Section 2 – Temporary Structures

SINGLE SPAN TEMPORARY BRIDGE DETAILS

XS Sheet Numbers:

xs2-010-1 to xs2-010-6

Description of Component:

Temporary Bridge Details for Single Span Steel Rolled Wide Flange Girders with Timber Decking

Standard Drawing Features:

The five possible girder types for this single span temporary bridge configuration are presented by girder, diaphragm and span length ranging from 20-70 ft. The bridge is designed to have only 1 lane of traffic open at any given time, regulated through automated signals and reversing traffic control.

Bridge Design

1. Designers must select the appropriate design based on span length in Table 1.
2. All call-outs shall be updated on the XS sheet details per the Girder/Diaphragm Type selected.
3. General Plan, Deck Contours, Foundation Plan and Log of Test Boring sheets should be added to each set of plans in addition to other project-specific details.
4. It is the responsibility of the designer to renumber the sheets accordingly and add project specific details.
5. It is the responsibility of the designer to consider and incorporate project-specific site conditions in the final design of the temporary bridge.

Table 1 – Girder and Diaphragm Selection by Span Length (Specify Project-specific design information from Table 1 into Contract plan sheets)

Design L (ft)*	Girder	Diaphragm	Maximum Diaphragm Spacing S (ft)*
20-29	W18x86	C10x15.3	
30-39	W24x104	C15x33.9	(1) Spaced @ 1/2L
40-49	W33x130		
50-59	W36x160		(2) Spaced @ 1/3L
60-70	W36x194	MC18x42.7	(3) Spaced @ 1/4L

(*) Designer must specify the Maximum Diaphragm Spacing S in feet in the Contract plans based on selected Design L that satisfies the Maximum Diaphragm Spacing S required in Table

Drawing Package:

Abutment Layout

Typical Section

Timber Deck Details

**Diaphragm Details, Simple Span L = 20'-30

**Diaphragm Details, Simple Span L = 30'-40'

**Diaphragm Details, Simple Span L = 40'-70'

(**) Designer will choose only one that applies.

Design/General Notes:

The design and details are based on AASHTO LRFD Bridge Design Specifications, 8th Edition, with CA Amendments Section 3.10.10 and AISC Steel Construction Manual, 15th Edition. Section 48 of Caltrans Standard Specifications provides information on Temporary Bridge/Structure fabrication and construction. Designers must read this section of the Standard Specifications. Live loads used for design were Strength I (HL-93) and Strength II (P15).

¾" ∅ high-strength bolts shall be ASTM F3125 Grade A325X.

Threaded rods and carriage bolts shall be ASTM F1554 A307 Grade 36.

Additional Drawings Needed to Complete PS&E:

General Plan

Deck Contours

Foundation Plan

Log of Test Borings

Contract Specifications:

Caltrans Standard Specifications:

Section 12 Temporary Traffic Control

Section 48 Temporary Structures

Section 57 Wood and Plastic Lumber Structures

Section 55 Steel Structures

Section 75 Miscellaneous Metal

Restrictions on Use of Standard Drawings:

The PE is responsible for applying the pre-designed Table 1 of this User Guide to the bridge and stamping the XS sheets used with a valid California Professional Engineer License Stamp.

Single Span Temporary Bridge abutments were designed using minimum soil bearing capacity of 3 ksf for Service Level Load. If project-specific site conditions do not meet the minimum soil bearing capacity, the PE and/or Geoprofessional are responsible for ensuring adequate soil bearing capacity to support the selected single span temporary bridge.

Design calculations were done using 2” average depth of HMA over the timber deck. If project-specific design requires more than 2” average HMA thickness, the PE is responsible for ensuring the temporary bridge design can accommodate project-specific design HMA thickness.

Special Considerations:

Selected design verification calculations are provided in the attached Appendix for single spans up to 70 ft.

APPENDIX

I. Wood Deck and Calculations for Span L = 20-30 ft

II. Calculations for Span L = 30-40 ft

III. Calculations for Span L = 40-50 ft

IV. Calculations for Span L = 50-60 ft

V. Calculations Span L = 60-70 ft

VI. Abutment Calculations Span L = 70 ft

I. Design Verification for Steel Rolled Wide Flange Girders with Timber Decking

Span Length = 20'- 30'

Wide Flange Girder (WFG), Girder spacing, S = 4'-0"

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Wood Deck Plank Calculations

1.1) Plank Demand

1.2) Plank Capacity

2.) Steel Girder Calculations

2.1) Girder Demand

2.2) Girder Capacity

3.) Steel Bearing Stiffener Calculations

3.1) Stiffener Demand

3.2) Stiffener Capacity

4.) Welded Connection Capacity

5.) Bolted Connection Capacity

6.) Steel Diaphragm Demand

7.) Steel Diaphragm Capacity

Temporary Bridge Calculations – Single Span, 1 Lane

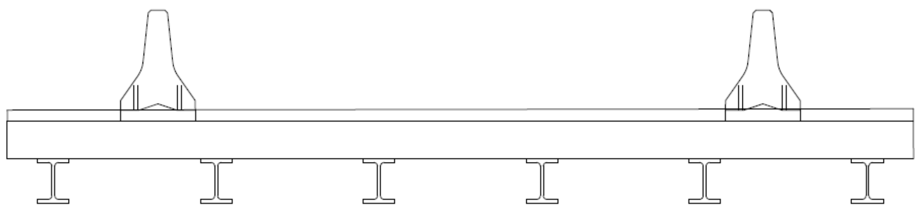

Figure 1 – Typical Section

Materials:

- 12 x 16 Spike Laminated Decking, DFL No.2 or better [AASHTO Art. 9.9.6]
- Steel Wide Flange Girder
- Temporary Barriers
- HMA Overlay
- Misc. Connection Hardware
- 24 ft total deck width, 6 girders total

1.) <u>Wood Deck Plank Calcs:</u>

1.1) Wood Deck Plank Demand:

Dead Loads (D_L):

[AASHTO Table 3.5.1-1]

w1 (Timber Deck) = 0.05 kcf x 1’ x 1’ = 0.05 klf

w2 (HMA) = 0.14 kcf x (2/12)’ x 1’ = 0.023 klf

Live Loads (L_L, HL93 Transverse):

Assuming simply supported between girders for Live Load

Moment Demand:

$$M_{u\_LL} = \frac{PL}{4} = \frac{16\ kips\ x\ 4\ ft}{4} = 16\ kft/plank$$

- No need to apply dynamic Load Allowance [AASHTO Art. 3.6.2.3]
- No need to apply lane load [AASHTO Art. 4.6.2.1.3]

- Assumed D_L is applied uniformly over contact area

$$M_{u_{factored}} = \frac{\Sigma \gamma_i w_i L^2}{8} = 1.25\left(0.05\ klf\ x\ \frac{4^2 ft^2}{8}\right) + 1.50\left(0.023\ klf\ x\ \frac{4^2 ft^2}{8}\right) + 1.75\ (16\ kft)$$

$$\boxed{M_{u\_factored} = 28.20\ kft}$$

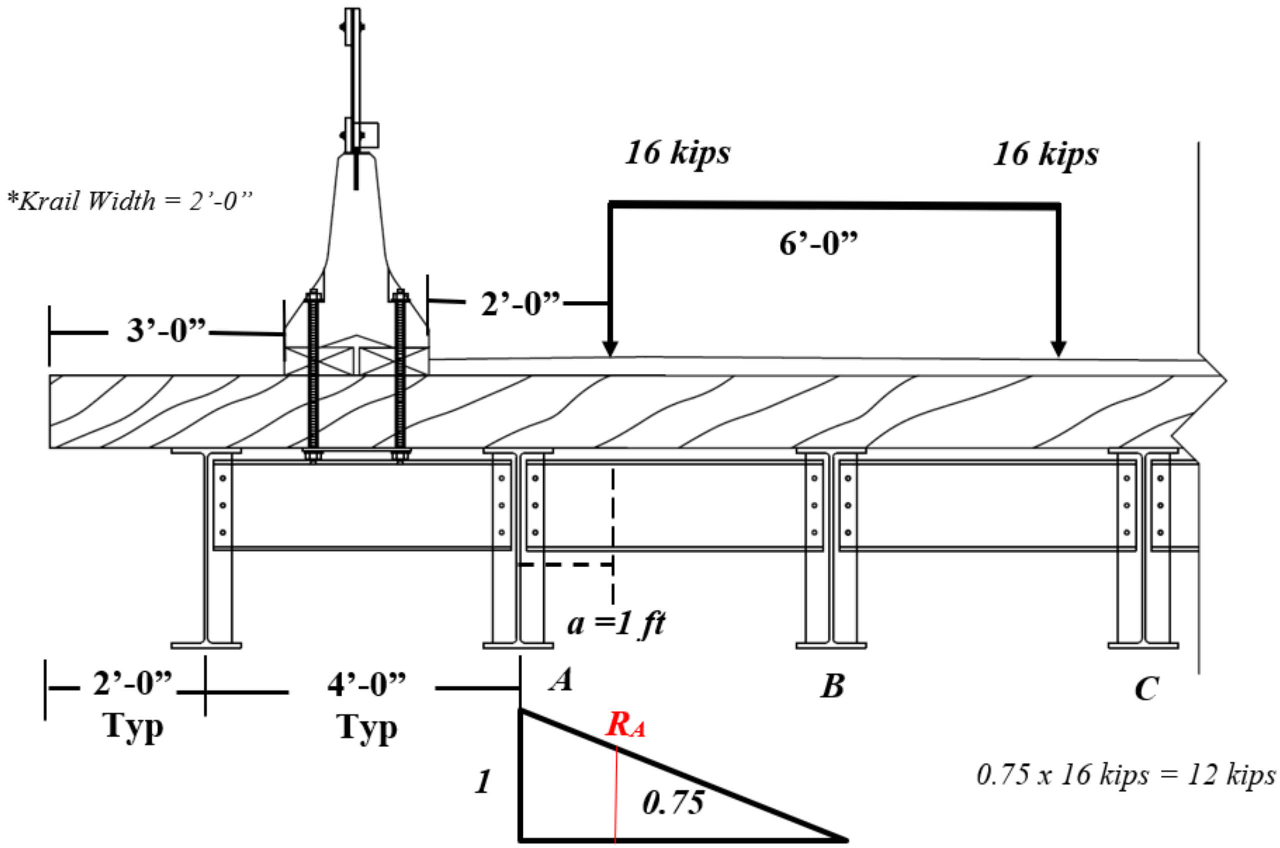

Figure 2 – Part Typical Section, HL-93 Rxn's

From Structural Analysis:

