{cg}	25.852	42.616	51.996	25.852
Total Iz	22114.8	51584.2	68447.3	22115.1
y_topdeck(d) [in]	-	23.759	14.379	-
y_topbar(c) [in]	-	21.759	12.379	38.523
y_topgrd(b) [in]	29.648	12.884	3.504	29.648
y_botgrd(a) [in]	25.852	42.616	51.996	25.852
S_topdeck(d) [in^3]	-	2171.1	4760.1	-
S_topbar(c) [in^3]	-	2370.7	5529.2	574.1
S_topgrd(b) [in^3]	745.9	4003.7	19532.7	745.9
S_botgrd(a) [in^3]	855.5	1210.4	1316.4	855.5
S_topdeck(d) [ft^3]	-	1.256440	2.754716	-
S_topbar(c) [ft^3]	-	1.371926	3.199770	0.332217
S_topgrd(b) [ft^3]	**0.431656**	**2.316949**	**11.303644**	**0.431663**
S_botgrd(a) [ft^3]	**0.495054**	**0.700490**	**0.761807**	**0.495057**

Neutral Axis Check
OK - neutral axis is within girder



Section Properties about Weak Axis

I_y =	343.6	*in^4*
$S_{y[TOP\ FLANGE]}$ =	49.1	*in^3*
$S_{y[BOT\ FLANGE]}$ =	49.1	*in^3*

Comment

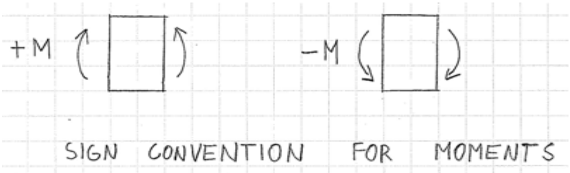
In progress...

D E S I G N S P R E A D S H E E T S . C O M

I-Girder Section Analysis, AASHTO LRFD
3rd Ed. (2004), Art. 6.10; ver. 0.15-dev
((c) DesignSpreadsheets.com 2005-2008)

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: Stress sign convention for this sheet - compressive stresses are reported as negative, tensile stresses as positive (sign convention for moments - see sketch below).



G2 @ 0.417L - pos. moment (Model A)

INPUT - Moments

DC1	1050.8	*kip-ft*
DC2	182.4	*kip-ft*
DW	250	*kip-ft*
LL+I	1963.1	*kip-ft*
LL+I fat range	0	*kip-ft*
LL+I permit	0	*kip-ft*
SE	0	*kip-ft*

OUTPUT - Stresses

Positive Moment Region (stress at the top of deck is reported as stress in concrete: $f_c=f_s/n$ or $f_c=f_s/(3n)$)

	Load Acting on	Grd Only	Composite (3n)			Composite (n)							
	Load Type	DC1	DC2	DW	SE	LL+I	LL+I fat	LL+I p	Service II	Strength I	Strength II	Fatigue	Governing
STRESS [ksi]	@ topdeck(d)	0	0.0	-0.1	0.0	-0.6	0.0	0.0	-0.9	-1.2	-0.1	0.0	-1.2
	@ topbar(c)	0	-0.9	-1.3	0.0	-4.3	0.0	0.0	-7.7	-10.5	-3.1	0.0	-10.5
	@ topgrd(b)	-16.9	-0.5	-0.7	0.0	-1.2	0.0	0.0	-19.8	-25.0	-22.9	0.0	-25.0
	@ botgrd(a)	14.7	1.8	2.5	0.0	17.9	0.0	0.0	42.3	55.7	24.4	0.0	55.7

Moment [kip-ft] → 4035.2 5351.9 1916.5 0.0

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

BASIC INPUTS

G2 @ 0.417L - pos. moment (Model A)

	Variable/Formula	Value	Units	Comment	AASHTO Page
||				● general	
||		POS		positive moment region (POS), negative moment region (NEG)	
||		COMPOSITE		composite section (COMPOSITE), noncomposite section (NONCOMPOSITE)	
||					
||				● geometry	
||	t_w =	0.500	*in*	web thickness	
||	b_{fc} =	14.000	*in*	full width of compression flange	
||	t_{fc} =	0.625	*in*	thickness of compression flange	
||	D =	54.000	*in*	web depth	
||					
||				● material properties	
||	E =	29,000	*ksi*	steel Young's modulus	
||	F_{yf} =	50.0	*ksi*	specified minimum yield strenth of flange	
||	F_{yw} =	50.0	*ksi*	specified mininun yield stress of web	
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement	
||	F_{yc} =	50.0	*ksi*	specified minimum yield strength of compression flange	
||	F_{yt} =	50.0	*ksi*	specified minimum yield strength of tension flange	
||	F_{yr} =	35.0	ksi	compression flange stress at the onset of nominal yielding	6-108
||	note: F_{yr} = min $(0.7F_{yc}, F_{yw})$, $Fyr \geq 0.5F_{yc}$				
||					
||				● effect of applied loads	
||					
||				● load factors [AASHTO 6.5.4.2]	6-27
||	Φ_f =	1.00	*[-]*	resistance factor for flexure	
||	Φ_v =	1.00	*[-]*	resistance factor for shear	
||					
||				● slenderness ratios for local buckling resistance [AASHTO 6.10.8.2.2]	6-107
||	**$\lambda_f = b_{fc} / (2t_{fc})$ =**	**11.200**	***[-]***	slenderness ratio for compression flange	
||	$\lambda_{pf} = 0.38 \text{ sqrt } (E/F_{yc})$ =	9.152	*[-]*	limiting slenderness ratio for a compact flange	
||	$\lambda_{rf} = 0.56 \text{ sqrt } (E/F_{yc})$ =	13.968	*[-]*	limiting slenderness ratio for a noncompact flange	
||					
||				● reduction factors	
||	**R_h =**	**1.000**	***[-]***	○ hybrid factor to account for reduced contribution of web to nominal flexural resistance at first yield in flange element; use 1.0 for girders with same steel strength for flange and web	6-80
||	**R_b =**	**1.000**	***[-]***	○ web load-shedding factor; accounts for increase in compression flange stress due to web local buckling	6-81
||	$R_b = 1 - [a_{wc}/(1200+300a_{wc})]\ [2D_c/t_w-\lambda_{rw}]$ =	1.000	*[-]*		
||	$\lambda_{rw} = 5.7 \text{ sqrt}(E/F_{yc})$ =	137.3	*[-]*	limiting slenderness ratio for noncompact web	
||	$a_{wc} = 2D_ct_w/b_{fc}t_{fc}$ =	3.317	*[-]*		
||					
||				● Nominal Shear Resistance of Unstiffened Webs [AASHTO 6.10.9.2]	6-115

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

|| **$V_n = V_{cr} = CV_p$ =**	**306**	***kip***	V_n = nominal shear resistance, V_{cr} = shear-buckling resistance	
|| $V_p = 0.58F_{yw}Dt_w$ =	783	*kip*	plastic shear force	
|| C =	0.390	*[-]*	ratio of the shear buckling resistance to the shear yield strength	
|| if $D/t_w \le 1.12\ \mathrm{sqrt}\ (Ek/F_{yw})$ then C =	1.000	*[-]*		
|| if $1.12\ \mathrm{sqrt}\ (Ek/F_{yw}) < D/t_w \le 1.40\ \mathrm{sqrt}\ (Ek/F_{yw})$ then C =				
|| $= 1.12 / (D/t_w)\ \mathrm{sqrt}\ (Ek/F_{yw})$ =	0.558	*[-]*		
|| if $D/t_w > 1.40\ \mathrm{sqrt}\ (Ek/F_{yw})$ then C =				
|| $= 1.57 / (D/t_w)^2\ (Ek/F_{yw})$ =	0.390	*[-]*		
|| note: D/t_w =	108.0	*[-]*		
|| note: $1.12\ \mathrm{sqrt}\ (Ek/F_{yw})$ =	60.3	*[-]*		
|| note: $1.40\ \mathrm{sqrt}\ (Ek/F_{yw})$ =	75.4	*[-]*		
|| k =	5.0	*[-]*	shear buckling coefficient (k=5 for unstiffened webs)	

CONSTRUCTIBILITY

6-86

			● Basic Inputs	
|| $f_{bu[C]}$ =	25.4	*ksi*	stress in compression flange due to DC1 (no lateral bending), factored by 1.5 (Strength IV load combination)	
|| $f_{bu[T]}$ =	22.1	*ksi*	stress in tension flange due to DC2 (no lateral bending), factored by 1.5 (Strength IV load combination)	
|| $f_{l[C]}$ =		*ksi*	stress in compression flange due to lateral bending (wind, from exterior girder bracket during construction, etc.)	
|| $f_{l[T]}$ =		*ksi*	stress in tension flange due to lateral bending	
|| V_u =	96	*kips*	factored shear in web, factored by 1.5 (Strength IV load combination)	
||				
|| D_c =	29.023	*in*	depth of web in compression in the elastic range	6-69
|| $r_t = b_{fc} / \mathrm{sqrt}\{ 12 [1+(D_c t_w)/(3b_{fc}t_{fc})] \}$ =	3.243	*in*	effective radius of gyration for lateral torsional buckling	6-109
|| C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||				
||			● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
|| **L_b =**	**240.0**	***in***	unbraced length	
|| $L_p = 1.0\ r_t\ \mathrm{sqrt}(E/F_{yc})$ =	78.1	*in*	limiting unbraced length 1 (for compact)	
|| $L_r = \pi\ r_t\ \mathrm{sqrt}\ (E/F_{yr})$ =	293.3	*in*	limiting unbraced length 2 (for noncompact)	
||				
||			● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106
|| **$F_{nc} = \min (F_{nc[1]}, F_{nc[2]})$ =**	**38.7**	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
|| $F_{nc[1]}$ =	43.6	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2]	
|| if $\lambda_f \le \lambda_{pf}$ then $F_{nc[1]} = R_b R_h F_{yc}$ =	50.0	*ksi*		
|| if $\lambda_f > \lambda_{pf}$ then $F_{nc[1]}$ =				
|| $= \{1-[1-F_{yr}/(R_h F_{yc})][(\lambda_f-\lambda_{pf})/(\lambda_{rf}-\lambda_{pf})] \}\ R_b R_h F_{yc}$ =	43.6	*ksi*		
|| $F_{nc[2]}$ =	38.7	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
|| indicates that corresponding checks are applicable for section under consideration (based on user's input)				
|| if $L_b \le L_p$ then $F_{nc[2]} = R_bR_hF_{yc}$ =	50.0	*ksi*		
|| if $L_p < L_b \le L_r$ then $F_{nc[2]}$ =				
|| $= C_b \{1\text{-}[1\text{-}F_{yr}/(R_hF_{yc})][(L_b\text{-}L_p)/(L_r\text{-}L_p)]\} R_bR_hF_{yc}$ =	38.7	ksi	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
|| if $L_b > L_r$ then $F_{nc[2]} = F_{cr}$ =	52.3	*ksi*	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
|| $F_{cr} = C_bR_b\pi^2E / (L_b/r_t)^2$ =	52.3	*ksi*		
||				
|| **F_{crw} = min ($F_{crw[1]}$, $F_{crw[2]}$, $F_{crw[3]}$) =**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
|| $F_{crw[1]} = 0.9Ek / (D/t_w)^2$ =	69.7	*ksi*		
|| $k = 9 / (D_c/D)^2$ =	31.2	*[-]*	bend buckling coefficient	
|| $F_{crw[2]} = R_hF_{yc}$ =	50.0	*ksi*		
|| $F_{crw[3]} = F_{yw}/0.7$ =	71.4	*ksi*		
||				
||			● Flexure - Discretely Braced Flanges in Compression [AASHTO 6.10.3.2.1]	6-87
|| **$f_{bu} + f_l \le \Phi_fR_hF_{yc}$**	**OK**			
|| $f_{bu} + f_l$ =	25.4	*ksi*		
|| $\Phi_fR_hF_{yc}$ =	50.0	*ksi*		
||				
|| **$f_{bu} + (1/3)f_l \le \Phi_fF_{nc}$**	**OK**			
|| $f_{bu} + (1/3)f_l$ =	25.4	*ksi*		
|| Φ_fF_{nc} =	38.7	*ksi*		
||				
|| **$f_{bu} \le \Phi_fF_{crw}$**	**OK**			
|| f_{bu} =	25.4	*ksi*		
|| Φ_fF_{crw} =	50.0	*ksi*		
||				
||			● Flexure - Discretely Braced Flanges in Tension [AASHTO 6.10.3.2.2]	6-89
|| **$f_{bu} + f_l \le \Phi_fR_hF_{yt}$**	**OK**			
|| $f_{bu} + f_l$ =	22.1	*ksi*		
|| $\Phi_fR_hF_{yt}$ =	50.0	*ksi*		
||			● Shear	6-90
|| **$V_u \le \Phi_vV_{cr}$**	**OK**			
|| V_u =	96	*kips*	factored shear in the web	
|| V_{cr} =	306	*kips*	shear buckling resistance	

