(Service III) exceeded the allowable tension in the section with additional strands.

Future Work

Future work will include efforts to determine if Precast Concrete “Multi Girder” Bridges – specifically Precast Concrete Channel Sections with Shear Keys, Precast Concrete Double Tee Sections with Shear Keys, and Precast Concrete Tee Sections with Shear Keys – can be made continuous for live load.

- To determine the number of strands required to meet the flexural capacity of the section, a spreadsheet was developed to facilitate the iterative process. In practice, advanced computational aids typically determine the appropriate cable path and strand layout.
- In the absence of advanced methods, the cable path can be assumed to run as closely through the geometric center of the section as possible.
- The location of the center of gravity of the strands was assumed to be 12 inches at the low point of the strands.
- Since the cable path was not designed, the friction losses and anchor set losses were not determined in this design. If time permits, full cable path design will be pursued. To complete the design of the path, friction and anchor set loss of 0.9330 were assumed at the critical locations analyzed.
- The resistance provided by secondary moments was conservatively neglected in the determination of the flexural resistance of the section.
- The required post-tensioning properties are shown in Table 102.

Table 102: Post-Tension Properties

Post-Tension Properties		
Type	Low Lax, Rigid Galvanzied Metal Duct	
f_pu	270.00	ksi
f_py	243.00	ksi
E_p	28500.0	ksi
f_pj	207.90	ksi
# Strands	66	ksi
0.6" Strand	0.217	in^2
A_ps	14.322	in^2
P_j	2977.54	kips
k	0.0002	
mu	0.250	

- The flexural resistance of the section was determined to be M_n = 10,574 ft-k > M_u = 9,961 ft-k at 0.4 Span 1 and 0.6 Span 2. However, 1.2M_{cr} = 20,323 ft-k > M_n = 10,574 ft-k. Additional mild steel reinforcement of #7 bars spaced at 12" in. C/C was needed to prevent cracking of the section. Thus, M_u = 62,125 ft-k > 1.2M_{cr} = 20,323 ft-k.
- The flexural resistance of the section was determined to be M_n = 9,534 ft-k < M_u = 10,828 ft-k < 1.2M_{cr} = 21,658 ft-k over the pier. Additional mild steel reinforcement of #7 bars spaced at 12" in. C/C was needed to prevent cracking of the section. Thus, M_u = 64,377 ft-k > 1.2M_{cr} = 21,658 ft-k.
- Note: Additional strands were not an efficient solution to increase the capacity of the section because the final concrete tension over the pier

M+ (0.4 Span 1)	4103.60	ft-k
M- (Pier)	-3350.55	ft-k
M+ (0.6 Span 2)	4103.60	ft-k

- The maximum live load shear effects, including dynamic load allowance and impact factor, are shown in Table 99. LEAP Bridge Concrete applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Table 99: Live Load Shear Effects

Shear - LL+IM		
V_+ (Pier)	619.73	k
V- (Pier)	-619.73	k

Investigate Service Limit State

- The factored service force effects were not directly calculated, but were investigated in the design of the post-tensioning, deck, and overhang.

Investigate Strength Limit State

- The factored moments used to check the Strength I Limit State are displayed in Table 100.

Table 100: Factored Moments - Strength I Limit State

Factored Moments - Strength I		
M+ (0.4 Span 1)	9961.36	ft-k
M- (Pier)	-10827.85	ft-k
M+ (0.6 Span 2)	9961.36	ft-k

- The factored shear used to check the Strength I Limit State are displayed in Table 101.

Table 101: Factored Shear - Strength I Limit State

Factored Shear - Strength I		
V+ (Pier)	1580.95	k
V- (Pier)	-1580.95	k

Check Flexure

Table 97: Unfactored Shear

Unfactored Shear			
1.0 Span 1	*DW*	7.93	k
0.0 Span 2		-7.93	k
1.0 Span 1	*Superstructure*	302.7	k
0.0 Span 2		-302.7	k
1.0 Span 1	*Barrier*	84.93	k
0.0 Span 2		-84.93	k
1.0 Span 1	*HL-93 Design Truck*	62	k
0.0 Span 2		-62	k
1.0 Span 1	*HL-93 Design Tandem*	48.88	k
0.0 Span 2		-48.88	k
1.0 Span 1	*HL-93 Dual Truck*	62.01	k
0.0 Span 2		-62.01	k
1.0 Span 1	*HL-93 Design Lane*	19.97	k
0.0 Span 2		-19.97	k

- The maximum live load moment effects, including dynamic load allowance and impact factor, are shown in Table 98. LEAP Bridge Concrete applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Table 98: Live Load Moment Effects

Moment - LL+IM

Fatigue (HL-93)			
Moment	60 ft	1933.00	ft-k
Moment	150 ft	-1083.00	ft-k
Moment	240 ft	1933.00	ft-k
Shear	150 ft	70.33	k
Shear	150 ft	-70.33	k
Construction Live Load			
Moment	60 ft	170.80	ft-k
Moment	150 ft	-303.80	ft-k
Moment	240 ft	170.80	ft-k
Shear	150 ft	10.13	k
Shear	150 ft	-10.13	k

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Table 96: Unfactored Moments

Unfactored Moments			
0.4 Span 1	*DW*	44.43	ft-k
Pier		-79.34	ft-k
0.6 Span 2		44.43	ft-k
0.4 Span 1	*Superstructure*	1695.13	ft-k
Pier		-3027.02	ft-k
0.6 Span 2		1695.13	ft-k
0.4 Span 1	*Barrier*	475.6	ft-k
Pier		-849.28	ft-k
0.6 Span 2		475.6	ft-k
0.4 Span 1	*HL-93 Design Truck*	501.51	ft-k
Pier		-316.37	ft-k
0.6 Span 2		501.51	ft-k
0.4 Span 1	*HL-93 Design Tandem*	472.1	ft-k
Pier		-237.75	ft-k
0.6 Span 2		472.1	ft-k
0.4 Span 1	*HL-93 Dual Truck*	501.15	ft-k
Pier		-353.91	ft-k
0.6 Span 2		501.15	ft-k
0.4 Span 1	*HL-93 Design Lane*	152.34	ft-k
Pier		-198.29	ft-k
0.6 Span 2		152.34	ft-k

- For the previously defined cross-section, LEAP Bridge Concrete software determined magnitude and location of the unfactored shear.

Table 10: Unfactored Live Load Shear and Moment Effects

Shear and Moment Effects from Visual Analysis			
HL-93 Design Truck (Short)			
Load Type	Location	Magnitude	Units
Moment	60 ft	1962.00	ft-k
Moment	150 ft	-1021.00	ft-k
Moment	240 ft	1962.00	ft-k
Shear	150 ft	69.16	k
Shear	150 ft	-69.46	k
HL-93 Design Truck (Long)			
Moment	60 ft	1743.00	ft-k
Moment	150 ft	-980.30	ft-k
Moment	240 ft	1743.00	ft-k
Shear	150 ft	64.26	k
Shear	150 ft	-64.26	k
HL-93 Dual Truck			
Moment	60 ft	3367.00	ft-k
Moment	150 ft	-3798.00	ft-k
Moment	240 ft	3367.00	ft-k
Shear	150 ft	154.30	k
Shear	150 ft	-154.30	k
HL-93 Design Tandem			
Moment	60 ft	2884.00	ft-k
Moment	150 ft	-2519.00	ft-k
Moment	240 ft	2884.00	ft-k
Shear	150 ft	109.50	k
Shear	150 ft	-109.50	k

DC_1 Total (per 4 Girders)		21.049	k/ft
DC_1 Load per 1 Girder		5.262	k/ft
DC_2	Barrier	1.040	k/ft
DC_2 per 1 Girder		0.260	k/ft
DW	FWS	1.000	k/ft
DW per 1 Girder		0.250	k/ft

Table 8: Construction Live Loads

Live Load			
CLL	Construction LL	0.430	k/ft
	Construction LL per Girder	0.108	k/ft

Table 9: Unfactored Dead Load Moments

Dead Load			
DC_1			
Moment	60 ft	8347.00	ft-k
Moment	150 ft	-14842.00	ft-k
Moment	240 ft	8347.00	ft-k
Shear	150 ft	494.70	k
Shear	150 ft	-494.70	k
DC_2			
Moment	60 ft	411.30	ft-k
Moment	150 ft	-731.30	ft-k
Moment	240 ft	411.30	ft-k
Shear	150 ft	24.38	k
Shear	150 ft	-24.38	k
DW			
Moment	60 ft	395.40	ft-k
Moment	150 ft	-703.10	ft-k
Moment	240 ft	395.40	ft-k
Shear	150 ft	23.44	k
Shear	150 ft	-23.44	k

- Applicable resistance factors for the Strength I Limit State were determined using AASHTO Article 5.5.4.2.1.

Select Load Modifiers

- The applicable load modification factors were determined based on the guidelines of AASHTO Articles 1.3.3 through 1.3.5. Table 95 displays the modification factors used in this design.

Table 95: Load Modification Factors

Modification Factors	
Ductility	1.0
Redundancy	1.0
Operational Importance	1.0

Select Applicable Load Combinations and Load Factors

- The applicable load combinations and load factors were determined using AASHTO Table 3.4.1-1.

- $\gamma_{DC} = 1.25$
- $\gamma_{DW} = 1.50$
- $\gamma_{LL} = 1.75$

$$M_u = \gamma_{DC} M_{DC} + \gamma_{DW} M_{DW} + \gamma_{LL} M_{LL}$$

$$V_u = \gamma_{DC} V_{DC} + \gamma_{DW} V_{DW} + \gamma_{LL} V_{LL}$$

Calculate Live Load Force Effects

- For the previously defined cross-section, LEAP Bridge Concrete software determined magnitude and location of the unfactored moments.
- . The dead loads, construction live load, and results from the HL-93 vehicular design load are shown below.

Table 7: Dead Loads

Dead Loads			
DC_1	Deck	5.106	k/ft
	Overhang Tapers	1.150	k/ft
	Haunches	0.131	k/ft
	SIP Forms	0.081	k/ft
	Girder Self-Weight	14.471	k/ft
	Cross Frames & Details	0.110	k/ft

Low Relaxation Prestressing Strand		
0.6" Diameter Strand		
Tensile Strength	270	ksi
Yield Strength	243	ksi
Modulus of Elasticity	28500	ksi
Concrete		
f'_c	4.5	ksi
f'_{ci}	3.5	ksi
Unit Weight for E_c	0.145	kcf
Unit Weight for DL Calculation	0.15	kcf
E_c	3681	ksi
E_{ci}	3405	ksi
n	8	
β_1	0.825	

Deck Design

- Note: Deck design was omitted for this design. Determination of the flexural capacity was the primary concern for design of the CIP MCB Girder Bridge with two 50 ft. spans. If sufficient time permits, future work will include full deck design.
- Deck design should be completed using the effective length of the section, as defined in AASHTO Article 9.7.2.3. The *strip method* (an elastic method of analysis) should be used to determine the required mild steel reinforcement in the section.
- Unfactored live loads used in this portion of the design should be determined based on the effective length and AASHTO Table A4-1.
- The 0.5-inch thickness of the future wearing surface should be assumed to contribute no strength in the deck design.
- Strength I Limit State, Service I Limit State, Cracking Criteria, and Minimum Reinforcement Limit must be satisfied in the design of the conventionally reinforced deck.

Overhang Design

- Note: Overhang design was omitted for this design. Determination of the flexural capacity was the primary concern for design of the CIP MCB Girder Bridge with two 50 ft. spans. If sufficient time permits, future work will include full overhang design.
- In each of the three design cases specified by AASHTO, the overhang should satisfy the Strength, Service, spacing, and minimum reinforcement requirements

Select Resistance Factors – Strength I Limit State

Roadway Width	44	ft
Total Bridge Width	47.167	ft
t_c	12	in
L_c	3.75	ft
Web	12	in
Web Spacing	12.9	ft
t_slab (top)	8	in
t_slab (bot)	7	in
Depth	4	ft
Dist. from Bot. Slab to OH	1.5	ft
N_w	5	
N_c	4	
Span	100	ft
Bottom Slab Width	36.667	ft
Fillet	4	in

- The following section properties were determined by modeling the section in LEAP Bridge Concrete software:

Table 93: Section Properties Determined in LEAP Bridge Concrete

A	66.80	ft^2	9619.66	in^2
I	158.34	ft^4	3283342.39	in^4
y_t	2.25	ft	27.04	in
y_b	1.75	ft	20.96	in
Vol/Area	4.38		in	

Table 94: Material Properties

Material Properties		
Reinforcing Steel		
Yield Strength	60	ksi
Modulus of Elasticity	29000	ksi
Prestressing Strand		

- Width and Thickness of Overhang
  - The thickness of the cantilever overhang wing root, t_c was determined using the following equation:

$$t_c = \frac{L_c}{5}(12) \geq t_{tip}$$

  - The thickness of the cantilever overhang tip, t_{tip}, was taken to equal 9 inches, as per the minimum for "short" cantilevers less than 5 feet in length.
  - The length of the cantilever overhang was determined as 3'-9" for this design.