Neglecting LL effect on 1st girder and assuming 2 span 8 ft cont. beam with 2 concentrated loads at "a"

R_A = R_C = 10.12 kips, R_B = 11.75 kips (use 12 kips for shear design, assuming simply supported between girders from influence line R_A)

Shear Demand:

$$V_{u\_LL} = 12\ kips$$

$$V_{u_{factored}} = \Sigma \gamma_i w_i = 1.25(0.05\ klf\ x\ 1\ ft) + 1.50(0.023\ klf\ x\ 1\ ft) + 1.75\ (12\ kips)$$

$$\boxed{V_{u\_factored} = 21.10\ kips}$$

1.2) Wood Deck Plank Capacity:

From AASHTO, Ch.8 – Wood Structures:

AASHTO Table 8.5.2.2 – Resistance Factors, Ø	
Flexure	0.85
Shear	0.75
Compression Perpendicular to the Grain	0.90

Plank Nominal Size (12" x 12"): 11.5" x 11.5"

Species: DFL No. 2 or better

<u>Adjusted Bending Stress Design</u> [AASHTO Table 4.6.2.2.2a-1]

$F_b = F_{bo} C_{KF} C_M C_F C_{fu} C_d C_\lambda$ [AASHTO Art. 8.4.4.1-1]

$C_{KF} = \frac{2.5}{\emptyset} = 2.94$ for bending and shear (LRFD) [AASHTO Art. 8.4.4.2]

$C_{KF} = \frac{2.1}{\emptyset} = 2.33$ for compression perpendicular to grain (LRFD) [AASHTO Art. 8.4.4.2]

$F_{bo}C_{KF} = 0.75 \text{ x } (\frac{2.5}{0.85}) = 2.20 > 1.15,\ C_M = 1.0$ [AASHTO Table 8.4.4.3-1]

$C_F = 1.0$ [AASHTO Table 8.4.4.4-1]

$C_{fu} = 1.0$ Since Member is > 10" x 10" [AASHTO Art. 8.4.4.6]

$C_d = 1.0$ [AASHTO Table 8.4.4.8-1]

$C_\lambda = 0.80$ (Str I) [AASHTO Table 8.4.4.9-1]

$F_{bo} = 0.75$ ksi [AASHTO Table 8.4.1.1.4-1]

$F_b = 0.75 \text{ ksi x } 2.94 \text{ x } 1 \text{ x } 1 \text{ x } 1 \text{ x } 1 \text{ x } 0.80 = \underline{1.76 \text{ ksi}}$

Adjusted Shear Stress Design

$F_v = F_{vo}C_{KF}C_MC_\lambda$ [AASHTO Art. 8.4.4.1-2]

$F_{vo} = 0.17$ ksi [AASHTO Table 8.4.1.1.4-1]

$F_v = 0.17$ ksi x 3.33 x 1 x 0.80 = 0.45 ksi

Adjusted Perpendicular (⊦) Compression Stress Design

$F_{cp} = F_{cpo}C_{KF}C_MC_\lambda$ [AASHTO Art. 8.4.4.1-5]

$F_{cpo} = 0.625$ ksi [AASHTO Table 8.4.1.1.4-1]

$F_{cp} = 0.625$ ksi x 2.33 x 1 x 0.80 = 1.17 ksi

Nominal Moment Capacity

$M_r = \emptyset M_n$ [AASHTO Art. 8.6.1-1]

$M_n = F_bS$ [AASHTO Art. 8.6.2-1]

$S = \frac{bh^2}{6} = \frac{11.5\text{x } 11.5^2}{6} = 253.28\ in.^3$

$M_r = 0.85$ x 1.76 ksi x 253.28 in.^{3} = 378.91 kip*in. = 31.58 kip*ft > 28.20 kip*ft OK

Shear Capacity

$V_r = \emptyset V_n$ [AASHTO Art. 8.7-1]

$V_n = \frac{F_v bd}{1.5}$ [AASHTO Art. 8.7-2]

$V_r = 0.75\ x \frac{0.45x11.5\text{x}11.5}{1.5} =$ 29.76 kips > 21.10 kips OK

Compressive Capacity – Perpendicular to the Grain

$P_r = \emptyset P_n$ [AASHTO Art. 8.8.1-1]

$P_n = F_{cpo}A_bC_b$ [AASHTO Art. 8.8.3-1]

A_b = 11.5" x 6" = 69 in.^{2} (Assuming 6" bearing length)

$P_r = 0.90\ x\ 1.17\ ksi\ x\ 69\ in.^2\ x\ 1 =$ 72.66 kips > 21.10 kips OK

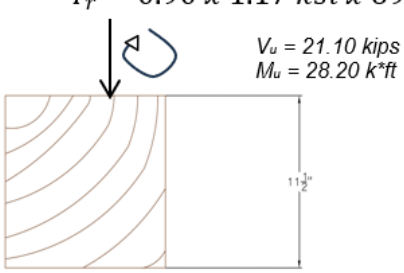

Figure 3 – 12 x 12 Nominal Timber plank

Span Length = 20'- 30'

2.) Steel Girder Calcs:

2.1) Steel Girder Demand:

Use upper bound of 30' for longitudinal analysis

C-C Girder Spacing = 4'-0"

Total Deck Width = 24'-0"

L_L Distribution Factor = S/8.3 = 4'/8.3 = 0.48 LL Lanes [AASHTO Table 4.6.2.2.2a-1]

$R_A\ from\ LL = \frac{0.5(4'-1')}{4} = 0.38\ Lanes,$ use 0.48 lanes for design

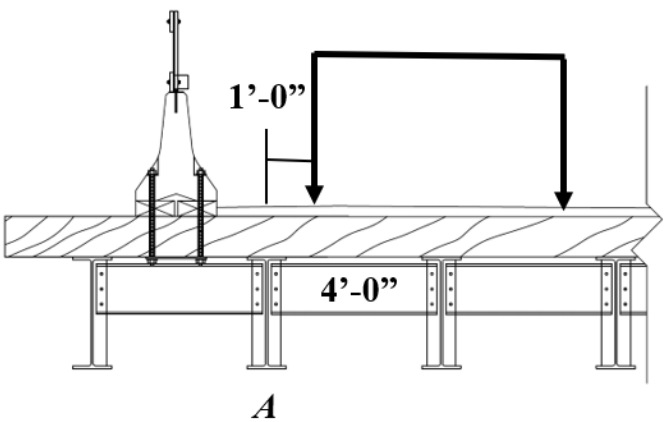

Figure 4 – Live Load Lane Rxn's

Assume an initial depth to span ratio of 0.045

30' x 12"/ft x 0.045 = 16.2" trial Superstructure depth

Demand Calcs:

Dead Loads (D_L): [AASHTO Table 3.5.1-1]

w_1 (Timber Deck) = 0.05 kcf x 4' x 1' = 0.2 klf

w_2 (HMA) = 0.14 kcf x (2/12)' x 4' = 0.093 klf

K_{rail} = 2 (barriers) x 0.145 kcf x 2.67 ft^2 x 1/6 (total girders) = 0.13 klf

Assumed Initial Steel Girder Self-Weight = 0.100 klf (conservative since W18x86)

D_C = 0.2 + 0.13 + 0.10 = 0.43 klf

D_W = 0.093 klf

Live Loads (CTBridge)

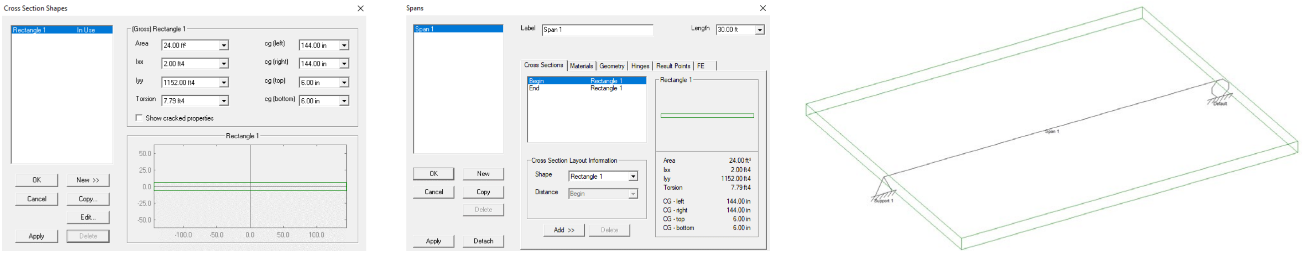

Figure 5 – CTBridge Input

Live Load - Controlling Unfactored Span Forces

LRFD Design Vehicle - Span 1

Dynamic Load Allowance (Included) = 1.3300

Location ft	# Lanes	MZ+ kip·ft	assoc VY kip	# Lanes	# Lanes	MZ- kip·ft	assoc VY kip	# Lanes
0.00	0.480	0.00	33.74	0.480	0.480	0.00	0.00	0.480
3.00	0.480	94.16	10.27	0.480	0.480	0.00	0.00	0.480
6.00	0.480	162.57	9.98	0.480	0.480	0.00	0.00	0.480
9.00	0.480	210.97	5.87	0.480	0.480	0.00	0.00	0.480
12.00	0.480	237.47	1.76	0.480	0.480	0.00	0.00	0.480
15.00	0.480	242.04	-4.49	0.480	0.480	0.00	0.00	0.480
18.00	0.480	237.47	-18.18	0.480	0.480	0.00	0.00	0.480
21.00	0.480	210.97	-22.29	0.480	0.480	0.00	0.00	0.480
24.00	0.480	162.57	-26.40	0.480	0.480	0.00	0.00	0.480
27.00	0.480	94.16	-31.16	0.480	0.480	0.00	0.00	0.480
30.00	0.480	0.00	-33.74	0.480	0.480	0.00	0.00	0.480

LRFD Permit Vehicle - Span 1

Dynamic Load Allowance (Included) = 1.2500

Location ft	# Lanes	MZ+ kip·ft	assoc VY kip	# Lanes	# Lanes	MZ- kip·ft	assoc VY kip	# Lanes
0.00	0.480	0.00	42.12	0.480	0.480	0.00	0.00	0.480
3.00	0.480	116.64	6.48	0.480	0.480	0.00	0.00	0.480
6.00	0.480	194.40	-0.00	0.480	0.480	0.00	0.00	0.480
9.00	0.480	233.28	-6.48	0.480	0.480	0.00	0.00	0.480
12.00	0.480	233.28	-12.96	0.480	0.480	0.00	0.00	0.480
15.00	0.480	243.00	-16.20	0.480	0.480	0.00	0.00	0.480
18.00	0.480	233.28	-19.44	0.480	0.480	0.00	0.00	0.480
21.00	0.480	233.28	-25.92	0.480	0.480	0.00	0.00	0.480
24.00	0.480	194.40	-32.40	0.480	0.480	0.00	0.00	0.480
27.00	0.480	116.64	-38.88	0.480	0.480	0.00	0.00	0.480
30.00	0.480	0.00	-42.12	0.480	0.480	0.00	0.00	0.480

