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Engineering Review
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dsdnijkamp
EPC Planning & Community Development Department

BRIDGE DESIGN CALCULATIONS

FOR

FOREST LAKE VEHICULAR BRIDGE

ELITE PROPERTIES OF AMERICA, INC.

100' LONG X 30' WIDE VEHICULAR Modular BRIDGE WITH CAST IN PLACE CONCRETE DECK

FOUTNAIN, CO

CONTECH JOB NO. 621715

Design Specifications: AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 8TH EDITION, 2017 (ABDS)

Structural Materials:

Structural Steel:	ASTM A588 Weathering Steel
Deck Concrete:	f'c = 4 ksi, maximum unit weight = 145 pcf
Elastomeric Pads:	Grade 4, 60 Durometer
Anchor Bolts:	ASTM F1554 Grade 55 or Better

COLORADO LICENSED PROFESSIONAL ENGINEER
NATHAN EVANS
53643
NBE
8-11-21

August 11, 2021

Structural Steel: F_y = 50 ksi F_u = 70 ksi
Finish = Weathering

Reinforced Concrete: Concrete: f'c = 4 ksi
Reinforcing Steel: fy = 60 ksi

Loading: HL-93 Vehicle with None & None Owner-Specified Vehicle 2 Lane of Traffic
Maximum ADTT is 172
TL-1 Design Rated Bridge Rail Non Crash Tested
37.5 psf Future Wearing Surface

Bridge Layout:
Clear Width = 30 ft
Deck Out to Out Width = 30 ft
Horizontal Length @ CL Bridge = 100 ft
Skew at End 1 = -45 °
Skew at End 2 = -45 °

CL Girder Length (ft):

B_1	B_2	B_3	B_4	B_5	B_6	B_7	B_8	B_9	B_{10}	B_{11}	B_{12}	B_{13}	B_{14}
100	100	100	100	100	0	0	0	0	0	0	0	0	0

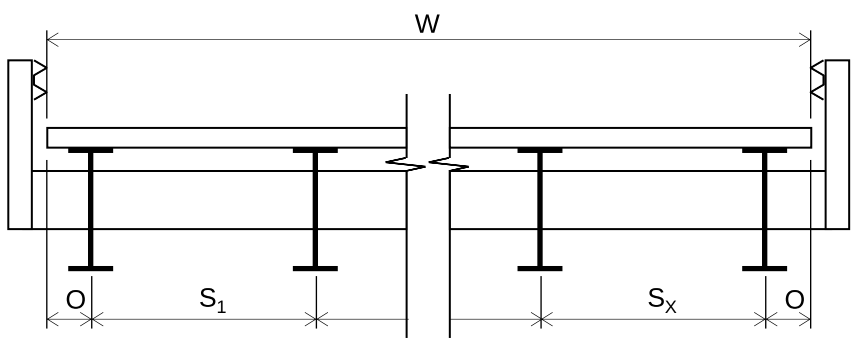
Girder Spacing (ft):

S_1	S_2	S_3	S_4	S_5	S_6	S_7	S_8	S_9	S_{10}	S_{11}	S_{12}	S_{13}
7.1667	7.1667	7.1667	7.1667	0	0	0	0	0	0	0	0	0

Grade = 1.00%

DL Camber = 6.25 in Use 6 1/4 in
Beam Splice Location = 40 ft
Number of Diaphragm Lines = 5
Number of Bridge Rail Post = 17 per side
Bridge Rail Post Spaced 1 at 5.25 ft, 14 at 6.25 ft, 1 at 5.25 ft and 1.002 ft from each end

Deck Layout: Use a Cast In Place Concrete Deck
8 in Thick at Edge and 8 in at Centerline with 7.33 inch by 5.5 feet Concrete Sidewalk
with #4 top transverse bars at 6 in spacing and 2 in of top cover
with #4 top longitudinal bars at 12 in spacing
with #4 bottom transverse bars at 6 in spacing and 1.5 in of bottom cover
with #5 bottom longitudinal bars at 12 in spacing



Unfactored Bearing Reactions in Kips		Max at Interior Girder			Max at Exterior Girder			Total @ Abutment		
		P	H	L	P	H	L	P	H	L
DC		61.25			57.50			298.76		
DW		13.44			7.97			56.25		
PL		0.00			0.00			0.00		
HL-93	LL	90.13			68.60			193.83		
	LL+IM	104.64			79.64			236.87		
None	LL	0.00			0.00			0.00		
	LL+IM	0.00			0.00			0.00		
None	LL	0.00			0.00			0.00		
	LL+IM	0.00			0.00			0.00		
None	LL	0.00			0.00			0.00		
	LL+IM	0.00			0.00			0.00		
None	LL	0.00			0.00			0.00		
	LL+IM	0.00			0.00			0.00		
WS		-30.00	3.00		0.00	3.00		-30.00	15.00	
TU				9.46			9.46			47.29
BR				7.20			7.20			36.00
EQ			13.70	27.40		13.70	27.40		68.49	136.98

Notes: DC = Dead Load
DW = Wearing Surface Load
LL = Vehicle Live Load
LL+IM = Vehicle Live Load including Dynamic Load Allowance
WS = Wind Load (Use a minimum of 0.30 klf on exterior girder per 3.8.1.2.1)
TU = Thermal Load
BR = Breaking Force
EQ = Seismic Load
Wind Load Uplift assumes full 20 psf of deck is applied to one girder line.

P is vertical load
H is horizontal load transverse to the structure
L is horizontal load longitudinal to the structure

Reference

				Vehicle	P	IM
Reinforcing Steel: f_y =	60	ksi		HL-93	16	1.33
Concrete: f'_c =	4	ksi		None	0	1
Concrete Weight =	150	pcf		None	0	1

Future Wearing Surface (FWS) = 37.5 psf

Stay-in-Place Forms (SIP) = 15 psf (includes weight of concrete in ribs)

Pedestrian Live Load (PL) = 0 psf

Design Slab Thickness =	8 in	Flange Width =	12.1	in	
Total Slab Thickness =	8 in	Girder Web Thickness =	0.87	in	
Top Clearance =	2 in	Girder Spacing =	7.1667	ft	
Bottom Clearance =	1.5 in	Effective Span = S =	6.6988	ft	ABDS 9.7.3.2

	Size	Bar Diameter	Bar Area	Bar Spacing
Top Bar:	4	0.5 in	0.1963 in^2	6 in
Bottom Bar:	4	0.5 in	0.1963 in^2	6 in

$w_{strip(+)}$ = 26.0+6.6S = 70.212 in $w_{strip(-)}$ = 48.0+3.0S = 68.096 in ABDS Tbl 4.6.2.1.3-1

w = SIP + Slab = 115 psf

w = FWS = 37.5 psf

w = PL = 0 psf

$M_{SIP+Slab} = wS^2/10$ = 0.516 k-ft/ft

$M_{FWS} = wS^2/10$ = 0.1683 k-ft/ft

$M_{PL} = wS^2/10$ = 0 k-ft/ft

$M_{LL+I} = PS/(6w_{strip(+)})$ = 4.0606 k-ft/ft (Design Vehicle)

$M_{LL+I} = PS/(6w_{strip(+)})$ = 0 k-ft/ft (Owner Specified Design Vehicle)

$M_{LL+I} = PS/(6w_{strip(+)})$ = 0 k-ft/ft (Owner Specified Design Vehicle)

$M_u(+)$ = 1.25 $M_{SIP+Slab}$ + 1.5 M_{FWS} + 1.75 M_{PL} = 0.8975 k-ft/ft ABDS Tbl 3.4.1-1

$M_u(+)$ = 1.25 $M_{SIP+Slab}$ + 1.5 M_{FWS} + 1.75 M_{LL+I} = 8.0034 k-ft/ft

$M_u(+)$ = 1.25 $M_{SIP+Slab}$ + 1.5 M_{FWS} + 1.35 M_{LL+I} = 0.8975 k-ft/ft

$M_u(+)$ = 1.25 $M_{SIP+Slab}$ + 1.5 M_{FWS} + 1.35 M_{LL+I} = 0.8975 k-ft/ft

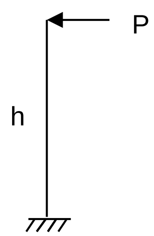
Use $M_u(+)$ = 8.0034 k-ft/ft

Reference

Deck Cantilever Overhang Moments

Deck Overhang = O =	0.6667 ft	Overhang inside the Rail = O_i =	0.6667 ft		
Curb/Rail Moment Arm = X_c =	0.6667 ft	Curb/Rail Load = P_c =	0 plf		
$w_{stripover}$ = 45.0+10.0X =	45 in	LL Moment Arm = X = O_i-1 =	0 ft	ABDS Tbl 4.6.2.1.3-1	
$M_{slab+Rail} = wO^2/2+P_cX_c$ =	0.0256 k-ft/ft				
$M_{FWS} = wO_i^2/2$ =	0.0083 k-ft/ft				
$M_{PL} = wO_i^2/2$ =	0 k-ft/ft				
$M_{LL+I} = (PI/w_{stripover})X$ =	0 k-ft/ft	(Design Vehicle)			
$M_{LL+I} = (PI/w_{stripover})X$ =	0 k-ft/ft	(Owner Specified Design Vehicle)			
$M_{LL+I} = (PI/w_{stripover})X$ =	0 k-ft/ft	(Owner Specified Design Vehicle)			

Deck/Curb Mounted Rail Impact Moments: Rail Test Level Loads = N/A



F_t =	0 k	$L_t = L_L$ =	0 ft	ABDS Tbl A13.2-1
F_L =	0 k	L_v =	0 ft	ABDS Tbl A13.2-1
F_v =	0 k	H_e (min) =	0 in	ABDS Tbl A13.2-1

$h = H_e + a$ = 0 in

Post Spacing = L_P = 6.25 ft   a = ***0*** in

$P_t = F_t(L_P-L_t/4)/L_P$ = 0 k

$P_L = F_L/3$ = 0 k

$P_v = F_vL_P/L_v$ = 0 k

$w_{striprail}$ = 45.0+10.0X = 51.667 in   LL Moment Arm = X = O_i = 0.6667 ft   ABDS Tbl 4.6.2.1.3-1

$M_{IM} = (P_th+P_vX)/w_{striprail}$ = 0.1548 ft-k

$M_u(-)$ =	1.25 $M_{slab+Rail}$	+	1.5 M_{FWS}	+	1.75 M_{PL} =	0.8975	k-ft/ft
$M_u(-)$ =	1.25 $M_{slab+Rail}$	+	1.5 M_{FWS}	+	1.75 M_{LL+I} =	8.0034	k-ft/ft
$M_u(-)$ =	1.25 $M_{slab+Rail}$	+	1.5 M_{FWS}	+	1.35 M_{LL+I} =	0.8975	k-ft/ft
$M_u(-)$ =	1.25 $M_{slab+Rail}$	+	1.5 M_{FWS}	+	1.35 M_{LL+I} =	0.8975	k-ft/ft
$M_u(-)$ =	1.25 $M_{slab+Rail}$	+	1.0 M_{FWS}	+	1.0 M_{IM} =	0.1887	k-ft/ft
					Use $M_u(-)$ =	8.0034	k-ft/ft

								Reference
Deck Capacity (+):	d_s =	6.25	in	$c = A_sf_s/(0.85f'_c\beta_1 b)$ =	0.6794			
	b =	12	in	β_1 =	0.85			ABDS 5.7.2.2
	A_s =	0.3927	in^2	c/d_s = 0.1087 <=	$0.003/(0.003 + \varepsilon_{cl})$		**OK**	ABDS 5.7.2.1
	$a = \beta_1 c$ =	0.5775	in	ε_{cl} =	0.002			ABDS Tbl C5.7.2.1-1
				$0.003/(0.003 + \varepsilon_{cl})$ =	0.6			
	$\phi M_n = 0.9A_sf_s(d_s\text{-}a/2)$ =	10.534	k-ft/ft	> $M_u(+)$			**OK**	ABDS 5.7.3.2.3
Deck Capacity (-):	d_s =	5.75	in	$c = A_sf_s/(0.85f'_c\beta_1 b)$ =	0.6794			
	b =	***12***	in	β_1 =	0.85			ABDS 5.7.2.2
	A_s =	0.3927	in^2	c/d_s = 0.1182 <=	$0.003/(0.003 + \varepsilon_{cl})$		**OK**	ABDS 5.7.2.1
	$a = \beta_1 c$ =	0.5775	in	ε_{cl} =	0.002			ABDS Tbl C5.7.2.1-1
				$0.003/(0.003 + \varepsilon_{cl})$ =	0.6			
	$\phi M_n = 0.9A_sf_s(d_s\text{-}a/2)$ =	9.6508	k-ft/ft	> $M_u(-)$			**OK**	ABDS 5.7.3.2.3

Longitudinal Steel

Bottom

% of Bottom Steel = $220/S^{1/2}$ = 85.001 % > 67%, use 67 % — ABDS 9.7.3.2

$As_{long\ bottom\ required}$ = 0.2631 in^2/ft

Bar Size = ***5***

Bar Area = 0.3068 in^2

Bar Spacing = 13.993 in max use 12 in

Top

Use # ***4*** bars at ***12*** in spacing

Slab Summary

Use a 8 in thick slab with a 0 in crown

with #4 top transverse bars at 6 in spacing
with #4 bottom transverse bars at 6 in spacing

with #4 top longitudinal bars at 12 in spacing
with #5 bottom longitudinal bars at 12 in spacing