SERVICE LIMIT STATE

[AASHTO 6.10.4] 6-93

||			● Basic Inputs	
|| **$f_{f[TOP]}$ =**	19.8	*ksi*	stress in top flange for Service II load combination	
|| **$f_{f[BOT]}$ =**	42.3	*ksi*	stress in bottom flange for Service II load combination	
|| **f_l =**		*ksi*		
||				
||			● Flexural Checks	6-93
|| **$f_f \le 0.95R_hF_{yf}$**	**OK**		○ top steel flange of composite sections	
|| f_f =	19.8	*ksi*		
|| $0.95R_hF_{yf}$ =	47.5	*ksi*		
||				
|| **$f_f + f_l/2 \le 0.95R_hF_{yf}$**	**OK**		○ bottom steel flange of composite sections	
|| $f_f + f_l/2$ =	42.3	*ksi*		

Project: *Check CSI Bridge composite steel design*	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	Value	Units	Comment	AASHTO Page

|| indicates that corresponding checks are applicable for section under consideration (based on user's input)

|| $0.95R_hF_{yf}$ = 47.5 *ksi*

● Stress in Concrete Deck [AASHTO 6.10.3.2.4] 6-89

FATIGUE AND FRACTURE LIMIT STATES [AASHTO 6.10.5] 6-95

	Variable/Formula	Value	Units	Comment	AASHTO Page
||	fatigue detail description =	tension flange			
||	fatigue detail category =	C			
||	**$\gamma(\Delta f) \le (\Delta F)_n$**	**OK**			6-29
||	$\gamma(\Delta f)$ =	0.0	*ksi*	live load stress range due to passage of fatigue load multiplied by load factor γ = 0.75	
||	$(\Delta F)_{n\,=}$ (A/N)^(1/3) =	5.0	*ksi*	nominal fatigue resistance [AASHTO 6.6.1.2.5]	6-40
||	A =	44.00	*10^8 ksi^3*	constant from [AASHTO Tab. 6.6.1.2.5-1]	6-42
||	N = (365)(75)n$(ADTT)_{SL}$ =	232,687,500			
||	n =	1		number of stress range cycles per truck passage taken from [AASHTO Tab. 6.6.1.2.5-2]	
||	$(ADTT)_{SL}$ = (p)(ADTT) =	8,500		single lane ADTT (number of trucks per day in one direction averaged over the design life) [AASHTO 3.6.1.4]	3-24
||	p =	0.85		reduction factor for number of trucks for multiple lanes taken from [AASHTO Tab. 3.6.1.4.2-1]	
||	ADTT = (ftt) (ADT) =	10,000		number of trucks per day in one direction averaged over the design life	
||	ftt =	0.25		fraction of trucks in traffic	
||	ADT = (nl) $(ADT)_{SL}$ =	40,000		average daily traffic per whole bridge	
||	$(ADT)_{SL}$ =	20,000		average daily traffic per single lane (20,000 is considered maximum)	
||	nl =	2		number of lanes	
||	note: $(\Delta F)n \ge (1/2)(\Delta F)_{TH}$ =	5.0	*ksi*		
||	$(\Delta F)_{TH}$ =	10.00	*10^8 ksi^3*	constant amplitude fatigue threshold taken from [AASHTO Tab. 6.6.1.2.5-3]	

STRENGTH LIMIT STATE 6-96

	Variable/Formula	Value	Units	Comment	AASHTO Page
||				● Basic Inputs	
||	$f_{bu[C]}$ =	25.0	*ksi*	stress in compression flange (no lateral bending)	
||	$f_{bu[T]}$ =	55.7	*ksi*	stress in tension flange (no lateral bending)	
||	$f_{l[C]}$ =		*ksi*	stress in compression flange due to lateral bending	
||	$f_{l[T]}$ =		*ksi*	stress in tension flange due to lateral bending	
||	M_u =	5,352	*kip-ft*	bending moment about major axis	
||	note: M_u =	64,223	*kip-in*		
||	V_u =	389	*kips*	factored shear in web	
||					
||	D_c =	16.588	*in*	depth of web in compression in the elastic range	6-69
||	D_{cp} =	0.000	*in*	depth of web in compression at the plastic moment	6-68
||	$r_t = b_{fc}$ / sqrt{ 12 [1+$(D_ct_w)/(3b_{fc}t_{fc})$] } =	3.523	*in*	effective radius of gyration for lateral torsional buckling	6-109
||	C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||					

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
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Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
|| indicates that corresponding checks are applicable for section under consideration (based on user's input)				
||			● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
|| **L_b =**	**300.0**	***in***	unbraced length	
|| $L_p = 1.0\ r_t$ sqrt (E/F_{yc}) =	84.8	*in*	limiting unbraced length 1 (for compact)	
|| $L_r = \pi\ r_t$ sqrt (E/F_{yr}) =	318.6	*in*	limiting unbraced length 2 (for noncompact)	
||				
||			● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106
|| **F_{nc} = min ($F_{nc[1]}$, $F_{nc[2]}$) =**	36.2	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
|| $F_{nc[1]}$ =	43.6	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2], same as for constructibility - see calculations above	
|| $F_{nc[2]}$ =	36.2	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	
|| if $L_b \le L_p$ then $F_{nc[2]} = R_bR_hF_{yc}$ =	50.0	*ksi*		
|| if $L_p < L_b \le L_r$ then $F_{nc[2]}$ =				
|| = C_b {1-[1-F_{yr}/(R_hF_{yc})] [(L_b-L_p)/(L_r-L_p)]} $R_bR_hF_{yc}$ =	36.2	ksi		
|| if $L_b > L_r$ then $F_{nc[2]} = F_{cr} = C_bR_b\pi^2E / (L_b/r_t)^2$ =	39.5	*ksi*		
|| note: $F_{nc[2]} \le R_bR_hF_{yc}$				
||				
|| **F_{crw} = min ($F_{crw[1]}$, $F_{crw[2]}$, $F_{crw[3]}$) =**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
|| $F_{crw[1]} = 0.9Ek / (D/t_w)^2$ =	213.4	*ksi*		
|| $k = 9 / (D_c/D)^2$ =	95.4	*[-]*	bend buckling coefficient	
|| $F_{crw[2]} = R_hF_{yc}$ =	50.0	*ksi*		
|| $F_{crw[3]} = F_{yw}/0.7$ =	71.4	*ksi*		
|| **Composite Section in Positive Flexure**				
||				
||			○ Compact Section Criteria	6-98
||	COMPACT		Compact/Noncompact (To qualify as compact, section must meet all the criteria listed below).	
|| Is $F_{yf} \le$ 70ksi satisfied?	YES			
|| Is $D/t_w \le 150$ satisfied?	YES			
|| D/t_w =	108.0	*[-]*		
|| Is $2D_{cp}/t_w \le 3.76$ sqrt(E/F_{yc}) satisfied?	YES			
|| $2D_{cp}/t_w$ =	0.0	*[-]*		
|| 3.76 sqrt(E/F_{yc}) =	90.6	*[-]*		
||			○ Flexural Resistance for Compact Section [AASHTO 6.10.7.1]	6-101
|| **$M_u + 1/3f_lS_{xt} \le \Phi_fM_n$**	**OK**			
|| $M_u + 1/3f_lS_{xt}$ =	64,223	*kip-in*		
|| Φ_fM_n =	73,702	*kip-in*		
|| note: M_n =	6,142	*kip-ft*		
||				
|| $S_{xt} = M_{yt}/F_{yt}$ =	1133.9	*in^3*	elastic section modulus about the major axis of the section to the tension flange	
|| M_{yt} =	56,694	*kip-in*	yield moment with respect to tension flange	6-252
|| M_n =	73,702	*kip-in*	nominal flexural resistance of the section	
|| note: M_n =	6,142	*kip-ft*		
|| if $D_p \le 0.1\ D_t$ then $M_n = M_p$ =	90,016	*kip-in*		
|| if $D_p > 0.1\ D_t$ then $M_n = M_p (1.07-0.7D_p/D_t)$ =	90,590	*kip-in*		
|| D_t =	66.375	*in*	total depth of composite section [imported from Mp tab]	

	Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||	D_p =	6.033	*in*	distance from top of concrete deck to the neutral axis of composite section at plastic moment [imported from Mp tab]	
||	M_p =	90,016	*kip-in*	plastic moment of composite section [imported from Mp tab]	
||	note: M_p =	7,501	*kip-ft*		
||	note: $M_n \leq 1.3R_hM_y$ =	73,702	*kip-in*		
||	M_y =	56,694	*kip-in*	yield moment [import from My tab]	6-102
	note: M_y =	4,724	*kip-ft*		

||				○ Ductility Requirement (For Both Compact and Noncompact Sections)	6-105
||	**$D_p \leq 0.42\ D_t$**	**OK**			
||	D_p =	6.033	*in*		
||	$0.42\ D_t$ =	27.878	*in*		

||	**Shear Resistance [AASHTO 6.10.9]**				6-114
||	**$V_u \leq \Phi_vV_n$**	**NG**			
||	V_u =	389	*kip*		
||	Φ_vV_n =	306	*kip*		

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments are reported as absolute values;
negative "P" forces are compressive, positive "P" forces are tensile.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF PLASTIC MOMENT Mp [AASHTO D6.1, p. 6-250]

G2 @ 0.417L - pos. moment (Model A)

Input Taken from Other Tabs

b_{bf} =	14	*in*	○ girder dimensions
b_{tf} =	14	*in*	
t_{bf} =	0.875	*in*	
t_{tf} =	0.625	*in*	
h_w =	54	*in*	
t_w =	0.5	*in*	
b_d =	117	*in*	○ deck dimensions
t_d =	8	*in*	
t_h =	2.875	*in*	○ haunch dimensions