Figure 6 – CTBridge Output

Moment Demand:

Str I (HL-93) Moment at Midspan (15'), M_u = 242 k*ft

Str II (P15) Moment at Midspan (15'), M_u = 243 k*ft

Shear Demand:

$V_{u\_str\ I}$ taken at X = 0, V_u = 33.74 kips

$V_{u\_str\ II}$ = 42.12 kips

Moment Demand:

$$M_{u_{str\,I}} = \frac{\Sigma\gamma_i w_i L^2}{8} = 1.25\left(0.43\,klf\ x\ \frac{30^2 ft^2}{8}\right) + 1.50\left(0.093\,klf\ x\ \frac{30^2 ft^2}{8}\right) + 1.75\,(242\,kft)$$

$$\boxed{M_{u_{str\,I}} = \ 499.66\,k - ft}$$ *Controls*

$$M_{u_{str\,II}} = \frac{\Sigma\gamma_i w_i L^2}{8} = 1.25\left(0.43\,klf\ x\ \frac{30^2 ft^2}{8}\right) + 1.50\left(0.093\,klf\ x\ \frac{30^2 ft^2}{8}\right) + 1.35\,(243\,kft)$$

$$M_{u_{str\,II}} = 404.21\,kft$$

Shear Demand:

$$V_{u_{str\,I}} = \Sigma\gamma_i w_i = 1.25(0.43\,klf\ x\ 30\,ft/2) + 1.50(0.093\,klf\ x\ 30\,ft/2) + 1.75\,(33.74\,kips)$$

$$\boxed{V_{u_{str\,I}} = 69.20\,kips}$$ *Controls*

$$V_{u_{str\,II}} = \Sigma\gamma_i w_i = 1.25(0.43\,klf\ x\ 30\,ft/2) + 1.50(0.093\,klf\ x\ 30\,ft/2) + 1.35\,(42.12\,kips)$$

$$V_{u_{str\,II}} = 67.02\,kips$$

2.2) *Steel Girder Capacity, AISC SCM 15th Edition and AASHTO BDS 8th as applicable:*

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
	in.^{3}	kip-ft	kip-ft	kip-ft	kip-ft	kips	kips	ft	ft	in.^{4}	kips	kips
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD
W30×90[f]	**283**	**706**	**1060**	**428**	**643**	**20.6**	**30.8**	**7.38**	**20.9**	**3610**	**249**	**374**
W24×103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404
W21×111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355
W27×94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395
W12×170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403
W18×119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373
W14×145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302
W24×94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375
W21×101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321
W27×84	**244**	**609**	**915**	**372**	**559**	**17.6**	**26.4**	**7.31**	**20.8**	**2850**	**246**	**368**
W12×152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358
W14×132	234	584	878	365	549	5.15	7.74	13.3	55.8	1530	190	284
W18×106	230	574	863	356	536	9.73	14.6	9.40	31.8	1910	221	331
W24×84	**224**	**559**	**840**	**342**	**515**	**16.2**	**24.2**	**6.89**	**20.3**	**2370**	**227**	**340**
W21×93	221	551	829	335	504	14.6	22.0	6.50	21.3	2070	251	376
W12×136	214	534	803	325	488	4.02	6.06	11.2	63.2	1240	212	318
W14×120	212	529	795	332	499	5.09	7.65	13.2	51.9	1380	171	257
W18×97	211	526	791	328	494	9.41	14.1	9.36	30.4	1750	199	299
W24×76	**200**	**499**	**750**	**307**	**462**	**15.1**	**22.6**	**6.78**	**19.5**	**2100**	**210**	**315**
W16×100	198	494	743	306	459	7.86	11.9	8.87	32.8	1490	199	298
W21×83	196	489	735	299	449	13.8	20.8	6.46	20.2	1830	220	331
W14×109	192	479	720	302	454	5.01	7.54	13.2	48.5	1240	150	225
W18×86	186	464	698	290	436	9.01	13.6	9.29	28.6	1530	177	265

Figure 7 – AISC SCM 15th (Table 3-2)

$L_p < L_b \leq L_r$ Beam is braced at L/2 = 15 ft

Moment Capacity:

$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(L_b - L_p)] \leq \emptyset_b M_{px}$ [AISC SCM 3-4a (LRFD)]

C_b = 1.0 (Conservative)

$\emptyset_b = 0.90$

L_b = 15 ft

L_p = 9.29 ft

$\emptyset_b M_{px} = \emptyset_b F_Y Z_x$ = 0.90 x 50 ksi x 186 in.^{3} = 8,370 k*in = 698 k*ft (same as in table)

$\emptyset_b BF = 13.6\ kips$

$\emptyset_b M_n = 1.0[698 - 13.6(15 - 9.29)] = 620\ k*ft > 499.66\ k*ft$ OK

Alternative Method: Use Table 3-10 for L_b = 15ft and W18x86, $\emptyset_b M_n$ ~ 620 k*ft

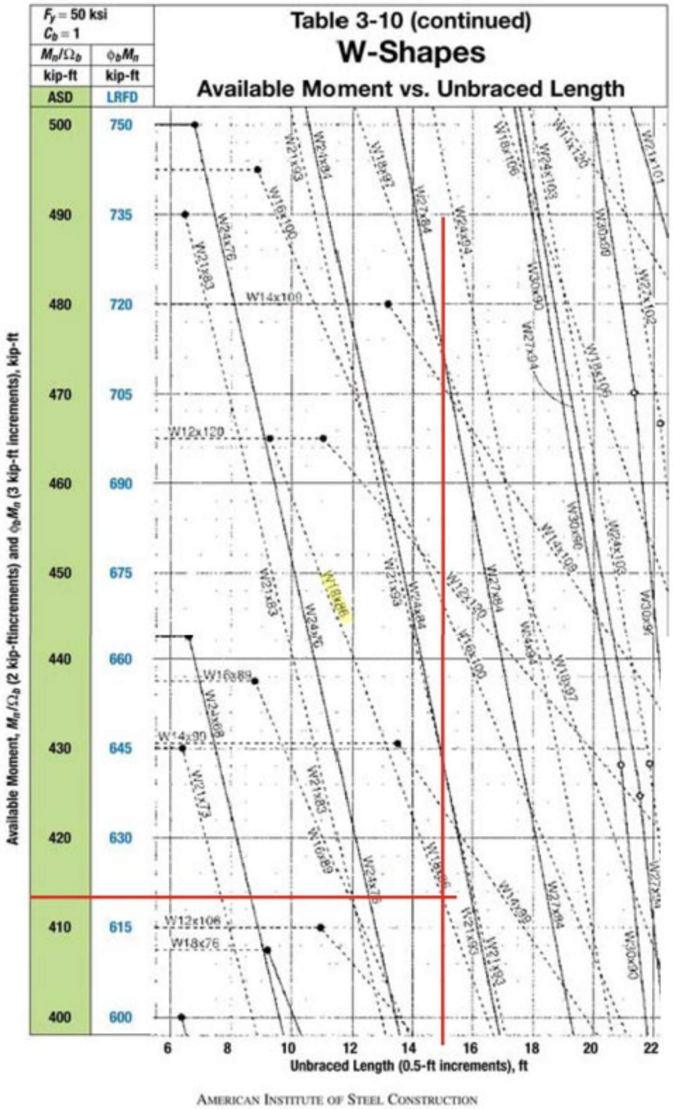

Figure 8 – AISC SCM 15th (Table 3-10)

Shear Capacity:

$\emptyset_v V_{vx} =$ 265 kips [AISC SCM Table 3-2 (LRFD)]

$\emptyset_v V_{vx} \geq V_u$ (69.2 kips) OK

Check:

$\emptyset_v V_n = 0.58 F_{yw} D t_w$ [AASHTO Art. 6.10.9.2-2]

D = 16.86 in. (Web depth)

t_w = 0.48

$\emptyset_v$ =1.0

$\emptyset_v V_n =$ 1.0 x 0.58 x 50 ksi x 16.86” x 0.48” = 234.7 kips > V_u (69.2 kips) OK

3.) <u>Bearing Stiffener Calcs:</u>

3.1) Bearing Stiffener Demand:

$P_{HL-93\_Abut} = 55.12\ kips$ x 1.33 (*IM*) = 73.31 kips

$$P_{u_{str\ I}} = \Sigma\gamma_i w_i = 1.25(0.43\ klf\ x\ 30\ ft/2) + 1.50(0.093\ klf\ x\ 30\ ft/2) + 1.75\ (\ 73.31\ kips)$$

$\boxed{P_{u_{str\ I}} = 138.45\ kips}$ *Controls*

$P_{P15\_Abut} = 70.20\ kips$ x 1.25 (*IM*) = 87.75 kips

$$P_{u_{str\ II}} = \Sigma\gamma_i w_i = 1.25(0.43\ klf\ x\ 30\ ft/2) + 1.50(0.093\ klf\ x\ 30\ ft/2) + 1.35\ (\ 87.75\ kips)$$

$P_{u_{str\ II}} = 128.62\ kips$

3.2) Bearing Stiffener - Capacity:

(Helps to prevent Local Buckling)

Projecting Width, b_t

$b_t \le 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$ [AASHTO Art. 6.10.11.2.2-1]

Try t_{pl} = 3/8" = 0.375":

0.48 x 3/8" x $\sqrt{\frac{29{,}000}{50}}$ =4.33"

Try 4" Plate:

4" ≤ 4.33" OK

$(R_{sb})r = \emptyset_b (R_{sb})n$ [AASHTO Art. 6.10.11.2.3-1]

$\emptyset_b = 1.0$ [AASHTO Art. 6.5.4.2]

$(R_{sb})n = 1.4 A_{pn} F_{ys}$ [AASHTO Art. 6.10.11.2.3-2]

b_{brg} = 4 - 1" (web-flange weld) = 3"

$\emptyset_b (R_{sb})n$ = 1.0 x 1.4 x 2 plates per girder x 3" x 3/8" x 50 ksi = 157.5 kips