Reference

Stay In Place Form Deck

								Reference
Design Slab Thickness = t =	8	in						
Girder Spacing = S =	7.1667	ft						
Girder Size =	W36x232							
Flange Width = b_f =	12.1	in						
Assumed Rib Fill Thickness = t_r =	***1***	in						
Unit Weight of Deck = w_c =	150	pcf						
Weight of Form = w_f =	1.79	psf						
Construction LL = w_{LL} =	***50***	psf						
Galvanizing Designation =	***G165***							
Deflection Load = w_d = $((t+t_r)w_c/12+w_f)$ =	114.29	psf						
Stress Load = $w_s = w_d+w_{LL}$ =	164.29	psf						
Form Span = L = $S-(b_f+2)/12$ =	5.9917	ft						
Allowable Deflection = Δ = 12L/ 180 =	0.3994	in <= 0.50	use	0.3994	in			ABDS 9.7.4.1
M = $12w_sL^2/8$ =	8847	in-lb						

Try: DMAC 2x9 22 - Gauge

S =	0.266	in^3
I =	0.329	in^4
Weight =	1.79	psf
Grade = F_y =	50	ksi
Actual Rib Fill =	0.67	in

$S_{req'd} = M/(.75F_y)$ =	0.2359	in^3	<= S	**OK**	ABDS 9.7.4.1
$I_{req'd} = (1728)5w_dL^4/(384E\Delta)$ =	0.2861	in^4	<= I	**OK**	
Total SIP Form wt =	10.165	psf	<= Assumed SIP Load	**OK**	

Crack Control

β_s= $1+d_c/(.7(h-d_c))$ — ABDS 5.6.7-1

= 1.559

d_c= 2.25 in

h= 8 in

Υ_e= 1

f_{ss}= 36 ksi

s= $700\Upsilon_e/(\beta_s f_{ss})-2d_c$ — ABDS 5.6.7-2

= 7.9723 in max spacing Use 6" spacing

Reference

Girder Spacing = S =	7.1667 ft	Number of Girders = NS =	5		
Overhang Width = O =	0.6667 ft	Number of Lanes = N_L =	2		
Girder Length = L =	99.057 ft	L_b =	22.71 ft		

Dead Loads (DC_1)

Weight of Decking =	15 psf	Girder Self Wt. =	232 lb/ft
Concrete Deck =	100 psf	Diaphragm Wt (Int) =	19.474 lb/ft
	115 psf	Diaphragm Wt (Ext) =	9.7369 lb/ft
		Bridge Rail and Curb (Ext) =	65 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DC_I =	1075.6 lb/ft	Dead Loads = DC_E =	795.49 lb/ft	
Shear = V_{DCI} =	53.275 k	Shear = V_{DCE} =	39.399 k	at support
Moment = M_{DCI} =	1319.3 ft-k	Moment = M_{DCE} =	975.7 ft-k	at midpoint
Deflection = Δ_{DCI} =	5.3568 in	Deflection = Δ_{DCE} =	3.9616 in	at midpoint
Rotation = θ_{DCI} =	0.0144 rad	Rotation = θ_{DCE} =	0.0107 rad	at support

Dead Loads (DC_2)

Bridge Rail and Curb (Ext) =	0 lb/ft
Sidewalk (Int) =	149.33 lb/ft
Sidewalk (Ext) =	354.6 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DL_I =	149.33 lb/ft	Dead Loads = DL_E =	354.6 lb/ft	
Shear = V_{DLI} =	7.3963 k	Shear = V_{DLE} =	17.563 k	at support
Moment = M_{DLI} =	183.16 ft-k	Moment = M_{DLE} =	434.94 ft-k	at midpoint
Deflection = Δ_{DLI} =	0.432 in	Deflection = Δ_{DLE} =	1.1864 in	at midpoint
Rotation = θ_{DLI} =	0.0012 rad	Rotation = θ_{DLE} =	0.0032 rad	at support

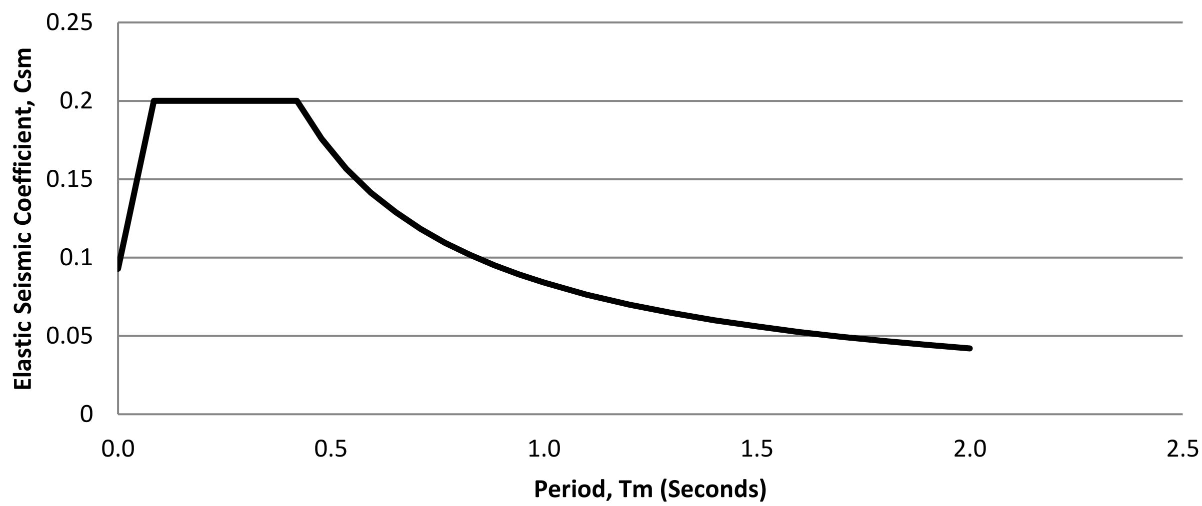
Wearing Surface (DW) Total =	37.5 psf	Miscellaneous (Ext) =	0 lb/ft
		Miscellaneous (Int) =	0 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DW_I =	268.75 lb/ft	Dead Loads = DW_E =	159.38 lb/ft	
Shear = V_{DWI} =	13.311 k	Shear = V_{DWE} =	7.8936 k	at support
Moment = M_{DWI} =	329.63 ft-k	Moment = M_{DWE} =	195.48 ft-k	at midpoint
Deflection = Δ_{DWI} =	0.7774 in	Deflection = Δ_{DWE} =	0.5332 in	at midpoint
Rotation = θ_{DWI} =	0.0021 rad	Rotation = θ_{DWE} =	0.0014 rad	at support

DL Camber = 6.2122 in **use** 6.375 **in**

Reference

Design Response Spectrum, Fig 3.10.4.1-1



Site Class = D

$A_s = F_{PGA}PGA =$	0.0928					(3.10.4.2-2)
$S_{DS} = F_aS_S =$	0.2					(3.10.4.2-3)
$S_{D1} = F_vS_1 =$	0.084					(3.10.4.2-6)
$T_o = 0.2T_s =$	0.084	sec				
$T_s = S_{D1}/S_{DS} =$	0.42	sec	$PGA =$	0.058		Figure 3.10.2.1-1
			$S_S =$	0.125		Figure 3.10.2.1-2
			$S_1 =$	0.035		Figure 3.10.2.1-3
			$F_{PGA} =$	1.6		Table 3.10.3.2-1
Use $C_{sm} =$ 0.1929 at $T_m =$	0.0785	sec	$F_a =$	1.6		Table 3.10.3.2-2
			$F_v =$	2.4		Table 3.10.3.2-3

Connection Between Superstructure and Abutment

Seismic Load = $C_{sm}W/R =$ 34.245 k (for Abutment Connection)

Seismic Load = $C_{sm}W =$ 27.396 k (for Bridge Reaction)

Tributary Permanent Load = W = 142 k ((Total Bridge Weight + gPL(Total PL)) / Number of Girders)

Modification Factor, R = ***0.8*** (Table 3.10.7.1-2) — Table 3.10.7.1-2

$\gamma_{PL} =$ 0

			Reference
3-Second Gust Wind Speed = V =	115	mph	ABDS Fig. 3.8.1.1.2-1
Wind Exposure Category =	C		ABDS 3.8.1.1.5
Wind Pressure = $P_Z = 2.56x10^{-6}V^2K_ZGC_D$ =	0.0441	ksf	ABDS 3.8.1.2.1-1
Wind Pressure = $P_Z = 2.56x10^{-6}V^2K_ZK_{zt}K_dGC_f$ =	N/A	ksf	
K_z (AASHTO) = $(2.5\ln(Z/a)+b)^2/c$ =	1.0014	K_z (IBC) = $2.01(Z/z_g)^{2/\alpha}$ = N/A	ABDS 3.8.1.2.1-2, -3, -4
a =	0.0984	α = N/A	
b =	7.35	z_g = N/A	
c =	478.4	K_{zt} = *N/A*	
		K_d = N/A	
Structure Height = Z =	33	ft (use 33 ft minimum)	
Gust Effect Factor = G =	*1.00*		ABDS Tbl 3.8.1.2.1-1
Drag Coefficient = $C_D = C_f$ =	*1.300*		ABDS Tbl 3.8.1.2.1-2
Structure Depth = d =	4.0083	ft	
Structure Effective Area = A_e =	397.05	ft^2	
Railing Effective Area = $A_{e,rails}$ =	185.11	ft^2	
Wind Load = $w_{WS} = P_Z(A_e+A_{e,rails})$ =	25.66	k	
Vertical Wind Pressure = P_V =	***0.020***	ksf (Strength III)	
*Strength V Load Factor = γ_V =	0.4839		ABDS Tbl 3.8.1.2-1
*Service I Load Factor = γ_{SerI} =	0.3705		

*Load combinations are adjusted for 70 mph and 80 mph wind speeds for Strength V and Service I load combinations

Reference

Pedestrian or Snow Live Load (PL) Unreduced = 0 psf

PL Reduction = $(0.25+15/A_T^{1/2})$ = N/A

Total = 0 psf

Interior Girders			Exterior Girders			
Dead Loads = PL_I =	0	lb/ft	Dead Loads = PL_E =	0	lb/ft	
Shear = V_{PLI} =	0	k	Shear = V_{PLE} =	0	k	at support
Moment = M_{PLI} =	0	ft-k	Moment = M_{PLE} =	0	ft-k	at midpoint
Deflection = Δ_{PLI} =	0	in	Deflection = Δ_{PLE} =	0	in	at midpoint
Rotation = θ_{PLI} =	0	rad	Rotation = θ_{PLE} =	0	rad	at support

Δ_{PL+I} = 0 in at midpoint <= L/800 = 1.4859 **OK**

Vehicle LL Distribution Factors (g)
Concrete Deck on Steel Beams

	Design Lanes	HL-93		None		None		None		None		Reference
		Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	
Moment	1	0.3682	0.5406	0	0	0	0	0	0	0	0	
	All	0.518	0.4368	0	0	0	0	0	0	0	0	ABDS 4.6.2.2.2
	Use	0.518	0.5406	0	0	0	0	0	0	0	0	
Shear	1	0.7962	0.7078	0	0	0	0	0	0	0	0	
	All	0.93	0.62	0	0	0	0	0	0	0	0	ABDS 4.6.2.2.3
	Use	0.93	0.7078	0	0	0	0	0	0	0	0	

Design Vehicle (LL): HL-93 — IM = 1.33 ABDS 3.6.2.1

Lane Load = 0.64 k/ft — f = 0.95 ABDS C3.6.1.1.2

ADTT<= 1000

	Truck (T)	Tandem (A)	Lane (L)	Controling			Reference
$V_{LL\ (total)}$ =	65.22	48.99	31.698	118.44	k	(Max(T or A)IM + L)	ABDS 3.6.1.3.1
$M_{LL\ (total)}$ =	1506.97	1188.72	784.99	2789.26	k-ft	(Max(T or A)IM + L)	ABDS 3.6.1.3.1
$\Delta_{LL\ (total)}$ =	2.5371	1.853	1.4729	3.3743	in	$(Max(IM_f(T)$ or $IM_f(0.25T)) + L)$	ABDS 3.6.1.3.2
$\theta_{LL\ (total)}$ =	0.0066	0.0047	0.004	0.0128	rad	(Max(T or A)IM + L)	ABDS 3.6.1.3.1

	Interior Girder		Exterior Girder			
V_{LL+I} =	104.64	k	79.642	k	at support	(fgV_{LL})
M_{LL+I} =	1372.5	ft-k	1432.6	ft-k	near midpoint	(fgM_{LL})

Δ_{LL+I} = 1.3497 in near midpoint $(N_L\Delta_{LL}/N_S)$ <= L/800 = 1.4859 **OK**