- Total and Individual Web Thickness
  - The total web thickness was approximated using the following equation, in which the allowable shear stress and superstructure dead load were estimated using the full bridge width method:

$$b_{total} = \frac{\rho_{DL} L_{max}}{2(0.8h)\tau_{allow}}$$

$$5\sqrt{f_c'} \leq \tau_{allow} \leq 7\sqrt{f_c'}$$

  - For webs with post-tensioning, the minimum thickness recommended for practical design is 10 inches. This design uses four 12-inch webs, spaced at L_{clear} = 12'-10 ½ " C/C.

- Top Slab Thickness
  - The top slab thickness was determined using the following equation:

$$\frac{L_{clear}}{14} \leq t_{slab} \leq \frac{L_{clear}}{17}$$

  - The top slab thickness chosen for this design is 8 inches.

- Bottom Slab Thickness
  - The practical minimum thickness of bottom slabs of CIP multi-cell box girder is 7 inches. This is due to cover requirements for mild steel reinforcement.

- The cross-section properties are summarize in the table below:

Table 92: Cross-Section Properties

Lane Width	12	ft
Shoulder Width	10	ft
Barrier Width	1.583	ft

Cast-in-Place Multi-Cell Box (50 ft. spans)

Limit States to be Satisfied:

- Strength I
- Service I
- Service III
- Any additional limit states, as required

Design Parameters

- Span length
  - Typical span lengths are shown in Figure 1. The figure was provided by FHWA *Post-Tensioned Box Girder Design Manual* (2015), in accordance with recommendations by the California Department of Transportation (CALTRANS).
  - Since the model used for NCHRP 12-103 determined that shorter spans control design of CIP Multi-Cell Box (MCB) Girder bridges, a practical minimum span length of 100 feet is recommended.
  - To present the panel with an alternative, shorter design, the span length chosen for this design is 50 feet.

- Width
  - The width of the bridge is determined based on the number and width of traffic lanes provided, the width of shoulders, and width of other features, such as pedestrian sidewalks, bicycle lanes, or observation overhangs.
  - In this design, the simplest practical design includes two 12-foot traffic lanes and two 10-foot shoulders, and Arizona DOT Standard F-Shape barriers (1'-7" width) for a total bridge width of 47'-2".
  - As permissible, the outside safety barriers are located on the outermost edge on each side of the bridge, along the entire bridge length.

- Skew
  - Skew is not considered in this design, but may be investigated in future work.

- Depth
  - As per AASHTO LRFD 2.5.2.6.3.1-1, the minimum depth of CIP Reinforced Concrete Box Girder continuous bridges can be determined the equation shown, where D is the total depth of the section, and L is the span length:
  $$D = 0.055L$$
  - The design uses D = 4.0 ft.

Table 91: Post-Tension Properties

Post-Tension Properties		
Type	Low Lax, Rigid Galvanzied Metal Duct	
f_{pu}	270.00	ksi
f_{py}	243.00	ksi
E_p	28500.0	ksi
f_{pj}	207.90	ksi
# Strands	150	ksi
0.6" Diameter Strand	0.217	in^2
A_{ps}	32.550	in^2
P_j	6767.15	kips
k	0.0002	
μ	0.250	

- The flexural resistance of the section was determined to be M_n = 37,376 ft-k > M_u = 25,955 ft-k at 0.4 Span 1 and 0.6 Span 2.
- The flexural resistance of the section was determined to be M_n = 52,274 ft-k > M_u = 36,627 ft-k over the pier. Additional mild steel reinforcement of #5 bars spaced at 12" in. C/C was needed to prevent cracking of the section.

V_- (Pier)	-710.95	k

Investigate Service Limit State

- The factored service force effects were not directly calculated, but were investigated in the design of the post-tensioning, deck, and overhang.

Investigate Strength Limit State

- The factored moments used to check the Strength I Limit State are displayed in Table 89.

Table 89: Factored Moments - Strength I Limit State

Factored Moments - Strength I		
M_+ (0.4 Span 1)	25955.22	ft-k
M_- (Pier)	-36626.55	ft-k
M_+ (0.6 Span 2)	25955.22	ft-k

- The factored shear used to check the Strength I Limit State are displayed in Table 90.

Table 90: Factored Shear - Strength I Limit State

Factored Shear - Strength I		
V_+ (Pier)	2230.69	k
V_- (Pier)	-2230.69	k

Check Flexure

- To determine the number of strands required to meet the flexural capacity of the section, a spreadsheet was developed to facilitate the iterative process. In practice, advanced computational aids typically determine the appropriate cable path and strand layout.
- In the absence of advanced methods, the cable path can be assumed to run as closely through the geometric center of the section as possible.
- The location of the center of gravity of the strands was assumed to be 12 inches at the low point of the strands.
- Since the cable path was not designed, the friction losses and anchor set losses were not determined in this design. If time permits, full cable path design will be pursued. To complete the design of the path, friction and anchor set loss of 0.9330 were assumed at the critical locations analyzed.
- The resistance provided by secondary moments was conservatively neglected in the determination of the flexural resistance of the section.
- The required post-tensioning properties are shown in Table 91.

Table 86: Unfactored Shear

Unfactored Shear			
1.0 Span 1	*DW*	17.3	k
0.0 Span 2		-17.3	k
1.0 Span 1	*Superstructure*	708.28	k
0.0 Span 2		-708.28	k
1.0 Span 1	*Barrier*	60.18	k
0.0 Span 2		-60.18	k
1.0 Span 1	*HL-93 Design Truck*	67.71	k
0.0 Span 2		-67.71	k
1.0 Span 1	*HL-93 Design Tandem*	49.97	k
0.0 Span 2		-49.97	k
1.0 Span 1	*HL-93 Dual Truck*	79.61	k
0.0 Span 2		-79.61	k
1.0 Span 1	*HL-93 Design Lane*	39.93	k
0.0 Span 2		-39.93	k

- The maximum live load moment effects, including dynamic load allowance and impact factor, are shown in Table 87. LEAP Bridge Concrete applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Table 87: Live Load Moment Effects

Moment - LL+IM		
M_+ (0.4 Span 1)	8517.73	ft-k
M_- (Pier)	-9654.79	ft-k
M_+ (0.6 Span 2)	8517.73	ft-k

- The maximum live load shear effects, including dynamic load allowance and impact factor, are shown in Table 88. LEAP Bridge Concrete applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Table 88: Live Load Shear Effects

Shear - LL+IM		
V_+ (Pier)	710.95	k

Moment	60 ft	170.80	ft-k
Moment	150 ft	-303.80	ft-k
Moment	240 ft	170.80	ft-k
Shear	150 ft	10.13	k
Shear	150 ft	-10.13	k

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Table 85: Unfactored Moments

Unfactored Moments			
0.4 Span 1	*DW*	193.74	ft-k
Pier		-345.96	ft-k
0.6 Span 2		193.74	ft-k
0.4 Span 1	*Superstructure*	7932.79	ft-k
Pier		-14165.69	ft-k
0.6 Span 2		7932.79	ft-k
0.4 Span 1	*Barrier*	674.07	ft-k
Pier		-1203.69	ft-k
0.6 Span 2		647.07	ft-k
0.4 Span 1	*HL-93 Design Truck*	1233.68	ft-k
Pier		-660.72	ft-k
0.6 Span 2		1233.68	ft-k
0.4 Span 1	*HL-93 Design Tandem*	988.1	ft-k
Pier		-477.75	ft-k
0.6 Span 2		988.1	ft-k
0.4 Span 1	*HL-93 Dual Truck*	1233.68	ft-k
Pier		-1321.32	ft-k
0.6 Span 2		1233.68	ft-k
0.4 Span 1	*HL-93 Design Lane*	609.36	ft-k
Pier		-793.18	ft-k
0.6 Span 2		609.36	ft-k

- For the previously defined cross-section, LEAP Bridge Concrete software determined magnitude and location of the unfactored shear.

Table 10: Unfactored Live Load Shear and Moment Effects

Shear and Moment Effects from Visual Analysis			
HL-93 Design Truck (Short)			
Load Type	Location	Magnitude	Units
Moment	60 ft	1962.00	ft-k
Moment	150 ft	-1021.00	ft-k
Moment	240 ft	1962.00	ft-k
Shear	150 ft	69.16	k
Shear	150 ft	-69.46	k
HL-93 Design Truck (Long)			
Moment	60 ft	1743.00	ft-k
Moment	150 ft	-980.30	ft-k
Moment	240 ft	1743.00	ft-k
Shear	150 ft	64.26	k
Shear	150 ft	-64.26	k
HL-93 Dual Truck			
Moment	60 ft	3367.00	ft-k
Moment	150 ft	-3798.00	ft-k
Moment	240 ft	3367.00	ft-k
Shear	150 ft	154.30	k
Shear	150 ft	-154.30	k
HL-93 Design Tandem			
Moment	60 ft	2884.00	ft-k
Moment	150 ft	-2519.00	ft-k
Moment	240 ft	2884.00	ft-k
Shear	150 ft	109.50	k
Shear	150 ft	-109.50	k
Fatigue (HL-93)			
Moment	60 ft	1933.00	ft-k
Moment	150 ft	-1083.00	ft-k
Moment	240 ft	1933.00	ft-k
Shear	150 ft	70.33	k
Shear	150 ft	-70.33	k
Construction Live Load			

Table 8: Construction Live Loads

Live Load			
CLL	Construction LL	0.430	k/ft
	Construction LL per Girder	0.108	k/ft

Table 9: Unfactored Dead Load Moments

Dead Load			
DC_1			
Moment	60 ft	8347.00	ft-k
Moment	150 ft	-14842.00	ft-k
Moment	240 ft	8347.00	ft-k
Shear	150 ft	494.70	k
Shear	150 ft	-494.70	k
DC_2			
Moment	60 ft	411.30	ft-k
Moment	150 ft	-731.30	ft-k
Moment	240 ft	411.30	ft-k
Shear	150 ft	24.38	k
Shear	150 ft	-24.38	k
DW			
Moment	60 ft	395.40	ft-k
Moment	150 ft	-703.10	ft-k
Moment	240 ft	395.40	ft-k
Shear	150 ft	23.44	k
Shear	150 ft	-23.44	k

Table 84: Load Modification Factors

Modification Factors	
Ductility	1.0
Redundancy	1.0
Operational Importance	1.0

Select Applicable Load Combinations and Load Factors

- The applicable load combinations and load factors were determined using AASHTO Table 3.4.1-1.

- $\gamma_{DC} = 1.25$
- $\gamma_{DW} = 1.50$
- $\gamma_{LL} = 1.75$

$$M_u = \gamma_{DC} M_{DC} + \gamma_{DW} M_{DW} + \gamma_{LL} M_{LL}$$

$$V_u = \gamma_{DC} V_{DC} + \gamma_{DW} V_{DW} + \gamma_{LL} V_{LL}$$

Calculate Live Load Force Effects

- For the previously defined cross-section, LEAP Bridge Concrete software determined magnitude and location of the unfactored moments.. The dead loads, construction live load, and results from the HL-93 vehicular design load are shown below.

Table 7: Dead Loads

Dead Loads			
DC_1	Deck	5.106	k/ft
	Overhang Tapers	1.150	k/ft
	Haunches	0.131	k/ft
	SIP Forms	0.081	k/ft
	Girder Self-Weight	14.471	k/ft
	Cross Frames & Details	0.110	k/ft
DC_1 Total (per 4 Girders)		21.049	k/ft
DC_1 Load per 1 Girder		5.262	k/ft
DC_2	Barrier	1.040	k/ft
DC_2 per 1 Girder		0.260	k/ft
DW	FWS	1.000	k/ft
DW per 1 Girder		0.250	k/ft

- Case II considered the Extreme Event Limit State in which a vehicle with a 16-kip axle rests on the barrier and TL-5 railing.