$\emptyset_b (R_{sb})n > 138.45\ kips$ OK

Live Load - Controlling Unfactored Abutment Reactions

LRFD Design Vehicle

No Dynamic Load Allowance - Single Lane

Abutment	# Lanes	MZ+ kip-ft	assoc VY kip	# Lanes	MZ- kip-ft	assoc VY kip
First Abutment	1.000	0.00	55.12	1.000	0.00	0.00
Last Abutment	1.000	0.00	-55.12	1.000	0.00	0.00

Abutment	# Lanes	VY+ kip	assoc MZ kip-ft	assoc TX kip-ft	# Lanes	VY- kip	assoc MZ kip-ft	assoc TX kip-ft
First Abutment	1.000	55.12	0.00	-0.00	1.000	-0.00	0.00	-0.00
Last Abutment	1.000	0.00	0.00	0.00	1.000	-55.12	0.00	0.00

Abutment	# Lanes	TX+ kip-ft	# Lanes	AX- kip	# Lanes	TX- kip-ft
First Abutment	1.000	0.00	1.000	0.00	1.000	0.00
Last Abutment	1.000	0.00	1.000	0.00	1.000	0.00

LRFD Permit Vehicle

No Dynamic Load Allowance - Single Lane

Abutment	# Lanes	MZ+ kip-ft	assoc VY kip	# Lanes	MZ- kip-ft	assoc VY kip
First Abutment	1.000	0.00	70.20	1.000	0.00	0.00
Last Abutment	1.000	0.00	-70.20	1.000	0.00	0.00

Abutment	# Lanes	VY+ kip	assoc MZ kip-ft	assoc TX kip-ft	# Lanes	VY- kip	assoc MZ kip-ft	assoc TX kip-ft
First Abutment	1.000	70.20	0.00	-0.00	1.000	-0.00	0.00	-0.00
Last Abutment	1.000	0.00	0.00	0.00	1.000	-70.20	0.00	0.00

Abutment	# Lanes	TX+ kip-ft	# Lanes	AX- kip	# Lanes	TX- kip-ft
First Abutment	1.000	0.00	1.000	0.00	1.000	0.00
Last Abutment	1.000	0.00	1.000	0.00	1.000	0.00

Figure 9 – CTBridge Load Abutment Reactions

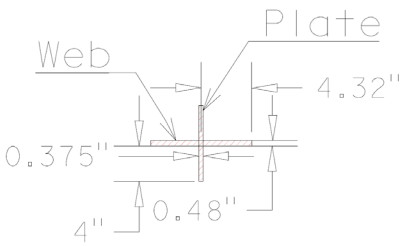

Figure 10 – Bearing Stiffener Effective Dimensions

Check Axial Resistance of Stiffener:

$P_r = \emptyset_c P_n$ [AASHTO Art. 6.9.2.1-1]

$\emptyset_c = 0.95$ [AASHTO Art. 6.5.4.2]

Effective Plate Width = 4” + 4” + 0.48” (t_w) = 8.48”

Effective Web Strip = 2 x 9t_w – t_p = 2 x 9(0.48”) – 3/8” = 8.27” [AASHTO Art. 6.10.11.2.4b]

$A_g = (0.375" \text{ x } 8.48") + (0.48" \text{ x } 8.27") = 7.15 \text{ in.}^2$

$I_s = ([0.375" \text{ x } (8.48")^3] + [0.48" \text{ x } (8.27")^3])\dfrac{1}{12} = 41.68 \text{ in.}^4$

$r_s = \sqrt{\dfrac{I_s}{A_g}} = 2.41"$

$K \text{ x } l = 0.75 \text{ x } 16.86" (D_w) = 12.65 \text{ in.}$

$\dfrac{k\ x\ l}{r_s} = (0.75 \text{ x } 16.86")/2.41" = 5.25 \leq 120$ OK [AASHTO Art. 6.9.3]

Check Critical Buckling Resistance:

$P_e = \dfrac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_g$ [AASHTO Art. 6.9.4.1.2-1]

$P_e = \dfrac{\pi^2\ x\ 29{,}000\ ksi}{(5.25)^2}\ x\ 7.15\ in.^2 = 74{,}248 \text{ kips}$

$P_o = F_y A_g$ [AASHTO Art. 6.9.4.1.1]

$P_o = 50\ ksi\ x\ 7.15\ in.^2 = 357.5 \text{ kips}$

$\dfrac{P_e}{P_o} = \dfrac{74{,}248}{357.5} = 207.7 > 0.44$

$P_n = 0.658^{\frac{P_o}{P_e}} P_o$ [AASHTO Art. 6.9.4.1.1-1]

$P_n = 356.8 \text{ kips}$

$P_r = \emptyset_c P_n = 0.95\ x\ 356.8\ kips = 338.9 \text{ kips} > 138.45 \text{ kips}$ OK

PL 3/8 x 4 is sufficient for axial loading

4.) Welded Connection - Capacity Calcs

Assume t_{Weld} = 0.25" for initial calculation.

$R_r = 0.6\emptyset_{e1}F_{EXX}$ [AASHTO Art. 6.13.3.2.2b-1]

$\emptyset_{e1} = 0.80$ [AASHTO Art. 6.5.4.2]

R_r = 0.60 x 0.80 x 70 ksi = 33.6 ksi

L_{eff} = 4 x (16.86" – 2") = 59.44" [AASHTO Art. 6.13.3.6]

$Throat_{eff} = \frac{t_{Weld}}{\sqrt{2}}$ = 0.18"

$Area_{eff} = L_{eff}$ x $Throat_{eff}$ = 10.51 in^2 [AASHTO Art. 6.13.3.3]

Weld Capacity = R_r x $Area_{eff}$ = 353.06 kips > 138.45 kips OK

For material 0.25" or more in thickness, the maximum size of the fillet weld is 0.0625" less than the materials thickness. [AASHTO Art. 6.13.3.4]

0.375" – 0.0625" = 0.3125" > 0.25" OK

¼" Weld is sufficient for factored Abutment reaction.

5.) Bolted Connection - Capacity Calcs

Assume ASTM F3125 Grade A325 high strength ¾"Ø, threads excluded from the shear plane.

Single Shear

$R_n = 0.56A_bF_{ub}N_s = 0.56\ x\ \pi\ x\ \left(\frac{0.75"}{2}\right)^2 x\ 120\ ksi\ x\ 1 = 29.69\ kips$ [AASHTO Art. 6.13.2.7-1]

Use edge distance of 1.5"

$L_c = 1.5" + 1"(\text{clr}) - (0.75"/2) = 2.13"$

Bearing

$R_n = 1.2L_ctF_u = 1.2\ x\ 2.13\ \text{x}\ \frac{3}{8}"\ x\ 58\ ksi\ =\ 55.46\ kips$ take as 29.69 kips [AASHTO Art. 6.13.2.9-2]

$\emptyset R_n = 0.8\ x\ 29.69\ kips = 23.75\ kips$ per bolt, for L connection to web - double shear

$$Number\ of\ bolts = \frac{138.45\ kips}{2\ x\ 23.75\ kips} =\ 2.91\ use\ 3\ bolts$$

6.) Steel Diaphragm - Demand Calcs

Live load reaction taken at midspan (CTBridge)

$P_{u_{str\ I}} = \Sigma\gamma_i w_i = 1.25(0.43\ klf\ x\ 30\ ft/2) + 1.50(0.093\ klf\ x\ 30\ ft/2) + 1.75\ (13.2\ kips)$

$P_{u_{str\ I}} = \boxed{33.30\ kips}$ *Controls*

$P_{u_{str\ II}} = \Sigma\gamma_i w_i = 1.25(0.43\ klf\ x\ 30\ ft/2) + 1.50(0.093\ klf\ x\ 30\ ft/2) + 1.35\ (16.2\ kips)$

$P_{u_{str\ II}} = 32.03\ kips$

7.) Steel Diaphragm - Capacity Calcs

Assume trial Channel height, H = ½H_{girder} = ½ x 18.4" = 9.2" [AASHTO Art. 6.7.4.2]

Try C10x15.3, H = 10":

Check Elastic Torsional Buckling Resistance:

$A_g = 4.48\ in.^2$ [AISC SCM Table 1-5]

$P_e = (\frac{P_{ey}+P_{ez}}{2H})[1-\sqrt{1-\frac{4P_{ey}P_{ez}H}{(P_{ey}+P_{ez})^2}}]$ [AASHTO Art. 6.9.4.1.1.3.2]

r_y = 0.711 in. [AISC SCM Table 1-5]

$P_{ey} = \frac{\pi^2 E}{\left(\frac{Kyly}{ry}\right)^2} A_g = \frac{\pi^2 x\ 29{,}000\ ksi\ x\ 4.48\ in.^2}{\left(1\ x\ 4' x \frac{12}{0.711}\right)^2} = 281\ kips$ [AASHTO Art. 6.9.4.1.3-4]

J = 0.209 in^4, r_o = 4.19 in., C_w = 45.5 in.^{6}, H = 0.884 [AISC SCM Table 1-5]

$$P_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z l_z)^2} + GJ\right)}{r_o^2} = \frac{\left[\left(\frac{\pi^2 x\ 29{,}000\ ksi\ x\ 45.5}{(1\ x\ 4' x\ 12)^2}\right) + (0.385\ x\ 29{,}000\ ksi\ x\ 0.209\ in^4)\right]}{4.19^2 in} = 455\ kips$$

[AASHTO Art. 6.9.4.1.1]

$$P_e = \left(\frac{281\ kips + 455\ kips}{2(0.884)}\right)\left[1 - \sqrt{1 - \frac{4\ x\ 281\ kips\ x\ 455\ kips\ x\ 0.884}{(281\ kips + 455\ kips)^2}}\right] = 68.8\ kips$$

$P_o = 50\ ksi\ x\ 4.48\ in.^2 = 224$ kips

$\frac{P_e}{P_o} = \frac{68.8\ kips}{224\ kips} = 0.31 < 0.44$ c

[AASHTO Art. 6.9.4.1.1-2]

$\emptyset_c P_n = 0.95\ x\ 0.877 P_e = 0.95\ x\ 0.877\ x\ 68.8\ kips = 57.3\ kips > 33.30\ kips$

C10x15.3 is sufficient for axial/torsional loading.