θ_{LL} = 0.0051 rad at support $(N_L\theta_{LL}/N_S)$

Reference

Fatigue Truck

$V_{LL\ (total)}$ = 58.76 k
$M_{LL\ (total)}$ = 1272.23 k-ft

	Interior	Exterior			
V_{LL+I} =	44.833	39.856	k	at support	($IM_f gV_{LL}/1.2$)
M_{LL+I} =	448.88	659.15	ft-k	near midpoint	($IM_f gM_{LL}/1.2$)

IM_f = ***1.15*** ABDS 3.6.2.1

Owner Specified Vehicle (LL_O): None

Truck
$V_{LL\ (total)}$ = 0.00 k — IM = 1 ABDS 3.6.2.1
$M_{LL\ (total)}$ = 0.00 k-ft — N_L = 1

	Interior Girder	Exterior Girder		
V_{LL+I} =	0 k	0 k	at support	($IMgV_{LL}$)
M_{LL+I} =	0 ft-k	0 ft-k	near midpoint	($IMgM_{LL}$)

Owner Specified Vehicle (LL_O): None

Truck
$V_{LL\ (total)}$ = 0.00 k — IM = 1 ABDS 3.6.2.1
$M_{LL\ (total)}$ = 0.00 k-ft — N_L = 1

	Interior Girder	Exterior Girder		
V_{LL+I} =	0 k	0 k	at support	($IMgV_{LL}$)
M_{LL+I} =	0 ft-k	0 ft-k	near midpoint	($IMgM_{LL}$)

Owner Specified Vehicle (LL_O): None

Truck
$V_{LL\ (total)}$ = 0.00 k — IM = 1 ABDS 3.6.2.1
$M_{LL\ (total)}$ = 0.00 k-ft — N_L = 1

	Interior Girder	Exterior Girder		
V_{LL+I} =	0 k	0 k	at support	($IMgV_{LL}$)
M_{LL+I} =	0 ft-k	0 ft-k	near midpoint	($IMgM_{LL}$)

Owner Specified Vehicle (LL_O): None

Truck
$V_{LL\ (total)}$ = 0.00 k — IM = 1 ABDS 3.6.2.1
$M_{LL\ (total)}$ = 0.00 k-ft — N_L = 1

	Interior Girder	Exterior Girder		
V_{LL+I} =	0 k	0 k	at support	($IMgV_{LL}$)
M_{LL+I} =	0 ft-k	0 ft-k	near midpoint	($IMgM_{LL}$)

Construction Vehicle (LL_C): None

Truck
$V_{LL\ (total)}$ = 0.00 k — g (Interior Girder) = ***0.5***
$M_{LL\ (total)}$ = 0.00 k-ft — g (Exterior Girder) = ***0.5***

	Interior Girder	Exterior Girder	
V_{LL} =	0 k	0 k	(gV_{LL})
M_{LL} =	0 ft-k	0 ft-k	(gM_{LL})

Construction Loading:

Overhang Forming Load (CEL) = W_{CEL} = 250 lb/lf
Deck Machine Load (CLL) = R_{CLL} = 3000 lbs

	Int. Girders		Ext. Girders	
	M (k-ft)	V (k)	M (k-ft)	V (k)
CEL =	0	0	306.64	12.382
CLL =	0	0	74.293	3

Reference

Deck Clear Width = W = 30 ft
Girder Spacing = S = 7.1667 ft
LL Overhang = d_e = 0.6667 ft
Span Length = L = 99.057 ft
Deck Type = Cast-In-Place Concrete

Number of Girders = NS = 5
Number of Lanes = N_L = 2
Skew End 1 = θ_1 = -45
Skew End 2 = θ_2 = -45
Use Skew = θ = 45

Multiple Pressence Factor = m_1 = ***1.2*** — Use with Design Vehicle Only — ABDS Tbl 3.6.1.1.2-1
m_2 = ***1***
m_3 = ***0.85***
$m_{>3}$ = ***0.65***

$K_g = n(I+Ae_g^2)$ = 374980 — ABDS (4.6.2.2.1-1)
$n = E_B/E_D$ = 7.5634 — ABDS (4.6.2.2.1-2)
$e_g = (d+t_s)/2$ = 22.55 in
t_s or t_g = 8 in

Beam = W36x232 A = 68 in^2
I = 15000 in^4 d = 37.1 in

Simplified Equations:

Type of Superstructure, Beam Line & Force Type	# Lanes Loaded	Distribution Factors		Reference
Wood Planks on Steel Beams, Interior Moment and Shear:	1	$g = S/6.7 =$	1.0697	ABDS Tbl 4.6.2.2.2a-1
	2+	$g = S/7.5 =$	0.9556	
Big R Steel Deck on Steel Beams, Interior Moment:	1	$g = S/9.2 =$	0.779	ABDS Tbl 4.6.2.2.2c-1
	2+	$g = S/9.0 =$	0.7963	
Open Steel Grid or Steel Ties on Steel Beams, Interior Moment:	1	$g = S/10.0 =$	0.7167	ABDS Tbl 4.6.2.2.2b-1
	2+	$g = S/10.0 =$	0.7167	
Concrete Deck on Steel Beams, Interior Moment:	1	$g = 0.06+(S/14)^{0.4}(S/L)^{0.3}(K_g/(12Lt_s^3))^{0.1} =$	0.3915	ABDS Tbl 4.6.2.2.2b-1
	2+	$g = 0.075+(S/9.5)^{0.6}(S/L)^{0.2}(K_g/(12Lt_s^3))^{0.1} =$	0.5508	
Concrete Deck on Steel Beams, Exterior Moment:	2+	$g = (0.77+d_e/9.1)g_{interior} =$	0.4645	ABDS Tbl 4.6.2.2.2d-1
Concrete Deck on Steel Beams, Interior Shear:	1	$g = 0.36+S/25.0 =$	0.6467	ABDS Tbl 4.6.2.2.3a-1
	2+	$g = 0.2+(S/12)-(S/35)^{2.0} =$	0.7553	
Concrete Deck on Steel Beams, Exterior Shear:	2+	$g = (0.6+d_e/10)g_{interior} =$	0.5035	ABDS Tbl 4.6.2.2.3b-1

Skew Moment Reduction Concrete Deck on Steel Beams: — ABDS Tbl 4.6.2.2.2e-1
$1-(0.25(K_g/(12.0Lt_s^3)^{0.25}(S/L)^{0.5})(\tan\theta)^{1.5} =$ 0.9404

Skew Shear Correction Factor Concrete Deck on Steel Beams: — ABDS Tbl 4.6.2.2.3c-1
$1+0.20((12.0Lt_s^3)/K_g)^{0.3}\tan\theta =$ 1.2313

Reference

Lever Rule

ABDS C4.6.2.2.1

Exterior Beams: $m(S+d_e\text{-}C\text{-}T/2)/S$ or $m0.5(S+d_e\text{-}C)/S$

Interior Beams (One Lane Loaded): $m_1(1+(S\text{-}T)/S)/2$ Use $0.5m_1$ min

Interior Beams (All Lanes Loaded): $m_x(1+(S\text{-}X)/S+(S\text{-}T)/S)/2$ Use $0.5m_x$ min

	HL-93	None	None	None	None
Lane Width (ft)	12	30	30	30	30
Number of Lanes	2	1	1	1	1
m_x	1	***1***	***1***	***1***	***1***
Wheel to Edge of Lane = C (ft)	2	0	0	0	0
Track Width = T (ft)	6	0	0	0	0
Track Width Between Trucks = X (ft)	4	0	0	0	0

Rigid Cross-Section (C4.6.2.2.2d): $R = N_L/N_S+X_{ext}\Sigma e/\Sigma x^2$

ABDS C4.6.2.2.2d

X_{ext} = 14.333

Σx^2 = 513.61

Girder	ΣS	x	x^2	Σx^2
1	0	14.333	205.44	205.44
2	7.1667	7.1667	51.361	256.81
3	14.333	0	0	256.81
4	21.5	7.1667	51.361	308.17
5	28.667	14.333	205.44	513.61
6	28.667	0	0	513.61
7	28.667	0	0	513.61
8	28.667	0	0	513.61
9	28.667	0	0	513.61
10	28.667	0	0	513.61

N_L	m	HL-93			
		e	Σe	R	mR
1	1.2	10	10	0.4791	0.5749
2	1	-2	8	0.6233	0.6233
3	0.85	0	8	0.8233	0.6998
4	0.65	0	8	1.0233	0.6651
5	0.65	0	8	1.2233	0.7951
6	0.65	0	8	1.4233	0.9251
7	0.65	0	8	1.6233	1.0551

N_L	None			None			None			None		
	e	Σe	R	e	Σe	R	e	Σe	R	e	Σe	R
1	15	15	0.6186	15	15	0.6186	15	15	0.6186	15	15	0.6186
2	0	15	0.8186	0	15	0.8186	0	15	0.8186	0	15	0.8186
3	0	15	1.0186	0	15	1.0186	0	15	1.0186	0	15	1.0186
4	0	15	1.2186	0	15	1.2186	0	15	1.2186	0	15	1.2186
5	0	15	1.4186	0	15	1.4186	0	15	1.4186	0	15	1.4186
6	0	15	1.6186	0	15	1.6186	0	15	1.6186	0	15	1.6186
7	0	15	1.8186	0	15	1.8186	0	15	1.8186	0	15	1.8186

Summary of Distribution Factors	Design Lanes	HL-93		None		None		None		None	
		Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
Lever Rule	1	0.6977	0.4884	0	0	0	0	0	0	0	0
	All	0.8023	0.407	0	0	0	0	0	0	0	0
Minimum Using Rigid Cross-Section	1		0.5749		0		0		0		0
	All		0.6233		0		0		0		0

Design Truck = HL-93

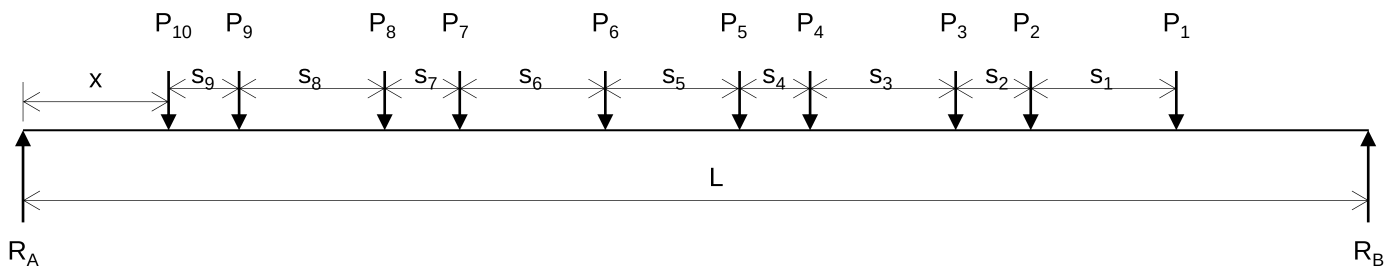
Alternate or Tandem Design Truck = HL-93 Tandem

Fatigue Design Truck = HL-93 Fatigue

L = 99.057 ft

E = 29000 ksi

I = 32459 in^4

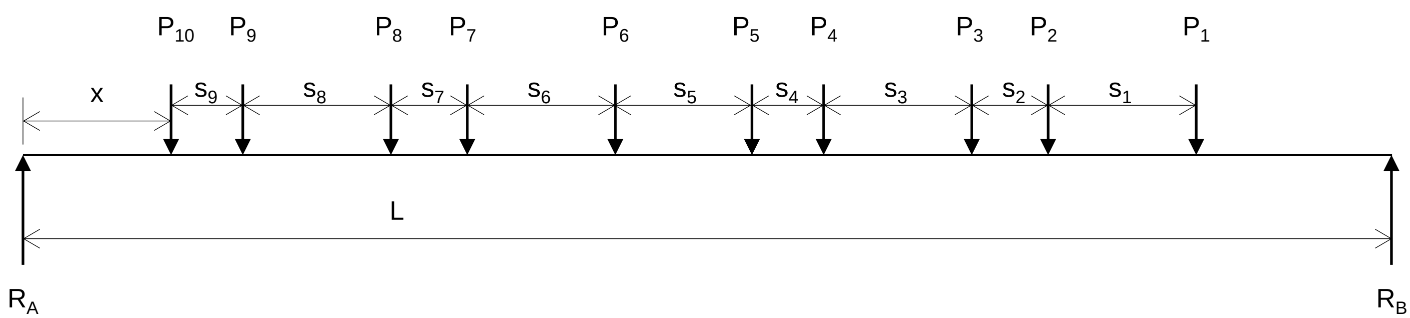


	Design Truck	Alt or Tandem	Fatigue Truck	
s_1 =	14	4	14	ft
s_2 =	14	0	30	ft
s_3 =	0	0	0	ft
s_4 =	0	0	0	ft
s_5 =	0	0	0	ft
s_6 =	0	0	0	ft
s_7 =	0	0	0	ft
s_8 =	0	0	0	ft
s_9 =	0	0	0	ft
Total Length =	28	4	44	ft