Table 82: Case II Dimensions and Properties

Case II - Dimensions and Properties		
Distances		
From Collision Load to Outer Edge	5.5	in
Edge to Far Edge	0.25	in
From Collision Load to Inner Edge	5.25	in
Vehicle Collision Load	18	k
Distribution of Load	18	ft

- Case III considers the Strength Limit State at the same locations considered in Case I.

Table 83: Case III Dimensions and Properties

Case III - Dimensions and Properties		
Edge of Bridge to Location 1	19	in
Location 1 to Location 2	26	in
From Location 1 to Live Load	12	in
From Live Load to Location 2	14	in
From Edge of Bridge to Location 2	45	in
LL Axle Load	16	k
Multiple Presence Factor	1.2	
X, Dist. From Location 2 to Live Load	1.167	ft
Width of Primary Strip	56.667	in
Width of Primary Strip	4.722	ft

- In each case, the overhang was deemed to satisfy the Strength, Service, spacing, and minimum reinforcement requirements

<u>Select Resistance Factors</u> – Strength I Limit State

- Applicable resistance factors for the Strength I Limit State were determined using AASHTO Article 5.5.4.2.1.

<u>Select Load Modifiers</u>

- The applicable load modification factors were determined based on the guidelines of AASHTO Articles 1.3.3 through 1.3.5. Table 84 displays the modification factors used in this design.

Table 78: TL-5 Railing Properties

TL-5 Rail with 42-in. Height		
F_t, transverse	124	k
F_r, longitudinal	41	k
F_v, vertical down	80	k
L_t & L_l	8	ft
L_v	40	ft
H_e, min.	42	in

Table 79: Barrier Test Forces (from Arizona DOT)

Barrier Test Forces (from ADOT)		
M_b	0	ft-k
M_c	15.16	ft-k
M_w	56.42	ft-k
R_w	129.6	k
Barrier Weight	0.538	k

Table 80: Barrier Reinforcement Details (from Arizona DOT)

Barrier Reinforcing		
A_s (#5 Bar)	0.31	in^2
d_{bar}	0.625	in
Cover	1.5	in
Rebar Angle	26	degrees

- The cantilever overhang design was checked for three cases.
- Case I considered the Service Limit State at three critical locations, which are at the face of the barrier, at the exterior support, and at the interior support.

Table 81: Case I Distances

Case I - Distances		
Edge of Bridge to Location 1	19	in
Location 1 to Location 2	26	in
Edge of Bridge to Location 2	45	in
Location 2 to Location 3	13	in
R_w to Top of Slab	42	in
Barrier Load to Face of Barrier	11.352	in

Table 77: Summary of Mild Steel Reinforcement in Deck

Reinforcement Summary	
Bar No.	**Spacing**
Positive Moment Region	
#6 Bars	8 in.
Negative Moment Region	
#6 Bars	6 in.
Distribution Reinforcement	
#6 Bars	10 in.
Bottom Slab Reinforcement	
Longitudinal	
#6 Bars	12 in.
Transverse	
#6 Bars	12 in.
Temperature and Shrinkage	
Exterior Web	
#5 Bars	12 in.
Top Slab	
#5 Bars	12 in.
Bottom Slab	
#5 Bars	12 in.
Additional Reinforcement	
Mild Steel Over Pier	
#5 Bars	12 in.

- The 0.5-inch thickness of the future wearing surface was assumed to contribute no strength in the deck design.
- Strength I Limit State, Service I Limit State, Cracking Criteria, and Minimum Reinforcement Limit were satisfied in the design of the conventionally reinforced deck.

<u>Overhang Design</u>

- The railing considered for this design was a TL-5 railing measuring 42 inches in height. Tables 78-80 summarize the railing and barrier properties used in the design.

Table 76: Material Properties

Material Properties		
Reinforcing Steel		
Yield Strength	60	ksi
Modulus of Elasticity	29000	ksi
Prestressing Strand		
Low Relaxation Prestressing Strand		
0.6" Diameter Strand		
Tensile Strength	270	ksi
Yield Strength	243	ksi
Modulus of Elasticity	28500	ksi
Concrete		
f'_c	4.5	ksi
f'_{ci}	3.5	ksi
Unit Weight for E_c	0.145	kcf
Unit Weight for DL Calculation	0.15	kcf
E_c	3681	ksi
E_{ci}	3405	ksi
n	8	
β_1	0.825	

Deck Design

- Deck design was completed using the effective length of the section, as defined in AASHTO Article 9.7.2.3. The *strip method* (an elastic method of analysis) was used to determine the required mild steel reinforcement in the section.
- Unfactored live loads used in this portion of the design were determined based on the effective length and AASHTO Table A4-1.
- Table 77 summarizes the design of mild steel reinforcement in the deck using the *strip method*.

- Bottom Slab Thickness
  - The practical minimum thickness of bottom slabs of CIP multi-cell box girder is 7 inches. This is due to cover requirements for mild steel reinforcement.

- The cross-section properties are summarize in the table below:

Table 74: Cross-Section Properties

Section Properties		
Lane Width	12	ft
Shoulder Width	10	ft
Barrier Width	1.583	ft
Roadway Width	44	ft
Total Bridge Width	47.167	ft
t_c	12	in
L_c	3.75	ft
Web	12	in
Web Spacing	9.75	ft
t_{slab} (top)	8	in
t_{slab} (bot)	7	in
Total Depth	5.75	ft
Dist. from Bot. Slab to OH	1.75	ft
N_w (Number of Webs)	5	
N_c (Number of Cells)	4	
Span Length	100	ft
Bottom Slab Width	34.167	ft
Fillet	4	in

- The following section properties were determined by modeling the section in LEAP Bridge Concrete software:

Table 75: Section Properties Determined in LEAP Bridge Concrete

LEAP Bridge Concrete Section Properties				
Area	78.1556	ft^2	11254.4064	in^2
Moment of Inertia (Strong Axis)	386.6289	ft^4	8017136.87	in^4
y_t (from top)	2.5258	ft	30.3096	in
y_b (from bottom)	3.2242	ft	38.6904	in
Vol./Area	4.6289		in	

- Skew
  - Skew is not considered in this design, but may be investigated in future work.

- Depth
  - As per AASHTO LRFD Table 2.5.2.6.3.1-1, the minimum depth of CIP Reinforced Concrete Box Girder continuous bridges can be determined the equation shown, where D is the total depth of the section, and L is the span length:

$$D = 0.055L$$

  - The design uses D=5.75 ft.

- Width and Thickness of Overhang
  - The thickness of the cantilever overhang wing root, t_c was determined using the following equation:

$$t_c = \frac{L_c}{5}(12) \geq t_{tip}$$

  - The thickness of the cantilever overhang tip, t_{tip}, was taken to equal 9 inches, as per the minimum for "short" cantilevers less than 5 feet in length.
  - The length of the cantilever overhang was determined as 3'-9" for this design.

- Total and Individual Web Thickness
  - The total web thickness was approximated using the following equation, in which the allowable shear stress and superstructure dead load were estimated using the full bridge width method:

$$b_{total} = \frac{\rho_{DL} L_{max}}{2(0.8h)\tau_{allow}}$$

$$5\sqrt{f_c'} \leq \tau_{allow} \leq 7\sqrt{f_c'}$$

  - For webs with post-tensioning, the minimum thickness recommended for practical design is 10 inches. This design uses five 12-inch webs, spaced at L_{clear} = 9'-9" C/C.

- Top Slab Thickness
  - The top slab thickness was determined using the following equation:

$$\frac{L_{clear}}{14} \leq t_{slab} \leq \frac{L_{clear}}{17}$$

  - The top slab thickness chosen for this design is 8 inches.

Cast-in-Place Multi-Cell Box (100 ft. spans)

Limit States to be Satisfied:

- Strength I
- Service I
- Service III
- Any additional limit states, as required

Design Parameters

- Span length

  - Typical span lengths are shown in the figure below. The figure was provided by FHWA *Post-Tensioned Box Girder Design Manual* (2015), in accordance with recommendations by the California Department of Transportation (CALTRANS).

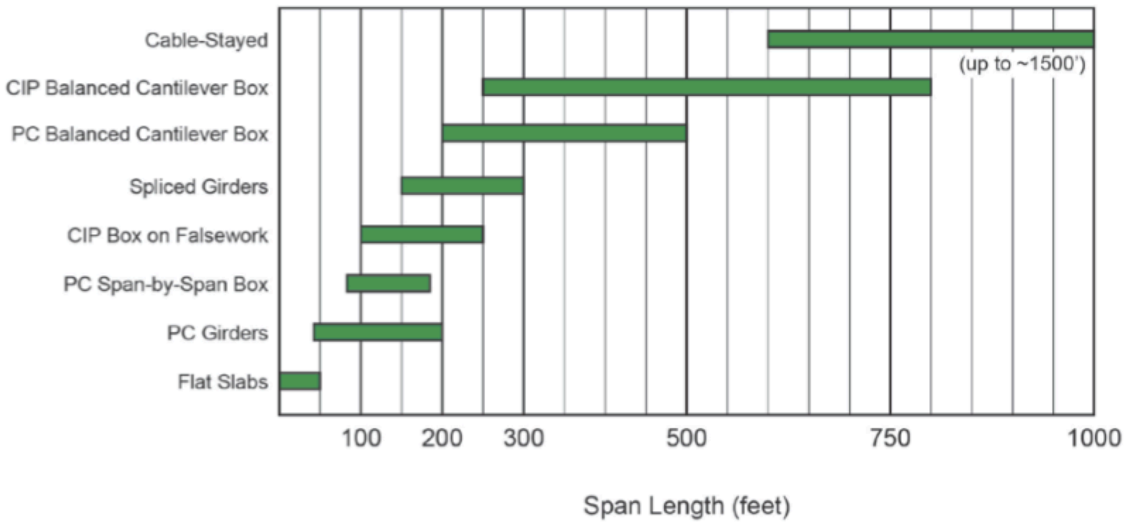

Figure 1: Typical Span Lengths

  - Since the model used for NCHRP 12-103 determined that shorter spans control design of CIP Multi-Cell Box (MCB) Girder bridges, a practical minimum span length of 100 feet is recommended.
  - For the design of this two-span continuous bridge each span equals 100 feet.

- Width

  - The width of the bridge is determined based on the number and width of traffic lanes provided, the width of shoulders, and width of other features, such as pedestrian sidewalks, bicycle lanes, or observation overhangs.
  - In this design, the simplest practical design includes two 12-foot traffic lanes and two 10-foot shoulders, and Arizona DOT Standard F-Shape barriers (1'-7" width) for a total bridge width of 47'-2".
  - As permissible, the outside safety barriers are located on the outermost edge on each side of the bridge, along the entire bridge length.

Table 72: Web Bend-Buckling Resistance - Negative Bending

Web Bend-Buckling Resistance - Negative Bending (6.10.1.9.1)			
D_c	23.974	in	
k	86.246		
F_{cr}	159.642	ksi	
F_{cr} USE	50.000	ksi	OK
f_{bu}	-12.200	ksi	

- In checking the strength limit state of composite sections in negative flexure, Article 6.11.6.2.3 directs the Engineer to Article 6.11.8. Furthermore, Article 6.11.6.2.3 states the provisions of Appendix A shall not apply, nor is redistribution of negative moment per Appendix B.
- At the strength limit state, tub flanges (bottom flanges) in compression shall satisfy:

$$f_{bu} \leq \phi_f F_{nc}$$

- At the Strength Limit State, the top flanges in tension continuously braced by the deck, shall satisfy:

$$f_{bu} \leq \phi_f F_{nt}$$

- Determine the nominal flexural resistance of the bottom flange in compression, Fnc, in accordance with Article 6.11.8.2.
- The results of the aforementioned process are shown below:

Table 73: Strength Limit State Check - Negative Bending

Strength Limit State - Negative Bending			
f_{bu} (top flange)	33.286	ksi	
f_{bu} (bot. flange)	-21.734	ksi	
D_c	29.575	in	
$2D_c/t_w$	94.640		
λ_{rw}	137.274		
R_b	1.0		
a_{wc}	0.2071		
F_{nc}	49.0703535	ksi	OK
F_{nt}	50.000	ksi	OK

- Thus, the negative bending region criteria were satisfied.