II. Design Verification for Steel Rolled Wide Flange Girders with Timber Decking

Span Length = 30’- 40’

Wide Flange Girder (WFG), Girder spacing, S = 4’-0”

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Spreadsheet Calculations for L = 40’

Temp Bridge, Single Span

Girder = W24x104

Span L =	40	ft	w1 =	0.2	klf
d/s =	0.045		w2 =	0.093	klf
C-C =	4	ft			
LL DF =	0.48	lanes	Krail =	0.13	klf
SS d =	21.6	in	SW_{girder} =	0.104	klf

Girders

D_C =	0.434	klf
D_W =	0.093	klf

CTBridge

Str I Moment Midspan =	348.72	k*ft
Str II Moment Midspan =	388.80	k*ft
Str I Shear @ Abut =	38.78	kips
Str II Shear @ Abut =	48.6	kips

Factored Moment

$M_{u\_Str\ I}$ =	747	k*ft	Controls
$M_{u\_Str\ II}$ =	661	k*ft	Does Not Control

Factored Shear

$V_{u\_Str\ I}$ =	82	kips	Controls
$V_{u\_Str\ II}$ =	79	kips	Does Not Control

Table 3-2 AISC 15th Brace pt = L/2

Lp = 10.3 ft

Lr = 29.2 ft

For 40 ft total length braced at midpoint, Lb = 20 ft

Lp < Lb < Lr

Moment Capacity

$$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(L_b - L_p)] \leq \emptyset_b M_{px}$$

AISC 15th 3-4a (LRFD)

C_b =	1			F_y =	50	ksi	
$\emptyset_b$ =	0.9			Z_x =	289	in^3	
$\emptyset_b M_{px}$ =	1,084	k*ft		$\emptyset_b BF$ =	21.3	kips	
$\emptyset_b M_n$ =	877	k*ft	<	$\emptyset_b M_{px}$ 1,084 k*ft	OK		
			>	$M_{u\_Str\ I}$ 747 k*ft	OK		

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W33×130	**467**	**1170**	**1750**	**709**	**1070**	**29.3**	**43.1**	**8.44**	**24.2**	**6710**	**384**	**576**
W27×146	464	1160	1740	723	1090	19.9	29.5	11.3	33.3	5660	332	497
W18×192	442	1100	1660	664	998	10.6	16.1	9.85	51.0	3870	392	588
W30×132	437	1090	1640	664	998	26.9	40.5	7.95	23.8	5770	373	559
W14×233	436	1090	1640	655	984	5.40	8.15	14.5	95.0	3010	342	514
W21×166	432	1080	1620	664	998	14.2	21.2	10.6	39.9	4280	338	506
W12×252[h]	428	1070	1610	617	927	4.43	6.68	11.8	114	2720	431	647
W24×146	418	1040	1570	648	974	17.0	25.8	10.6	33.7	4580	321	482
W33×118[c]	**415**	**1040**	**1560**	**627**	**942**	**27.2**	**40.6**	**8.19**	**23.4**	**5900**	**325**	**489**
W30×124	408	1020	1530	620	932	26.1	39.0	7.88	23.2	5360	353	530
W18×175	398	993	1490	601	903	10.6	15.8	9.75	46.9	3450	356	534
W27×129	395	986	1480	603	906	23.4	35.0	7.81	24.2	4760	337	505
W14×211	390	973	1460	590	887	5.30	7.94	14.4	86.6	2660	308	462
W12×230[h]	386	963	1450	561	843	4.31	6.51	11.7	105	2420	390	584
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
W18×158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414
W12×210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520
W30×108	**346**	**863**	**1300**	**522**	**785**	**23.5**	**35.5**	**7.59**	**22.1**	**4470**	**325**	**487**
W27×114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425
W24×117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	267	401
W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.4**	**7.42**	**21.3**	**3990**	**309**	**463**
W12×190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458
W21×122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391
W27×102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419
W18×130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388
W24×104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362

Shear Capacity

$\emptyset_v V_{vx=}$ 362 kips

V_u = 82 kips

$\emptyset_v V_{vx} > V_u$

$\emptyset_v$ = 1

$\emptyset_v V_n = 0.6 F_y A_w C_{v1} =$ 362 *kips* ≥ 82 kips OK

AISC 15th G2-2

$\frac{h}{t_w} \leq 2.24 \sqrt{E/F_y}$

45.2 ≤ 53.9 kips OK

C_v = 1

AISC 15th Table 3-2 (LRFD)

E/Fy = 580

Bearing Stiffeners

Abutment Axial Rxn, IM included (CTBridge)

$P_{y\_HL\text{-}93}$ = 85 kips

P_{y_P15} = 101 kips

Factored Axial Demand

$P_{u\_Str\ I}$ = 162 kips Controls

$P_{u\_Str\ II}$ = 150 kips Does Not Control

Bearing Capacity

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

AASHTO BDS 8th 6.10.11.2.2-1

t_p = 0.5 in (1/2")

E = 29,000 ksi

F_{ys} = 50 ksi

~ 5.78 in $(0.48 t_p \sqrt{E/F_{ys}})$

Try 4" plate

$\varnothing_b$ = 1

$R_{sb\,n} = 1.4 A_{pn} F_{ys}$

b_{brg} = 3 in

$\varnothing_b R_{sb\,n}$ = 210 kips > $P_{u\_Str\ I}$ 162 kips OK

Axial Resistance

$$P_r = \varnothing_c P_n$$

t_w = 0.5 in Web thickness

$\varnothing_c$ = 0.95

Effective Plate Width = 4" + 4" + 0.5" (t_w) = 8.5 in

Effective Strip Width = 2 x $9t_w - t_p$ = 8.5 in

A_g = 8.5 in^2 $D_w = l$ = 22.6 in depth of web

I_s = 51.2 in^4 k = 0.75

r_s = 2.45 in

$k \times l$ = 16.95 in

$\frac{k\,x\,l}{r_s}$ = 6.91 ≤ 120 OK AASHTO BDS 8th 6.9.3

$$P_e = \frac{\pi^2 E}{\left(\frac{kl}{r_s}\right)^2} A_g$$

AASHTO BDS 8th 6.9.4.1.2-1

P_e = 50,984 kips

$P_o = F_y A_g$

P_o = 425 kips

$\frac{P_e}{P_o} =$ 120 ≥ 0.44 AASHTO BDS 8th 6.9.4.1.1-1

$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$ 424 kips

$P_r = \emptyset_c P_n =$ 402 kips > 162 kips OK

Welded Connection

Assume t_{weld} = 0.25 in

$R_r = 0.6\emptyset_{e1} F_{EXX}$ F_{EXX} = 70 ksi

$\emptyset_{e1}$ = 0.8

R_r = 33.6 ksi

L_{eff} = 4 x (22.6 in - 2 in) = 82.4 in

$T_{eff} = \frac{t_{weld}}{\sqrt{2}} =$ 0.18 in Throat

$A_{eff} = L_{eff} x\, T_{eff} =$ 14.6 in^2

$R_r\, x\, A_{eff} =$ 489.43 kips > 162 kips OK

Bolted Connection

Assume ASTM F3125 Grade A325 High Stength 3/4" ∅, threads excluded from the shear plane

F_{ub} = 120 ksi

Single Shear A_b = 0.44 in^2

$R_n = 0.56 A_b F_{ub} N_s =$ 29.69 kips

$L_c = 1.5\, in + 1"\,(clr) - (0.75\, in/2) =$ 2.13 in

Bearing F_u = 58 ksi t = 0.5 in

$R_n = 1.2 L_c t F_u =$ 73.95 kips

take as the lesser 29.69 kips

$\emptyset$ = 0.8

$\emptyset R_n =$ 23.75 kips per bolt

Abutment Rxn = 162 kips

$Number\ of\ Bolts = \frac{63.76}{2x\ 23.75} =$ 3.41 use 4 bolts

Diaphragm

Assume trial Channel size = 1/2H_{girder} = 12.05 in

Try C15x33.9

Torsional Buckling Resistance

A_g =	10 in^2	k_y =	1	AISC SCM Table 1-5
r_y =	0.901 in	l_y =	48 in	(4 ft girder spacing)

$$P_{ey} = \frac{\pi^2 E}{\left(\frac{k_y l_y}{r_y}\right)^2} A_g =$$ 1,008 kips — AASHTO BDS 8th 6.9.4.1.3-4

AISC SCM Table 1-5

J =	1.01 in^4	r_o =	5.94 in	C_w = 358 in^6
				H = 0.92

$$P_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z l_z)^{\wedge}2}\right) + GJ}{r_o^2} =$$ 1,580 kips — AASHTO BDS 8th 6.9.4.1.1

AASHTO BDS 8th 6.9.4.1.1.3.2

$$P_e = \left(\frac{P_{ey} + P_{ez}}{2H}\right)[1 - \sqrt{1 - \frac{4 P_{ey} P_{ez} H}{(P_{ey} + P_{ez})^{\wedge}2}} =$$ 910 kips

$$P_o = F_y A_g =$$ 500 kips

$$\frac{P_e}{P_o} =$$ 1.82 > 0.44

Therefore

$$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$$ 397 kips

$\varnothing_c$ = 0.95

$\varnothing_c P_n =$ 377 kips > 38 kips OK

Factored Axial load at Midspan			HL-93	P15
$P_{u\_Str\ I}$ =	38 kips	Controls	14.1 kips	16.2 kips
$P_{u\_Str\ II}$ =	36 kips	Does Not Control		

III. Design Verification for Steel Rolled Wide Flange Girders with Timber Decking

Span Length = 40’- 50’

Wide Flange Girder (WFG), Girder spacing, S = 4’-0”

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Spreadsheet Calculations for L = 50’

Temp Bridge, Single Span

Girder =	W33x130				
Span L =	50	ft	w1 =	0.2	klf
d/s =	0.045		w2 =	0.093	klf
C-C =	4	ft			
LL DF =	0.48	lanes	Krail =	0.13	klf
SS d =	27	in	SW_{girder} =	0.13	klf
Girders			D_C =	0.46	klf
CTBridge			D_W =	0.093	klf

Str I Moment Midspan =	491.81	k*ft
Str II Moment Midspan =	631.80	k*ft
Str I Shear @ Abut =	43.28	kips
Str II Shear @ Abut =	58.97	kips

Factored Moment

$M_{u\_Str\ I}$ =	1084	k*ft	Controls
$M_{u\_Str\ II}$ =	1076	k*ft	Does Not Control

Factored Shear

$V_{u\_Str\ I}$ =	94	kips	Does Not Control
$V_{u\_Str\ II}$ =	97	kips	Controls

Table 3-2 AISC 15th Brace pt = L/3

Lp = 8.44 ft

Lr = 24.2 ft

For 50 ft total length braced at L/3, Lb = 16.7 ft

Lp < Lb < Lr

Moment Capacity

$$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(L_b - L_p)] \leq \emptyset_b M_{px}$$

