Bridge Design Calculations

for

Mesa Top Drive Bridge

100' Long X 30' Wide

Vehicular Modular Rolled Girder Bridge

With Cast In Place Concrete Deck

Fountain, CO

CONTECH Project No. 621715

Design Specifications:

AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (ABDS)
Steel Construction Manual by AISC, 15th Edition (AISC)

Structural Materials:

Structural Steel: ASTM A588 Weathering Steel
Deck Concrete: f'c = 4.5 ksi, maximum unit weight = 145 pcf
Elastomeric Pads: Grade 4, 60 Durometer
Anchor Bolts: ASTM F1554 Grade 55 or Better

COLORADO LICENSED PROFESSIONAL ENGINEER CAMERON D. ABERNATHY 39705

Cameron Abernathy 7/13/21

July 13, 2021

Structural Steel: F_y = 50 ksi F_u = 70 ksi

Finish = Weathering

Reinforced Concrete: Concrete: f'c = 4.5 ksi

Reinforcing Steel: fy = 60 ksi

Loading: HL-93 Design Vehicle — 2 Lane of Traffic

No Owner-Specified Vehicle

Maximum ADTT is 61

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 = 6.375 in Use 6 3/8 0

Beam Splice Location = 40 ft

Number of Diaphragm Lines = 5

Number of Bridge Rail Post = 17 per side

Bridge Rail Post Spaced 1 at 5.252 ft, 14 at 6.25 ft, 1 at 5.252 ft and 1 ft from each end

Deck Layout: Use a Cast In Place Concrete Deck

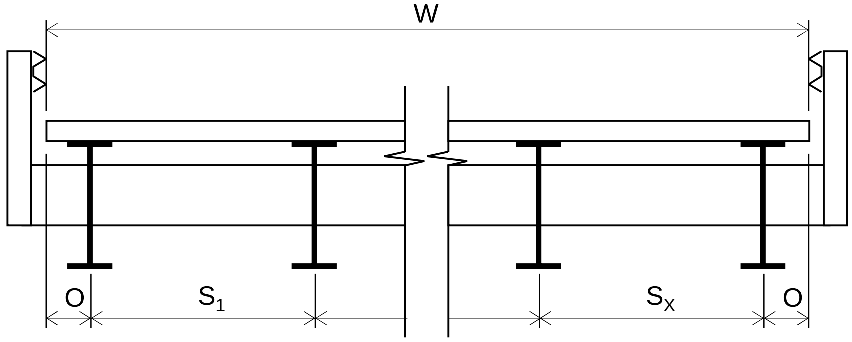
8 in Thick at Edge and 8 in at Centerline with 7.125 inch by 5.5 feet Concrete Sidewalk

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		61.05			57.77			298.69		
DW		13.44			7.97			56.25		
PL		0.00			0.00			0.00		
HL-93	LL	90.43			68.83			193.83		
	LL+IM	110.51			84.11			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	
WA		0.00	0.00		0.00	0.00		0.00	0.00	
TU				13.89			13.89			69.43
BR				7.20			7.20			36.00
EQ			13.44	26.87		13.44	26.87		67.19	134.37

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)
WA = Stream Overtopping Load applied to the Superstructure
TU = Thermal Load
BR = Braking 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

Materials:

					Reference
Concrete: f'_c =	4.5 ksi	β_1 =	0.825		ABDS 5.6.2.2
Reinforcing Steel: f_y =	60 ksi	α_1 =	0.85		ABDS 5.6.2.2
Concrete Weight =	150 pcf	ε_{cl} =	0.002		ABDS 5.6.2.1
Design Section Width =	***12*** in	ε_{tl} =	0.005		ABDS 5.6.2.1
		$0.003/(0.003 + \varepsilon_{cl})$ =	0.6		

Loading:

Vehicle	HL-93	None	None	None	None
P	16 k	0 k	0 k	0 k	0 k
IM	1.33	1	1	1	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 9.7.2.3

	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

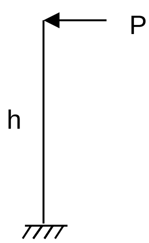
Use $M_u(+) =$ 8.0034 k-ft/ft

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
$M_u(-)$ =	1.0	$M_{slab+Rail}$	+	1.0	M_{FWS}	+	1.0	M_{IM} =	0.1887	k-ft/ft
								Use $M_u(-)$ =	8.0034	k-ft/ft