	Design Truck	Alt or Tandem	Fatigue Truck	
P_1 =	8	25	8	k
P_2 =	32	25	32	k
P_3 =	32	0	32	k
P_4 =	0	0	0	k
P_5 =	0	0	0	k
P_6 =	0	0	0	k
P_7 =	0	0	0	k
P_8 =	0	0	0	k
P_9 =	0	0	0	k
P_{10} =	0	0	0	k
Total Weight =	72	50	72	k

	Design Loading						Splice Loading:					
Design Truck	M_{Max} =	1506.97	ft-k	with x =	37.71	ft	M_{Max} =	1464.3	ft-k	with x =	45.5286	ft
	$R_{Max} = V_{Max}$ =	65.22	k	with x =	0	ft	V_{Max} =	36.485	k	with x =	39.529	ft
	Δ_{Max} =	2.5371	in	with x =	37.71	ft						
	θ_A =	0.0066	rad	with x =	37.71	ft						
	θ_B =	0.0064	rad	with x =	37.71	ft						
Alt or Tandem	M_{Max} =	1188.72	ft-k	with x =	46.52	ft	M_{Max} =	1147.83	ft-k	with x =	39.5286	ft
	$R_{Max} = V_{Max}$ =	48.99	k	with x =	0	ft	V_{Max} =	29.04	k	with x =	39.529	ft
	Δ_{Max} =	1.8530	in	with x =	46.52	ft						
	θ_A =	0.0047	rad	with x =	46.52	ft						
	θ_B =	0.0046	rad	with x =	46.52	ft						
Fatigue Truck	M_{Max} =	1272.23	ft-k	with x =	25.34	ft	M_{Max} =	1259.95	ft-k	with x =	29.5286	ft
	$R_{Max} = V_{Max}$ =	58.76	k	with x =	0	ft	V_{Max} =	30.02	k	with x =	39.529	ft
	Δ_{Max} =	2.2347	in	with x =	25.34	ft						
	θ_A =	0.0060	rad	with x =	25.34	ft						
	θ_B =	0.0057	rad	with x =	25.34	ft						

Stage 1 Construction Vehicle = None



s_1 =	0 ft	P_1 =	0 k	L =	99.057 ft
s_2 =	0 ft	P_2 =	0 k		
s_3 =	0 ft	P_3 =	0 k	E =	29000 ksi
s_4 =	0 ft	P_4 =	0 k	I =	15000 in^4
s_5 =	0 ft	P_5 =	0 k		
s_6 =	0 ft	P_6 =	0 k		
s_7 =	0 ft	P_7 =	0 k		
s_8 =	0 ft	P_8 =	0 k		
s_9 =	0 ft	P_9 =	0 k		
Total Length =	0 ft	P_{10} =	0 k		
		Total Weight =	0 k		

M_{Max} =	0.00 ft-k	with x =	0.00 ft
$R_{Max} = V_{Max}$ =	0.00 k	with x =	0 ft
Δ_{Max} =	0.0000 in	with x =	0.00 ft
θ_A =	0.0000 rad	with x =	0.00 ft
θ_B =	0.0000 rad	with x =	0.00 ft

Splice Loading:

M_{Max} =	0 ft-k	with x =	39.5286 ft
V_{Max} =	0 k	with x =	39.529 ft

Reference

Lateral Stress:

Girder Depth = d = 37.1 in — $L =$ 99.057 ft

Flange Thickness = t_f = 1.57 in — $L_b =$ 22.71 ft

Flange Width = b_f = 12.1 in — $O_D =$ 0.6667 ft

Dead Loading

$f_{lDC} = 72M_{DC}/(t_f b_f^2) =$ 0.0968 ksi

$M_{DC} = W_{DC}L_b^2/12 =$ 0.3089 k-ft

$W_{DC} = (w_d O_D^2/2)/d =$ 0.0072 k/ft

Overhang Wet Concrete Load = w_d = 100 psf

Wind Loading

$f_{lWS} = 72M_{WS}/(t_f b_f^2) =$ 6.3359 ksi

$M_{WS} = WL_b^2/10 + WL^2/8N_b =$ 20.228 k-ft — ABDS (C4.6.2.7.1-3)

$N_b =$ 5 (Number of Beams that load is distributed to)

$W = \eta_i \gamma P_D d/2 =$ 0.0681 k/ft — ABDS (C4.6.2.7.1-1)

$\gamma =$ ***1*** — ABDS Tbl 3.4.1-1

$\eta_i =$ ***1*** — ABDS 1.3.2.1

$P_D =$ 0.0441 ksf — ABDS Tbl 3.8.1

Construction Loading

$f_{lCEL} = 72M_{CEL}/(t_f b_f^2) =$ 0.7257 ksi

$M_{CEL} = F_{CEL}L_b^2/12 =$ 2.3168 k-ft — ABDS (C6.10.3.4.1-2)

$F_{CEL} = W_{CEL}O_D/d =$ 0.0539 k/ft

$f_{lCLL} = 72M_{CLL}/(t_f b_f^2) =$ 0.5752 ksi

$M_{CLL} = P_{CLL}L_b/8 =$ 1.8364 k-ft — ABDS (C6.10.3.4.1-3)

$P_{CLL} = R_{CLL}O_D/d =$ 0.6469 k

Reference

Load Summary

Girder	Load	M_u (k-ft)	V_u (k)	S_{xtf} (in³)	f_{ftf} (ksi)	S_{xbf} (in³)	f_{fbf} (ksi)	f_l (ksi)	f_lS_x (k-ft)
Exterior	DC_1	975.7	39.399	809	14.473	809	14.473	0.0968	6.5232
	DC_2	434.94	17.563	1613.9	3.2339	959.66	5.4386	***0***	0
	DW	195.48	7.8936	1613.9	1.4534	959.66	2.4444	***0***	0
	PL	0	0	3566.8	0	1072.2	0	***0***	0
	LL	1432.6	79.642	3566.8	4.8197	1072.2	16.034	***0***	0
	LL_O	0	0	3566.8	0	1072.2	0	***0***	0
	WS	***0***	***0***	3566.8	0	1072.2	0	6.3359	566.1
	CEL	306.64	12.382	809	4.5484	809	4.5484	0.7257	48.924
	CLL	74.293	3	809	1.102	809	1.102	0.5752	38.778
	LL_C	0	0	809	0	809	0	***0***	0
Interior	DC_1	1319.3	53.275	809	19.57	809	19.57	***0***	0
	DC_2	183.16	7.3963	2227.1	0.9869	1012.6	2.1707	***0***	0
	DW	329.63	13.311	2227.1	1.7761	1012.6	3.9064	***0***	0
	PL	0	0	6247	0	1121.4	0	***0***	0
	LL	1372.5	104.64	6247	2.6365	1121.4	14.687	***0***	0
	LL_O	0	0	6247	0	1121.4	0	***0***	0
	WS	***0***	0	6247	0	1121.4	0	6.3359	592.08
	CEL	0	0	809	0	809	0	***0***	0
	CLL	0	0	809	0	809	0	***0***	0
	LL_C	0	0	809	0	809	0	***0***	0

Constructability Loading

ABDS Tbl 3.4.1-1

Exterior Girder

	f_{bu} (ksi)	f_l (ksi)	$f_{bu}+f_l$ (ksi)	$f_{bu}+1/3f_l$ (ksi)
Strength I ($1.25DC_1+1.5(CEL+CLL)+1.75LL_C$):	26.566	2.0723	28.639	27.257
Strength III ($1.25DC_1+1.5CEL+1.0WS$):	24.913	7.5454	32.459	27.428
Strength IV ($1.5DC_1+1.5(CEL+CLL)$):	30.184	2.0965	32.281	30.883
Strength V ($1.25DC_1+1.5CEL+1.35(CLL+LL_C)+1.0\gamma_V WS$):	24.913	4.2756	29.189	26.339
	30.184	7.5454	32.459	30.883

Interior Girder

	f_{bu} (ksi)	f_l (ksi)	$f_{bu}+f_l$ (ksi)	$f_{bu}+1/3f_l$ (ksi)
Strength I ($1.25DC_1+1.75LL_C$):	24.462	0	24.462	24.462
Strength III ($1.25DC_1+1.0WS$):	24.462	6.3359	30.798	26.574
Strength IV ($1.5DC_1$):	29.354	0	29.354	29.354
Strength V ($1.25DC_1+1.35LL+1.0\gamma_V WS$):	24.462	3.0661	27.528	25.484
	29.354	6.3359	30.798	29.354

Reference

Service Limit State Loading

ABDS Tbl 3.4.1-1

Exterior Girders

	f_{ftf} (ksi)	f_{fbf} (ksi)	f_l (ksi)	$f_{ftf}+f_l/2$ (ksi)	$f_{fbf}+f_l/2$ (ksi)
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00PL+1.00\gamma_{SerI}WS$):	19.16	22.356	2.4443	20.382	23.578
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	23.98	38.389	2.4443	25.202	39.612
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	25.426	43.2	0.0968	25.474	43.248
	25.426	43.2	2.4443	25.474	43.248

Interior Girders

	f_{ftf} (ksi)	f_{fbf} (ksi)	f_l (ksi)	$f_{ftf}+f_l/2$ (ksi)	$f_{fbf}+f_l/2$ (ksi)
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00PL+1.00\gamma_{SerI}WS$):	22.333	25.647	2.3475	23.506	26.82
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	24.969	40.334	2.3475	26.143	41.508
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	25.76	44.74	0	25.76	44.74
	25.76	44.74	2.3475	26.143	44.74

Strength Limit State Loading

ABDS Tbl 3.4.1-1

Exterior Girders

	M_u (k-ft)	V_u (k)	f_l (ksi)	f_lS_x (k-ft)	Mu + $1/3f_lS_{xt}$
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75PL$):	2056.5	83.043	0.1209	8.154	2059.2
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	4563.5	222.42	0.1209	8.154	4566.2
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	2056.5	83.043	0.1209	8.154	2059.2
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	2056.5	83.043	6.4569	574.25	2247.9
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	2409.2	97.284	0.1451	9.7848	2412.4
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	3990.5	190.56	3.1871	282.11	4084.5
	4563.5	222.42	6.4569	574.25	4566.2

Interior Girders

	M_u (k-ft)	V_u (k)	f_{lbf} (ksi)	f_lS_x (k-ft)	Mu + $1/3f_lS_{xt}$
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75PL$):	2372.6	95.805	0	0	2372.6
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	4774.4	278.92	0	0	4774.4
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	2372.6	95.805	0	0	2372.6
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	2372.6	95.805	6.3359	592.08	2569.9
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	2748.2	110.97	0	0	2748.2
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	4225.4	237.06	3.0661	286.53	4321
	4774.4	278.92	6.3359	592.08	4774.4

Max f_l = 7.5454 ksi <= $0.6F_y$ = 30 ksi **OK** ABDS (6.10.1.6-1)

Reference

Beam Size = W36x232 — $F_y = F_{yc} = F_{yt} = F_{yw} =$ 50 ksi

$Z_x =$ 936 in^3 — $E =$ 29000 ksi

$S_x =$ 809 in^3

$b_f =$ 12.1 in — $D = d-2t_f =$ 33.96 in — $M_p = F_yZ_x =$ 46800 k-in

$t_f =$ 1.57 in — $D_c = D_{cp} = D/2 =$ 16.98 in — $M_y = F_yS_x =$ 40450 k-in

$d =$ 37.1 in — $h = d-t_f =$ 35.53 in — $F_{yr} = 0.7F_{yc} =$ 35 ksi

$t_w =$ 0.87 in

$I_x =$ 15000 in^4

$D/t_w =$	39.034	<=	150	**OK**	ABDS (6.10.2.1.1-1)
$b_f/2t_f =$	3.8535	<=	12	**OK**	ABDS (6.10.2.2-1)
$b_f =$	12.1	>=	$D/6 =$ 5.66	**OK**	ABDS (6.10.2.2-2)
$t_f =$	1.57	>=	$1.1t_w =$ 0.957	**OK**	ABDS (6.10.2.2-3)
$I_{yc}/I_{yt} =$	***1***	>=	0.1 and <= 10	**OK**	ABDS (6.10.2.2-4)

Shear Capacity — ABDS 6.10.9.2

$V_p = 0.58F_yDt_w =$ 856.81 k — ABDS (6.10.9.2-2)

$1.12(Ek/F_{yw})^{1/2} =$ 60.314 — $1.4(Ek/F_{yw})^{1/2} =$ 75.392

If $D/t_w \le 1.12(Ek/F_{yw})^{1/2}$, C = 1.0 — C = 1 — ABDS (6.10.9.3.2-4)

If $D/t_w > 1.12(Ek/F_{yw})^{1/2}$ and $\le 1.40(Ek/F_{yw})^{1/2}$, $C = (1.12/(D/t_w))(Ek/F_{yw})^{1/2} =$ 1.5451 — ABDS (6.10.9.3.2-5)

If $D/t_w > 1.40(Ek/F_{yw})^{1/2}$, $C = (1.57/(D/t_w)^2)(Ek/F_{yw})$ — C = 2.9881 — ABDS (6.10.9.3.2-6)

k = ***5*** — Use C = 1

$V_n = V_{cr} = CV_p =$ 856.81 k — ABDS (6.10.9.2-1)