Additional Input

F_{yf} =	50.0	*ksi*	specified minimum yield stress of flange
F_{yw} =	50.0	*ksi*	specified minimum yield stress of web
$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield stress of reinforcement
f_c' =	4.0	*ksi*	minimum specified 28-day compressive strength of concrete
β_1 =	1.000	*[-]*	use β_1 = 1 to consider whole concrete block in compression

Outputs - Positive Moment Region

	top coord [in]	bot coord [in]	Height [in]	Force [kips]	Arm to PNA [in]	Moment @ PNA [kip-in]	
Ps compression	0.000	6.033	6.033	-2400.0	-3.017	7239.8	concrete
Pc compression	10.875	10.875	0.000	0.0	4.842	0.0	top flange
Pc tension	10.875	11.500	0.625	437.5	5.154	2255.0	
Pw compression	11.500	11.500	0.000	0.0	5.467	0.0	web
Pw tension	11.500	65.500	54.000	1350.0	32.467	43830.2	
Pt tension	65.500	66.375	0.875	612.5	59.904	36691.4	bottom flange
Total				0.0		90016.4	

$y = D_p$ =	6.033	*in*	distance of PNA from the top of section
M_p =	90,016	*kip-in*	plastic moment
note: M_p =	7,501	*kip-ft*	plastic moment
D_{cp} =	0.000	*in*	depth of web in compression at plastic moment
D_t =	66.375	*in*	total depth of composite section

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF YIELD MOMENT My (for strength check) [AASHTO D6.2, p. 6-252]

G2 @ 0.417L - pos. moment (Model A)

Input

			● general
M_{D1} =	15,762	*kip-in*	factored moment applied to noncomposite section (1.25 DC1)
M_{D2} =	7,236	*kip-in*	factored moment applied to longterm composite section (1.25 DC2 + 1.5 DW)
note: M_{D1} =	1313.5	*kip-ft*	
note: M_{D2} =	603.0	*kip-ft*	
f_c =	25.0	*ksi*	sum of compression flange stresses [import from Stresses tab, for Service II load group]
f_t =	55.7	*ksi*	sum of tension flange stresses [import from Stresses tab, for Service I| load group]
F_{yf} =	50.0	*ksi*	specified minimum yield strength of flange
$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement

Composite Section in Positive Flexure

$M_Y = \min(M_{Y[T]}, M_{Y[C]})$ =	**56,694**	***kip-in***	
note: M_Y =	4,724	*kip-ft*	
			● determine moment to cause yielding in tension (bottom) flange
$M_{Y[T]} = M_{D1} + M_{D2} + M_{AD[T]}$ =	56,694	*kip-in*	
$M_{AD[T]} = (F_{yf} - M_{D1}/S_{NC[T]} - M_{D2}/S_{LT[T]})\ S_{ST[T]}$ =	33,696	*kip-in*	additional moment applied to short term composite section to cause nominal yielding in tension flange
$S_{NC[T]}$ =	855.5	in^3	noncomposite section modulus
$S_{LT[T]}$ =	1,210.4	in^3	short-term composite section modulus
$S_{ST[T]}$ =	1,316.4	in^3	long-term composite section modulus
			● determine moment to cause yielding in compression (top) flange
$M_{Y[C]} = M_{D1} + M_{D2} + M_{AD[C]}$ =	551,576	*kip-in*	
$M_{AD[C]} = (F_{yf} - M_{D1}/S_{NC[C]} - M_{D2}/S_{LT[C]})\ S_{ST[C]}$ =	528,578	*kip-in*	
$S_{NC[C]}$ =	745.9	in^3	
$S_{LT[C]}$ =	4,003.7	in^3	
$S_{ST[C]}$ =	19,532.7	in^3	
$D_c = \lvert f_c\rvert / (\lvert f_c\rvert + f_t)\ d - t_{fc}$ =	16.588	*in*	depth of web in compression in the elastic range
d =	55.500	*in*	depth of steel section
t_{fc} =	0.625	*in*	thickness of top flange

DETERMINATION OF YIELD MOMENT My (for constructibility check) [AASHTO D6.2, p. 6-252]

Input

f_c =	16.9	*ksi*	sum of compression flange stresses [import from Stresses tab, for DC1]

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
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Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

|| f_t = 14.7 *ksi* — sum of tension flange stresses [import from Stresses tab, for DC1]

|| F_{yf} = 50.0 *ksi* — specified minimum yield strength of flange

|| Noncomposite Section in Positive Flexure

||

|| $M_Y = \min(S_{NC[T]}F_{yf}, S_{NC[C]}F_{yf})$ = 37,295 *kip-in*

|| note: M_Y = 3,108 *kip-ft*

|| $S_{NC[T]}$ = 855.5 in^3 — section modulus for tension flange

|| $S_{NC[C]}$ = 745.9 in^3 — section modulus for compression flange

||

||

|| $D_c = |f_c| / (|f_c|+f_t) d - t_{fc}$ = 29.023 *in* — depth of web in compression in the elastic range

|| d = 55.500 *in* — depth of steel section

|| t_{fc} = 0.625 *in* — thickness of top flange

G2 Girder at 120'

DESIGNSPREADSHEETS.COM

I-Girder Section Analysis, AASHTO LRFD 3rd Ed. (2004), Art. 6.10; ver. 0.15-dev ((c) DesignSpreadsheets.com 2005-2008)

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

G2 @ 1.0L - neg. moment (Model A)

INPUT

Girder

b_{bf}	14	*in*
b_{tf}	14	*in*
t_{bf}	2.75	*in*
t_{tf}	2.5	*in*
h_w	54	*in*
t_w	0.5	*in*

Deck

b_d	117	*in*
t_d	8	*in*
A_{s1}	0.0001	*in^2*
A_{s2}	0.0001	*in^2*
d_1	2	*in*
d_2	2	*in*

Hauch

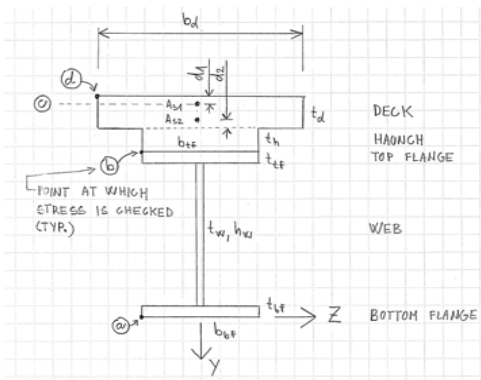
b_h	14	*in*
t_h	1	*in*

Modular ratio

n	8.047	*[-]*

OUTPUT - Sectional Properties
(section transformed into steel)

	Girder Only	Composite (3n)	Composite (n)	Composite (rebar)
Girder A [in^2]	100.50	100.50	100.50	100.50
Girder y_{cg} [in]	28.718	28.718	28.718	28.718
Girder I_z [in^4]	65426.6	65426.6	65426.6	65426.6
Haunch A [in^2]	-	0.58	1.74	-
Haunch y_{cg} [in]	-	59.75	59.75	-
Haunch I_z [in^4]	-	0.0	0.1	-
Deck A [in^2]	-	38.77	116.32	-
Deck y_{cg} [in]	-	64.25	64.25	-
Deck I_z [in^4]	-	206.8	620.4	-
Rebar A	-	-	-	0.0002
Rebar y_{cg}	-	-	-	64.250
Rebar I_z	-	-	-	0.0
Total A	100.50	139.85	218.56	100.50
Total y_{cg}	28.718	38.698	47.875	28.718
Total Iz	65426.6	101214.3	134363.1	65426.9
y_topdeck(d) [in]	-	29.552	20.375	-
y_topbar(c) [in]	-	27.552	18.375	37.532
y_topgrd(b) [in]	30.532	20.552	11.375	30.532
y_botgrd(a) [in]	28.718	38.698	47.875	28.718
S_topdeck(d) [in^3]	-	3424.9	6594.7	-
S_topbar(c) [in^3]	-	3673.5	7312.5	1743.2
S_topgrd(b) [in^3]	2142.9	4924.7	11812.6	2142.9
S_botgrd(a) [in^3]	2278.2	2615.5	2806.5	2278.2
S_topdeck(d) [ft^3]	-	1.982013	3.816349	-
S_topbar(c) [ft^3]	-	2.125886	4.231743	1.008823
S_topgrd(b) [ft^3]	**1.240108**	**2.849950**	**6.835992**	**1.240116**
S_botgrd(a) [ft^3]	**1.318415**	**1.513607**	**1.624141**	**1.318417**



Section Properties about Weak Axis

I_y =	1201.1	*in^4*
$S_{y[TOP\ FLANGE]}$ =	171.6	*in^3*
$S_{y[BOT\ FLANGE]}$ =	171.6	*in^3*

Neutral Axis Check
OK - neutral axis is within girder

Comment

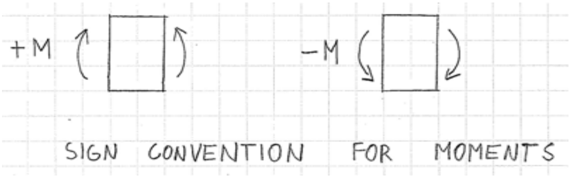
D E S I G N S P R E A D S H E E T S . C O M

I-Girder Section Analysis, AASHTO LRFD
3rd Ed. (2004), Art. 6.10; ver. 0.15-dev
((c) DesignSpreadsheets.com 2005-2008)

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: Stress sign convention for this sheet - compressive stresses are reported as negative, tensile stresses as positive (sign convention for moments - see sketch below).

G2 @ 1.0L - neg. moment (Model A)



INPUT - Moments

DC1	-2909.5	*kip-ft*
DC2	-423.5	*kip-ft*
DW	-580.5	*kip-ft*
LL+I	-2345.32	*kip-ft*
LL+I fat range	0	*kip-ft*
LL+I permit	0	*kip-ft*
SE	0	*kip-ft*

OUTPUT - Stresses

Negative Moment Region (stress at the top of deck is reported as stress in concrete using short-term composite section: $f_c=f_s/n$)

	Load Acting on	Grd Only	Composite (rebar)										
	Load Type	DC1	DC2	DW	SE	LL+I	LL+I fat	LL+I p	Service II	Strength I	Strength II	Fatigue	Governing
STRESS [ksi]	@ topdeck(d)	0	0.1	0.1	0.0	0.5	0.0	0.0	0.9	1.2	0.3	0.0	1.2
	@ topbar(c)	0	2.9	4.0	0.0	16.1	0.0	0.0	27.9	37.9	9.6	0.0	37.9
	@ topgrd(b)	16.3	2.4	3.3	0.0	13.1	0.0	0.0	39.0	51.2	28.2	0.0	51.2
	@ botgrd(a)	-15.3	-2.2	-3.1	0.0	-12.4	0.0	0.0	-36.7	-48.1	-26.5	0.0	-48.1
								Moment [kip-ft] →	-6962.4	-9141.3	-5037.0	0.0	

Project: ***Check CSI Bridge composite steel design***	Made By: *ok*	Date: *5/7/2011*	Job No:
Subject:	Checked By:	Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

BASIC INPUTS

G2 @ 1.0L - neg. moment (Model A)