AISC 15th 3-4a (LRFD)

C_b =	1			F_y =	50	ksi	
$\emptyset_b$ =	0.9			Z_x =	467	in^3	
$\emptyset_b M_{px}$ =	1,751	k*ft		$\emptyset_b BF$ =	43.1	kips	
$\emptyset_b M_n$ =	1,397	k*ft	<	$\emptyset_b M_{px}$ 1,751 k*ft	OK		
			>	$M_{u\_Str\ I}$ 1084 k*ft	OK		

F_y = 50 ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W33×130	**467**	**1170**	**1750**	**709**	**1070**	**29.3**	**43.1**	**8.44**	**24.2**	**6710**	**384**	**576**
W27×146	464	1160	1740	723	1090	19.9	29.5	11.3	33.3	5660	332	497
W18×192	442	1100	1660	664	998	10.6	16.1	9.85	51.0	3870	392	588
W30×132	437	1090	1640	664	998	26.9	40.5	7.95	23.8	5770	373	559
W14×233	436	1090	1640	655	984	5.40	8.15	14.5	95.0	3010	342	514
W21×166	432	1080	1620	664	998	14.2	21.2	10.6	39.9	4280	338	506
W12×252[h]	428	1070	1610	617	927	4.43	6.68	11.8	114	2720	431	647
W24×146	418	1040	1570	648	974	17.0	25.8	10.6	33.7	4580	321	482
W33×118[v]	**415**	**1040**	**1560**	**627**	**942**	**27.2**	**40.6**	**8.19**	**23.4**	**5900**	**325**	**489**
W30×124	408	1020	1530	620	932	26.1	39.0	7.88	23.2	5360	353	530
W18×175	398	993	1490	601	903	10.6	15.8	9.75	46.9	3450	356	534
W27×129	395	986	1480	603	906	23.4	35.0	7.81	24.2	4760	337	505
W14×211	390	973	1460	590	887	5.30	7.94	14.4	86.6	2660	308	462
W12×230[h]	386	963	1450	561	843	4.31	6.51	11.7	105	2420	390	584
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
W18×158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414
W12×210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520
W30×108	**346**	**863**	**1300**	**522**	**785**	**23.5**	**35.5**	**7.59**	**22.1**	**4470**	**325**	**487**
W27×114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425
W24×117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	267	401
W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.4**	**7.42**	**21.3**	**3990**	**309**	**463**
W12×190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458
W21×122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391
W27×102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419
W18×130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388
W24×104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362

Shear Capacity

$\emptyset_v V_{vx} =$ 576 kips — AISC 15th Table 3-2 (LRFD)

V_u = 97 kips

$\emptyset_v V_{vx} > V_u$

$\emptyset_v$ = 1

$\emptyset_v V_n = 0.58 F_{yw} D t_w =$ 557 *kips* ≥ 97 kips OK

AASHTO BDS 8th 6.10.9.2-2

Bearing Stiffeners

Abutment Axial Rxn, IM included (CTBridge)

$P_{y\_HL\text{-}93}$ = 95 kips
P_{y_P15} = 123 kips

Factored Axial Demand

$P_{u\_Str\ I}$ = 185 kips Controls
$P_{u\_Str\ II}$ = 184 kips Does Not Control

Bearing Capacity

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

AASHTO BDS 8th 6.10.11.2.2-1

t_p = 0.5 in (1/2")
E = 29,000 ksi
F_{ys} = 50 ksi
~ 5.78 in $(0.48 t_p \sqrt{E/F_{ys}})$
Try 4" plate
$\emptyset_b$ = 1
$R_{sb\,n} = 1.4 A_{pn} F_{ys}$
b_{brg} = 3 in
$\emptyset_b R_{sb\,n}$ = 210 kips > $P_{u\_Str\ I}$ OK
185 kips

Axial Resistance

$$P_r = \emptyset_c P_n$$

t_w = 0.58 in Web thickness
$\emptyset_c$ = 0.95
Effective Plate Width = 4" + 4" + 0.58" (t_w) = 8.58 in
Effective Strip Width = 2 x $9t_w$ - t_p = 9.94 in
A_g = 10.055 in^2
D_w = l = 22.6 in depth of web
I_s = 73.8 in^4
k = 0.75
r_s = 2.71 in
$k \times l$ = 16.95 in

$$\frac{k\ x\ l}{r_s} = 6.26 \leq 120$$ OK

AASHTO BDS 8th 6.9.3

$$P_e = \frac{\pi^2 E}{\left(\frac{kl}{r_s}\right)^2} A_g$$

AASHTO BDS 8th 6.9.4.1.2-1

P_e = 73,508 kips

$P_o = F_y A_g$

P_o = 502.76 kips

$\frac{P_e}{P_o} =$ 146 ≥ 0.44 AASHTO BDS 8th 6.9.4.1.1-1

$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$ 501 kips

$P_r = \emptyset_c P_n =$ 476 kips > 185 kips OK

Welded Connection

Assume t_{weld} = 0.25 in

$R_r = 0.6\emptyset_{e1} F_{EXX}$ F_{EXX} = 70 ksi

$\emptyset_{e1}$ = 0.8

R_r = 33.6 ksi

L_{eff} = 4 x (31.4 in - 2 in) = 117.56 in

$T_{eff} = \frac{t_{weld}}{\sqrt{2}} =$ 0.18 in Throat

$A_{eff} = L_{eff} x\, T_{eff} =$ 20.8 in^2

$R_r\, x\, A_{eff} =$ 698.27 kips > 185 kips OK

Bolted Connection

Assume ASTM F3125 Grade A325 High Stength 3/4" ∅, threads excluded from the shear plane

F_{ub} = 120 ksi

Single Shear A_b = 0.44 in^2

$R_n = 0.56 A_b F_{ub} N_s =$ 29.69 kips

$L_c = 1.5\, in + 1"\,(clr) - (0.75\, in/2) =$ 2.13 in

Bearing F_u = 58 ksi t = 0.5 in

$R_n = 1.2 L_c t F_u =$ 73.95 kips

take as the lesser 29.69 kips

$\emptyset$ = 0.8

$\emptyset R_n =$ 23.75 kips per bolt

Abutment Rxn = 185 kips

$Number\ of\ Bolts = \frac{185}{2x\ 23.75} =$ 3.89 use 4 bolts

Diaphragm

Assume trial Channel size = 1/2H_{girder} = 16.55 in

Try MC18x42.7

Torsional Buckling Resistance

A_g =	12.6 in^2	k_y =	1	AISC SCM Table 1-5
r_y =	1.07 in	l_y =	48 in	(4 ft girder spacing)

$$P_{ey} = \frac{\pi^2 E}{\left(\frac{k_y l_y}{r_y}\right)^2} A_g = \quad 1{,}792 \text{ kips}$$
AASHTO BDS 8th 6.9.4.1.3-4

AISC SCM Table 1-5

J =	1.23 in^4	r_o =	6.97 in	C_w =	852 in^6
				H =	0.93

$$P_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z l_z)^{\wedge}2}\right) + GJ}{r_o^2} = \quad 2{,}461 \text{ kips}$$
AASHTO BDS 8th 6.9.4.1.1

AASHTO BDS 8th 6.9.4.1.1.3.2

$$P_e = \left(\frac{P_{ey} + P_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4 P_{ey} P_{ez} H}{(P_{ey} + P_{ez})^{\wedge}2}} = \quad 1589 \text{ kips}\right.$$

$$P_o = F_y A_g = \quad 630 \text{ kips}$$

$$\frac{P_e}{P_o} = \quad 2.52 \quad > \quad 0.44$$

Therefore

$$P_n = 0.658^{\frac{P_o}{P_e}} P_o = \quad 534 \text{ kips}$$

$$\varnothing_c = \quad 0.95$$

$$\varnothing_c P_n = \quad 507 \text{ kips} \quad > \quad 65 \text{ kips}$$
OK

See below for demand

Controlling Factored Axial load at L/3			HL-93	P15
$P_{u\_Str\ I}$ =	65 kips	Controls	26.9 kips	33.7 kips
$P_{u\_Str\ II}$ =	63 kips	Does Not Control		

IV. Design Verification for Steel Rolled Wide Flange Girders with Timber Decking

Span Length = 50’- 60’

Wide Flange Girder (WFG), Girder spacing, S = 4’-0”

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Spreadsheet Calculations for L = 60’

Temp Bridge, Single Span

Girder =	W36x160				
Span L =	60 ft	w1 =	0.2 klf	t_f =	1.02 in
d/s =	0.045	w2 =	0.093 klf	d =	36 in
C-C =	4 ft				
LL DF =	0.48 lanes	Krail =	0.13 klf		
SS d =	32.4 in	SW_{girder} =	0.16 klf		

Girders

CTBridge		D_C =	0.49 klf
		D_W =	0.093 klf
Str I Moment Midspan =	648.96 k*ft		
Str II Moment Midspan =	874.80 k*ft		
Str I Shear @ Abut =	46.19 kips		
Str II Shear @ Abut =	66.96 kips		

Factored Moment

$M_{u\_Str\ I}$ =	1474 k*ft	Does Not Control
$M_{u\_Str\ II}$ =	1519 k*ft	Controls

Factored Shear

$V_{u\_Str\ I}$ =	103 kips	Does Not Control
$V_{u\_Str\ II}$ =	113 kips	Controls

Table 3-2 AISC 15th Brace pt = L/3

Lp = 8.83 ft

Lr = 25.8 ft

For 60 ft total length braced at L/3, Lb = 20.0 ft

Lp < Lb < Lr

Moment Capacity

$$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(L_b - L_p)] \le \emptyset_b M_{px}$$

AISC 15th 3-4a (LRFD)

C_b =	1		F_y =	50	ksi
$\emptyset_b$ =	0.9		Z_x =	624	in^3
$\emptyset_b M_{px}$ =	2,340 k*ft		$\emptyset_b BF$ =	54.2	kips
$\emptyset_b M_n$ =	1,735 k*ft	<	$\emptyset_b M_{px}$ 2,340 k*ft	OK	
		>	$M_{u\_Str\ I}$ 1519 k*ft	OK	