Non-Composite Section Flexural Capacity (Use the Provisions of Appendix A6)

F_y		<=	70 ksi	**OK**	ABDS A6.1
$2D_c/t_w =$	39.034	<	$5.7(E/F_{yc})^{1/2} =$ 137.27	**OK**	ABDS (A6.1-1)
$I_{yc}/I_{yt} =$	***1***	>=	0.3	**OK**	ABDS (A6.1-2)

Web Plastification Factors

$2D_{cp}/t_w =$ 39.034 <= $\lambda_{pw(Dcp)} =$ 84.213 — ABDS (A6.2.1-1)

$\lambda_w = 2D_c/t_w =$ 39.034 < $\lambda_{rw} =$ 137.27 — ABDS (A6.2.2-1)

Section is Compact

$\lambda_{pw(Dcp)} = (E/F_{yc})^{1/2}/(0.54(M_p/(R_hM_y))-.09)^2 =$ 84.213 <= $\lambda_{rw}(D_{cp}/D_c)$ Use $\lambda_{pw(Dcp)} =$ 84.213 — ABDS (A6.2.1-2)

$\lambda_{pw(Dc)} = \lambda_{pw(Dcp)}(D_c/D_{cp}) =$ 84.213 <= λ_{rw} Use $\lambda_{pw(Dc)} =$ 84.213 — (A6.2.2-6)

$R_h =$ ***1*** — ABDS 6.10.1.10.1

$\lambda_{rw} = 5.7(E/F_{yc})^{1/2} =$ 137.27 — (A6.2.1-3)

$R_{pc} = R_{pt} =$ 1.157 — (A6.2.1-4 & A6.2.1-5)

Compression Flange — ABDS A6.3

Local Buckling Resistance

λ_f <= λ_{pf} $M_{nc} = R_{pc}M_{yc} =$ 46800 k-in — ABDS (A6.3.2-1)

$M_{nc}= [1-(1-(F_{yr}S_x)/(R_{pc}M_{yc}))((\lambda_f-\lambda_{pf})/(\lambda_{rf}-\lambda_{pf}))]R_{pc}M_{yc} =$ N/A k-in — ABDS (A6.3.2-2)

$\lambda_f = b_{fc}/(2t_{fc}) =$ 3.8535 — ABDS (A6.3.2-3)

$\lambda_{pf} = 0.38(E/F_{yc})^{1/2} =$ 9.1516 — ABDS (A6.3.2-4)

$\lambda_{rf} = 0.95(Ek_c/F_{yr})^{0.5} =$ 19.945 — ABDS (A6.3.2-5)

$k_c =$ ***0.76*** — ABDS A6.3.2

Lateral Torsional Buckling Resistance

		S1	S2 & S3		
If $L_b <= L_p$,	$M_{nc} = R_{pc}M_{yc} =$	46800	46800	k-in	ABDS (A6.3.3-1)
If $L_p < L_b <= L_r$,	$M_{nc} =C_b[1-(1-F_{yr}S_{xc}/(R_{pc}M_{yc}))((L_b-L_p)/(L_r-L_p))]R_{pc}M_{yc} =$	N/A	N/A	k-in	ABDS (A6.3.3-2)
If $L_b > L_r$,	$M_{nc} = F_{cr}S_x =$	N/A	N/A	k-in	ABDS (A6.3.3-3)
	$F_{cr} = (C_b\pi^2E/(L_b/r_t)^2)(1+0.078J(L_b/r_t)^2/(S_xh))^{0.5} =$	19273	19273	ksi	ABDS (A6.3.3-8)
	$L_b =$	12	12	in	
	$L_p = 1.0r_t(E/F_y)^{1/2} =$	74.965	in		ABDS (A6.3.3-4)
	$L_r = 1.95r_t(E/F_{yr})(J/(S_{xc}h))^{.5}(1+(1+6.76(F_{yr}S_{xc}h/(EJ))^2)^{1/2})^{1/2} =$	342.47	in		ABDS (A6.3.3-5)
	$r_t = b_{fc}/(12(1+D_ct_w/(3b_{fc}t_{fc})))^{1/2} =$	3.1128	in		ABDS (A6.3.3-10)
	$J = Dt_w^3/3+b_{fc}t_{fc}^3(1-0.63t_{fc}/b_{fc})/3+b_{ft}t_{ft}^3(1-0.63t_{ft}/b_{ft})/3 =$	36.12	in^4		ABDS (A6.3.3-9)
	$C_b =$	***1***			

Use M_{nc} = 3900 k-ft Constructability Condition (S1)

Use M_{nc} = 3900 k-ft Final Condition (S2 & S3 without composite deck)

Tension Flange

$M_{nt} = R_{pt}M_{yt} =$ 46800 k-in — ABDS (A6.4-1)

Use M_{nt} = 3900 k-ft

Reference

Effective Width of Concrete Slab — ABDS 4.6.2.6.1

Span = 99.057 ft
Girder Spacing = 7.1667 ft
Use Effective Width = 7.1667 ft

Concrete Slab Width = 86 in
Concrete Slab Thickness= 8 in
f'_c = 4 ksi

Beam = W36x232		S_x =	809 in^3	A =	68 in^2	b_f =	12.1 in
F_y =	50 ksi	Z_x =	936 in^3	d =	37.1 in	t_f =	1.57 in
E =	29000 ksi	I_x =	15000 in^4	t_w =	0.87 in		

$n = E/E_c$ = 7.5634 $E_c = 33000K_1w_c^{1.5}(f'_c)^{1/2}$ = 3834.3 ksi K_1 = ***1*** — ABDS 6.10.1.1.1

ELASTIC COMPOSITE SECTION PROPERTIES - Modular Ratio = 3n

	A (in^2)	y (in)	A*y	d (in)	Ad^2 (in^4)	I (in^4)
Slab:	30.321	41.1	1246.2	-15.596	7375	161.71
W-Beam:	68	18.55	1261.4	6.9542	3288.6	15000
	98.321		2507.6		10664	15162

Long Term

y_b = 25.504 in
I = 25825 in^4
S top slab = 1317.9 in^3
S top beam = 2227.1 in^3
S bot beam = 1012.6 in^3

ELASTIC SECTION PROPERTIES - Modular Ratio = n

	A (in^2)	y (in)	A*y	d (in)	Ad^2 (in^4)	I (in^4)
Slab:	90.964	41.1	3738.6	-9.6462	8464.1	485.14
W-Beam:	68	18.55	1261.4	12.904	11323	15000
	158.96		5000		19787	15485

Short Term

y_b = 31.454 in
I = 35272 in^4
S top slab = 2584.7 in^3
S top beam = 6247 in^3
S bot beam = 1121.4 in^3

PLASTIC SECTION PROPERTIES (Adjust b_n dimensions to make equivalent section)

Non-Composite

	n	b_n	h_n	F_{yn}	y_n	P_n	h_i (Sum(i=1 to n-1))	P_i (Sum(i=1 to n-1))	h_n'	Y_n'	d_n'	M_{pn}
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	-1.1907	0	-17.765	17175
Web	2	***0.8636***	33.96	50	18.55	1466.4	1.57	966.79	16.98	18.55	N-Axis	12450
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	2.7607	0	17.765	17175

P_{total} = 3400
$P_{total}/2$ = 1700

Y' = 18.55 in
M_p = 3900 ft-k

Composite

	n	b_n	h_n	F_{yn} or $0.85F_c'$	y_n	P_n	h_i (Sum(i=1 to n-1))	P_i (Sum(i=1 to n-1))	h_n'	Y_n'	d_n'	M_{pn}
Slab	4	86	8	3.4	41.1	2339.2	37.1	3400	-3.6279	0	-4.8613	11372
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	0.7087	36.239	N-Axis	383.05
Web	2	0.8636	33.96	50	18.55	1466.4	1.57	966.79	44.066	0	17.689	25939
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	4.66	0	35.454	34276

P_{total} = 5739.2
$P_{total}/2$ = 2869.6

Y' = 36.239 in
M_p = 5997.5 ft-k

Reference

Effective Width of Concrete Slab — ABDS 4.6.2.6.1

Span =	99.057	ft	Concrete Slab Width =	51	in
Overhang =	0.6667	ft	Concrete Slab Thickness=	8	in
Use Effective Width =	4.25	ft	f'_c =	4	ksi

Beam =	W36x232		S_x =	809	in^3	A =	68	in^2	b_f =	12.1	in
F_y =	50	ksi	Z_x =	936	in^3	d =	37.1	in	t_f =	1.57	in
E =	29000	ksi	I_x =	15000	in^4	t_w =	0.87	in			

$n = E/E_c =$ 7.5634 $E_c = 33000K_1w_c^{1.5}(f'_c)^{1/2} =$ 3834.3 ksi $K_1 =$ ***1*** ABDS 6.10.1.1.1

ELASTIC COMPOSITE SECTION PROPERTIES - Modular Ratio = 3n

	A (in^2)	y (in)	A*y	d (in)	Ad^2 (in^4)	I (in^4)
Slab:	17.981	41.1	739.03	-17.834	5719.1	95.9
W-Beam:	68	18.55	1261.4	4.7159	1512.3	15000
	85.981		2000.4		7231.4	15096

Long Term

y_b =	23.266	in
I =	22327	in^4
S top slab =	1022.6	in^3
S top beam =	1613.9	in^3
S bot beam =	959.66	in^3

ELASTIC SECTION PROPERTIES - Modular Ratio = n

	A (in^2)	y (in)	A*y	d (in)	Ad^2 (in^4)	I (in^4)
Slab:	53.944	41.1	2217.1	-12.575	8529.7	287.7
W-Beam:	68	18.55	1261.4	9.9754	6766.5	15000
	121.94		3478.5		15296	15288

Short Term

y_b =	28.525	in
I =	30584	in^4
S top slab =	1845.2	in^3
S top beam =	3566.8	in^3
S bot beam =	1072.2	in^3

PLASTIC SECTION PROPERTIES (Adjust b_n dimensions to make equivalent section)

Non-Composite

	n	b_n	h_n	F_{yn}	y_n	P_n	h_i (Sum(i=1 to n-1))	P_i (Sum(i=1 to n-1))	h_n'	Y_n'	d_n'	M_{pn}
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	-1.1907	0	-17.765	17175
Web	2	***0.8636***	33.96	50	18.55	1466.4	1.57	966.79	16.98	18.55	N-Axis	12450
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	2.7607	0	17.765	17175

P_{total} = 3400

$P_{total}/2$ = 1700 Y' = 18.55 in

M_p = 3900 ft-k

Composite

	n	b_n	h_n	F_{yn} or $0.85F_c'$	y_n	P_n	h_i (Sum(i=1 to n-1))	P_i (Sum(i=1 to n-1))	h_n'	Y_n'	d_n'	M_{pn}
Slab	4	51	8	3.4	41.1	1387.2	37.1	3400	-11.608	0	-6.4873	8999.1
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	-0.0643	0	-1.7023	1645.7
Web	2	0.8636	33.96	50	18.55	1466.4	1.57	966.79	33.043	34.613	N-Axis	23591
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	3.887	0	33.828	32704

P_{total} = 4787.2

$P_{total}/2$ = 2393.6 Y' = 34.613 in

M_p = 5578.4 ft-k

CONTECH ENGINEERED SOLUTIONS / BIG R BRIDGE	Project Name:	FOREST LAKE VEHICULAR BRIDGE	By:	NBE
	Project No.:	621715	Date:	8/11/2021
	Subject:	SHEAR CONNECTORS	Page:	

Reference

Fatigue

						Reference
$d =$	0.875 in	Spacing = $p =$	18 in		18	ABDS (6.10.10.1.2-1)
$\alpha = Z_r/d^2 =$	6.234	$Z_r = pV_{sr}/n =$	4.7729 k >	$5.5d^2 =$	4.2109 k	ABDS (6.10.10.2-1)
$n =$	3 /group			Use $Z_r =$	4.773 k	
Min Spacing =	5.25 in	$V_{sr} = V_fQ/I =$	0.7955 k			ABDS (6.10.10.1.2-2)
$V_f = \gamma V_{LL(fatigue)} =$	35.866 k	$Q = yA =$	678.33 in^3			
$\gamma =$	***0.8*** Fatigue II - Finite Life	$N = 10^{(34.5-\alpha)/4.28} =$	4E+06 cycles			ABDS (6.10.10.2-3)
$V_{LL(fatigue)} =$	44.833 k			Stress cycles, $n =$	***1***	ABDS Tbl 6.6.1.2.5-2
$I =$	30584 in^4	$(ADTT)_{SL\ Max} = N/((365)(75)n) =$			146.84	ABDS (6.6.1.2.5-3)
$y =$	12.575 in			$p =$	0.85	ABDS Tbl 3.6.1.4.2-1
$A =$	54 in^2					

Maximum ADTT = $ADTT_{SL}/p$ = 172 — ABDS (3.6.1.4.2-1)

Ultimate Strength

				Reference
$F_y =$ 50 ksi		$b =$ 86 in		
$f'_c =$ 4 ksi		$t_s =$ 8 in		
$A_s =$ 68 in^2		$H =$ 4 in		
$E_c =$ 3834.3 ksi				ABDS 5.4.2.4
$D =$ 49.529 ft <-----	Distance between points of maximum positive moment and end support			