Table 70: Check Flange Stress - Negative Bending

Check Flange Stresses - Negative Bending		
$f_{bu\ (DC1)}$ (top flange)	22.883	ksi
$f_{bu\ (DC1)}$ (bottom flange)	-11.334	ksi
$f_{bu\ (CLL)}$ (top flange)	1.749	ksi
$f_{bu\ (CLL)}$ (bottom flange)	-0.866	ksi
f_{bu} (top flange)	24.632	ksi
f_{bu} (bottom flange)	-12.200	ksi

- For the flexural resistance of unstiffened flanges in compression, calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2. In computing F_{nc} for constructability, the web load-shedding factor, R_b, shall be taken as 1.0.

Table 71: Constructability Check - Negative Bending

Constructability Checks - Negative Bending			
$f_{bu}+f_{lat}$ (6.10.3.2.2-1)	27.184	ksi	OK
Ratio	0.5437		
Bottom Flange - Unstiffened Flange (6.11.8.2.2)			
λ_f	29.143		
k	4.00		
k_s	5.34		
f_v	0.0		
Δ	1.0		
λ_p	27.45483564		
F_{yr}	35	ksi	
λ_r	54.6913417		
F_{cb}	49.0703535	ksi	
$1.12\sqrt{(E^*k_s)/F_{yc}}$	62.33079239		
$1.40\sqrt{(E^*k_s)/F_{yc}}$	77.91349049		
F_{cv}	29	ksi	
F_{nc}	49.0703535	ksi	OK
Ratio	0.248632101		

- The web bend-buckling resistance shall be compared with the maximum compressive stress in the bottom flange. Determine the nominal elastic web bend-buckling resistance over the girder over the pier according to the provisions of Article 6.10.1.9.1.

- Thus:

Table 67: Plastic Dimensions - Positive Bending

Plastic Dimensions (Composite)	
Distance from Top of Deck to PNA, D_p (in.)	13.700
Total Depth of Composite Section, D_t (in.)	88.000
Depth of Web in Comp. at Plastic Moment, D_{cp} (in.)	0.000
Depth of Web in Comp. in Elastic Range, D_c (in.)	50.241

Table 68: Nominal Moment Capacity - Compact Sections (Positive Bending)

Nominal Moment Capacity - Compact Sections (6.10.7.1)	
Ratio of D_p/D_t	0.156
M_n if $D_p/D_t <= 0.1$	N/A
M_n if $D_p/D_t > 0.1$	33712.99

Table 69: Flexural Capacity - Positive Bending

Check Flexural Capacity - Positive Bending			
M_n	33712.99	ft-k	
M_u	16726.17	ft-k	OK
Ratio	0.4961		

<u>Check Flexure and Constructability – Negative Bending</u>

- The bottom flange, in regions of negative flexure, shall satisfy the requirements of Eqs. 6.11.3.2-1 and 6.11.3.2-2 for critical stages of construction. Generally these provisions will not control because the size of the bottom flange in negative flexure regions is normally governed by the Strength Limit State. In regard to construction loads, the maximum negative moment reached during the deck-placement analysis, plus the moment due to the self-weight, typically do not differ significantly from the calculated DC_1 negative moments assuming a single stage deck pour.
- The deck pour sequence and the application of wind loads are not considered in this example. It is assumed for this design that the application of the concrete deck occurs all at once for the purpose of the constructability checks.

10)The section satisfies the following web-slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$

Table 66: Compactness Check - Positive Bending

Check Compactness in Composite Condition (Output is 1 for Yes, 0 for No)	
Is the yield stress of the compression flange less than 70ksi?	1
Is the yield stress of the tension flange less than 70ksi?	1
Is $2D_{cp}/t_w$ less than or equal to $3.76\sqrt{E/F_y}$?	1
Is D/t_w less than or equal to 150?	1
Is the girder straight (not curved)?	1
Does section meet all compactness criteria?	Yes

- At the strength limit state, compact composite sections in positive flexure must satisfy the provisions of Article 6.11.7.1.
- The nominal flexural resistance of the section shall be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance shall always be subject to the limitation of Eq. (6.10.7.1.2-3). According to the provisions of Article 6.10.7.1.2, the nominal flexural resistance of compact composite sections in positive flexure is determined as follows:

$$M_n = M_p \; when \; D_p \leq 0.1D_t$$

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right) \; for \; all \; other \; cases$$

- In continuous spans, the nominal flexural resistance of the section is also limited to the following:

$$M_n \leq 1.3R_hM_y$$

- With the exception of composite sections in positive flexure in which the web satisfies the requirement of Articles 6.11.2.1.2 and 6.10.2.1.1 ($D/t_w \leq 150$), web bend-buckling of all sections under the SERVICE II load combination is to be checked as follows:

$$f_c \leq F_{crw}$$

- The term fc is the compression-flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because Section 2-2 is a composite section subject to positive flexure satisfying Article 6.11.2.1.2, Eq. (6.10.4.2.2-4) need not be checked. An explanation as to why these particular sections are exempt from the above web bend-buckling check is given in Article C6.10.1.9.1.
- It should be noted that in accordance with Article 6.11.4 redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous span flexural members using the procedures specified in Appendix B shall not apply to tub girder sections. The applicability of the Appendix B provisions to tub girder sections has not been demonstrated; hence the procedures are not permitted for the design of tub girder sections.

Table 65: Service Limit State Check - Positive Bending

Service Limit State Check - Positive Bending			
Top Flange			
$0.95 R_h F_{yf}$	47.5	ksi	
f_f	13.192	ksi	OK
Ratio	0.2777		
Bottom Flange			
f_l	0	ksi	
$f_f+(1/2) f_l$	9.900	ksi	OK
Ratio	0.2084		

- Determine if Section 2-2 qualifies as a compact section. According to Article 6.11.6.2.2, composite sections in positive flexure qualify as compact when:

6) The specified minimum yield strengths of the flanges and web do not exceed 70 ksi
7) The web satisfies the requirement of Article 6.11.2.1.2 such that longitudinal stiffeners are not required (i.e. D/tw £ _150)
8) The section is part of a bridge that satisfies the requirements of Article 6.11.2.3 (Special Restrictions for use of live load distribution factors)
9) The tub flange (bottom flange) is fully effective as specified in Article 6.11.1.1 (i.e. bottom flange bf less than one-fifth effective span)

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2}$$

- The term f_v is the St. Venant torsional shear stress in the flange due to factored loads at the section under consideration. However, in accordance with Article C6.11.2.3, if the provisions of Article 6.11.2.3 are satisfied, shear due to St. Venant torsion and secondary distortional bending stress effects may be neglected if the width of the tub flange does not exceed one-fifth the effective span defined in Article 6.11.1.1. For continuous spans, the effective span length is to be taken as the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. Since the above conditions were satisfied, f_v=0 in this case.

Table 64: Bottom Flange Constructability Check - Positive Bending

Bottom Flange Constructability Checks (6.11.3.2)			
f_v	0.0	ksi	
Δ	1.0		
f_{bu}	6.862	ksi	
R_h*F_{yf}*Δ	50	ksi	OK

- The service limit state was then checked for positive moment. Article 6.11.4 directs the Engineer to Article 6.10.4, which contains provisions related to the control of elastic and permanent deformations at the Service Limit State. For the sake of brevity, only the calculations pertaining to permanent deformations will be presented for this example.
- Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the SERVICE II load combination.
- Article 6.10.4.2.2 requires that, flanges of composite sections must satisfy AASHTO Equations 6.10.4.2.2-1 and 6.10.4.2.2-2.

Top Flange: $f_f \leq 0.95 R_h F_{yf}$

Bottom Flange: $f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf}$

- The term f_f is the flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending. The fl _term, the flange lateral bending stress, in Eq. (6.10.4.2.2-2) shall be taken equal to zero, in accordance with Article 6.11.4. A resistance factor is not included in these equations because Article 1.3.2.1 specifies that the resistance factor be taken equal to 1.0 at the service limit state.

- Lateral torsional buckling resistance was also considered, as per AASHTO Article 6.10.8.2.3.

Table 61: Lateral Torsional Buckling Resistance - Positive Bending

Top Flange Lateral Torsional Buckling Resistance (6.10.8.2.3)		
L_r	33.925	ft.
F_{nc} (6.10.8.2.3)	46.405	ksi
R_h*R_b*F_{yc}	50.000	ksi
F_{nc}	46.405	ksi
F_{nc} (governing)	46.405	ksi

- Web bend-buckling resistance was also considered, as per AASHTO Article 6.10.1.9.1.

Table 62: Web Bend-Buckling Resistance - Positive Bending

Web Bend Buckling Resistance (6.10.1.9)		
k	19.639	
F_{crw}	36.351	ksi
F_{crw} Limit	50.000	ksi
F_{crw} USE	36.351	ksi

- Since all the criteria were determined, the top flange constructability checks were made according to AASHTO Article 6.10.3.2.1:

Table 63: Top Flange Constructability Check - Positive Bending

Constructability Checks - Positive Bending			
Top Flange Constructability Checks (6.10.3.2.1)			
f_{bu}+f_{LAT} (6.10.3.2.1-1)	16.405	ksi	OK
Ratio	0.3281		
f_{bu}+(1/3)*f_{LAT} (6.10.3.2.1-2)	14.704	ksi	OK
Ratio	0.3169		
$f_{bu} < F_{crw}$	13.8532562	ksi	OK
Ratio	0.3811		

- Non-composite tub flanges (bottom flanges) in tension, must satisfy AASHTO Eq. 6.11.3.2-3:

$$f_{bu} \leq \phi_f R_h F_{yf} \Delta$$

Table 59: Top Flange Lateral Bending Amplification

Top Flange Lateral Bending Amplification		
Depth of Web in Compression, D_c	50.241	in.
r_t	4.502	in.
L_p	9.035	ft.
C_b (conservative)	1.0	
L_b (Assumed)	15.000	ft.
L_b (Limit) 6.10.1.6-2	20.597	ft.
F_{cr}	179.027	ksi
Amplification Factor, AF	1.000	OK
f_{LAT}, including AF	4.122	ksi
Stress Limit (6.10.1.6-1)	30.000	ksi

- Article 6.11.3.2 directs the engineer to the provisions of Article 6.10.3.2 for top flange constructability checks. Article 6.10.3.2.1 requires that discretely braced flanges in compression satisfy the following requirements, except that for slender-web sections, Eq. (6.10.3.2.1-1) need not be checked when f_l is equal to zero.

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$

$$f_{bu} \leq \phi_f F_{crw}$$

- Article 6.11.3.2 requires that the non-composite tub flange (bottom flange) in tension satisfy:

$$f_{bu} \leq \phi_f R_h F_{yt} \Delta$$

Table 60: Local Buckling Resistance Check - Positive Bending

$2D_c/t_w$	160.773	SLENDER
$5.7\sqrt{(E/F_{yc})}$	137.274	
Top Flange Local Buckling Resistance		
λ_f	5.143	
λ_{pf}	9.152	NONCOMPACT
λ_{rf}	13.487	
F_{yr}	35.000	ksi
F_{nc}	63.871	ksi

- Construction loads, or dead loads and temporary loads that act on the overhang only during construction, were assumed as follows.

Table 58: Top Flange Lateral Bending due to Deck Overhang Loads

Top Flange Lateral Bending due to Deck Overhang Loads		
Deck Overhang Weight	286.328	lb/ft
Overhang Deck Forms	40.000	lb/ft
Screed Rail	85.000	lb/ft
Railing	25.000	lb/ft
Walkway	125.000	lb/ft
Finishing Machine	3000.000	lb
F_{LAT}/P	0.917	
Strength I - Dead Loads		
P	770.410	lb/ft
F_{LAT}	706.209	lb/ft
M_{LAT}	13.241	k-ft
Finishing Machine		
P	4500.000	lb
F_{LAT}	4125.000	lb
M_{LAT}	7.734	ft-k
f_{LAT} (top flange)	0.982	ksi
Deck Overhang Total	2.552	ksi

- Top flange lateral bending amplification was also considered in this design. As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, the stress, fl, is to be determined as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. For design checks where the flexural resistance is based on yielding or flange local buckling, f_l may be determined as the stress at the section under consideration. For simplicity in this example, the largest value of f_l within the unbraced length will conservatively be used in all design checks. f_l was taken as positive in sign in all resistance equations. AASHTO Article 6.10.1.6 was used to determine the stress.

consideration. The brace points were assumed to be located at intervals of 0.1L=15 ft., as previously stated.