3-22 DESIGN OF FLEXURAL MEMBERS

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×167	693	1730	2600	1050	1580	41.7	62.5	8.48	24.8	11600	502	753
W18×283[h]	676	1690	2540	987	1480	11.1	16.7	10.3	73.6	6170	613	920
W30×191	675	1680	2530	1050	1580	25.6	38.6	12.2	36.8	9200	436	654
W24×229	675	1680	2530	1030	1540	19.0	28.9	11.0	45.2	7650	499	749
W14×342[h]	672	1680	2520	975	1460	5.73	8.62	15.0	138	4900	539	809
W21×248	671	1670	2520	1010	1510	14.3	21.9	10.9	57.1	6830	521	782
W36×170	668	1670	2510	1010	1530	37.8	56.1	8.94	26.4	10500	492	738
W27×194	631	1570	2370	976	1470	22.3	33.8	11.6	38.2	7860	422	632
W33×169	629	1570	2360	959	1440	34.2	51.5	8.83	26.7	9290	453	679
W36×160	624	1560	2340	947	1420	36.1	54.2	8.83	25.8	9760	468	702

Shear Capacity

$\emptyset_v V_{vx=}$ 702 kips — AISC 15th Table 3-2 (LRFD)

V_u = 113 kips

$\emptyset_v V_{vx} > V_u$

$\emptyset_v$ = 1

$\emptyset_v V_n = 0.58 F_{yw} D t_w =$ 640 *kips* ≥ 113 kips **OK**

AASHTO BDS 8th 6.10.9.2-2

Bearing Stiffeners

Abutment Axial Rxn, IM included (CTBridge)

$P_{y\_HL\text{-}93}$ = 102 kips

P_{y_P15} = 140 kips

Factored Axial Demand

$P_{u\_Str\ I}$ = 202 kips — Does Not Control

$P_{u\_Str\ II}$ = 211 kips — Controls

Bearing Capacity

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

AASHTO BDS 8th 6.10.11.2.2-1

t_p = 0.5 in (1/2")

E = 29,000 ksi

F_{ys} = 50 ksi

~ 5.78 in *($0.48 t_p \sqrt{E/F_{ys}}$)*

Try 4" plate

$\emptyset_b$ = 1

$R_{sb\,n} = 1.4 A_{pn} F_{ys}$

b_{brg} = 3 in

$\emptyset_b R_{sb\,n}$ = 210 kips > $P_{u\_Str\ I}$ **OK**

202 kips

Axial Resistance

$P_r = \emptyset_c P_n$ t_w = 0.65 in Web thickness

$\emptyset_c$ = 0.95

Effective Plate Width = 4" + 4" + 0.65" (t_w) = 8.65 in

Effective Strip Width = 2 x $9t_w$ - t_p = 11.2 in

A_g = 11.605 in^2 D_w = l = 33.96 in depth of web

I_s = 103.1 in^4 k = 0.75

r_s = 2.98 in

k x l = 25.47 in

$\frac{k\,x\,l}{r_s} =$ 8.55 ≤ 120 OK AASHTO BDS 8th 6.9.3

$$P_e = \frac{\pi^2 E}{\left(\frac{kl}{r_s}\right)^2} A_g$$ AASHTO BDS 8th 6.9.4.1.2-1

P_e = 45,474 kips

$P_o = F_y A_g$

P_o = 580.25 kips

$\frac{P_e}{P_o} =$ 78 ≥ 0.44 AASHTO BDS 8th 6.9.4.1.1-1

$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$ 577 kips

$P_r = \emptyset_c P_n =$ 548 kips > 211 kips OK

Welded Connection

Assume t_{weld} = 0.25 in

$R_r = 0.6\emptyset_{e1} F_{EXX}$ F_{EXX} = 70 ksi

$\emptyset_{e1}$ = 0.8

R_r = 33.6 ksi

L_{eff} = 4 x (33.96 in - 2 in) = 127.84 in

$T_{eff} = \frac{t_{weld}}{\sqrt{2}} =$ 0.18 in Throat

$A_{eff} = L_{eff} x\, T_{eff} =$ 22.6 in^2

$R_r\, x\, A_{eff} =$ 759.33 kips > 211 kips OK

Bolted Connection

Assume ASTM F3125 Grade A325 High Stength 3/4" ∅, threads excluded from the shear plane

F_{ub} = 120 ksi

Single Shear A_b = 0.44 in^2

$R_n = 0.56 A_b F_{ub} N_s =$ 29.69 kips

$L_c = 1.5\ in + 1"\ (clr) - (0.75\ in/2) =$ 2.13 in

Bearing F_u = 58 ksi t = 0.5 in

$R_n = 1.2 L_c t F_u =$ 73.95 kips

take as the lesser 29.69 kips

$\emptyset =$ 0.8

$\emptyset R_n =$ 23.75 kips per bolt

Abutment Rxn = 211 kips Multiply resistance by 2 since angle is in double shear

$Number\ of\ Bolts = \frac{211}{2x\ 23.75} =$ 4.44 use 5 bolts

Diaphragm

Assume trial Channel size = 1/2H_{girder} = 18 in

Try MC18x42.7

Torsional Buckling Resistance

A_g = 12.6 in^2 k_y = 1 AISC SCM Table 1-5

r_y = 1.07 in l_y = 48 in (4 ft girder spacing)

$$P_{ey} = \frac{\pi^2 E}{\left(\frac{k_y l_y}{r_y}\right)^2} A_g =$$ 1,792 kips AASHTO BDS 8th 6.9.4.1.3-4

AISC SCM Table 1-5

J = 1.23 in^4 r_o = 6.97 in C_w = 852 in^6

H = 0.93

$$P_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z l_z)^\wedge 2}\right) + GJ}{r_o^2} =$$ 2,461 kips AASHTO BDS 8th 6.9.4.1.1

AASHTO BDS 8th 6.9.4.1.1.3.2

$$P_e = \left(\frac{P_{ey} + P_{ez}}{2H}\right)[1 - \sqrt{1 - \frac{4 P_{ey} P_{ez} H}{(P_{ey} + P_{ez})^\wedge 2}} =$$ 1589 kips

$P_o = F_y A_g =$ 630 kips

$\frac{P_e}{P_o} =$ 2.52 > 0.44

Therefore

$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$ 534 kips

$\emptyset_c =$ 0.95

$\emptyset_c P_n =$ 507 kips > 75 kips OK

See below for demand

Controlling Factored Axial load at L/3 HL-93 P15

$P_{u\_Str\ I}$ = 73 kips Does Not Control 29.02 kips 38.88 kips

$P_{u\_Str\ II}$ = 75 kips Controls

V. Design Verification for Steel Rolled Wide Flange Girders with Timber Decking

Span Length = 60'- 70'

Wide Flange Girder (WFG), Girder spacing, S = 4'-0"

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Spreadsheet Calculations for L = 70'

Temp Bridge, Single Span

Girder =	W36x194							
Span L =	70	ft	w1 =	0.2	klf	t_f =	1.26	in
d/s =	0.045		w2 =	0.093	klf	d =	36.5	in
C-C =	4	ft						
LL DF =	0.48	lanes	Krail =	0.13	klf			
SS d =	37.8	in	SW_{girder} =	0.194	klf			
Girders			D_C =	0.524	klf			
CTBridge			D_W =	0.093	klf			

Str I Moment Midspan =	813.79	k*ft
Str II Moment Midspan =	1127.52	k*ft
Str I Shear @ Abut =	49.17	kips
Str II Shear @ Abut =	76.37	kips

Factored Moment

$M_{u\_Str\ I}$ =	1911	k*ft	Does Not Control
$M_{u\_Str\ II}$ =	2009	k*ft	Controls

Factored Shear

$V_{u\_Str\ I}$ =	114	kips	Does Not Control
$V_{u\_Str\ II}$ =	131	kips	Controls

Table 3-2 AISC 15th Brace pt = L/4

Lp = 9.04 ft

Lr = 27.6 ft

For 70 ft total length braced at L/4, Lb = 17.5 ft

Lp < Lb < Lr

Moment Capacity

$$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(L_b - L_p)] \le \emptyset_b M_{px}$$

AISC 15th 3-4a (LRFD)

C_b =	1			F_y =	50	ksi
$\emptyset_b$ =	0.9			Z_x =	767	in^3
$\emptyset_b M_{px}$ =	2,876	k*ft		$\emptyset_b BF$ =	61.4	kips
$\emptyset_b M_n$ =	2,357	k*ft	<	$\emptyset_b M_{px}$ 2,876 k*ft	OK	
			>	$M_{u\_Str\ I}$ 2009 k*ft	OK	

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W40×215	**964**	**2410**	**3620**	**1500**	**2250**	**39.4**	**59.3**	**12.5**	**35.6**	**16700**	**507**	**761**
W36×231	963	2400	3610	1490	2240	35.7	53.7	13.1	38.6	15600	555	832
W30×261	943	2350	3540	1450	2180	29.1	44.0	12.5	43.4	13100	588	882
W33×241	940	2350	3530	1450	2180	33.5	50.2	12.8	39.7	14200	568	852
W36×232	936	2340	3510	1410	2120	44.8	67.0	9.25	30.0	15000	646	968
W27×281	936	2340	3510	1420	2140	24.8	36.9	12.0	49.1	11900	621	932
W14×455[h]	936	2340	3510	1320	1980	6.24	9.36	15.5	179	7190	768	1150
W24×306[h]	922	2300	3460	1380	2070	19.7	29.8	11.3	57.9	10700	683	1020
W40×211	**906**	**2260**	**3400**	**1370**	**2060**	**48.6**	**73.1**	**8.87**	**27.2**	**15500**	**591**	**887**
W40×199	**869**	**2170**	**3260**	**1340**	**2020**	**37.6**	**56.1**	**12.2**	**34.3**	**14900**	**503**	**755**
W14×426[h]	869	2170	3260	1230	1850	6.16	9.23	15.3	168	6600	703	1050
W33×221	857	2140	3210	1330	1990	31.8	47.8	12.7	38.2	12900	525	788
W27×258	852	2130	3200	1300	1960	24.4	36.5	11.9	45.9	10800	568	853
W30×235	847	2110	3180	1310	1960	28.0	42.7	12.4	41.0	11700	520	779
W24×279[h]	835	2080	3130	1250	1880	19.7	29.6	11.2	53.4	9600	619	929
W36×210	833	2080	3120	1260	1890	42.3	63.4	9.11	28.5	13200	609	914
W14×398[h]	801	2000	3000	1150	1720	5.95	8.96	15.2	158	6000	648	972
W40×183	**774**	**1930**	**2900**	**1180**	**1770**	**44.1**	**66.5**	**8.80**	**25.8**	**13200**	**507**	**761**
W33×201	773	1930	2900	1200	1800	30.3	45.6	12.6	36.7	11600	482	723
W27×235	772	1930	2900	1180	1780	24.1	36.0	11.8	42.9	9700	522	784
W36×194	767	1910	2880	1160	1740	40.4	61.4	9.04	27.6	12100	558	838