$H/d =$ 4.5714 **> 4, OK** — ABDS 6.10.10.1.1

$Q_r = \phi_{sc}Q_n =$ 30.667 k

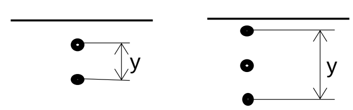
$\phi_{sc} =$ ***0.85*** — ABDS 6.5.4.2

$Q_n = 0.5A_{sc}(f'_cE_c)^{0.5} <$ $A_{sc}F_u =$ 37.235 k — Use $Q_n =$ 36.079 k — ABDS 6.10.10.4.3

$A_{sc} =$ 0.6013 in^2

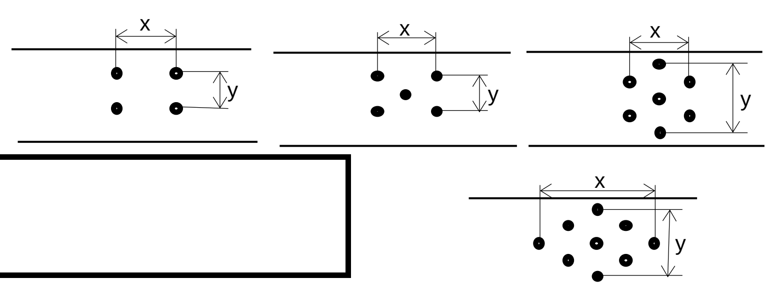
$F_u =$ ***60*** ksi — ABDS 6.4.4

			Reference
$n_{min} = P/Qr =$	76.277 studs	< Total Studs = 99 **OK**	ABDS (6.10.10.4.1-2)
$P = (P_P^2 + F_P^2)^{0.5} =$	2339.2 k		ABDS (6.10.10.4.2-1)
$P_{1P} = 0.85f'_cb_st_s =$	2339.2 k		ABDS (6.10.10.4.2-2)
$P_{2P} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} =$	3377 k		ABDS (6.10.10.4.2-3)
$F_p =$	***0*** k		



$x =$ 0 in

$y =$ 7 in



Use 0.875 in diameter by 4 in high studs
With 3 studs per Grouping
At 18 in between groups

							Reference
Load Modifiers							ABDS 1.3.2.1
$\eta_{iS} = \eta_D \eta_R \eta_I$ =	1	(Strength Limit State)		η_D =	***1***		ABDS 1.3.3
$\eta_{iE} = \eta_D \eta_I$ =	1	(Extreme Limit State)		η_R =	1		ABDS 1.3.4
				η_I =	***1***		ABDS 1.3.5

				Reference
Resistance Factors				ABDS 6.5.4.2
ϕ_f =	***1***	ϕ_v =	***1***	

Constructability Checks — ABDS 6.10.3

Compression Flange

							Reference
$f_{bu}+f_l$ =	32.459 ksi	<=	$\phi_f R_h F_{yc}$ =	50	ksi	**OK**	ABDS (6.10.3.2.1-1)
$f_{bu}+1/3f_l$ =	30.883 ksi	<=	$\phi_f F_{nc} = \phi_f M_{nc}/S_x$ =	57.849	ksi	**OK**	ABDS (6.10.3.2.1-2)
f_{bu} =	30.184 ksi	<=	$\phi_f F_{crw}$ =	616.66	ksi	**OK**	ABDS (6.10.3.2.1-3)

			Reference
$F_{crw} = 0.9Ek/(D/t_w)^2$ =	616.66	ksi	ABDS (6.10.1.9.1-1)
$k = 9/(D_c/D)^2$ =	36		ABDS (6.10.1.9.1-2)

Tension Flange

							Reference
$f_{bu}+f_l$ =	32.459 ksi	<=	$\phi_f R_h F_{yt}$ =	50	ksi	**OK**	ABDS (6.10.3.2.2-1)

Service Limit State Checks — ABDS 6.10.4.2

								Reference
Top Flange								
f_{ftf} + 0 or $f_l/2$ =	25.76 ksi	<=	0.95	$R_h F_{ytf}$ =	47.5	ksi	**OK**	ABDS (6.10.4.2.2-1 or 3)
Bottom Flange								
$f_{fbf}+f_l/2$ =	44.74 ksi	<=	0.95	$R_h F_{ybf}$ =	47.5	ksi	**OK**	ABDS (6.10.4.2.2-2 or 3)

Strength Limit State Checks — ABDS 6.10.7

							Reference
Compression Flange:							
Exterior Girder							
$\eta_{iS}(M_u+(1/3)f_l S_{xt})$ =	4563.51 ft-k	<	$\phi_f M_n$ =	5060.8	ft-k	**OK**	ABDS 6.10.7.1
Interior Girder							
$\eta_{iS}(M_u+(1/3)f_l S_{xt})$ =	4774.45 ft-k	<	$\phi_f M_n$ =	5592.4	ft-k	**OK**	ABDS 6.10.7.1
Tension Flange:							
Exterior Girder							
$\eta_{iS}(M_u+(1/3)f_l S_{xt})$ =	4566.22 ft-k	<	$\phi_f M_n$ =	5060.8	ft-k	**OK**	ABDS 6.10.7.1
Interior Girder							
$\eta_{iS}(M_u+(1/3)f_l S_{xt})$ =	4774.45 ft-k	<	$\phi_f M_n$ =	5592.4	ft-k	**OK**	ABDS 6.10.7.1

Shear Capacity Checks — ABDS 6.10.9

							Reference
V_u =	278.92 k	<=	$\phi_v V_n$ =	856.81	k	**OK**	ABDS (6.10.9.1-1)

Reference

Girder Tension Flange:

$\gamma(\Delta f)_{FI} = \gamma M_{LL} d/I_x$ (Fatigue I - Infinite Life)
$\gamma(\Delta f)_{FII} = \gamma M_{LL} d/I_x$ (Fatigue II - Finite Life)

γ = ***1.75*** Fatigue I - Infinite Life — ABDS Tbl 3.4.1-1
γ = ***0.8*** Fatigue II - Finite Life — ABDS Tbl 3.4.1-1

Maximum ADTT = $ADTT_{SL}/p$ (for Finite Life Checks)

Max N = $A/(\gamma(\Delta F)_{FII})^3$ — ABDS (6.6.1.2.5-2)
N = $(365)(75)n(ADTT)_{SL}$ or $(ADTT)_{SL} = N/((365)(75)n)$ — ABDS (6.6.1.2.5-3)

n = ***1*** — ABDS Tbl 6.6.1.2.5-2
Number of Lanes = p = 0.85 — ABDS Tbl 3.6.1.4.2-1

M_{LL} = 659.15 ft-k
I_x = 35272 in^4 (Interior Girder)
I_x = 30584 in^4 (Exterior Girder)

Detail Category (Cat), $(\Delta F)_{TH}$ & A — ABDS Tbl 6.6.1.2.5-1

	Location	Cat.	$(\Delta F)_{TH}$	A	d	$\gamma(\Delta f)_{FI}$	$\gamma(\Delta f)_{FII}$	Max ADTT
Interior Girder	Top of TF	C	10	4400000000	-11.596	-4.5507	-2.0803	OK for Infinite Life
	Top of BF	C'	12	4400000000	23.934	9.3928	4.2939	OK for Infinite Life
	Top of Diaphragm	E	4.5	1100000000	-6.5958	-2.5885	-1.1833	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	14.104	5.5351	2.5303	2480
Exterior Girder	Top of TF	C	10	4400000000	-13.834	-6.2613	-2.8623	OK for Infinite Life
	Top of BF	C'	12	4400000000	21.696	9.8195	4.4889	OK for Infinite Life
	Top of Post Block	E	4.5	1100000000	-8.8341	-3.9983	-1.8278	OK for Infinite Life
	Bottom of Post Block	E	4.5	1100000000	4.8659	2.2023	1.0068	OK for Infinite Life
	Top of Diaphragm	E	4.5	1100000000	-8.8341	-3.9983	-1.8278	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	11.866	5.3705	2.4551	2715

Use Maximum ADTT: Finite Life **Use ADTT = 2480** — ABDS (3.6.1.4.2-1)

Deck: Decking connection is only for stability of deck and supporting member.
Fatigue check not needed. — ABDS C9.8.5.1 LRFD

Reference

Diaphragm Spacing = L_b = 22.71 ft   Beam Depth = d_b = 37.1 in
Number of Diaphragms = N_b = 5 F_y = 50 ksi
Diaphragm Length = L_c = 85.13 in   E = 29000 ksi

Diaphragm Section = W21x44 t_f = 0.45 in C_w = 2110 in^6
Diaphragm Depth = d_d = 20.7 in b_f = 6.5 in   J = 0.77 in^4
Diaphragm Area = A_g = 13 in^2 t_w = 0.35 in I_x = 843 in^4
$r_s = r_{y\text{-}y}$ = 1.26 in   h = 18.76 in I_y = 20.7 in^4

Minimum Diaphragm Depth = $0.5d_b$ = 18.55 in <= d_d **OK** — ABDS 6.7.4.2

Check Total Load on Diaphragms

P_u = 20.00 k <= $\phi_c P_n$ = 390.91 k OK
ϕ_c = 0.95 — ABDS 6.5.4.2

W = 0.0681 k/ft
$P_w = WL_b$ = 1.5473 k P_u = 1.4 P_w = 2.1662 k — ABDS (C4.6.2.7.1-4)
P_u = 20 k — ABDS 6.6.1.3.1
KL_c/r = 50.673 <= 140 **OK** — ABDS 6.9.3
K = ***0.75*** — ABDS 4.6.2.5

$P_n = (0.658^{(Po/Pe)})P_o$ = 411.49 k, or $P_n = 0.877P_e$ = 1217.6 k — ABDS (6.9.4.1.1-1 & 2)
P_e/P_o = 2.9125 >= 0.44   Use P_n = 411.49 k

$P_e = (\pi^2 E/(KL_c/r_s)^2)A_g$ = 1449.1 k, or $P_e = (\pi^2 EC_w/(K_zL_z)^2 + GJ)A_g/(I_x+I_y)$ = 1383.7 k — ABDS (6.9.4.1.2-1 & 3-1)
Use P_e = 1383.7 k

$P_o = QF_yA_g$ = 475.08 k $Q = Q_sQ_a$ = 0.7309

$b_f/(2t_f)$ = 7.22 <= $0.56(E/F_y)^{1/2}$ = 13.487 $Q_s = 1.415-0.74(b_f/(2t_f))(F_y/E)^{1/2}$ = 1.1932 — ABDS (6.9.4.2.2-1)
<= $1.03(E/F_y)^{1/2}$ = 25.287 $Q_s = 0.69E/(F_y(b_f/(2t_f))^2)$ = 7.6772 — ABDS (6.9.4.2.2-2)
Use Q_s = 1

$Q_a = A_{eff}/A = b_e/h$ = 0.7309   Use Q_a = 0.7309 — ABDS (6.9.4.2.2-9)

$b_e = 1.92t_w(E/f)^{1/2}(1-(0.34/(h/t_w))(E/f)^{1/2})$ = 13.712 in <= h   Use b_e = 13.712 in — ABDS (6.9.4.2.2-11)

$f = Q_sF_y$ = 50 ksi

Reference

Loading Data

$R_u = V_u =$ 278.92 k (Total Factored Load)

$R_{LL} =$ 90.13 k (Unfactored w/o IM)
$R_{DL} =$ 74.69 k (Unfactored)
$R =$ 164.81 k (Unfactored)

Beam Data

Beam Size =	W36x232		$E =$	29000	ksi
$d =$	37.1	in	$F_y =$	50	ksi
$t_w =$	0.87	in	$b_f =$	12.1	in
$t_f =$	1.57	in	$k_1 =$	1.625	in

Check Bearing Stiffeners

Web Local Yielding — ABDS D6.5.2

$\phi_b R_n = \phi_b(2.5k+N)F_y t_w =$ 663.38 k >= R_u **OK**

Web Crippling — ABDS D6.5.3

$\phi_w R_n = \phi_w 0.4 t_w^2(1+3(N/d)(t_w/t_f)^{1.5})(EF_y t_f/t_w)^{1/2} =$ 496.35 k >= R_u **N/A**

$\phi_w R_n = \phi_w 0.4 t_w^2(1+(4N/d-0.2)(t_w/t_f)^{1.5})(EF_y t_f/t_w)^{1/2} =$ 498.88 k >= R_u **OK**

$N/d =$ 0.2156 > 0.2

$k = b_f/2-k_1 =$ 4.425 — ABDS D6.5.2

$N =$ 8 in

$\phi_b =$ ***0.8*** $\phi_w =$ ***0.8*** — ABDS 6.5.4.2

No Bearing Stiffeners are Required

Movements

Expansion Coefficient = $\alpha =$ 0.0000065 /°F
Expansion Length = $L =$ 99.057 ft

Mean High Temperature = $T_H =$ 115 °F
Mean Low Temperature = $T_L =$ -30 °F
Max Base = $T_{BMax} =$ 55 °F   Min Base = $T_{BMin} =$ 30 °F