- In addition to the applied steel, permanent metal deck forms, and concrete self-weight loads, it is pertinent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in the load calculations section. In the STRENGTH I load combination; a load factor of 1.5 is applied to all construction loads, in accordance with Article 3.4.2.

Table 56: Check Flange Stress - Positive Bending

Check Flange Stresses - Positive Bending		
$f_{bu\,(DC1)}$ (top flange)	-12.870	ksi
$f_{bu\,(DC1)}$ (bottom flange)	6.374	ksi
$f_{bu\,(CLL)}$ (top flange)	-0.984	ksi
$f_{bu\,(CLL)}$ (bottom flange)	0.487	ksi
f_{bu} (top flange)	-13.853	ksi
f_{bu} (bottom flange)	6.862	ksi

- The change in the horizontal component of the web shear in the inclined web along the span acts as a lateral force in the flanges of the tub girder. Under initial non-composite dead load DC_1, the lateral force due to shear is assumed to be distributed to the top flanges of the open tub girder. To simplify the calculations for this example, it will conservatively be assumed that the entire DC_1 horizontal component of web shear is applied to the top flanges. The change in vertical shear force, equal to the lateral load on the top flanges, is constant and is equal to the change in DC_1 shear force in the girder measured at adjacent supports divided by the span length.

Table 57: Top Flange Lateral Bending due to Web Shear

Top Flange Lateral Bending due to Horizontal Component of Web Shear		
Change in DC_1 Girder Shear over Span	4.220	k/ft
Horizontal Component of Web Shear per Top Flange	0.528	k/ft
M_{LAT} (assuming bracing at 0.1L)	9.891	k-ft
Section Modulus of Top Flange	94.500	in.^{3}
f_{LAT}	1.570	ksi

- The top flange lateral bending due to overhang loads was also considered. Although the brackets are typically spaced at 3 to 4 feet along the exterior girder, all bracket loads except for the finishing machine load are assumed to be applied uniformly. For this example, the bracket is assumed to extend near the edge of the deck overhang. Therefore it is assumed that half the deck overhang weight is placed on the exterior girder web and half the weight is placed on the overhang brackets. Conservatively, one-half the deck haunch weight was included in the total overhang weight.

Table 54: Factored Loadings

Loadings	M_{D1} (in-k)	Positive Bending	80100.000
		Negative Bending	-142425.000
	M_{D2} (in-k)	Positive Bending	26573.700
		Negative Bending	-47253.000
	M_{LL+IM} (in-k)	Positive Bending	7836.693
		Negative Bending	-8839.845
	Yield Strength (ksi) for Goal Seek		50
	M_{AD} (Positive Bending)	Calc. M_{AD} (k-in) (Goal Seek)	644469.703
		M_{AD} (k-ft)	53705.809
	Yield Strength (ksi) for Goal Seek		50
	M_{AD} (Negative Bending)	Calc. M_{AD} (k-in) (Goal Seek)	-121518.872
		M_{AD} (k-ft)	-10126.573

- The yield moment of the composite section in positive bending is shown below. Also, M_{yt} in negative bending was taken about the tension flange since mild steel reinforcement was not considered.

Table 55: Yield Moments

Yield Moment (0.4L) - Positive Bending		
M_y	62595.283	ft-k
Yield Moment (1.0L) - Negative Bending		
M_y	-25933.073	ft-k

Girder Constructability Check – Positive Moment Region

- Article 6.11.3 directs the engineer to Article 6.10.3 for the constructability checks of tub girders. For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 shall be applied to the top flanges of the tub girder. The non-composite bottom tub flange in compression or tension shall satisfy requirements specified in Article 6.11.3.2
- Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and the assumed full deck-placement (DC_1). As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, f_{bu} is to be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. For design checks where the flexural resistance is based on yielding, flange local buckling or web bend buckling, f_{bu} may be determined as the stress at the section under

Table 51: Component Forces - Positive Bending

Component Forces & Details – Positive Bending	
Top Flange, P_c (kips)	3150.000
Web, P_w (kips)	4638.494
Bottom Flange, P_t (kips)	8925.000
Slab, P_s (kips)	7894.800
Top Reinf., P_{rt} (kips)	0.000
Bottom Reinf., P_{rb} (kips)	0.000

Table 52: Location of Component Forces - Positive Bending

Location of Component Forces – Positive Bending	
P_s (from PNA, in.)	8.700
P_t (from PNA, in.)	72.175
P_w (from PNA, in.)	35.300
P_c (from PNA, in.)	20.423

Table 53: Plastic Moment Capacity - Positive Bending

Plastic Moment Capacity	
878109.69	k-in
73175.81	k-ft

- In composite bending, it was necessary to determine M_{AD}, which is the additional moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange, as per AASHTO Eq. D6.2.2-1:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

- The yield moment of the composite section in positive bending was determined using AASHTO Eq. D6.2.2-2:

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

- Similarly, M_{AD} and the yield moment of the composite section in negative bending were determined using AASHTO Eq. D6.2.2-1 and D6.2.2-2 with S_{LT} and S_{ST} representing the steel section only, neglecting the strength contribution of the concrete slab. These calculations were facilitated using the *Data > What If > Goal Seek* function in *Microsoft Excel*. The factored M_{D1}, M_{D2}, and M_{LL+IM} and *Goal Seek* results are tabulated below.

Effective Width of Concrete Deck

- As specified in Article 6.10.1.1.1e, the effective flange width is to be determined as specified in Article 4.6.2.6. The individual webs of the tub girder must be initially considered separately since one web is an exterior web and the other is an interior web. According to Article 4.6.2.6, for an exterior web, the effective flange width may be taken as one-half the effective width of the adjacent interior girder, plus the full width of the overhang.

Table 49: Effective Width of Concrete Deck

Effective Width of Concrete Deck		
$b_{eff.\ int.\ web}$	141.00	in.
$b_{eff.\ ext.\ web}$	117.00	in.
$b_{eff.\ girder}$	258.00	in.

Investigate Service Limit State

- The factored service force effects were not directly calculated, but were investigated in the design of the post-tensioning, deck, and overhang.

Investigate Strength Limit State

- The factored moments used to check the Strength I Limit State are displayed in Table 50.
-

Table 50: Factored Moments - Strength I Limit State

Factored Moments - Strength I		
M_+ (0.4 Span 1)	16,726.17	ft-k
M_- (Pier)	-24646.35	ft-k
M_+ (0.6 Span 2)	16,726.17	ft-k

Check Flexure and Constructability – Positive Bending

- The area of the concrete haunch was neglected in the calculation of girder section properties, but the height of the concrete haunch was considered.
- The unbraced length was assumed to be 0.1L = 15 ft. Conventional mild deck reinforcement, diaphragms, and stiffeners were not considered in this design.
- The plastic moment capacity of the girder was determined using AASHTO Article D6.1. The plastic neutral axis was calculated to be in the web. Thus, Case I in Table D6.1-1 was used to determine the plastic moment capacity and location of the plastic neutral axis.

HL-93 Design Truck (Long)			
Moment	60 ft	1743.00	ft-k
Moment	150 ft	-980.30	ft-k
Moment	240 ft	1743.00	ft-k
Shear	150 ft	64.26	k
Shear	150 ft	-64.26	k
HL-93 Dual Truck			
Moment	60 ft	3367.00	ft-k
Moment	150 ft	-3798.00	ft-k
Moment	240 ft	3367.00	ft-k
Shear	150 ft	154.30	k
Shear	150 ft	-154.30	k
HL-93 Design Tandem			
Moment	60 ft	2884.00	ft-k
Moment	150 ft	-2519.00	ft-k
Moment	240 ft	2884.00	ft-k
Shear	150 ft	109.50	k
Shear	150 ft	-109.50	k

- The maximum live load moment and shear effects, including dynamic load allowance and impact factor as applicable, are shown. *IES Visual Analysis* applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Live Load Distribution Factors

- Live loads are distributed to individual girders according to the approximate methods specified in AASHTO 4.6.2.2. The number of lanes, N_L, was assumed to equal two.

Table 48: Distribution Factors

Distribution Factors		
DF, 2 Lanes (Strength/Service)	1.113	Lanes
DF, 1 Lane (Fatigue)	0.900	Lanes
LL+IM (Strength)	1.33	
IM (Fatigue)	1.15	

Table 45: Construction Live Loads

Live Load			
CLL	Construction LL	0.430	k/ft
	Construction LL per Girder	0.215	k/ft

Table 46: Unfactored Dead Load Moments

Dead Load			
DC_1			
Moment	60 ft	5340.00	ft-k
Moment	150 ft	-9495.00	ft-k
Moment	240 ft	5340.00	ft-k
Shear	150 ft	316.50	k
Shear	150 ft	-316.50	k
DC_2			
Moment	60 ft	822.50	ft-k
Moment	150 ft	-1463.00	ft-k
Moment	240 ft	822.50	ft-k
Shear	150 ft	48.75	k
Shear	150 ft	-48.75	k
DW			
Moment	60 ft	790.90	ft-k
Moment	150 ft	-1406.00	ft-k
Moment	240 ft	790.90	ft-k
Shear	150 ft	46.88	k
Shear	150 ft	-46.88	k

Table 47: Unfactored Live Load Shear and Moment Effects

Shear and Moment Effects from Visual Analysis			
HL-93 Design Truck (Short)			
Load Type	Location	Magnitude	Units
Moment	60 ft	1962.00	ft-k
Moment	150 ft	-1021.00	ft-k
Moment	240 ft	1962.00	ft-k
Shear	150 ft	69.16	k
Shear	150 ft	-69.46	k

Table 43: Load Modification Factors

Modification Factors	
Ductility	1.0
Redundancy	1.0
Operational Importance	1.0

Select Applicable Load Combinations and Load Factors

- The applicable load combinations and load factors were determined using AASHTO Table 3.4.1-1 and 3.4.1-2.
- $\gamma_{DC} = 1.25$
- $\gamma_{DW} = 1.50$
- $\gamma_{LL} = 1.75$

$$M_u = \gamma_{DC} M_{DC} + \gamma_{DW} M_{DW} + \gamma_{LL} M_{LL}$$

$$V_u = \gamma_{DC} V_{DC} + \gamma_{DW} V_{DW} + \gamma_{LL} V_{LL}$$

Calculate Force Effects

- For the previously defined cross-section, *IES Visual Analysis* software determined magnitude and location of the unfactored moments. The dead loads, construction live load, and results from the HL-93 vehicular design load are shown below.

Table 44: Dead Loads

Dead Loads			
DC_1	Deck	5.106	k/ft
	Overhang Tapers	0.163	k/ft
	Haunches	0.197	k/ft
	Stay-in-Place Forms	0.458	k/ft
	Girder Self-Weight	0.826	k/ft
	Cross Frames & Details	0.110	k/ft
DC_1 Total (per Two Girders)		6.859	k/ft
DC_1 Load (per One Girder)		3.430	k/ft
DC_2	Barrier	1.040	k/ft
DC_2 per One Girder		0.520	k/ft
DW	FWS	1.000	k/ft
DW per One Girder		0.500	k/ft

- Additionally, the moment of inertia of each inclined web was determined:

Table 40: Moment of Inertia of Inclined Web

Moment of Inertia of Inclined Web		
I_{ow}	20038.29	in.^{4}

- The bridge properties are summarized in the table below:

Table 41: Bridge Properties

Bridge Properties		
Number of Girders	2	
Top Flange Spacing	11.50	ft
Adjacent Flange Spacing	12.00	ft
Overhang Length	4.00	ft
Roadway Width	40.00	ft
Barrier Width	1.50	ft
Out-to-out Width	43.00	ft
Deck Thickness	9.00	in
FWS Thickness	0.50	in
Total Deck Thickness	9.50	in
Haunch Thickness	3.50	in
Haunch Width	18.00	in

- The weight of bridge components is tabulated below:

Table 42: Weight of Bridge Components

Weight of Components		
Weight of Parapets	0.520	klf
Weight of FWS	0.025	ksf
Weight of Conc. Deck	0.150	kcf
Weight of SIP Forms	0.015	kcf
Weight of Steel	0.490	kcf
Weight of Cross Frames & Details	0.110	k/ft

<u>Select Resistance Factors</u> – Strength I Limit State

- Applicable resistance factors were determined using AASHTO Article 6.5.4.2.

<u>Select Load Modifiers</u>

- The applicable load modification factors were determined based on the guidelines of AASHTO Articles 1.3.3 through 1.3.5. Table 43 displays the modification factors used in this design.