	Variable/Formula	Value	Units	Comment	AASHTO Page
||				● general	
||		NEG		positive moment region (POS), negative moment region (NEG)	
||		COMPOSITE		composite section (COMPOSITE), noncomposite section (NONCOMPOSITE)	
||					
||				● geometry	
||	t_w =	0.500	*in*	web thickness	
||	b_{fc} =	14.000	*in*	full width of compression flange	
||	t_{fc} =	2.750	*in*	thickness of compression flange	
||	D =	54.000	*in*	web depth	
||					
||				● material properties	
||	E =	29,000	*ksi*	steel Young's modulus	
||	F_{yf} =	50.0	*ksi*	specified minimum yield strenth of flange	
||	F_{yw} =	50.0	*ksi*	specified mininun yield stress of web	
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement	
||	F_{yc} =	50.0	*ksi*	specified minimum yield strength of compression flange	
||	F_{yt} =	50.0	*ksi*	specified minimum yield strength of tension flange	
||	F_{yr} =	35.0	ksi	compression flange stress at the onset of nominal yielding	6-108
||	note: F_{yr} = min $(0.7F_{yc}, F_{yw})$, $F_{yr} \geq 0.5F_{yc}$				
||					
||				● effect of applied loads	
||					
||				● load factors [AASHTO 6.5.4.2]	6-27
||	Φ_f =	1.00	*[-]*	resistance factor for flexure	
||	Φ_v =	1.00	*[-]*	resistance factor for shear	
||					
||				● slenderness ratios for local buckling resistance [AASHTO 6.10.8.2.2]	6-107
||	**$\lambda_f = b_{fc} / (2t_{fc})$ =**	**2.545**	***[-]***	slenderness ratio for compression flange	
||	$\lambda_{pf} = 0.38$ sqrt (E/F_{yc}) =	9.152	*[-]*	limiting slenderness ratio for a compact flange	
||	$\lambda_{rf} = 0.56$ sqrt (E/F_{yc}) =	13.968	*[-]*	limiting slenderness ratio for a noncompact flange	
||					
||				● reduction factors	
||	**R_h =**	**1.000**	***[-]***	○ hybrid factor to account for reduced contribution of web to nominal flexural resistance at first yield in flange element; use 1.0 for girders with same steel strength for flange and web	6-80
||	**R_b =**	**1.000**	***[-]***	○ web load-shedding factor; accounts for increase in compression flange stress due to web local buckling	6-81
||	$R_b = 1 - [a_{wc}/(1200+300a_{wc})]\ [2D_c/t_w-\lambda_{rw}]$ =	1.000	*[-]*		
||	$\lambda_{rw} = 5.7$ sqrt(E/F_{yc}) =	137.3	*[-]*	limiting slenderness ratio for noncompact web	
||	$a_{wc} = 2D_ct_w/b_{fc}t_{fc}$ =	0.675	*[-]*		
||					
||				● Nominal Shear Resistance of Unstiffened Webs [AASHTO 6.10.9.2]	6-115

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
|| indicates that corresponding checks are applicable for section under consideration (based on user's input)				
|| **$V_n = V_{cr} = CV_p$ =**	**306**	***kip***	V_n = nominal shear resistance, V_{cr} = shear-buckling resistance	
|| $V_p = 0.58F_{yw}Dt_w$ =	783	*kip*	plastic shear force	
|| C =	0.390	*[-]*	ratio of the shear buckling resistance to the shear yield strength	
|| if $D/t_w \le 1.12$ sqrt (Ek/F_{yw}) then C =	1.000	*[-]*		
|| if 1.12 sqrt $(Ek/F_{yw}) < D/t_w \le 1.40$ sqrt (Ek/F_{yw}) then C =				
|| = 1.12 / (D/t_w) sqrt (Ek/F_{yw}) =	0.558	*[-]*		
|| if $D/t_w > 1.40$ sqrt (Ek/F_{yw}) then C =				
|| = 1.57 / $(D/t_w)^2$ (Ek/F_{yw}) =	0.390	*[-]*		
|| note: D/t_w =	108.0	*[-]*		
|| note: 1.12 sqrt (Ek/F_{yw}) =	60.3	*[-]*		
|| note: 1.40 sqrt (Ek/F_{yw}) =	75.4	*[-]*		
|| k =	5.0	*[-]*	shear buckling coefficient (k=5 for unstiffened webs)	

CONSTRUCTIBILITY

6-86

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
			● Basic Inputs	
|| $f_{bu[C]}$ =	23.0	*ksi*	stress in compression flange due to DC1 (no lateral bending), factored by 1.5 (Strength IV load combination)	
|| $f_{bu[T]}$ =	24.4	*ksi*	stress in tension flange due to DC2 (no lateral bending), factored by 1.5 (Strength IV load combination)	
|| $f_{l[C]}$ =		*ksi*	stress in compression flange due to lateral bending (wind, from exterior girder bracket during construction, etc.)	
|| $f_{l[T]}$ =		*ksi*	stress in tension flange due to lateral bending	
|| V_u =	96	*kips*	factored shear in web, factored by 1.5 (Strength IV load combination)	
||				
|| D_c =	25.968	*in*	depth of web in compression in the elastic range	6-69
|| $r_t = b_{fc}$ / sqrt{ 12 $[1+(D_ct_w)/(3b_{fc}t_{fc})]$ } =	3.832	*in*	effective radius of gyration for lateral torsional buckling	6-109
|| C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||				
||			● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
|| **L_b =**	**240.0**	***in***	unbraced length	
|| $L_p = 1.0\ r_t$ sqrt(E/F_{yc}) =	92.3	*in*	limiting unbraced length 1 (for compact)	
|| $L_r = \pi\ r_t$ sqrt (E/F_{yr}) =	346.5	*in*	limiting unbraced length 2 (for noncompact)	
||				
||			● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106
|| **F_{nc} = min $(F_{nc[1]}, F_{nc[2]})$ =**	**41.3**	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
|| $F_{nc[1]}$ =	50.0	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2]	
|| if $\lambda_f \le \lambda_{pf}$ then $F_{nc[1]} = R_bR_hF_{yc}$ =	50.0	*ksi*		
|| if $\lambda_f > \lambda_{pf}$ then $F_{nc[1]}$ =				
|| = $\{1-[1-F_{yr}/(R_hF_{yc})][(\lambda_f-\lambda_{pf})/(\lambda_{rf}-\lambda_{pf})]\}\ R_bR_hF_{yc}$ =	70.6	*ksi*		
|| $F_{nc[2]}$ =	41.3	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

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Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

|| if $L_b \le L_p$ then $F_{nc[2]} = R_bR_hF_{yc}$ =	50.0	*ksi*		
|| if $L_p < L_b \le L_r$ then $F_{nc[2]}$ =				
|| = C_b {1-[1-F_{yr}/(R_hF_{yc})][(L_b-L_p)/(L_r-L_p)]} $R_bR_hF_{yc}$ =	41.3	ksi	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
|| if $L_b > L_r$ then $F_{nc[2]} = F_{cr}$ =	73.0	*ksi*	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
|| $F_{cr} = C_bR_b\pi^2E / (L_b/r_t)^2$ =	73.0	*ksi*		
||				
|| **F_{crw} = min ($F_{crw[1]}$, $F_{crw[2]}$, $F_{crw[3]}$) =**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
|| $F_{crw[1]} = 0.9Ek / (D/t_w)^2$ =	87.1	*ksi*		
|| $k = 9 / (D_c/D)^2$ =	38.9	*[-]*	bend buckling coefficient	
|| $F_{crw[2]} = R_hF_{yc}$ =	50.0	*ksi*		
|| $F_{crw[3]} = F_{yw}/0.7$ =	71.4	*ksi*		
||				
||			● Flexure - Discretely Braced Flanges in Compression [AASHTO 6.10.3.2.1]	6-87
|| **$f_{bu} + f_l \le \Phi_fR_hF_{yc}$**	**OK**			
|| $f_{bu} + f_l$ =	23.0	*ksi*		
|| $\Phi_fR_hF_{yc}$ =	50.0	*ksi*		
||				
|| **$f_{bu} + (1/3)f_l \le \Phi_fF_{nc}$**	**OK**			
|| $f_{bu} + (1/3)f_l$ =	23.0	*ksi*		
|| Φ_fF_{nc} =	41.3	*ksi*		
||				
|| **$f_{bu} \le \Phi_fF_{crw}$**	**OK**			
|| f_{bu} =	23.0	*ksi*		
|| Φ_fF_{crw} =	50.0	*ksi*		
||				
||			● Flexure - Discretely Braced Flanges in Tension [AASHTO 6.10.3.2.2]	6-89
|| **$f_{bu} + f_l \le \Phi_fR_hF_{yt}$**	**OK**			
|| $f_{bu} + f_l$ =	24.4	*ksi*		
|| $\Phi_fR_hF_{yt}$ =	50.0	*ksi*		
||			● Shear	6-90
|| **$V_u \le \Phi_vV_{cr}$**	**OK**			
|| V_u =	96	*kips*	factored shear in the web	
|| V_{cr} =	306	*kips*	shear buckling resistance	
||			● Stress in Concrete Deck [AASHTO 6.10.3.2.4]	6-89
|| Shall 1% longitudinal reinforcement be provided?	YES		If the actual tensile stress in concrete deck exceeds Φf_r, to control cracking, the area of longitudinal reinforcement shall be at least 1% of the concrete deck area. [AASHTO 6.10.1.7]	6-75
|| σ_c = (1/n) (M/S) =	1.0	*ksi*	actual tensile stress in concrete deck	
|| M =	52,371	*kip-in*	moment due to construction loads, DC1 , factored by 1.5 (Strength IV load combination - high DL to LL ratio) [imported from Stresses tab] [AASHTO 6.10.1.7]	6-75
|| note: M =	4,364	*kip-ft*		
|| S =	6594.7	*in^3*	section modulus for top of deck using n = 1 (section transformed in steel) [imported from Stresses tab]	
|| Φf_r =	0.0	*ksi*	factored concrete tensile resistance	
|| Φ =		*[-]*	resistance factor for concrete in tension	5-53
|| f_r = 0.24 sqrt (f_c') =	0.00	*ksi*	modulus of rupture for concrete deck [AASHTO 5.4.2.6]	5-16
|| f_c' =		*ksi*	specified compressive strength of concrete	

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

SERVICE LIMIT STATE [AASHTO 6.10.4] 6-93

Variable/Formula	Value	Units	Comment	AASHTO Page
||			● Basic Inputs	
|| $f_{f[TOP]}$ =	39.0	ksi	stress in top flange for Service II load combination	
|| $f_{f[BOT]}$ =	36.7	ksi	stress in bottom flange for Service II load combination	
|| f_l =		ksi		
||				
||			● Flexural Checks	6-93
|| **$f_f \le 0.95R_hF_{yf}$**	**OK**		○ top steel flange of composite sections	
|| f_f =	39.0	ksi		
|| $0.95R_hF_{yf}$ =	47.5	ksi		
||				
|| **$f_f + f_l/2 \le 0.95R_hF_{yf}$**	**OK**		○ bottom steel flange of composite sections	
|| $f_f + f_l/2$ =	36.7	ksi		
|| $0.95R_hF_{yf}$ =	47.5	ksi		
|| **$f_c \le F_{crw}$**	**OK**		○ this check applies to all sections except for composite sections in positive flexure with web proportions such that $D/t_w \le 150$ [AASHTO Eq. 6.10.4.2.2-4]	6-94
|| $f_c = f_{f[TOP]}$ =	39.0	ksi	compression flange stress due to Service II loads	
|| F_{crw} =	50.0	ksi	nominal bend buckling resistance for webs [see "Strength Limit State" section below for the calculation of F_{crw}] [AASHTO 6.10.1.9.1]	6-77
			● Stress in Concrete Deck [AASHTO 6.10.3.2.4]	6-89
|| Shall 1% longitudinal reinforcement be provided?	#DIV/0!		If the actual tensile stress in concrete deck exceeds Φf_r, to control cracking, the area of longitudinal reinforcement shall be at least 1% of the concrete deck area. [AASHTO 6.10.1.7]	6-75
|| $\sigma_c = (1/n)(M/S)$ =	#DIV/0!	ksi	actual tensile stress in concrete deck	
|| M =	83,549	kip-in	moment due to Service II load combination [imported from Stresses tab] [AASHTO 6.10.1.7]	6-75
|| S =		in^3	section modulus for top of deck using n = 1; section transformed in steel (same as for constructibility check above)	
|| Φf_r =	0.0	ksi	factored concrete tensile resistance (same as for constructibility check above)	