Shear Movement:
$\Delta_{Rise} = \alpha(T_H - T_{BMin})L =$ 0.6567 in
$\Delta_{Fall} = \alpha(T_{BMax} - T_L)L =$ 0.6567 in
$\Delta_s = 1.2(\mathrm{Max}(\Delta_{Rise} \text{ or } \Delta_{Fall})) =$ 0.7881 in

Rotation:

$\theta_D =$	0.0041	0.0041	rad	(assume girder mill or induced camber accounts for 75% of DL rotation
$\theta_L =$	0.0051	0.0051	rad	
$\theta_R =$	-0.0100	0.0100	rad	(Due to Rise in Bridge)
$\theta_T =$	0.0000	0.0000	rad	(Due to Taper in bearing Plate
$\theta_s = \theta_D + \theta_L + \theta_R + \theta_T =$	-0.0008	0.0192	rad, use	0.0192 rad

Reference

Check Elastomeric Bearing Pads

Use 60 Durometer Elastomer G_{Max} = 200 psi

G_{Min} = 130 psi

Try a Steel Reinforced Elastomeric Bearing Use AASHTO Method A Design — ABDS 14.7.6

Pad Dimensions: L_E = 6.75 in $A = L_EW_E - n_h\pi d^2/4$ = 135 in^2

W_E = 20 in $S_i = A/(h_{ri}(2(L_E+W_E)+n_h\pi d)$ = 9.8384

W_E' (Effective Pad Width) = 20 in

Include Holes in W_E' = Y $A' = L_EW_E' - n_h\pi d^2/4$ = 135 in^2

Hole Diameter = d = 0 in $S_i' = A'/(h_{ri}(2(L_E+W_E')+n_h\pi d)$ = 9.8384

Number of Holes = n_h = 0

Total Thickness = h = 2.25 in S_i^2/n = 19.359 < 22 **OK** ABDS 14.7.6.1

Number of Layers = n = 5

h_{ri} = 0.2565 in

Cover = 0.125 in

Shim Thickness = 0.1196 in

Check Compressive Stress: ABDS 14.7.6.3.2

$\sigma_s = R/(L_EW_E)$ = 1.2208 ksi

$\sigma_s' = R/(L_EW_E')$ = 1.2208 ksi < 1.25 ksi **OK**

1.25 $G_{Min}S_i'$ = 1.5987 > σ_s' **OK**

Check Deflection: ABDS 14.7.6.3.3

$\delta = \text{Max}(\varepsilon_i \text{ or } \varepsilon_i')h_{ri}$ = 0.0052 in < $.09h_{ri}$ = 0.0231 **OK**

$\varepsilon_i = \sigma_s/(4.8G_{Min}S_i^2)$ = 2.0213 % ABDS (C14.7.5.3.6-1)

$\varepsilon_i' = \sigma_s'/(4.8G_{Min}S_i'^2)$ = 2.0213 %

Check Shear: h_{rt} = 1.5324 in < $2\Delta_s$ = 1.5762 in **N.G.** ABDS 14.7.6.3.4

Check Stability: $L_E/3$ = 2.25 in >= h **OK** ABDS 14.7.6.3.6

$W_E'/3$ = 6.6667 in >= h **OK**

Check Shear at Bearing

Shear Force on Pad = $H_u = GA\Delta_u/h_{rt}$ = 13.886 k ABDS (14.6.3.1-2)

Friction Resistance = μV_{DL} = 9.4586 k < H_m **Use Restraint System**

Coefficient of Friction = μ = ***0.2***

Reference

Check Bearing Plates:

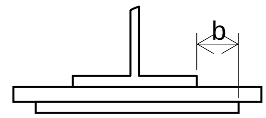


Plate Dimensions:

N_P =	8	in
W_P =	22	in
t_{Max} =	1.25	in
t_{Min} =	1.25	in
Taper =	0	in
t =	1.25	in

$S_P = N_P t^2/6 =$ 2.0833 in^3

Bottom Flange Width = b_f = 12.1 in

Pad Width Outside Flange = b = $(W_E' - b_f)/2$ = 3.95 in

$V_{u(plate)} = (V_u/W_E)b =$ 55.086 k

$M_{u(plate)} = (V_u/W_E)b^2/2 =$ 108.79 k-in

$\phi_f M_n = \phi_f F_y S_P =$ 104.17 k-in < $M_{u(plate)}$ **Say OK**

4.44% Overstressed

Use 20 in Wide by 6.75 in Long by 2.25 in Thick Layered Pad
with 6 - 11 Gage Shims and 0.125 in Cover

Pads do not meet AASHTO Design criteria for bearing pads
use as LEVELING PADS ONLY.

Reference

Anchor Bolt Checks

Breaking Force: BR = 7.2 k — Max[0.25*Truck,0.05(Truck+Lane*L)]N_Lm/N_S — ABDS 3.6.4

Horizontal Loads:
WS_H = 3 k — Strength III = $(1.0WS_H^2+1.2TU_L^2)^{1/2}$ = 11.74 k — ABDS Tbl 3.4.1-1
EQ_H = 17.12 k — Extreme Event I = $(EQ_L^2+(.3EQ_H)^2)^{1/2}$ = 34.628 k — ABDS 3.8.10
EQ_L = 34.245 k
TU_L = 9.4586 k — R_u = 34.628 k

Uplift Load:
DC = 57.504 k — Strength III = 0.90DC+ 0.65DW+ 1.0WS_H = 26.934 k — ABDS Tbl 3.4.1-1
DW = 7.9688 k
WS_H = -30 k — P_u = 0 k

Bearing Plate Thickness = t = 1.25 in
Bearing Plate Clear Distance = L_c = ***1.75*** in
Bearing Plate Tensile Strength = F_u = 70 ksi

Number of Bolts = n = ***2***
Anchor Bolt Diameter = d = 1.25 in
Area of Bolt = A_b = 1.2272 in^2
F_{ub} = 75 ksi (F1554 Gr 55 Bolts) — ABDS 6.4.3.1

Resistance Factors — ABDS 6.5.4.2

ϕ_{bb}	ϕ_t	ϕ_s	ϕ_{e2}
0.8	***0.8***	***0.75***	***0.8***

Bolt Capacity in Shear
$\phi_s R_n$ = (0.8)0.48 $\phi_s A_b F_{ub} N_s n$ = 53.014 k >= R_u **OK** — ABDS 6.13.2.12
N_s = ***1***

Bolt Capacity in Tension
$\phi_t T_n$ = 0.76 $\phi_t A_b F_{ub} n$ = 111.92 k >= P_u **OK** — ABDS 6.13.2.10

Bolt Capacity in Bearing
$\phi_{bb} R_{nb} = \phi_{bb} L_c t F_u n$ = 245 k >= R_u **OK** — ABDS 6.13.2.9

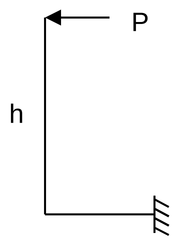
Capacity of Weld
$R_r = (0.6\phi_{e2}F_{EXX})(0.707t)l_w$ = 95.021 k >= γR_u **OK** — ABDS 6.13.3.2.4

Weld Length = l_w = 16 in
Weld Size = t = ***0.25*** in
F_{EXX} = ***70*** ksi

Reference

Rail Test Level Loads = TL-1

Post and Post Block Design



F_t =	13.5	k	$L_t = L_L$ =	4	ft	ABDS Tbl A13.2-1
F_L =	4.5	k	L_v =	18	ft	ABDS Tbl A13.2-1
F_v =	4.5	k	H_e (min) =	18	in	ABDS Tbl A13.2-1
			H (min) =	27	in	ABDS Tbl A13.2-1

$h = H_e + a$ = 32.85 in

Post Spacing = L_P = 6.25 ft　　a = 14.85 in

$P_t = F_t(L_P - L_t/4)/L_P$ =	11.34	k	$M_t = P_t h$ =	372.52	k-in
$P_L = F_L/3$ =	1.5	k	$M_L = P_L h$ =	49.275	k-in
$P_v = F_v L_P/L_v$ =	1.5625	k			

Beam Data

Beam Size = W8x18　　F_y = 50 ksi　　E = 29000 ksi

S_x =	15.2	in^3			
S_y =	15.2	in^3	$D = d - 2t_f$ =	7.48	in
t_f =	0.33	in	D/t_w =	32.522	
d =	8.14	in	$M_{yt} = F_y S_x$ =	760	k-in
t_w =	0.23	in	$M_{yL} = F_y S_y$ =	760	k-in

Load Factors (Extreme Limit State)

γ = ***1***　　ABDS 13.6.2

Resistance Factors　　ABDS 6.5.4.2

ϕ_f	ϕ_v	ϕ_{bb}	ϕ_t	ϕ_s
1	***1***	***0.8***	***0.8***	***0.8***

Check Flexure Capacity (Assume Braced Noncompact with no flange Lateral Bending Stress):

γM_t =	**372.52**	**k-in**	**<=**	**$\phi_f M_{yt}$ =**	**760**	**k-in**	**OK**
γM_L =	**49.275**	**k-in**	**<=**	**$\phi_f M_{yL}$ =**	**760**	**k-in**	**OK**

Check Shear (Unstiffened Webs)

γV_u =	**11.34**	**k**	**<=**	**$\phi_v V_n$ =**	**49.892**	**k**	**OK**

$V_n = V_{cr} = CV_p$ = 49.892 k　　ABDS (6.10.9.2-1)

V_p = 0.58 $F_y D t_w$ = 49.892 k　　ABDS (6.10.9.2-2)

1.12 $(Ek/F_{yw})^{1/2}$ = 60.314　　1.4 $(Ek/F_{yw})^{1/2}$ = 75.392

If D/t_w <= 1.12 $(Ek/F_{yw})^{1/2}$, C = 1.0　　C = 1　　ABDS (6.10.9.3.2-4)

If D/t_w > 1.12 $(Ek/F_{yw})^{1/2}$ and <= 1.4 $(Ek/F_{yw})^{1/2}$, C = $(1.12/(D/t_w))(Ek/F_{yw})^{1/2}$ = 1.8546　　ABDS (6.10.9.3.2-5)

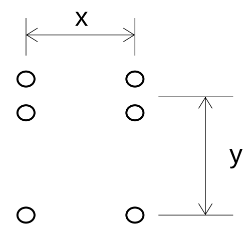
If D/t_w > 1.40 $(Ek/F_{yw})^{1/2}$, C = (1.57 $/(D/t_w)^2)(Ek/F_{yw})$　　C = 4.3048　　ABDS (6.10.9.3.2-6)

k = ***5***　　Use C = 1

Check Post Block to Post Connection

Post Block Section = W14x22
b_f = 5 in
t_f = 0.335 in
d = 13.7 in
t_w = 0.23 in

Plate Thickness = t_p = 0.5 in
Clear Distance L_c = 1.0313 in



Number of Top Bolts = n_t = 2
Number of Bottom Bolts = n_b = ***2***
x = ***3*** in y = 10 in
Bolt Diameter = d = ***0.875*** in
Area of Bolt = A_b = 0.6013 in^2
F_{ub} = 120 ksi (A325 Bolts) — ABDS 6.4.3.1

Bolt Capacity in Shear
$\phi_s R_n$ = 0.45 $\phi_s A_b F_{ub} N_s n$ = 25.977 k — ABDS (6.13.2.7-2)
N_s = ***1***

Bolt Capacity in Tension
$\phi_t T_n$ = 0.76 $\phi_t A_b F_{ub} n$ = 43.872 k — ABDS (6.13.2.10.2-1)

$\gamma R_u = \gamma((P_L(h+y)/(yn/2))+P_V/n)$ = 7.2088 k <= $\phi_s R_{ns}$ **OK**
$P_u = \gamma P_t(h+y)/(yn_t)$ = 24.296 k <= $\phi_s T_n$ **OK**
$P_u = \gamma P_t(h)/(yn_b)$ = 18.626 k <= $\phi_s T_n$ **OK**

Bolt Capacity in Bearing
$\phi_{bb} R_{nb} = \phi_{bb} 1.2 L_c t F_u$ = 34.65 ksi >= γR_u **OK** — ABDS (6.13.2.9-2)
F_u = 70 ksi

Check Rail Member

Rail Member = 12 Ga Thrie-Beam
S = 2.22 in^3
F_y = ***50*** ksi
F_u = 65 ksi

γM_u = 106.91 in-k

FS = $\phi_f F_y S/\gamma M_u$ = 1.0383 (For Steel Yielding)
FS = $\phi_f F_u S/\gamma M_u$ = 1.3497 (For Steel Rupture)

Reference

Beam Data	F_y (ksi)	A (in^2)	Z_x (in^3)	S_x (in^3)	b_f (in)	t_f (in)	d (in)	t_w (in)	I_x (in^4)	k_1 (in)
W36x232	50	68	936	809	12.1	1.57	37.1	0.87	15000	1.625

$D = d-2t_f =$ 33.96 in

Top Flange Splice Type = C (C - Conventional, B - Bearing Block, W - Welded)