- The steel cross-section properties are summarized in the table below:

Table 37: Steel Cross-Section Properties

Girder	Top Flange	b_f (in.)	18.000
		t_f (in.)	1.750
		F_{yc} (ksi)	50.000
	Web	D (in.)	72.000
		t_w (in.)	0.6250
		F_{yw} (ksi)	50.000
	Bottom Flange	b_f (in.)	102.000
		t_f (in.)	1.750
		F_{yt} (ksi)	50.000
	Hybrid Girder Factor	R_h	1.000
	Steel Modulus	E_s (ksi)	29000.000
	S (Ratio of Vertical to Horizontal)		4
	Length of Inclined Web		74.22
	Horizontal Between Flanges		18.000

- The slab and haunch properties are summarized in the table below:

Table 38: Slab and Haunch Properties

Slab	Structural Thickness (in.)	9.000
	w_c (lb/ft^3)	150.000
	f'_c (ksi)	4.000
	E_c (ksi)	3834.254
Haunch	Thickness of Haunch (in.)	3.500

- The section properties for the non-composite, *n* composite, and *3n* composite sections are tabulated below:

Table 39: Section Properties for Non-composite, Short-term Composite, and Long-term Composite Sections

Component	*I_x (in.^{4})*	*S_{top} (in.^{3})*	*S_{bottom} (in.^{3})*
Girder Only	314255.246	6223.937	12565.882
Composite n	931668.788	33948.07	15409.40
Composite 3n	556482.671	14739.39	14743.13

- Web Thickness
  - The webs of the girders were proportioned such that no longitudinal stiffeners were required, based on AASHTO Eq. 6.11.2.1.2-1:

$$\frac{D}{t_w} \leq 150$$

- Top Flange Width
  - The minimum width of the top flange was determined by AASHTO Eq. 6.11.2.2-2:

$$b_f \geq \frac{D}{6}$$

  - However, Article C6.10.3.4 suggested additional criteria to determine flange width, where *L* is the length of the girder:

$$b_f \geq \frac{L}{85}$$

- Flange Thickness
  - The minimum flange thickness guideline adheres to AASHTO Eq. 6.11.2.2-3:

$$t_f \geq 1.1t_w$$

  - However, AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design and Constructibility* recommend a minimum flange thickness of 0.75 inches to enhance girder stability during handling and erection.

  - The top flange must also satisfy AASHTO Eq. 6.11.2.2-1:

$$\frac{b_f}{2t_f} \leq 12.0$$

- Deck Thickness
  - The total thickness of the cast-in-place concrete deck is 9.5 inches, including a 0.5-inch thick integral wearing surface.

- Concrete Deck Haunches
  - The concrete deck haunch is 3.5 inches deep measured from the top of the web to the bottom of the deck. The width of the deck haunch is assumed to be 18.0 inches.

Open Steel Box

Limit States to be Satisfied:

- Strength I
- Service II
- Any additional limit states, as required

Design Parameters

- Span length
  - Typical span lengths for tub girder bridges range from 150 feet to 500 feet.
  - Since the model used for NCHRP 12-103 determined that shorter spans control design of tub girder bridges, a practical minimum span length of 150 feet is recommended.
  - For the design of this two-span continuous bridge each span equals 150 feet.

- Width
  - The width of the bridge is determined based on the number and width of traffic lanes provided, the width of shoulders, and width of other features, such as pedestrian sidewalks, bicycle lanes, or observation overhangs.
  - The bridge cross-section consists of two trapezoidal tub girders with top flanges spaced at 11.5 ft. on center, 12.0 ft. between the centerline of adjacent top tub flanges, and 4.0 ft. overhangs for a deck width of 43.0 ft., out-to-out.

- Skew
  - Skew is not considered in this design, but may be investigated in future work.

- Depth
  - The minimum recommended depth of tub girders is 5 ft., or L/25. For this design, the depth of the section equals 150'/25 = 6 ft.

- Width and Thickness of Overhang
  - If empirical live load distribution factors are to be employed, the final cross-section must meet the requirements of Article 6.11.2.3, which states that the deck overhang should not exceed 60 percent of the distance between centers of the top flanges of adjacent tub girders, or 6.0 feet. Also, the distance center-to-center of adjacent tub girders shall not be greater than 120 percent, nor less than 80 percent, of the top flange center-to-center distance of a single tub girder.
  - The deck overhangs are 33 percent of the adjacent tub girder spacing.
- Deck Parapets
  - Deck parapets are each assumed to weigh 520 pounds per linear foot, and rest on the outer edge of the roadway, parallel to the direction of traffic.

bend-buckling resistance over the girder over the pier according to the provisions of Article 6.10.1.9.1.

Table 35: Web Bend-Buckling Resistance - Negative Bending

Web Bend-Buckling Resistance - Negative Bending (6.10.1.9.1)			
D_c	36.000	in	
k	36.000		
F_{cr}	70.801	ksi	
F_{cr} USE	50.000	ksi	OK
f_{bu}	-23.862	ksi	

- In checking the strength limit state of composite sections in negative flexure, Article 6.11.6.2.3 directs the Engineer to Article 6.11.8. Furthermore, Article 6.11.6.2.3 states the provisions of Appendix A shall not apply, nor is redistribution of negative moment per Appendix B.
- At the strength limit state, box flanges (bottom flanges) in compression shall satisfy:

$$f_{bu} \leq \phi_f F_{nc}$$

- At the Strength Limit State, the top flanges in tension continuously braced by the deck, shall satisfy:

$$f_{bu} \leq \phi_f F_{nt}$$

- Determine the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2.
- The results of the aforementioned process are shown below:

Table 36: Strength Limit State Check - Negative Bending

Strength Limit State - Negative Bending			
f_{bu} (top flange)	32.179	ksi	
f_{bu} (bot. flange)	-35.717	ksi	
D_c	37.251	in	
$2D_c/t_w$	119.203		
λ_{rw}	137.274		
R_b	1.0		
a_{wc}	0.3512		
F_{nc}	50	ksi	OK
F_{nt}	50.000	ksi	OK

- Thus, the negative bending region criteria were satisfied.

- The deck pour sequence and the application of wind loads are not considered in this example. It is assumed for this design that the application of the concrete deck occurs all at once for the purpose of the constructability checks.

Table 33: Check Flange Stress - Negative Bending

Check Flange Stresses - Negative Bending		
$f_{bu\_DC\_1}$ (top flange)	23.290	ksi
$f_{bu\_DC\_1}$ (bottom flange)	-23.290	ksi
f_{bu_CLL} (top flange)	0.572	ksi
f_{bu_CLL} (bottom flange)	-0.572	ksi
f_{bu} (top flange)	23.862	ksi
f_{bu} (bottom flange)	-23.862	ksi

- For the flexural resistance of unstiffened flanges in compression, calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2. In computing F_{nc} for constructability, the web load-shedding factor, R_b, shall be taken as 1.0.

Table 34: Constructability Check - Negative Bending

Constructability Checks - Negative Bending			
$f_{bu} + f_{lat}$ (6.10.3.2.2-1)	24.696	ksi	OK
Ratio	0.4939		
Bottom Flange - Unstiffened Flange (6.11.8.2.2)			
λ_f	11.753		
k	4.00		
k_s	5.34		
f_v	0.0		
Δ	1.0		
λ_p	27.45483564		
F_{yr}	35	ksi	
λ_r	54.6913417		
F_{cb}	50	ksi	
$1.12*\sqrt{((E*k_s)/F_{yc})}$	62.33079239		
$1.40*\sqrt{((E*k_s)/F_{yc})}$	77.91349049		
F_{cv}	29	ksi	
F_{nc}	50	ksi	OK
Ratio	0.477239875		

- The web bend-buckling resistance shall be compared with the maximum compressive stress in the bottom flange. Determine the nominal elastic web follows:

$$M_n = M_p \; when \; D_p \leq 0.1 D_t$$

$$M_n = M_p \left(1.07 - 0.7 \frac{D_p}{D_t}\right) \; for \; all \; other \; cases$$

- In continuous spans, the nominal flexural resistance of the section is also limited to the following:

$$M_n \leq 1.3 R_h M_y$$

- Thus:

Table 30: Plastic Dimensions - Positive Bending

Plastic Dimensions (Composite)	
Distance from Top of Deck to PNA, D_p (in.)	12.927
Total Depth of Composite Section, D_t (in.)	89.250
Depth of Web in Comp. at Plastic Moment, D_{cp} (in.)	0.000
Depth of Web in Comp. in Elastic Range, D_c (in.)	36.000

Table 31: Nominal Moment Capacity - Compact Sections (Positive Bending)

Nominal Moment Capacity - Compact Sections (6.10.7.1)	
Ratio of D_p/D_t	0.145
M_n if $D_p/D_t <= 0.1$	N/A
M_n if $D_p/D_t > 0.1$	51778.28

Table 32: Flexural Capacity - Positive Bending

Check Flexural Capacity - Positive Bending			
M_n	51778.28	ft-k	
M_u	16928.70	ft-k	OK
Ratio	0.3269		

<u>Check Flexure and Constructability – Negative Bending</u>

- The bottom flange, in regions of negative flexure, shall satisfy the requirements of Eqs. 6.11.3.2-1 and 6.11.3.2-2 for critical stages of construction. Generally these provisions will not control because the size of the bottom flange in negative flexure regions is normally governed by the Strength Limit State. In regard to construction loads, the maximum negative moment reached during the deck-placement analysis, plus the moment due to the self-weight, typically do not differ significantly from the calculated DC_1 negative moments assuming a single stage deck pour.

- It should be noted that in accordance with Article 6.11.4 redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous span flexural members using the procedures specified in Appendix B shall not apply to box girder sections. The applicability of the Appendix B provisions to box girder sections has not been demonstrated; hence the procedures are not permitted for the design of box girder sections.

- Determine if Section 2-2 qualifies as a compact section. According to Article 6.11.6.2.2, composite sections in positive flexure qualify as compact when:

1) The specified minimum yield strengths of the flanges and web do not exceed 70 ksi
2) The web satisfies the requirement of Article 6.11.2.1.2 such that longitudinal stiffeners are not required (i.e. D/tw £ _150)
3) The section is part of a bridge that satisfies the requirements of Article 6.11.2.3 (Special Restrictions for use of live load distribution factors)
4) The tub flange (bottom flange) is fully effective as specified in Article 6.11.1.1 (i.e. bottom flange bf less than one-fifth effective span)

5) The section satisfies the following web-slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$

Table 29: Compactness Check - Positive Bending

Check Compactness in Composite Condition (Output is 1 for Yes, 0 for No)	
Is the yield stress of the compression flange less than 70ksi?	1
Is the yield stress of the tension flange less than 70ksi?	1
Is $2D_{cp}/t_w$ less than or equal to $3.76\sqrt{E/F_y}$?	1
Is D/t_w less than or equal to 150?	1
Is the girder straight (not curved)?	1
Does section meet all compactness criteria?	Yes

- At the strength limit state, compact composite sections in positive flexure must satisfy the provisions of Article 6.11.7.1.
- The nominal flexural resistance of the section shall be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance shall always be subject to the limitation of Eq. (6.10.7.1.2-3). According to the provisions of Article 6.10.7.1.2, the nominal flexural resistance of compact composite sections in positive flexure is determined as

For the sake of brevity, only the calculations pertaining to permanent deformations will be presented for this design.

- Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the SERVICE II load combination.
- Article 6.10.4.2.2 requires that, flanges of composite sections must satisfy AASHTO Equations 6.10.4.2.2-1 and 6.10.4.2.2-2.

Top Flange: $f_f \leq 0.95 R_h F_{yf}$

Bottom Flange: $f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf}$

- The term f_f is the flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending. The f_l term, the flange lateral bending stress, in Eq. (6.10.4.2.2-2) shall be taken equal to zero, in accordance with Article 6.11.4. A resistance factor is not included in these equations because Article 1.3.2.1 specifies that the resistance factor be taken equal to 1.0 at the service limit state.
- With the exception of composite sections in positive flexure in which the web satisfies the requirement of Articles 6.11.2.1.2 and 6.10.2.1.1 ($D/t_w \leq 150$), web bend-buckling of all sections under the SERVICE II load combination is to be checked as follows:

$$f_c \leq F_{crw}$$

- The term f_c is the compression-flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because Section 2-2 is a composite section subject to positive flexure satisfying Article 6.11.2.1.2, Eq. (6.10.4.2.2-4) need not be checked. An explanation as to why these particular sections are exempt from the above web bend-buckling check is given in Article C6.10.1.9.1.