FATIGUE AND FRACTURE LIMIT STATES [AASHTO 6.10.5] 6-95

Variable/Formula	Value	Units	Comment	AASHTO Page
|| fatigue detail description =	tension flange			
|| fatigue detail category =	C			
|| **$\gamma(\Delta f) \le (\Delta F)_n$**	**OK**			6-29
|| $\gamma(\Delta f)$ =	0.0	ksi	live load stress range due to passage of fatigue load multiplied by load factor γ = 0.75	
|| $(\Delta F)_n = (A/N)^{(1/3)}$ =	5.0	ksi	nominal fatigue resistance [AASHTO 6.6.1.2.5]	6-40
|| A =	44.00	10^8 ksi^3	constant from [AASHTO Tab. 6.6.1.2.5-1]	6-42
|| $N = (365)(75)n(ADTT)_{SL}$ =	232,687,500			
|| n =	1		number of stress range cycles per truck passage taken from [AASHTO Tab. 6.6.1.2.5-2]	

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

	Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
||	$(ADTT)_{SL}$ = (p)(ADTT) =	8,500		single lane ADTT (number of trucks per day in one direction averaged over the design life) [AASHTO 3.6.1.4]	3-24
||	p =	0.85		reduction factor for number of trucks for multiple lanes taken from [AASHTO Tab. 3.6.1.4.2-1]	
||	ADTT = (ftt) (ADT) =	10,000		number of trucks per day in one direction averaged over the design life	
||	ftt =	0.25		fraction of trucks in traffic	
||	ADT = (nl) $(ADT)_{SL}$ =	40,000		average daily traffic per whole bridge	
||	$(ADT)_{SL}$ =	20,000		average daily traffic per single lane (20,000 is considered maximum)	
||	nl =	2		number of lanes	
||	note: $(\Delta F)n \geq (1/2)(\Delta F)_{TH}$ =	5.0	*ksi*		
||	$(\Delta F)_{TH}$ =	10.00	*10^8 ksi^3*	constant amplitude fatigue threshold taken from [AASHTO Tab. 6.6.1.2.5-3]	

STRENGTH LIMIT STATE

6-96

	Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
||				● Basic Inputs	
||	$f_{bu[C]}$ =	48.1	*ksi*	stress in compression flange (no lateral bending)	
||	$f_{bu[T]}$ =	51.2	*ksi*	stress in tension flange (no lateral bending)	
||	$f_{l[C]}$ =		*ksi*	stress in compression flange due to lateral bending	
||	$f_{l[T]}$ =		*ksi*	stress in tension flange due to lateral bending	
||	M_u =	9,141	*kip-ft*	bending moment about major axis	
||	note: M_u =	109,696	*kip-in*		
||	V_u =	389	*kips*	factored shear in web	
||					
||	D_c =	25.968	*in*	depth of web in compression in the elastic range	6-69
||	D_{cp} =	23.500	*in*	depth of web in compression at the plastic moment	6-68
||	$r_t = b_{fc}$ / sqrt{ 12 [1+$(D_c t_w)/(3b_{fc}t_{fc})$] } =	3.832	*in*	effective radius of gyration for lateral torsional buckling	6-109
||	C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||					
||				● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
||	**L_b =**	**300.0**	***in***	unbraced length	
||	$L_p = 1.0\ r_t$ sqrt (E/F_{yc}) =	92.3	*in*	limiting unbraced length 1 (for compact)	
||	$L_r = \pi\ r_t$ sqrt (E/F_{yr}) =	346.5	*in*	limiting unbraced length 2 (for noncompact)	
||					
||				● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106
||	**F_{nc} = min ($F_{nc[1]}$, $F_{nc[2]}$) =**	37.7	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
||	$F_{nc[1]}$ =	50.0	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2], same as for constructibility - see calculations above	
||	$F_{nc[2]}$ =	37.7	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	
||	if $L_b \leq L_p$ then $F_{nc[2]} = R_b R_h F_{yc}$ =	50.0	*ksi*		
||	if $L_p < L_b \leq L_r$ then $F_{nc[2]}$ =				
||	= C_b {1-[1-$F_{yr}/(R_h F_{yc})$] [$(L_b-L_p)/(L_r-L_p)$]} $R_b R_h F_{yc}$ =	37.7	ksi		
||	if $L_b > L_r$ then $F_{nc[2]} = F_{cr} = C_b R_b \pi^2 E / (L_b/r_t)^2$ =	46.7	*ksi*		
||	note: $F_{nc[2]} \leq R_b R_h F_{yc}$				

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||					
||	**F_{crw} = min ($F_{crw[1]}$, $F_{crw[2]}$, $F_{crw[3]}$) =**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
||	$F_{crw[1]} = 0.9Ek / (D/t_w)^2$ =	87.1	*ksi*		
||	$k = 9 / (D_c/D)^2$ =	38.9	*[-]*	bend buckling coefficient	
||	$F_{crw[2]} = R_hF_{yc}$ =	50.0	*ksi*		
||	$F_{crw[3]} = F_{yw}/0.7$ =	71.4	*ksi*		
	note: M_y =	10,748	*kip-ft*		
||	**Composite Sections in Negative Flexure and Noncomposite Sections**				
||					
||				○ Compact Section Criteria	
||		COMPACT		Compact/Noncompact (To qualify as compact, section must meet all the criteria listed below).	
||	Is $F_{yf} \leq$ 70ksi satisfied?	YES			
||	Is $2D_c/t_w \leq 5.7\ \mathrm{sqrt}(E/F_{yc})$ satisfied?	YES			
||	$2D_c/t_w$ =	103.9	*[-]*		
||	$5.7\ \mathrm{sqrt}(E/F_{yc})$ =	137.3	*[-]*		
||					
||				○ Flexural Resistance - Discretely Braced Flanges in Compression [AASHTO 6.10.8.1.1]	
||	**$f_{bu} + (1/3)f_l \leq \Phi_fF_{nc}$**	**NG**			
||	$f_{bu} + (1/3)f_l$ =	48.1	*ksi*		
||	Φ_fF_{nc} =	37.7	*ksi*		
||				○ Flexural Resistance - Continously Braced Flanges in T or C	
||	**$f_{bu} \leq \Phi_fR_hF_{yf}$**	**OK**			
||	f_{bu} =	48.1	ksi		
||	$\Phi_fR_hF_{yf}$ =	50.0	ksi		
||	**Shear Resistance [AASHTO 6.10.9]**				6-114
||	**$V_u \leq \Phi_vV_n$**	**NG**			
||	V_u =	389	*kip*		
||	Φ_vV_n =	306	*kip*		

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments are reported as absolute values;
negative "P" forces are compressive, positive "P" forces are tensile.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF PLASTIC MOMENT Mp [AASHTO D6.1, p. 6-250]

G2 @ 1.0L - neg. moment (Model A)

|| **Input Taken from Other Tabs**

||	b_{bf} =	14	*in*	○ girder dimensions
||	b_{tf} =	14	*in*	
||	t_{bf} =	2.75	*in*	
||	t_{tf} =	2.5	*in*	
||	h_w =	54	*in*	
||	t_w =	0.5	*in*	
||	b_d =	117	*in*	○ deck dimensions
||	t_d =	8	*in*	
||	t_h =	1	*in*	○ haunch dimensions

|| **Additional Input**

||	F_{yf} =	50.0	*ksi*	specified minimum yield stress of flange
||	F_{yw} =	50.0	*ksi*	specified minimum yield stress of web
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield stress of reinforcement
||	f_c' =	4.0	*ksi*	minimum specified 28-day compressive strength of concrete
||	β_1 =	1.000	*[-]*	use β_1 = 1 to consider whole concrete block in compression

|| **Outputs - Negative Moment Region**

||		bot coord [in]	top coord [in]	Height [in]	Force [kips]	Arm to PNA [in]	Moment @ PNA [kip-in]	
||	Pr	62.250	62.250	-	0.0	36.000	0.4	reinforcement
||	Pt tension	56.750	59.250	2.500	1750.0	31.750	55562.1	top flange
||	Pt compression	56.750	56.750	0.000	0.0	30.500	0.0	
||	Pw tension	26.250	56.750	30.500	762.5	15.250	11627.9	web
||	Pw compression	2.750	26.250	23.500	-587.5	-11.750	6903.3	
||	Pc compression	0.000	2.750	2.750	-1925.0	-24.875	47884.8	bottom flange
||	Total				0.0		121978.6	

||	$y = D_p$ =	26.250	*in*	distance of PNA from the bottom of section
||	M_p =	121,979	*kip-in*	plastic moment
||	note: M_p =	10,165	*kip-ft*	plastic moment
||	D_{cp} =	23.500	*in*	depth of web in compression at plastic moment
||	D_t =	62.250	*in*	total depth of composite section

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF YIELD MOMENT My (for strength check) [AASHTO D6.2, p. 6-252]

G2 @ 1.0L - neg. moment (Model A)

||	**Input**			
||				● general
||	M_{D1} =	43,643	*kip-in*	factored moment applied to noncomposite section (1.25 DC1)
||	M_{D2} =	16,802	*kip-in*	factored moment applied to longterm composite section (1.25 DC2 + 1.5 DW)
||	note: M_{D1} =	3636.9	*kip-ft*	
||	note: M_{D2} =	1400.1	*kip-ft*	
||	f_c =	48.1	*ksi*	sum of compression flange stresses [import from Stresses tab, for Service II load group]
||	f_t =	51.2	*ksi*	sum of tension flange stresses [import from Stresses tab, for Service I| load group]
||	F_{yf} =	50.0	*ksi*	specified minimum yield strength of flange
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement

||	**Composite Section in Negative Flexure**			
||				
||	**$M_Y = \min(M_{Y[T1]}, M_{Y[T2]}, M_{Y[C]})$ =**	77,754	***kip-in***	
||	note: M_Y =	6,479	*kip-ft*	
||				
||				● determine moment to cause yielding in tension (top) flange
||	$M_{Y[T1]} = M_{D1} + M_{D2} + M_{AD[T1]}$ =	107,146	*kip-in*	
||	$M_{AD[T1]} = (F_{yf} - M_{D1}/S_{NC[T1]} - M_{D2}/_{COMP[T1]})\, S_{COMP[T1]}$ =	46,702	*kip-in*	additional moment applied to short term composite section to cause nominal yielding in tension flange
||	$S_{NC[T1]}$ =	2,142.9	*in³*	noncomposite section modulus
||	$S_{COMP[T1]} = S_{LT[T1]} = S_{ST[T1]}$ =	2,142.9	*in³*	composite section modulus (concrete deck not effective)
||				
||				● determine moment to cause yielding in reinforcement
||	$M_{Y[T2]} = M_{D2} + M_{AD[T2]}$ =	77,754	*kip-in*	
||	$M_{AD[T2]} = (F_{y,reinf} - M_{D1}/S_{NC[T2]})\, S_{COMP[T2]}$ =	60,952	*kip-in*	
||	$S_{COMP[T2]} = S_{LT[T2]} = S_{ST[T2]}$ =	1,743.2	*in³*	
||				
||				● determine moment to cause yielding in compression (bottom) flange
||	$M_{Y[C]} = M_{D1} + M_{D2} + M_{AD[C]}$ =	113,911	*kip-in*	
||	$M_{AD[C]} = (F_{yf} - M_{D1}/S_{NC[C]} - M_{D2}/S_{COMP[C]})\, S_{COM[C]}$ =	53,467	*kip-in*	
||	$S_{NC[C]}$ =	2,278.2	*in³*	
||	$S_{COMP[C]} = S_{LT[C]} = S_{ST[C]}$ =	2,278.2	*in³*	
||				
||	$D_c = \|f_c\| / (\|f_c\|+f_t)\, d - t_{fc}$ =	25.968	*in*	depth of web in compression in the elastic range
||	d =	59.250	*in*	depth of steel section
||	t_{bc} =	2.750	*in*	thickness of bottom flange