Shear Capacity

$\emptyset_v V_{vx=}$ 838 kips — AISC 15[th] Table 3-2 (LRFD)

V_u = 131 kips

$\emptyset_v V_{vx} > V_u$

$\emptyset_v$ = 1

$\emptyset_v V_n = 0.58 F_{yw} D t_w =$ 754 *kips* ≥ 131 kips OK

AASHTO BDS 8th 6.10.9.2-2

Bearing Stiffeners

Abutment Axial Rxn, IM included (CTBridge)

$P_{y\_HL\text{-}93}$ = 110 kips

P_{y_P15} = 159 kips

Factored Axial Demand

$P_{u\_Str\ I}$ = 220 kips — Does Not Control

$P_{u\_Str\ II}$ = 243 kips — Controls

Bearing Capacity

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$ AASHTO BDS 8th 6.10.11.2.2-1

t_p = 0.625 in (1/2")
E = 29,000 ksi
F_{ys} = 50 ksi
~ 7.22 in ($0.48t_p\sqrt{E/F_{ys}}$)
Try 4" plate
$\varnothing_b$ = 1
$R_{sb\,n}$ = $1.4A_{pn}F_{ys}$
b_{brg} = 3 in
$\varnothing_b R_{sb\,n}$ = 262.5 kips > $P_{u\_Str\ I}$ 220 kips OK

Axial Resistance

$P_r = \varnothing_c P_n$ t_w = 0.765 in Web thickness
$\varnothing_c$ = 0.95
Effective Plate Width = 4" + 4" + 0.765" (t_w) = 8.765 in
Effective Strip Width = 2 x $9t_w - t_p$ = 13.145 in
A_g = 15.534 in^2 D_w = l = 33.98 in depth of web
I_s = 179.9 in^4 k = 0.75
r_s = 3.40 in
$k \times l$ = 25.485 in

$\frac{k\,x\,l}{r_s}$ = 7.49 ≤ 120 OK AASHTO BDS 8th 6.9.3

$$P_e = \frac{\pi^2 E}{\left(\frac{kl}{r_s}\right)^2} A_g$$ AASHTO BDS 8th 6.9.4.1.2-1

P_e = 79,266 kips

$P_o = F_y A_g$
P_o = 776.7 kips

$\frac{P_e}{P_o}$ = 102 ≥ 0.44 AASHTO BDS 8th 6.9.4.1.1-1

$P_n = 0.658^{\frac{P_o}{P_e}} P_o =$ 774 kips
$P_r = \varnothing_c P_n =$ 735 kips > 243 kips OK

Welded Connection

Assume t_{weld} = 0.25 in

$R_r = 0.6\phi_{e1} F_{EXX}$ F_{EXX} = 70 ksi

ϕ_{e1} = 0.8

R_r = 33.6 ksi

L_{eff} = 4 x (33.98 in - 2 in) = 127.92 in

$T_{eff} = \frac{t_{weld}}{\sqrt{2}} =$ 0.18 in Throat

$A_{eff} = L_{eff} x\ T_{eff} =$ 22.6 in^2

$R_r\ x\ A_{eff} =$ 759.81 kips > 243 kips OK

Bolted Connection

Assume ASTM F3125 Grade A325 High Stength 3/4" ø, threads excluded from the shear plane

F_{ub} = 120 ksi

Single Shear A_b = 0.44 in^2

$R_n = 0.56 A_b F_{ub} N_s =$ 29.69 kips

$L_c = 1.5\ in + 1"\ (clr)\ -\ (0.75\ in/2)\ =$ 2.13 in

Bearing F_u = 58 ksi t = 0.625 in

$R_n = 1.2 L_c t F_u =$ 92.438 kips

take as the lesser 29.69 kips

ϕ = 0.8

$\phi R_n =$ 23.75 kips per bolt

Abutment Rxn = 243 kips Multiply resistance by 2 since angle is in double shear

$Number\ of\ Bolts = \frac{243}{2x\ 23.75} =$ 5.11 use 5 bolts

Diaphragm

Assume trial Channel size = $1/2H_{girder}$ = 18.25 in

Try MC18x42.7

Torsional Buckling Resistance

A_g =	12.6 in^2	k_y =	1	AISC SCM Table 1-5
r_y =	1.07 in	l_y =	48 in	(4 ft girder spacing)

$$P_{ey} = \frac{\pi^2 E}{\left(\frac{k_y l_y}{r_y}\right)^2} A_g = \quad 1{,}792 \text{ kips}$$

AASHTO BDS 8th 6.9.4.1.3-4

AISC SCM Table 1-5

J =	1.23 in^4	r_o =	6.97 in	C_w =	852 in^6
				H =	0.93

$$P_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z l_z)^\wedge 2}\right) + GJ}{r_o^2} = \quad 2{,}461 \text{ kips}$$

AASHTO BDS 8th 6.9.4.1.1

AASHTO BDS 8th 6.9.4.1.1.3.2

$$P_e = \left(\frac{P_{ey} + P_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4P_{ey}P_{ez}H}{(P_{ey}+P_{ez})^\wedge 2}}\right] = \quad 1589 \text{ kips}$$

$$P_o = F_y A_g = \quad 630 \text{ kips}$$

$$\frac{P_e}{P_o} = \quad 2.52 \quad > \quad 0.44$$

Therefore

$$P_n = 0.658^{\frac{P_o}{P_e}} P_o = \quad 534 \text{ kips}$$

$\varnothing_c$ = 0.95

$\varnothing_c P_n$ = 507 kips > 100 kips OK

See below for demand

Controlling Factored Axial load at L/4			HL-93	P15
$P_{u\_Str\ I}$ =	93 kips	Does Not Control	37.36 kips	53.69 kips
$P_{u\_Str\ II}$ =	100 kips	Controls		

VI. Design Verification for Timber Stacked Abutment

Wide Flange Girder (WFG), Girder spacing, S = 4'-0"

Deck thickness - Timber = 11.5 inch, HMA = Varies

1.) Spreadsheet Calculations for L = 70'

Span Length = 70 ft
W36x194

$w1$ =	0.05	klf	Timber Decking
$w2$ =	0.02	klf	HMA Overlay

Krail =	0.77	klf/ft	
$SW_{girders}$ =	1.164	klf/ft	0.194 x 6
D_C =	1.99	klf/ft	say 2 klf for design
D_W =	0.023	klf/ft	say 0.03 klf for design
Rxn_{DC} =	3	kips/ft	
Rxn_{DW} =	1	kips/ft	
$LL_{HL\text{-}93}$ =	5	kips/ft	
LL_{P15} =	7	kips/ft	

Ftg dims
Width = 3.83 ft
Length = 24 ft

Settlement Check (Service I, all Load Factors = 1)

Factored Loads
DC = 3 kips/ft
DW = 1 kips/ft
$LL_{HL\text{-}93}$ = 5 kips/ft

Multiply loads by 24'
DC = 70 kips/ft
DW = 25 kips/ft
$LL_{HL\text{-}93}$ = 110 kips/ft LL_{P15} = 159 kips/ft Controls

Total Vertical Load = 254 kips
Surface Area of Bearing = (11.5"/12) x 24' x 4 pads = 92 SF
Vertical Pressure = 2.76 ksf OK assuming allowable = 3 ksf

Bearing Check
Strength I = 317 kips
Strength II = 340 kips Controls
Bearing Pressure = 3.70 ksf OK assuming allowable = 10 ksf
Ref. BDP Table 10.1.8-7

Moment
L = 4 ft
w = 3.54 ksf 3.70 ksf x 11.5/12

$M_u = \frac{wL^2}{8} =$ 85.0 k*in

F_b = 1.76 ksi S = 57.98 in^3

ϕ = 0.85

$M_r = \phi F_b S =$ 87 k*in > 85.0 k*in OK

Shear $V_u = \frac{PL}{4} =$ 19.5 kips

P = 1.62 kips
(3.70 ksf x 5.5" x 11.5")/144

F_v = 0.45 ksi b = 11.5 in
$\emptyset$ = 0.75 d = 5.5 in

$V_n = \emptyset \frac{F_v bd}{1.5} =$ 14.2 kips > 19.5 k*in NG

use 12 x 8 pad

Compression Perpendicular to the grain

R_{u_max} = 159 kips (LL_{P15})

A_c = 437 in^2 try 10" x 12" x 4'-0"

F_{cp} = 1.17 ksi

$P_r = 0.9F_{cp}A_c =$ 460 kips > 159 k*in OK

Check Cap Beam

Try 12 x 12 cap beam

$P_r = 0.9F_{cp}11.5\text{" x }11.5\text{"} =$ 139.26 < 159 k*in NG

$P_r = 0.9F_{cp}11.5\text{" x }(11.5\text{"}+3.5\text{"}) =$ 181.64 > 159 k*in OK

use 12 x 12 cap
fastened to
4 x 12 block

Backwall Check

Assume 4 stacks of 12 x 12 Timber beams

H = 46 in

Assumed Soil Properties:

K_a = 0.33

γ_s = 120 pcf

h_{eq} = 4 ft AASHTO BDS 8th Table 3.11.6.4-1

$P = K_a \gamma_s H$ AASHTO BDS 8th 3.11.5.1-1

P = 151.8 psf

Load factor, EH = 1.5 AASHTO BDS 8th Table 3.4.1-1 & Table 3.4.1-2

$\Delta_p = K_a \gamma_s h_{eq}$ AASHTO BDS 8th Table 3.11.6.4-1

Δ_p = 158.4 psf

Load factor, LS = 1.35 STP 17.1, Table 17.1.3-1

Total Factored Pressure = 0.44 klf

L = Cantilever shoring length on either side of bridge = 110 in
(assumed to be 18" + 2 x H)

Factored Moment

$M_u = \frac{wL^2}{2} =$	19 k*ft	=	223 k*in	
$\emptyset M_n = \emptyset F_b S =$	379 k*in	>	223 k*in	OK

Factored Shear

$V_u = wL =$	4 kips	b = d =	11.5 in	
$\emptyset V_n = \emptyset \frac{F_v bd}{1.5} =$	30 kips	>	4 kips	OK

Provide a minimum of (4) 12" x 12" for backwall

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