Table 28: Service Limit State Check - Positive Bending

Service Limit State Check - Positive Bending			
Top Flange			
$0.95*R_h*F_{yf}$	47.5	ksi	
f_f	12.667	ksi	OK
Ratio	0.2667		
Bottom Flange			
f_l	0	ksi	
$f_f+(1/2)*f_l$	14.795	ksi	OK
Ratio	0.3115		

Table 26: Top Flange Constructability Check - Positive Bending

Constructability Checks - Positive Bending			
Top Flange Constructability Checks (6.10.3.2.1)			
f_bu+f_lat (6.10.3.2.1-1)	14.254	ksi	OK
Ratio	0.2851		
f_bu+(1/3)f_lat (6.10.3.2.1-2)	13.698	ksi	OK
Ratio	0.3341		
f_bu < F_crw	13.4196568	ksi	OK
Ratio	0.2684		

- Non-composite box flanges (bottom flanges) in tension, must satisfy AASHTO Eq. 6.11.3.2-3:

$$f_{bu} \leq \phi_f R_h F_{yf} \Delta$$

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2}$$

- The term f_v is the St. Venant torsional shear stress in the flange due to factored loads at the section under consideration. However, in accordance with Article C6.11.2.3, if the provisions of Article 6.11.2.3 are satisfied, shear due to St. Venant torsion and secondary distortional bending stress effects may be neglected if the width of the box flange does not exceed one-fifth the effective span defined in Article 6.11.1.1. For continuous spans, the effective span length is to be taken as the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. Since the above conditions were satisfied, f_v=0 in this case.

Table 27: Bottom Flange Constructability Check - Positive Bending

Bottom Flange Constructability Checks (6.11.3.2)			
f_v	0.0	ksi	
Δ	1.0		
f_{bu}	13.420	ksi	
R_h*F_{yf}*Δ	50	ksi	OK

- The service limit state was then checked for positive moment. Article 6.11.4 directs the Engineer to Article 6.10.4, which contains provisions related to the control of elastic and permanent deformations at the Service Limit State.

Table 23: Local Buckling Resistance Check - Positive Bending

$2D_c/t_w$	115.200	NOT SLENDER
$5.7\sqrt{(E/F_{yc})}$	137.274	
Top Flange Local Buckling Resistance		
λ_f	11.753	
λ_{pf}	9.152	NONCOMPACT
λ_{rf}	13.487	
F_{yr}	35.000	ksi
F_{nc}	41.000	ksi

- Lateral torsional buckling resistance was also considered, as per AASHTO Article 6.10.8.2.3.

Table 24: Lateral Torsional Buckling Resistance - Positive Bending

Top Flange Lateral Torsional Buckling Resistance (6.10.8.2.3)		
L_r	118.147	ft.
F_{nc} (6.10.8.2.3)	43.332	ksi
R_h*R_b*F_{yc}	50.000	ksi
F_{nc}	43.332	ksi
F_{nc} (governing)	41.000	ksi

- Web bend-buckling resistance was also considered, as per AASHTO Article 6.10.1.9.1.

Table 25: Web Bend-Buckling Resistance - Positive Bending

Web Bend Buckling Resistance (6.10.1.9)		
k	36.000	
F_{crw}	70.801	ksi
F_{crw} Limit	50.000	ksi
F_{crw} USE	50.000	ksi

- Since all the criteria were determined, the top flange constructability checks were made according to AASHTO Article 6.10.3.2.1:

- Top flange lateral bending amplification was also considered in this design. As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, the stress, f_l, is to be determined as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. For design checks where the flexural resistance is based on yielding or flange local buckling, f_l may be determined as the stress at the section under consideration. For simplicity in this example, the largest value of f_l within the unbraced length will conservatively be used in all design checks. f_l was taken as positive in sign in all resistance equations. AASHTO Article 6.10.1.6 was used to determine the stress.

Table 22: Top Flange Lateral Bending Amplification

Top Flange Lateral Bending Amplification		
Depth of Web in Compression, D_c	36.000	in.
r_t	15.678	in.
L_p	31.465	ft.
C_b (conservative)	1.0	
L_b (Assumed)	70.000	ft.
L_b (Limit) 6.10.1.6-2	72.882	ft.
F_{cr}	99.705	ksi
Amplification Factor, AF	1.000	OK
f_{LAT}, including AF	1.586	ksi
Stress Limit (6.10.1.6-1)	30.000	ksi

- Article 6.11.3.2 directs the engineer to the provisions of Article 6.10.3.2 for top flange constructability checks. Article 6.10.3.2.1 requires that discretely braced flanges in compression satisfy the following requirements, except that for slender-web sections, Eq. (6.10.3.2.1-1) need not be checked when f_l is equal to zero.

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$

$$f_{bu} \leq \phi_f F_{crw}$$

- Article 6.11.3.2 requires that the non-composite tub flange (bottom flange) in tension satisfy:

$$f_{bu} \leq \phi_f R_h F_{yt} \Delta$$

Table 20: Top Flange Lateral Bending due to Web Shear

Top Flange Lateral Bending due to Horizontal Component of Web Shear		
Change in DC_1 Girder Shear over Span	6.596	k/ft
Horizontal Component of Web Shear per Top Flange	3.298	k/ft
M_{LAT} (assuming bracing at 70 ft.)	61.838	k-ft
Section Modulus of Top Flange	1233.587	in.^3
f_{LAT}	0.752	ksi

- The top flange lateral bending due to overhang loads was also considered. Although the brackets are typically spaced at 3 to 4 feet along the exterior girder, all bracket loads except for the finishing machine load are assumed to be applied uniformly. For this example, the bracket is assumed to extend near the edge of the deck overhang. Therefore it is assumed that half the deck overhang weight is placed on the exterior girder web and half the weight is placed on the overhang brackets. Conservatively, one-half the deck haunch weight was included in the total overhang weight.
- Construction loads, or dead loads and temporary loads that act on the overhang only during construction, were assumed as follows.

Table 21: Top Flange Lateral Bending due to Deck Overhang Loads

Top Flange Lateral Bending due to Deck Overhang Loads		
Deck Overhang Weight	418.529	lb/ft
Overhang Deck Forms	40.000	lb/ft
Screed Rail	85.000	lb/ft
Railing	25.000	lb/ft
Walkway	125.000	lb/ft
Finishing Machine	3000.000	lb
F_{LAT}/P	1.000	
Strength I - Dead Loads		
p	935.662	lb/ft
F_{LAT}	935.662	lb/ft
M_{LAT}	17.544	k-ft
Finishing Machine		
P	4500.000	lb
F_{LAT}	4500.000	lb
M_{LAT}	8.438	ft-k
f_{LAT} (top flange)	0.082	ksi
Deck Overhang Total	0.834	ksi

Girder Constructability Check – Positive Moment Region

- Article 6.11.3 directs the engineer to Article 6.10.3 for the constructability checks of tub girders. For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 shall be applied to the top flanges of the box girder. The non-composite bottom box flange in compression or tension shall satisfy requirements specified in Article 6.11.3.2
- Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and the assumed full deck-placement (DC_1). As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, f_{bu} is to be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. For design checks where the flexural resistance is based on yielding, flange local buckling or web bend buckling, f_{bu} may be determined as the stress at the section under consideration. The brace points were assumed to be located at intervals of 70 ft., as previously stated.
- In addition to the applied steel, permanent metal deck forms, and concrete self-weight loads, it is pertinent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in the load calculations section. In the STRENGTH I load combination; a load factor of 1.5 is applied to all construction loads, in accordance with Article 3.4.2.

Table 19: Check Flange Stress - Positive Bending

Check Flange Stresses - Positive Bending		
$f_{bu\_DC\_1}$ (top flange)	-13.098	ksi
$f_{bu\_DC\_1}$ (bottom flange)	13.098	ksi
f_{bu_CLL} (top flange)	-0.322	ksi
f_{bu_CLL} (bottom flange)	0.322	ksi
f_{bu} (top flange)	-13.420	ksi
f_{bu} (bottom flange)	13.420	ksi

- The change in the horizontal component of the web shear in the web along the span acts as a lateral force in the flanges of the box girder. Under initial non-composite dead load DC_1, the lateral force due to shear is assumed to be distributed to the top flanges of the closed box girder. To simplify the calculations for this example, it will conservatively be assumed that the entire DC_1 horizontal component of web shear is applied to the top flanges. The change in vertical shear force, equal to the lateral load on the top flanges, is constant and is equal to the change in DC_1 shear force in the girder measured at adjacent supports divided by the span length.

- In composite bending, it was necessary to determine M_{AD}, which is the additional moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange, as per AASHTO Eq. D6.2.2-1:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

- The yield moment of the composite section in positive bending was determined using AASHTO Eq. D6.2.2-2:

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

- Similarly, M_{AD} and the yield moment of the composite section in negative bending were determined using AASHTO Eq. D6.2.2-1 and D6.2.2-2 with S_{LT} and S_{ST} representing the steel section only, neglecting the strength contribution of the concrete slab. These calculations were facilitated using the *Data > What If > Goal Seek* function in *Microsoft Excel*. The factored M_{D1}, M_{D2}, and M_{LL+IM} and *Goal Seek* results are tabulated below.

Table 17: Factored Loadings

Loadings	M_{D1} (in-k)	Positive Bending	125205.000
		Negative Bending	-222630.000
	M_{D2} (in-k)	Positive Bending	13286.700
		Negative Bending	-23625.300
	M_{LL+IM} (in-k)	Positive Bending	7836.693
		Negative Bending	-8839.845
	Yield Strength (ksi) for Goal Seek		50
	M_{AD} (Positive Bending)	Calc. M_AD (k-in) (Goal Seek)	**388400.054**
		M_AD (k-ft)	**32366.671**
	Yield Strength (ksi) for Goal Seek		50
	M_{AD} (Negative Bending)	Calc. M_AD (k-in) (Goal Seek)	**-231698.056**
		M_AD (k-ft)	**-19308.171**

- The yield moment of the composite section in positive bending is shown below. Also, M_{yt} in negative bending was taken about the tension flange since mild steel reinforcement was not considered.

Table 18: Yield Moments

Yield Moment (0.4L) - Positive Bending		
M_y	43907.65	ft-k
Yield Moment (1.0L) - Negative Bending		
M_y	-39829.45	ft-k

Table 13: Factored Moments - Strength Limit State

Factored Moments - Strength I		
M_+ (0.4 Span 1)	16928.70	ft-k
M_- (Pier)	-26598.67	ft-k
M_+ (0.6 Span 2)	16928.70	ft-k

Check Flexure and Constructability – Positive Bending

- The area of the concrete haunch was neglected in the calculation of girder section properties, but the height of the concrete haunch was considered.
- The unbraced length was assumed to be 70 ft., based on the upper limit outlined in AASHTO 6.10.1.6-2. Conventional mild deck reinforcement, diaphragms, and stiffeners were not considered in this design.
- The plastic moment capacity of the girder was determined using AASHTO Article D6.1. The plastic neutral axis was calculated to be in the top flange. Thus, Case II in Table D6.1-1 was used to determine the plastic moment capacity and location of the plastic neutral axis.

Table 14: Component Forces - Positive Bending

Component Forces & Details	
Top Flange, P_c (kips)	6629.219
Web, P_w (kips)	4500.000
Bottom Flange, P_t (kips)	6629.219
Slab, P_s (kips)	4905.792

Table 15: Location of Component Forces - Positive Bending

Location of Component Forces	
P_s (from PNA)	7.927
P_t (from PNA)	73.260
P_w (from PNA)	36.073
P_c (from PNA)	1.115

Table 16: Plastic Moment Capacity - Positive Bending

Plastic Moment Capacity	
694279.73	k-in
57856.64	k-ft

- The maximum live load moment and shear effects, including dynamic load allowance and impact factor as applicable, are shown. *IES Visual Analysis* applied the HL-93 Design Load to the model (Design Truck, Design Tandem, Dual Truck, and Design Lane) such that the maximum effects were produced.

Live Load Distribution Factors

- Live loads are distributed to individual girders according to the approximate methods specified in AASHTO 4.6.2.2. The number of design lanes, N_L, was assumed to equal two.