Deck Data Composite Deck: Y $t_s =$ 8 in $t_{haunch} =$ 1.57 in

Splice Plate Material: $F_y =$ ***50*** ksi E = ***29000*** ksi

$F_u =$ ***70*** ksi Splice Bolts Grade: ***A325***

Splice Layout

Bolt Dia = ***0.875*** in Hole Dia = $d_h =$ 0.9375 in

Minimum Bolt Spacing = 2.625 in — ABDS 6.13.2.6.1

Longitudinal Spacing = $s_f =$ 2.75 in

Vertical Spacing = $s_w =$ 3.5 in

Minimum Edge Distance = $L_c =$ 1.125 in, use 1.25 in — ABDS Tb6.13.2.3.6-1

Maximum Gap Between Beams= ***0.125*** in

Distance from CL Splice to first Row on Flange = $s_{cf} =$ 1.5 in

Distance from CL Splice to first Row on Web = $s_{cw} =$ 1.75 in

Bottom Flange Layout

Number of Longitudinal Rows = $N_{vf} =$ 2

Number of Horizontal Rows per Side = $N_{hf} =$ 10 **(Meets minimum number of required Bolts)**

Outside Plate Width = $w_{(ofp)} =$ 12 in Min Inside Plate Width = 2.5 in **OK**

Inside Plate Width = $w_{(ifp)} =$ 4.375 in Max Width = $b_f/2-k_1 =$ 4.425 in **OK**

Outside Plate Thickness = $t_{(ofp)} =$ 1 in 9.3% Plate difference — C6.13.6.1.3b

Inside Plate Thickness = $t_{(ifp)} =$ 1.25 in Plate Length = 55 in

Conventional Top Flange Layout

Number of Longitudinal Rows = $N_{vf} =$ 2

Number of Horizontal Rows per Side = $N_{hf} =$ 10 **(Meets minimum number of required Bolts)**

Outside Plate Width = $w_{(ofp)} =$ 12 in Min Inside Plate Width = 2.5 in **OK**

Inside Plate Width = $w_{(ifp)} =$ 4.375 in Max Width = $b_f/2-k_1 =$ 4.425 in **OK**

Outside Plate Thickness = $t_{(ofp)} =$ 1 in 9.26% Plate difference **OK** — C6.13.6.1.3b

Inside Plate Thickness = $t_{(ifp)} =$ 1.25 in Plate Length = 55 in

Web Layout

Number of Vertical Rows per Side = $N_{vw} =$ 3

Number of Horizontal Rows = $N_{hw} =$ 8 **(Meets minimum number of required Bolts)**

Plate Depth = $d_{wp} =$ 27 in Max Depth = 28.46 in **OK**

Plate Width = 17 in

Plate Thickness = $t_{wp} =$ 0.75 in

Reference

Resistance Factors

ϕ_f	ϕ_v	ϕ_u	ϕ_y	ϕ_s	ϕ_{bb}	ϕ_c	ϕ_t	ϕ_{e2}	ϕ_{bs}
1	***1***	***0.8***	***0.95***	***0.8***	***0.8***	***0.95***	***0.8***	***0.8***	***0.8***

ABDS 6.5.4.2

Loading

Splice Location: 39.529 ft

	V (k)		M (ft-k)	
	Int	Ext	Int	Ext
DC_1	10.76	7.95	1265.53	935.92
DC_2	1.49	3.55	175.70	417.20
DW	2.69	1.59	316.20	187.51
$LL_{Des\ Veh}$	51.08	38.88	1328.82	1386.96
$LL_{Owner\ 1}$	0.00	0.00	0.00	0.00
$LL_{Owner\ 2}$	0.00	0.00	0.00	0.00
PL	0.00	0.00	0.00	0.00
Strength I	108.73	84.80	4601.27	4399.86
	19.34	16.77	2275.83	1972.68
Strength II	19.34	16.77	2275.83	1972.68
	19.34	16.77	2275.83	1972.68
Service II	81.34	63.64	3484.89	3343.69
	14.94	13.09	1757.43	1540.64

Table 3.4.1-1

	DC	DW	LL
Strength I	1.25	1.5	1.75
Strength II	1.25	1.5	1.35
Service II	1	1	1.3

ABDS Tbl 3.4.1-1

Factored Loads

	Strength I	Strength II	Service II
M (ft-k) =	4601.27	2275.83	3484.89
V (k) =	108.73	19.34	81.34

Bolt Capacity

Flange Shear Capacity = $\phi_s R_n = \phi_s 0.45 A_b F_{ub} N_{sf}$ =	51.954 k	ABDS (6.13.2.7-2)
Flange Slip Capacity = $R_n = K_h K_s N_{sf} P_t$ =	39 k	ABDS (6.13.2.8-1)
Web Shear Capacity = $\phi_s R_n = \phi_s 0.45 A_b F_{ub} N_{sw}$ =	51.954 k	ABDS (6.13.2.7-2)
Flange Bearing Capacity = $\phi_{bb} R_n = \phi_{bb} 1.2 L_c t F_u$ =	95.613 k	ABDS (6.13.2.9-2)
Flange Splice Pl Bearing Capacity = $\phi_{bb} R_{n(o)} = \phi_{bb} 1.2 L_c t_{(ofp)} F_u$ =	105 k	ABDS (6.13.2.9-2)
Flange Splice Pl Bearing Capacity = $\phi_{bb} R_{n(in)} = \phi_{bb} 1.2 L_c t_{(ifp)} F_u$ =	131.25 k	ABDS (6.13.2.9-2)
Web Bearing Capacity = $\phi_{bb} R_n = \phi_{bb} 1.2 L_c t F_u$ =	67.599 k	ABDS (6.13.2.9-2)
Web Splice Pl Bearing Capacity = $\phi_{bb} R_n = \phi_{bb} 1.2 L_c t F_u$ =	78.75 k	ABDS (6.13.2.9-2)

F_{ub} = ***120*** ksi (A325 Bolts) — ABDS Tbl 6.4.3.1.1-1

N_{sf} = *2* N_{sw} = ***2***

$K_h = h_f$ = ***1*** — ABDS Tbl 6.13.2.8-2

$K_s = \mu$ = 0.5 (All Faying Surfaces to be blast cleaned to SSPC-SP6) — ABDS Tbl 6.13.2.8-3

$P_t = T_b$ = 39 k — ABDS Tbl 6.13.2.8-1

Reference

Flange Splice Design

$P_{fy} = F_y A_e =$ 946.3 k — ABDS (6.13.6.1.3b-1)

Net Section Properties (Reduce Section to Account for Holes in Flange)

$A_e = (\phi_u F_u/\phi_y F_y)A_n =$ 18.926 in^2 <= A_g, use $A_e =$ 18.926 in^2 — ABDS (6.13.6.1.3b-2)

$A_g = b_f t_f =$ 18.997 in^2

$A_n = W_n t_f =$ 16.053 in^2 Use hole area = A_g - A_e = 0.0711 in^2 — ABDS 6.8.3

$W_n = b_f - N_{vf} d_h =$ 10.225 in

Minimum Number of Bolts Required = $n_t = P_{fy}/\phi_s R_n =$ 18.214

$A = D+t_f/2+t_{haunch}+t_s/2$ for Composite Deck and $D+t_f$ for Non-composite Deck — ABDS C6.13.6.1.3b

A = 3.3596 ft

Check Bearing

$M_{ubearing}$ = 4601.3 ft-k

$M_{nbearing} = N_{vf} N_{hf} \phi_{bb} R_n A =$ 6424.4 ft-k (Bottom Flange) **OK**

Check Flange Splice Plates Capacities

Bottom Flange Plates (Tension)

$P_u = P_{fy} =$ 946.3 k

$A_{g(in)} = 2t_{(ifp)} w_{(ifp)} =$ 10.938 in^2 $A_{n(in)} = 2t_{(ifp)}(w_{(ifp)} - N_{vf} d_h/2) =$ 8.5938 in^2 — ABDS 6.8.3

$A_{g(o)} = t_{(ofp)} w_{(ofp)} =$ 12 in^2 $A_{n(o)} = t_{(ofp)}(w_{(ofp)} - N_{fv} d_h) =$ 10.125 in^2

$A_{g(in+o)} =$ 22.938 in^2 $A_{n(in+o)} =$ 18.719 in^2

$\gamma_{g(in)} = A_{g(in)}/A_{g(in+o)} =$ 0.4768 $\gamma_{n(in)} = A_{n(in)}/A_{n(in+o)} =$ 0.4591 — ABDS C6.13.6.1.3b

$\gamma_{g(o)} = A_{g(o)}/A_{g(in+o)} =$ 0.5232 $\gamma_{n(o)} = A_{n(o)}/A_{n(in+o)} =$ 0.5409 — ABDS C6.13.6.1.3b

$\phi_y P_n = \phi_y F_y A_{g(in)} =$	519.53 k	$\gamma_{g(in)} P_u =$	451.23 k	**OK**	ABDS (6.8.2.1-1)
$\phi_u P_n = \phi_u F_u A_{n(in)} U =$	481.25 k	$\gamma_{n(in)} P_u =$	434.44 k	**OK**	ABDS (6.8.2.1-2)
$\phi_y P_n = \phi_y F_y A_{g(o)} =$	570 k	$\gamma_{g(o)} P_u =$	495.07 k	**OK**	ABDS (6.8.2.1-1)
$\phi_u P_n = \phi_u F_u A_{n(o)} U =$	567 k	$\gamma_{n(o)} P_u =$	511.85 k	**OK**	ABDS (6.8.2.1-2)

U = ***1***

Bottom Flange Plates (Bearing)

$M_{ubearing(o)} = M_{ubearing}\gamma_{n(o)} =$ 2488.83 ft-k

$M_{nbearing(o)} = N_{vf} N_{hf} \phi_{bb} R_{n(o)} A =$ 7055.1 ft-k (BF Splice Plate) **OK**

$M_{ubearing(in)} = M_{ubearing}\gamma_{n(in)} =$ 2112.43 ft-k

$M_{nbearing(in)} = N_{vf} N_{hf} \phi_{bb} R_{n(in)} A =$ 8818.9 ft-k (BF Splice Plate) **OK**

Reference

Conventional Top Flange Plates (Compression)

$R_u =$ 946.3 k

$A_{s(in)} = A_{g(in)} = 2t_{(ifp)}w_{(ifp)} =$ 10.938 in^2

$A_{s(o)} = A_{g(o)} = t_{(ofp)}w_{(ofp)} =$ 12 in^2

$A_s =$ 22.938 in^2

$R_r = \phi_c F_y A_s =$ 1089.5 k >= $R_{u(in)}$ **OK** ABDS 6.13.6.1.3b

Web Splice Design

$V_r = \phi_v V_n =$ 856.81 k

$A_w = D/2+t_{haunch}+t_s/2$ for Composite Deck and D/4 for Non-composite Deck — ABDS C6.13.6.1.3c

$A_w =$ 1.8792 ft

Moment Resistance of Flange Splice

$M_{nf} = P_{fy}A =$ 3179.2 ft-k

$M_u =$ 4601.27 ft-k

$H_{wshear} = (M_u - M_{nf})/A_w =$ 756.77 k

$R = (V_r^2 + H_{wshear}^2)^{1/2} =$ 1143.2 k

Minimum Number of Bolts Required = $n_t = R/\phi_s R_n =$ 22.003

Check Slip

$V_{uslip} =$ 81.34 k

$M_{nslip} = N_{vf}N_{hf}R_nA =$ 2620.5 ft-k

$M_{uslip} =$ 3484.9 ft-k

$H_{wslip} = (M_u - M_{nf})/A_w =$ 460 k

$R = (V_{uuslip}^2 + H_{wslip}^2)^{1/2} =$ 467.14 k

Minimum Number of Bolts Required = $n_t = R/\phi_s R_n =$ 11.978

Check Bearing

$R_{tmax} =$ 1143.2 k

$N_b =$ 24

$R_u = R_{tmax}/N_b =$ 47.632 k <= $\phi_{bb}R_n$ (Web) **OK**

$R_u = R_{tmax}/N_b =$ 47.632 k <= $\phi_{bb}R_n$ (Web Splice Plates) **OK**

Check Web Splice Plates Capacity

$R_r = \phi_v 0.58 F_y A_{vg} =$ 1174.5 k >= V_r **OK**

$A_{vg} = 2d_{wp}t_{wp} =$ 40.5 in^2

Reference

Web Splice:	Use 0.75 in thick by 27 in deep by 17 in wide plates with 48 - 0.875 in bolts in 8 horizontal rows at 3.5 in spacing and 3 vertical rows per side of splice with 1.75 in from CL to first row
Bottom Flange Splice:	Use 1 in thick by 12 in wide by 55 in long outside plates and 1.25 in thick by 4.375 in wide by 55 in longe inside plates with 40 - 0.875 in bolts in 2 longitudinal rows at 2.75 in spacing and 10 lateral rows per side of splice with 1.5 in from CL to first row
Top Flange Splice:	Use 1 in thick by 12 in wide by 55 in long outside plates and 1.25 in thick by 4.375 in wide by 55 in longe inside plates with 40 - 0.875 in bolts in 2 longitudinal rows at 2.75 in spacing and 10 lateral rows per side of splice with 1.5 in from CL to first row