DETERMINATION OF YIELD MOMENT My (for constructibility check) [AASHTO D6.2, p. 6-252]

Project: ***Check CSI Bridge composite steel design***	Made By: *ok* Date: *5/7/2011*	Job No:
Subject:	Checked By: Date:	Sheet No:

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

Input

||	f_c =	15.3	*ksi*	sum of compression flange stresses [import from Stresses tab, for DC1]
||	f_t =	16.3	*ksi*	sum of tension flange stresses [import from Stresses tab, for DC1]
||	F_{yf} =	50.0	*ksi*	specified minimum yield strength of flange

Noncomposite Section in Negative Flexure

||	$M_Y = \min(S_{NC[T]}F_{yf}, S_{NC[C]}F_{yf})$ =	107,145	*kip-in*	
||	note: M_Y =	8,929	*kip-ft*	
||	$S_{NC[T]}$ =	2,142.9	*in*^{3}	section modulus for tension flange
||	$S_{NC[C]}$ =	2,278.2	*in*^{3}	section modulus for compression flange
||	$D_c = \|f_c\| / (\|f_c\|+f_t) d - t_{fc}$ =	25.968	*in*	depth of web in compression in the elastic range
||	d =	59.250	*in*	depth of steel section
||	t_{bc} =	2.750	*in*	thickness of bottom flange

Model B

Model C

Model C

- Model C uses staged construction to simulate the erection of the bridge.
- The design request is based on the staged construction load case.
- This model is currently a part of model A and may be isolated as a separate model in the future.

Load Cases

The following are the most important load cases were setup for model C:

- Service (staged): staged construction load case to reflect the construction sequence and apply unfactored loads
- Strength 1 (staged): staged construction load case to reflect the construction sequence and apply factored loads
- Service (staged, with pours): similar to the “Service (staged)” load case, but the deck is added in 3 pours
- Strength 1 (staged, with pours): similar to the “Strength 1 (staged)” load case, but the deck is added in 3 pours
- LL - 1 lane (after staged): moving load case with factored live loads applied to a model with stiffness at the end of “Strength 1 (staged)” load case.

Stresses Due to Staged Construction – G2 at 1.000L (120')

		Spreadsheet Calculations				CSI Bridge Output for "Strength 1 (staged)" load case	
		Unfactored Stresses	Factor	Factored Stresses	Cumulative Factored Stresses	Factored Stresses	Cumulative Factored Stresses
Stresses at the Bottom of Girder [ksi]	DC1	-15.3	1.25	-19.1	-19.1	-18.9	-18.9
	DC2	-2.2	1.25	-2.8	-21.9		-21.4
	DW	-3.1	1.5	-4.7	-26.5		-25.5
	LL+I	-12.4	1.75	-21.7	-48.2	-10.3	

Comments:

(1) CSI Bridge LL+I factored stress is based on the distribution from the analysis for 1 lane loaded, while the spreadsheet value is based on user-calculated distribution factors.

Comparison of Models A, B and C

Strength Limit State - No Pour Sequence

Parameter	Units	Model A	Model B	Model C
Design Request Mdnc Combo Mdc Combo Mu Combo		a_ Strength 1 a- Mdnc (Strength) a- Mdnc (Strength) a- Strength 1	b_ Strength 1 b- Mdnc (Strength) b- Mdnc (Strength) b- Strength 1	c_ Strength 1 a- Mdnc (Strength) a- Mdnc (Strength) c- Strength 1
G2 Positive Flexure at 50ft Mu D/C ratio	 [kip-ft] [-]	 0.868	 0.906	 0.867
G2 Negative Flexure at 120ft Mu D/C ratio	 [kip-ft] [-]	 1.138	 1.094	 1.142
G2 Shear at 120ft Vu D/C ratio	 [kips] [-]	 1.301	 1.291	 1.301

Comments:
(1) The above results are for build V15.1.0_S

ok, 4/25/2011

Strength Limit State - With Pour Sequence

Parameter Status/Comment	Units	Model A	Model B Not Aplicable	Model C
Design Request Mdnc Combo Mdc Combo Mu Combo		a_ Strength 1 (pours) a- Mdnc (Strength) a- Mdnc (Strength) a- Strength 1 (pours)		c_ Strength 1 (pours) a- Mdnc (Strength) a- Mdnc (Strength) c- Strength 1 (pours)
G2 Positive Flexure at 50ft Mu D/C ratio	 [kip-ft] [-]	 0.863		 0.862
G2 Negative Flexure at 120ft Mu D/C ratio	 [kip-ft] [-]	 1.148		 1.150
G2 Shear at 120ft Vu D/C ratio	 [kips] [-]	 1.303		 1.303

Comments:
(1) The above results are for CsiBridge V15.1.0_S.

ok, 4/25/2011

Fatigue Limit State

Parameter Status/Comment	Units	Model A	Model B Not Checked	Model C Not Checked
Design Request Fatigue Combo		a_ Fatigue a- Fatigue		
G2 Shear at 120ft Vu Vcr D/C ratio	 [kips] [kips] [-]	 32.58 368.56 0.088		

Notes:
(1) The above results are for CSiBridge V15.0.1_W

ok, 4/25/2011

Service Limit State - No Pour Sequence

Parameter Status/Comments	Units	Model A	Model B	Model C
Design Request Mdnc Combo Mdc Combo Ms Combo		a_ Service a- Mdnc (Service 2) a- Mdc (Service 2) a- Service 2	b_Service b- Mdnc (Service 2) b- Mdc (Service 2) b- Service 2	c_ Service a- Mdnc (Service 2) a- Mdc (Service 2) c- Service 2
G2 Positive Flexure at 50ft D/C ratio	[-]	0.887	0.921	0.924
G2 Negative Flexure at 120ft D/C ratio	[-]	0.682	0.652	0.697
G2 Shear at 120ft D/C ratio	[-]	not calculated	not calculated	not calculated

Notes:

(1) The above results are for CsiBridge V15.1.0_S.

ok, 4/25/2011

Service Limit State - With Pour Sequence

Parameter Status/Comments	Units	Model A	Model B Not Applicable	Model C
Design Request Mdnc Combo Mdc Combo DSet1 (Ms Combo)		a_ Service (pours) a- Mdnc (Service 2) a- Mdc (Service 2) a- Service 2 (pours)		c_ Service (pours) a- Mdnc (Service 2) a- Mdc (Service 2) c- Service 2 (pours)
G2 Positive Flexure at 50ft D/C ratio	[-]	0.880		0.918
G2 Negative Flexure at 120ft D/C ratio	[-]	0.690		0.702
G2 Shear at 120ft D/C ratio	[-]	not calculated		not calculated

Notes:

(1) The above results are for CSiBridge V15.1.0_S.

ok, 4/25/2011

Constructability Limit State - No Pour Sequence

Parameter Status/Comments	Units	Model A	Model B	Model C
Design Request Demand Set 1 Demand Set 2 Demand Set 3 Demand Set 4		a_ Constr a- Strength IV		c_ Constr c. Strength 4 DL, stage DC1a c. Strength 4 DL, stage DC1b c. Strength 4 DL, stage DC1c c. Strength 4 DL, stage DC1d
G2 Positive Flexure at 50ft D/C ratio	 [-]	 0.649		 0.652
G2 Negative Flexure at 120ft D/C ratio	 [-]	 0.547		 0.551
G2 Shear at 120ft Vu D/C ratio	 [kips] [-]	 0.501		 0.504

Notes:

(1) The above results are for CsiBridge V15.1.0_S, except for shear ratio for Model A that was obtained from V15.1.0_Y.

ok, 5/7/2011

Constructability Limit State - With Pour Sequence

Parameter Status/Comments	Units	Model A	Model B Not Applicable	Model C
Design Request Demand Set 1 Demand Set 2 Demand Set 3 Demand Set 4 Demand Set 5 Demand Set 6 Demand Set 7 Demand Set 8 Demand Set 9		a_ Constr (pour) a- Strength 4 (pour)		c_ Constr (pour) c. Strength 4 DL, stage DC1a c. Strength 4 DL, stage DC1b c. Strength 4 DL, stage DC1c c. Strength 4 DL, stage DC1d pour 1 c. Strength 4 DL, stage DC1d harden 1 c. Strength 4 DL, stage DC1d pour 2 c. Strength 4 DL, stage DC1d harden 2 c. Strength 4 DL, stage DC1d pour 3 c. Strength 4 DL, stage DC1d harden 3
G2 Positive Flexure at 50ft D/C ratio	 [-]	 0.633		 0.807
G2 Negative Flexure at 120ft D/C ratio	 [-]	 0.558		 0.564
G2 Shear at 120ft Vu D/C ratio	 [kips] [-]	 0.503		 0.509

Notes:
(1) The above results were obtained in CsiBridge V15.1.0_S, except for shear ratio for Model A that was obtained from V15.1.0_Y.

ok, 5/7/2011

SUPERSEDED

Model A

SUPERSEDED

CSiBridge Tabular Output

SUPERSEDED

Strength Limit State

SUPERSEDED

G2 - Section at 0.417L

TABLE: Bridge Super Design 01 - Design Result Status			
Parameter	Unit	Value 1	Value 2
DesReqName	*Text*	Flexure (was changed to Strength 1)	Flexure
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or	Design was performed and results are available, whether or not the design passed or

- This model had diaphragms only at abutments and piers (it did not have any intermediate diaphragms).