Table 11: Distribution Factors

Distribution Factors		
DF, 2 Lanes (Strength/Service)	0.688	Lanes
DF, 1 Lane (Fatigue)	0.688	Lanes
LL+IM (Strength)	1.33	
IM (Fatigue)	1.15	

Effective Width of Concrete Deck

- As specified in Article 6.10.1.1.1e, the effective flange width is to be determined as specified in Article 4.6.2.6. The individual webs of the tub girder must be initially considered separately since one web is an exterior web and the other is an interior web. According to Article 4.6.2.6, for an exterior web, the effective flange width may be taken as one-half the effective width of the adjacent interior girder, plus the full width of the overhang.

Table 12: Effective Width of Concrete Deck

Effective Width of Concrete Deck		
$b_{eff.\ int.\ web}$	60.72	in.
$b_{eff.\ ext.\ web}$	99.60	in.
$b_{eff.\ girder}$	160.32	in.

Investigate Service Limit State

- The factored service force effects were not directly calculated, but were investigated in the design of the post-tensioning, deck, and overhang.

Investigate Strength Limit State

The factored moments used to check the Strength I Limit State are displayed in

- Table 13.

Table 10: Unfactored Live Load Shear and Moment Effects

Shear and Moment Effects from Visual Analysis			
HL-93 Design Truck (Short)			
Load Type	Location	Magnitude	Units
Moment	60 ft	1962.00	ft-k
Moment	150 ft	-1021.00	ft-k
Moment	240 ft	1962.00	ft-k
Shear	150 ft	69.16	k
Shear	150 ft	-69.46	k
HL-93 Design Truck (Long)			
Moment	60 ft	1743.00	ft-k
Moment	150 ft	-980.30	ft-k
Moment	240 ft	1743.00	ft-k
Shear	150 ft	64.26	k
Shear	150 ft	-64.26	k
HL-93 Dual Truck			
Moment	60 ft	3367.00	ft-k
Moment	150 ft	-3798.00	ft-k
Moment	240 ft	3367.00	ft-k
Shear	150 ft	154.30	k
Shear	150 ft	-154.30	k
HL-93 Design Tandem			
Moment	60 ft	2884.00	ft-k
Moment	150 ft	-2519.00	ft-k
Moment	240 ft	2884.00	ft-k
Shear	150 ft	109.50	k
Shear	150 ft	-109.50	k
Fatigue (HL-93)			
Moment	60 ft	1933.00	ft-k
Moment	150 ft	-1083.00	ft-k
Moment	240 ft	1933.00	ft-k
Shear	150 ft	70.33	k
Shear	150 ft	-70.33	k
Construction Live Load			
Moment	60 ft	170.80	ft-k
Moment	150 ft	-303.80	ft-k
Moment	240 ft	170.80	ft-k
Shear	150 ft	10.13	k
Shear	150 ft	-10.13	k

Table 9: Unfactored Dead Load Moments

Dead Load			
DC_1			
Moment	60 ft	8347.00	ft-k
Moment	150 ft	-14842.00	ft-k
Moment	240 ft	8347.00	ft-k
Shear	150 ft	494.70	k
Shear	150 ft	-494.70	k
DC_2			
Moment	60 ft	411.30	ft-k
Moment	150 ft	-731.30	ft-k
Moment	240 ft	411.30	ft-k
Shear	150 ft	24.38	k
Shear	150 ft	-24.38	k
DW			
Moment	60 ft	395.40	ft-k
Moment	150 ft	-703.10	ft-k
Moment	240 ft	395.40	ft-k
Shear	150 ft	23.44	k
Shear	150 ft	-23.44	k

Select Applicable Load Combinations and Load Factors

- The applicable load combinations and load factors were determined using AASHTO Table 3.4.1-1 and 3.4.1-2.
- $\gamma_{DC} = 1.25$
- $\gamma_{DW} = 1.50$
- $\gamma_{LL} = 1.75$

$$M_u = \gamma_{DC} M_{DC} + \gamma_{DW} M_{DW} + \gamma_{LL} M_{LL}$$

$$V_u = \gamma_{DC} V_{DC} + \gamma_{DW} V_{DW} + \gamma_{LL} V_{LL}$$

Calculate Force Effects

- For the previously defined cross-section, *IES Visual Analysis* software determined magnitude and location of the unfactored moments. The dead loads, construction live load, and results from the HL-93 vehicular design load are shown below.

Table 7: Dead Loads

Dead Loads			
DC_1	Deck	5.106	k/ft
	Overhang Tapers	1.150	k/ft
	Haunches	0.131	k/ft
	SIP Forms	0.081	k/ft
	Girder Self-Weight	14.471	k/ft
	Cross Frames & Details	0.110	k/ft
DC_1 Total (per 4 Girders)		21.049	k/ft
DC_1 Load per 1 Girder		5.262	k/ft
DC_2	Barrier	1.040	k/ft
DC_2 per 1 Girder		0.260	k/ft
DW	FWS	1.000	k/ft
DW per 1 Girder		0.250	k/ft

Table 8: Construction Live Loads

Live Load			
CLL	Construction LL	0.430	k/ft
	Construction LL per Girder	0.108	k/ft

Table 4: Bridge Properties

Bridge Properties		
Number of Girders	4	
Top Flange Spacing	4.60	ft
Adjacent Flange Spacing	5.520	ft
Overhang Length	4.00	ft
Roadway Width	40.00	ft
Barrier Width	1.50	ft
Out-to-out Width	43.00	ft
Deck Thickness	9.00	in
FWS Thickness	0.50	in
Total Deck Thickness	9.50	in
Haunch Thickness	3.50	in
Haunch Width	55.825	in

- The weight of bridge components is tabulated below:

Table 5: Weight of Bridge Components

Weight of Components		
Weight of Parapets	0.520	klf
Weight of FWS	0.025	ksf
Weight of Conc. Deck	0.150	kcf
Weight of SIP Forms	0.015	kcf
Weight of Steel	0.490	kcf
Weight of Cross Frames & Details	0.110	k/ft

<u>Select Resistance Factors</u> – Strength I Limit State

- Applicable resistance factors were determined using AASHTO Article 6.5.4.2.

<u>Select Load Modifiers</u>

- The applicable load modification factors were determined based on the guidelines of AASHTO Articles 1.3.3 through 1.3.5. Table 6 displays the modification factors used in this design.

Table 6: Load Modification Factors

Modification Factors	
Ductility	1.0
Redundancy	1.0
Operational Importance	1.0

- The steel cross-section properties are summarized in the table below:

Table 1: Steel Cross-Section Properties

Girder	Top Flange	b_f (in.)	55.825
		t_f (in.)	2.375
		F_{yc} (ksi)	50.000
	Web	D (in.)	72.000
		t_w (in.)	0.6250
		F_{yw} (ksi)	50.000
	Bottom Flange	b_f (in.)	55.825
		t_f (in.)	2.375
		F_{yt} (ksi)	50.000
	Hybrid Girder Factor	R_h	1.000
	Steel Modulus	E_s (ksi)	29000.000

- The slab and haunch properties are summarized in the table below:

Table 2: Slab and Haunch Properties

Slab	Structural Thickness (in.)	9.000
	w_c (lb/ft^3)	150.000
	f'_c (ksi)	4.000
	E_c (ksi)	3834.254
Haunch	Thickness of Haunch (in.)	3.500

- The section properties for the non-composite, *n* composite, and *3n* composite sections are tabulated below:

Table 3: Section Properties for Non-composite, Short-term Composite, and Long-term Composite Sections

Component	*I_x (in.^{4})*	*S_{top} (in.^{3})*	*S_{bottom} (in.^{3})*
Girder Only	366829.201	9559.067	9559.067
Composite n	670227.511	24219.36	10901.24
Composite 3n	467024.846	14592.13	10437.54

- The bridge properties are summarized in the table below. Note that the haunch width is cast to match the width of the top flanges.

$$\frac{D}{t_w} \leq 150$$

- Top Flange Width
  - The minimum width of the top flange was determined by AASHTO Eq. 6.11.2.2-2:

$$b_f \geq \frac{D}{6}$$

  - However, Article C6.10.3.4 suggested additional criteria to determine flange width, where *L* is the length of the girder:

$$b_f \geq \frac{L}{85}$$

- Flange Thickness
  - The minimum flange thickness guideline adheres to AASHTO Eq. 6.11.2.2-3:

$$t_f \geq 1.1t_w$$

  - However, AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design and Constructibility* recommend a minimum flange thickness of 0.75 inches to enhance girder stability during handling and erection.

  - The top flange must also satisfy AASHTO Eq. 6.11.2.2-1:

$$\frac{b_f}{2t_f} \leq 12.0$$

- Deck Thickness
  - The total thickness of the cast-in-place concrete deck is 9.5 inches, including a 0.5-inch thick integral wearing surface.

- Concrete Deck Haunches
  - The concrete deck haunch is 1-⅛ inches thick. The total haunch thickness, including the top flange, measures 3.5 inches.

Design Parameters

- Span length
  - Typical span lengths for closed steel box girder bridges range from 150 feet to 500 feet.
  - Since the model used for NCHRP 12-103 determined that shorter spans control design of closed steel box girder bridges, a practical minimum span length of 150 feet is recommended.
  - For the design of this two-span continuous bridge each span equals 150 feet.

- Width
  - The width of the bridge is determined based on the number and width of traffic lanes provided, the width of shoulders, and width of other features, such as pedestrian sidewalks, bicycle lanes, or observation overhangs.
  - The bridge cross-section consists of four rectangular steel girders with top flanges spaced at 4'-7 ¼" on center, 5'-6 ¼" between the centerline of adjacent top flanges, and 4' overhangs for a deck width of 43.0', out-to-out.

- Skew
  - Skew is not considered in this design, but may be investigated in future work.

- Depth
  - The minimum recommended depth of tub girders is 5 ft., or L/25. For this design, the depth of the section equals 150'/25 = 6 ft.

- Width and Thickness of Overhang

  - If empirical live load distribution factors are to be employed, the final cross-section must meet the requirements of Article 6.11.2.3, which states that the deck overhang should not exceed 60 percent of the distance between centers of the top flanges of adjacent box girders, or 6.0 feet. Also, the distance center-to-center of adjacent box girders shall not be greater than 120 percent, nor less than 80 percent, of the top flange center-to-center distance of a single box girder.

  - The deck overhang length is 72% percent of the adjacent box girder spacing.

- Deck Parapets

  - Deck parapets are each assumed to weigh 520 pounds per linear foot, and rest on the outer edge of the roadway, parallel to the direction of traffic.

- Web Thickness

  - The webs of the girders were proportioned such that no longitudinal stiffeners were required, based on AASHTO Eq. 6.11.2.1.2-1:

Task 2.4 – **Spot Checking Additional Bridge Types**

Overview
The following cross-sections were considered in Task 2.4:

1) *Closed Steel Boxes*
2) *Open Steel boxes*
3) *Prestressed Concrete Boxes with Integral Deck*
   a) *Prestressed Concrete Boxes*
   b) *Precast Solid, Voided, or Cellular Concrete Boxes with an Integral Concrete Deck*
   c) *Cast-in-Place Concrete Multi-cell Box*
4) *Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys with Cast-in-Place Concrete Overlay*
5) *Open Precast Concrete Boxes*
6) *Various Precast Concrete "Multi-Girder" Bridges*
   a) *Precast Concrete Channel Sections with Shear Keys*
   b) *Precast Concrete Double Tee Sections with Shear Keys*
   c) *Precast Concrete Tee Sections with Shear Keys*
7) *Cast-in-Place Concrete Tee Beams*
8) *Wood Beams*

The following cross-sections are commonly made continuous for live load, and thus have been omitted from Task 2.4 to date: Prestressed Concrete Boxes; Precast Solid, Voided, or Cellular Concrete Boxes with an Integral Concrete Deck; Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys with Cast-in-Place Concrete Overlay; Open Precast Concrete Boxes; Cast-in-Place Tee Beams; and Wood Beams.

Additional efforts will be made to formally declare these cross-sections as made continuous for live load.

Completed Designs
The researchers at University of Delaware designed the following cross-sections in a concerted effort with the researchers at Rutgers University: Closed Steel Boxes, Open Steel Boxes, and Cast-in-Place Multi-Cell Box. Two designs were completed for the Cast-in-Place Multi-Cell Box section to present results with bridges of different span lengths.

Closed Steel Boxes
<u>Limit States to be Satisfied</u>:
- Strength I
- Service II
- Any additional limit states, as required

Interim Summary of Work Completed

NCHRP 12-103

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