The results on the following 6 pages are for build V15.0.0-R

@ 0.417L for Build R

TABLE: Bridge Super Design 29 - AASHTOLRFD07 - SteelICompStrgth-Prop

Parameter	Unit	Value 1	Value 2
Request	Text	Flexure	Flexure
BridgeObj	Text	BOBJ1	BOBJ1
Station	ft	50	50
Location	Text	Before	After
Girder	Text	Interior Girder 1	Interior Girder 1
BeamProp	Text	I-Girder 0.625in T	I-Girder 0.625in T
LLDFactM	Unitless	0.696	0.696
LLDFactV	Unitless	0.935	0.935
ASlabTri	ft2	6.5	6.5
ThSlab	ft	0.66667 (8")	0.66667
WSlabEff	ft	(117") 9.75	9.75 ← say OK (average spacing)
fcConcSlab	Kip/ft2	576 (4ksi)	576
ESlab	Kip/ft2	519119.5 (3604ksi)	519119.5
nLongTerm	Unitless	3	3
ARebSlabTop	ft2	0	0
ARebSlabBot	ft2	0	0
YRebSlabTop	ft	0	0
YRebSlabBot	ft	0	0
fysLRebar	Kip/ft2	8640 (60ksi)	8640
ABeam	ft2	0.3335 (48in²)	0.3335
EBeam	Kip/ft2	4176000 (29,000ksi)	4176000
IxBeam	ft4	1.0668 (22121in⁴)	1.0668
BeamRolled	Yes/No	No	No
ThFlgTop	ft	0.0521 (0.625")	0.0521
WdthFlgTop	ft	1.1667 (14")	1.1667
fyFlgTop	Kip/ft2	7200 (50ksi)	7200
ThFlgBot	ft	0.0729 (0.875in)	0.0729
WdthFlgBot	ft	1.1667 (14")	1.1667
fyFlgBot	Kip/ft2	7200 (50ksi)	7200
fyrFlgBot	Kip/ft2	5040 (35ksi)	5040 0.7Fyc, p. 6-255
LamfBotFlg λ_f	Unitless	8.002058	8.002058 see hand calcs.
LampfBotFlg λ_{pf}	Unitless	9.151612	9.151612
LamrfBotFlg λ_{rf}	Unitless	16.119553 (vs. 16.96 by hand)	16.119553
kcBotFlg k_c	Unitless	0.385054	0.385054
CmpctFlgBot	Yes/No	Yes (8<9.15)	Yes
DepthWeb	ft	4.5 (54")	4.5
ThickWeb	ft	0.0417 (0.5")	0.0417
fyWeb	Kip/ft2	7200 (50ksi)	7200
DcpWebPos	ft	0	0 (checked against spreadsheet)
DcWebNeg	ft	2.08172 (25")	2.08172 This matches the
DcpWebNeg	ft	1.95902 (23.5")	1.95902 spreadsheet if all
LamwWeb λ_w	Unitless	99.842831	99.842831 moment are taken
LampwDcWeb $\lambda_{pw(D_c)}$	Unitless	24.794714	24.794714 as negative
LampwDcpWeb $\lambda_{pw(D_{cp})}$	Unitless	23.333292	23.333292
LamrwWeb λ_{rw}	Unitless	137.274178	137.274178
CmpctWebNeg	Yes/No	No	No
RpcWeb	Unitless	0	0
RptWeb	Unitless	0	0
rt	ft	0.29393 (3.527")	0.29393 (vs. 3.523 in spreadsheet)
J	ft4	0	0

No rebar was considered (p. 3-12 of FHWA Example)

Is this applicable for pos. moment?

@ 0.417L for Build R

	RbPos	Unitless	1	1	
	RbNeg	Unitless	1	1	
	RhPos	Unitless	1	1	
	RhNeg	Unitless	0.985987	0.985987	
	CmpctGrdPos	Yes/No	Yes	Yes	
	CmpctGrdNeg	Yes/No	Yes	Yes	
	SxSteelBot	ft3	0.495122 (855.5 in³) ✓	0.495122 ✓	
	SxSteelTop	ft3	0.431837 (746.2 in³) ✓	0.431837 ✓	
	SxLTermBPos	ft3	0.699063 (1207.9 in³) ✓	0.699063 ✓	(vs. 1210 in sprd.)
check →	SxLTermTPos	ft3	2.211966 (3822.3 in³)	2.211966	(vs. 4003 in sprd.)
	SxSTermBPos	ft3	0.761928 (1316.6 in³) ✓	0.761928	(vs. 1316.4 in sprd.)
check →	SxSTermTPos	ft3	10.383153 (17942.1 in³)	10.383153	(vs. 19532 in sprd.)
Not sure why this is needed	SxCompBNeg	ft3	0.495122 (855 in³)	0.495122	
	SxCompTNeg	ft3	0.315938 (545 in³)	0.315938	
	MpPos	Kip-ft	7506.8302 ✓	7506.8302 ✓	compared to spreadsheet
	MpNeg	Kip-ft	3892.4384 ✓	3892.4384 ✓	
	PNADistPos	ft	0.503 (6.03") ✓	0.503 ✓	
	PNADistLmt $= 0.42 D_t = (0.42)(60.375) = 25.36$	ft	2.2225 (26.67") ✓	2.2225 ✓	
check →	PNADistNeg	ft	3.49931 (42")	3.49931	vs. 36" by hand calc
	MuDNC	Kip-ft	1297.0749 ✓ (vs. 1313 in sprd.)	1297.0749 ✓	
	MuDCLTerm	Kip-ft	593.9712 ✓ (vs. 603 in sprd.)	593.9712 ✓	
	MyPos	Kip-ft	4735.7718 ✓ (vs. 4724 in sprd.)	4735.7718 ✓	
check →	MyNegCtr	Kip-ft	? 1927.5769 (vs. 3108 in sprd.)	1927.5769	
	MyNegBot	Kip-ft	3566.3448 ✓ (vs. 3564 in sprd.)	3566.3448 ✓	
check →	MyNegTop	Kip-ft	? 1927.5769	1927.5769	
	Lp	ft	7.07873 (84.94")	7.07873	(vs. 84.8" in sprd.)
	Lr	ft	26.58006 (318.96")	26.58006	(vs 318.6" in sprd.)
	Lb	ft	120 ✓	120	
	BeamPBklShr	Yes/No	No	No	

this appears to be taken from spacing of diaphragms

(a) 0.447L for Baild R

TABLE: Bridge Super Design 30 - AASHTOLRFD07 - SteelICompStrgth-FlexPos

Parameter	Unit	Value 1	Value 2
Request	*Text*	Flexure √	Flexure √
BridgeObj	*Text*	BOBJ1 √	BOBJ1 √
Station	*ft*	50 √	50 √
Location	*Text*	Before √	After √
Girder	*Text*	Interior Girder 1 √	Interior Girder 1 √
Combo	*Text*	c Strength 1 √	c Strength 1 √
StepType	*Text*	Max √	Min
Step	*Text*	0 √	0
DSet	*Text*	Mu Combo √	Mu Combo √
CodeEqtn	*Text*	6.10.7.1.1-1 Compact Section Positive Flexure	6.10.7.1.1-1 Compact Section Positive Flexure (6.6-11a)
MuPos	*Kip-ft*	5326.4245 √ (vs 5351 in sprd.)	5326.4245 √
fl	*Kip/ft2*	0	0
MrPos	*Kip-ft*	6156.5033 √	6156.5033 √ (vs. 6142 in sprd
Pu	*Kip*	0	0
MuNonComp	*Kip-ft*	0	0
MuLTerm	*Kip-ft*	0	0
MuSTerm	*Kip-ft*	0	0
fbuComp	*Kip/ft2*	0	0
fbuTens	*Kip/ft2*	0	0
FrcPos	*Kip/ft2*	0	0
FrtPos	*Kip/ft2*	0	0
DCRatio	*Unitless*	0.86517 √	0.86517 √

These values are reported as zero, because the section is compact and the resistance is compared directly to Mu Pos, therefore these values are not used.

@ 0.417 for Baild R

This table is not applicable, because the section is in positive flexure.

TABLE: Bridge Super Design 31 - AASHTOLRFD07 - SteelICompStrgth-FlexNeg			
Parameter	Unit	Value 1	Value 2
Request	*Text*	Flexure	Flexure
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	50	50
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 1	Interior Girder 1
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.8.2.3-3 Bottom Flange	6.10.8.2.3-3 Bottom Flange
MuNeg	*Kip-ft*	0	0
fl	*Kip/ft2*	0	0
MncFLB	*Kip-ft*	0	0
MncLTB	*Kip-ft*	0	0
MrcNeg	*Kip-ft*	0	0
MrtNeg	*Kip-ft*	0	0
Pu	*Kip*	-8.11E-08	-1.117E-07
MuNonComp	*Kip-ft*	1297.0749	1297.0749
MuLTerm	*Kip-ft*	593.9712	593.9712
MuSTerm	*Kip-ft*	3435.3783	-952.2962
fbuComp	*Kip/ft2*	0	0
fbuTens	*Kip/ft2*	0	0
FncFLB	*Kip/ft2*	7099.11	7099.11
FncLTB	*Kip/ft2*	247.27	247.27
FrcNeg	*Kip/ft2*	247.27	247.27
FrtNeg	*Kip/ft2*	7099.11	7099.11
DCRatio	*Unitless*	0	0

@ 0.417L for Build R

TABLE: Bridge Super Design 32 - AASHTOLRFD07 - SteelICompStrgth-Shear

Parameter	Unit	Value 1 √	Value 2 √
Request	Text	Flexure √	Flexure √
BridgeObj	Text	BOBJ1 √	BOBJ1 √
Station	ft	50 √	50 √
Location	Text	Before √	After √
Girder	Text	Interior Girder 1 √	Interior Girder 1 √
Combo	Text	c Strength 1 √	c Strength 1 √
StepType	Text	Max √	Min √
Step	Text	0 √	0 √
DSet	Text	Mu Combo √	Mu Combo √
CodeEqtn	Text	6.10.9.2-1 √	6.10.9.2-1 √
PanelType	Text	Internal Unstiffened √	Internal Unstiffened √
Vu	Kip	99.877 √ (vs. 84.7 in spread)	109.706 √
Vr	Kip	306.502 √	306.502 √
Vcr	Kip	306.502 √	306.502 √
Vp	Kip	783.949 √	783.949 √
C	Unitless	0.390971 √	0.390971 √
k	Unitless	5 √	5 √
d0	ft	0 √	0 √
d0req	ft	0 √	0 √
VrWithD0req	Kip	306.502 √	306.502 √
DCRatio	Unitless	0.325863	0.357929

} same as for section at 0.417 L

V_u ... CSI bridge output = 84.4 kip before STA 50

114.1 kip after STA 50

@ 0.417 L for Build R

G2 - section at 1.000L (negative moment goveuns)

Parameter	Unit	Value 1	Value 2
TABLE: Bridge Super Design 01 - Design Result Status ✓		was changed to	strength 1
Parameter	Unit	Value 1	Value 2
DesReqName	*Text*	Flexure	Flexure
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Status	*Unitless*	0	0
Message	*Text*	Design was performed and results are available, whether or not the design passed or failed.	Design was performed and results are available, whether or not the design passed or failed.

- This model had intermediate diaphragms.

The results on the following 6 pages are for build 15.0.0_R

TABLE: Bridge Super Design 29 - AASHTOLRFD07 - SteelICompStrgth-Prop

Parameter	Unit	Value 1	Value 2	
Request	Text	Flexure ✓	Flexure ✓	
BridgeObj	Text	BOBJ1 ✓	BOBJ1 ✓	
Station	ft	120 ✓	120 ✓	
Location	Text	Before ✓	After ✓	
Girder	Text	Interior Girder 2 ✓	Interior Girder 2 ✓	
BeamProp	Text	I-Girder 2.5in T ✓	I-Girder 2.5in T ✓	
LLDFactM	Unitless	0.696 ✓	0.696 ✓	
LLDFactV	Unitless	0.935 ✓	0.935 ✓	
ASlabTri	ft2	6.5 ✓	6.5 ✓	
ThSlab	ft	0.66667 (8")	0.66667 ✓	
WSlabEff	ft	9.75 (117") ✓	9.75 ✓ ← SAY	OK (average spacing, see calcs for details)
fcConcSlab	Kip/ft2	576 (4ksi) ✓	576 ✓	
ESlab	Kip/ft2	519119.5 (3604ksi) ✓	519119.5 ✓	
nLongTerm	Unitless	3 ✓	3 ✓	
ARebSlabTop	ft2	0	0	No Rebar was considered.
ARebSlabBot	ft2	0	0	
YRebSlabTop	ft	0	0	
YRebSlabBot	ft	0	0	
fysLRebar	Kip/ft2	8640 (60ksi) ✓	8640 ✓	
ABeam	ft2	0.6981	0.6981	
EBeam	Kip/ft2	4176000 (29,000ksi) ✓	4176000 ✓	
IxBeam	ft4	3.155525 (65432 in⁴) ✓	3.155525 ✓	(vs. 65426 in spud.)
BeamRolled	Yes/No	No ✓	No ✓	
ThFlgTop	ft	0.2083 (2.5") ✓	0.2083 ✓	
WdthFlgTop	ft	1.1667 (14") ✓	1.1667 ✓	
fyFlgTop	Kip/ft2	7200 (50ksi) ✓	7200 ✓	
ThFlgBot	ft	0.2292 (2.75") ✓	0.2292 ✓	
WdthFlgBot	ft	1.1667 (14") ✓	1.1667 ✓	
fyFlgBot	Kip/ft2	7200 (50ksi) ✓	7200 ✓	
fyrFlgBot	Kip/ft2	5040 (35ksi) ✓	5040	$0.7 F_{yc}$, p. 6-225
LamfBotFlg λ_f	Unitless	2.545157 ✓	2.545157 ✓	$\frac{14}{(2)(12.75)} = 2.54$
LampfBotFlg λ_{pf}	Unitless	9.151612 ✓	9.151612 ✓	same as for G2 at 0.417L
LamrfBotFlg λ_{rf}	Unitless	16.119553 ✓	16.119553 ✓	
kcBotFlg k_c	Unitless	0.385054	0.385054	
CmpctFlgBot	Yes/No	Yes (2.54 < 9.15) ✓	Yes ✓	
DepthWeb	ft	4.5 (54") ✓	4.5 ✓	
ThickWeb	ft	0.0417 (0.5") ✓	0.0417 ✓	
fyWeb	Kip/ft2	7200 (50ksi) ✓	7200 ✓	
DcpWebPos	ft	0 ✓	0 ✓	(vs. 0 in spud.)
DcWebNeg	ft	2.16377 (25.97")	2.16377	(vs. 25.96 in spud.)
DcpWebNeg	ft	1.95763 (23.50") ✓	1.95763	(vs. 23.5" in spud.)
LamwWeb λ_w	Unitless	103.777764	103.777764	
LampwDcWeb $\lambda_{pw}(D_c)$	Unitless	68.108233	68.108233	
LampwDcpWeb $\lambda_{pw}(D_{cp})$	Unitless	61.619599	61.619599	
LamrwWeb λ_{rw}	Unitless	137.274178 ✓	137.274178 ✓	(vs. 137.3 handcalc)
CmpctWebNeg	Yes/No	No ✓	No ✓	
RpcWeb	Unitless	0	0	
RptWeb	Unitless	0	0	
rt	ft	0.32814 (3.937") say ok	0.32814 ✓	(vs 3.832" in spud.)
J	ft4	0	0	

$$n = \frac{29{,}000\ ksi}{3{,}604\ ksi} = 8.047$$

RbPos	*Unitless*	1	1	
RbNeg	*Unitless*	1	1	
RhPos	*Unitless*	1	1	
RhNeg	*Unitless*	0.995361	0.995361	
CmpctGrdPos	*Yes/No*	Yes	Yes	
CmpctGrdNeg	*Yes/No*	Yes	Yes	
SxSteelBot	*ft3*	1.318667 (2278.7in³) ✓	1.318667 ✓	(vs. 2278.2 in spr.
SxSteelTop	*ft3*	1.240119 (2142.9in³) ✓	1.240119 ✓	(vs. 2142.9 in spr
SxLTermBPos	*ft3*	1.479132 (2555.94in³) say OK	1.513569 (2615.15in³) ✓	(vs. 2615.5in s
SxLTermTPos	*ft3*	2.661919 (4599.8in³) ?	2.831093 (4892.1in³) ✓	(vs. 4924 in sp
SxSTermBPos	*ft3*	1.573202 (2718.5in³) ?	1.624842 (2807.7in³) ✓	(vs. 2805.6ins
SxSTermTPos	*ft3*	5.897908 (10191.6in³) ?	6.768451 (11695in³) ✓	(vs. 11812 in sp
SxCompBNeg	*ft3*	1.318667 (2278.7in³) ✓	1.318667 (2278.7in³) ✓	(vs. 2278.2in sp
SxCompTNeg	*ft3*	0.957797 (1655.1in³) ?	0.957797 (1655.1in³) ?	(vs. 2142.9in³ in
MpPos	*Kip-ft*	14020.0069 ✓	14020.0069 ✓	(vs. 14010 in sprd.)
MpNeg	*Kip-ft*	10170.3594 ✓	10170.3594 ✓	(vs. 10165 in sprd.
PNADistPos	*ft*	0.85978 (10.3in) ✓	0.85978 ✓	(vs. 10.3 in sprd)
PNADistLmt 0.42D_t = (0.42)(68.25") = 28.7"	*ft*	2.35375 (28.24in) ✓	2.35375 ✓	
PNADistNeg	*ft*	3.50071 (42")	3.50071	(vs. 36" in sprd
MuDNC	*Kip-ft*	-3584.7734 ✓	-3584.7734 ✓	(vs. -3636 in sprd
MuDCLTerm	*Kip-ft*	-1348.398 ✓	-1348.398 ✓	(vs. -1400 in sprd
MyPos	*Kip-ft*	12109.4252	12635.1512	(vs. 10756 in sprd
MyNegCtr	*Kip-ft*	7715.08	7715.08	(vs. 6479 in sprd
MyNegBot	*Kip-ft*	9498.3148 ✓	9498.3148	(vs 9492 in sprd.
MyNegTop	*Kip-ft*	7715.08	7715.08	(vs. 6479 in sprd.
Lp	*ft*	7.90269 (94.8") ✓	7.90269 ✓	(vs. 92.3 in sprd
Lr	*ft*	29.674 (356") ✓	29.674 ✓	(vs. 346.5 in sprd
Lb	*ft*	20 ✓	20 ✓	
BeamPBklShr	*Yes/No*	Yes	Yes	

This table does not apply, because the section is in negative flexure.

TABLE: **Bridge Super Design 30 - AASHTOLRFD07 - SteelICompStrgth-FlexPos**

Parameter	Unit	Value 1	Value 2
Request	*Text*	Flexure	Flexure
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 2	Interior Girder 2
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.7.1.1-1 Compact Section	6.10.7.1.1-1 Compact Section
MuPos	*Kip-ft*	0	0
fl	*Kip/ft2*	0	0
MrPos	*Kip-ft*	14020.0069	14020.0069
Pu	*Kip*	0	0
MuNonComp	*Kip-ft*	0	0
MuLTerm	*Kip-ft*	0	0
MuSTerm	*Kip-ft*	0	0
fbuComp	*Kip/ft2*	0	0
fbuTens	*Kip/ft2*	0	0
FrcPos	*Kip/ft2*	0	0
FrtPos	*Kip/ft2*	0	0
DCRatio	*Unitless*	0	0

$M_{p\,(pos)}$ from previous page

TABLE: Bridge Super Design 31 - AASHTOLRFD07 - SteelICompStrgth-FlexNeg

	Parameter	Unit	Value 1 ✓	Value 2 ✓	
	Request	*Text*	Flexure ✓	Flexure ✓	
	BridgeObj	*Text*	BOBJ1 ✓	BOBJ1	
	Station	*ft*	120 ✓	120 ✓	
	Location	*Text*	Before ✓	After ✓	
	Girder	*Text*	Interior Girder 2 ✓	Interior Girder 2 ✓	
	Combo	*Text*	c Strength 1 ✓	c Strength 1 ✓	
	StepType	*Text*	Min ✓	Max ✓	
	Step	*Text*	0 ✓	0 ✓	
	DSet	*Text*	Mu Combo ✓	Mu Combo ✓	
	CodeEqtn	*Text*	6.10.8.2.3-2 Bottom Flange	6.10.8.2.3-2 Bottom Flange	
check	MuNeg	*Kip-ft*	0	0	not sure why this is zero
	fl	*Kip/ft2*	0	0	
check	MncFLB	*Kip-ft*	0	0	not sure why all these are set zero
	MncLTB	*Kip-ft*	0	0	
	MrcNeg	*Kip-ft*	0	0	
	MrtNeg	*Kip-ft*	0	0	
	Pu	*Kip*	-3.679E-08	-9.252E-08	
	MuNonComp	*Kip-ft*	-3584.7734 ✓	-3584.7734 ✓	(vs. -3636 in sprd.)
	MuLTerm	*Kip-ft*	-1348.398 ✓	-1348.398 ✓	(vs. -1400 in sprd
NG	MuSTerm	*Kip-ft*	-1179.398	-1179.398	(vs. -4404 in sprd
compression flange stress	fbuComp *	*Kip/ft2*	-4633.51 (-32.18 ksi)	-4633.51	should be -46.1 (from sprd.)
tensile flange stress	fbuTens *	*Kip/ft2*	5527.57 (38 ksi)	5527.57	
	FncFLB	*Kip/ft2*	7166.6 (49.8 ksi)	7166.6	
NG	FncLTB	*Kip/ft2*	2800.49 (19.4 ksi)	2800.49	(vs. 37.7 in sprd
	FrcNeg	*Kip/ft2*	2800.49 (19.4 ksi)	2800.49	
	FrtNeg	*Kip/ft2*	7166.6 (49.8 ksi)	7166.6	
	DCRatio	*Unitless*	1.654531	1.654531	

* provide legend

Note that the program uses $C_b = 1$

This was fixed for build V15.0.0_1F. see printout for build V15.0.0_1F for details.

TABLE: Bridge Super Design 32 - AASHTOLRFD07 - SteelICompStrgth-Shear

was changed to strength 1

Parameter	Unit	Value 1	Value 2
Request	*Text*	Flexure	Flexure
BridgeObj	*Text*	BOBJ1	BOBJ1
Station	*ft*	120	120
Location	*Text*	Before	After
Girder	*Text*	Interior Girder 2	Interior Girder 2
Combo	*Text*	c Strength 1	c Strength 1
StepType	*Text*	Max	Max
Step	*Text*	0	0
DSet	*Text*	Mu Combo	Mu Combo
CodeEqtn	*Text*	6.10.9.2-1	6.10.9.2-1
PanelType	*Text*	Internal Unstiffened	Internal Unstiffened
Vu	*Kip*	~~226.283~~ 391.2	~~226.284~~ 391.2
Vr	*Kip*	306.502	306.502
Vcr	*Kip*	306.502	306.502
Vp	*Kip*	783.949	783.949
C	*Unitless*	0.390971	0.390971
k	*Unitless*	5	5
d0	*ft*	0	0
d0req	*ft*	0	0
VrWithD0req	*Kip*	306.502	306.502
DCRatio	*Unitless*	$\frac{226.283}{306.5}$ = 0.738276	0.738279

in latest program tables as of 7/22/2010 →

(vs. 394 in sp

(vs. 306 in sp

(vs. 783.9 in sp

(vs. 0.390 in sp

Model B

SUPERSEDED

Model C

SUPERSEDED