CONTECH ENGINEERED SOLUTIONS / BIG R BRIDGE	Project Name: Mesa Top Drive Bridge	By: CDA	
	Project No.: 621715	Date: 7/13/2021	
	Subject: CONCRETE DECK DESIGN	Page: 6 of 39	

Reference

Deck Capacity (+): d_s = 6.25 in — $c = A_s f_s/(\alpha_1 f'_c \beta_1 b)$ = 0.6222

A_s = 0.3927 in^2 — c/d_s = 0.0996 <= $0.003/(0.003 + \varepsilon_{cl})$ — **OK** ABDS (5.6.2.1-1)

$a = \beta_1 c$ = 0.5133 in — ε_t = 0.0271 — ϕ = 0.9 — ABDS 5.6.2.1 & ABDS 5.5.4.2

$\phi M_n = \phi A_s f_s (d_s - a/2)$ = 10.591 k-ft/ft > $M_u(+)$ — **OK** ABDS 5.6.3.2.3

Deck Capacity (-): d_s = 5.75 in — $c = A_s f_s/(\alpha_1 f'_c \beta_1 b)$ = 0.6222

A_s = 0.3927 in^2 — c/d_s = 0.1082 <= $0.003/(0.003 + \varepsilon_{cl})$ — **OK** ABDS (5.6.2.1-1)

$a = \beta_1 c$ = 0.5133 in — ε_t = 0.0247 — ϕ = 0.9 — ABDS 5.6.2.1 & ABDS 5.5.4.2

$\phi M_n = \phi A_s f_s (d_s - a/2)$ = 9.7075 k-ft/ft > $M_u(-)$ — **OK** ABDS 5.6.3.2.3

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 (min of 1.5 x total thickness or 12 in)

Crack Control

$s <= 700\gamma_e/\beta_s f_{ss} - 2d_c$ = 13.966 in >= s — **OK** ABDS 5.6.7

$\beta_s = 1 + d_c/0.7(h - d_c)$ = 1.4762

γ_e = **1** Class 1 exposure condition

$f_{ss} = M_{Ser}(-)/A_s(d - a/2)$ = 26.394 ksi <= $0.6f_y$

M_{Ser} = 4.7449 ft-k

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

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						ABDS 9.7.4.1
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**	

Reference

Girder Spacing = S = 7.1667 ft   Number of Girders = N_S = 5

Overhang Width = O = 0.6667 ft   Girder Length = L = 99.057 ft

Traveled Way Overhang Width = O_T = 0.6667 ft

Dead Loads (DC_1)

Weight of Decking =	15 psf	Girder Self Wt. =	232 lb/ft
Concrete Deck =	100 psf	Diaphragm Wt (Int) =	19.746 lb/ft
	115 psf	Diaphragm Wt (Ext) =	9.873 lb/ft
		Bridge Rail and Curb (Ext) =	80 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DC_I =	1075.9 lb/ft	Dead Loads = DC_E =	810.62 lb/ft	
Shear = V_{DCI} =	53.288 k	Shear = V_{DCE} =	40.149 k	at support
Moment = M_{DCI} =	1319.7 ft-k	Moment = M_{DCE} =	994.26 ft-k	at midpoint
Deflection = Δ_{DCI} =	5.3581 in	Deflection = Δ_{DCE} =	4.037 in	at midpoint
Rotation = θ_{DCI} =	0.0144 rad	Rotation = θ_{DCE} =	0.0109 rad	at support

Dead Loads (DC_2)

Bridge Rail and Curb (Ext) =	0 lb/ft
Sidewalk (Int) =	145.16 lb/ft
Sidewalk (Ext) =	344.69 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DL_I =	145.16 lb/ft	Dead Loads = DL_E =	344.69 lb/ft	
Shear = V_{DLI} =	7.1895 k	Shear = V_{DLE} =	17.072 k	at support
Moment = M_{DLI} =	178.04 ft-k	Moment = M_{DLE} =	422.77 ft-k	at midpoint
Deflection = Δ_{DLI} =	0.4127 in	Deflection = Δ_{DLE} =	1.1355 in	at midpoint
Rotation = θ_{DLI} =	0.0011 rad	Rotation = θ_{DLE} =	0.0031 rad	at support

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

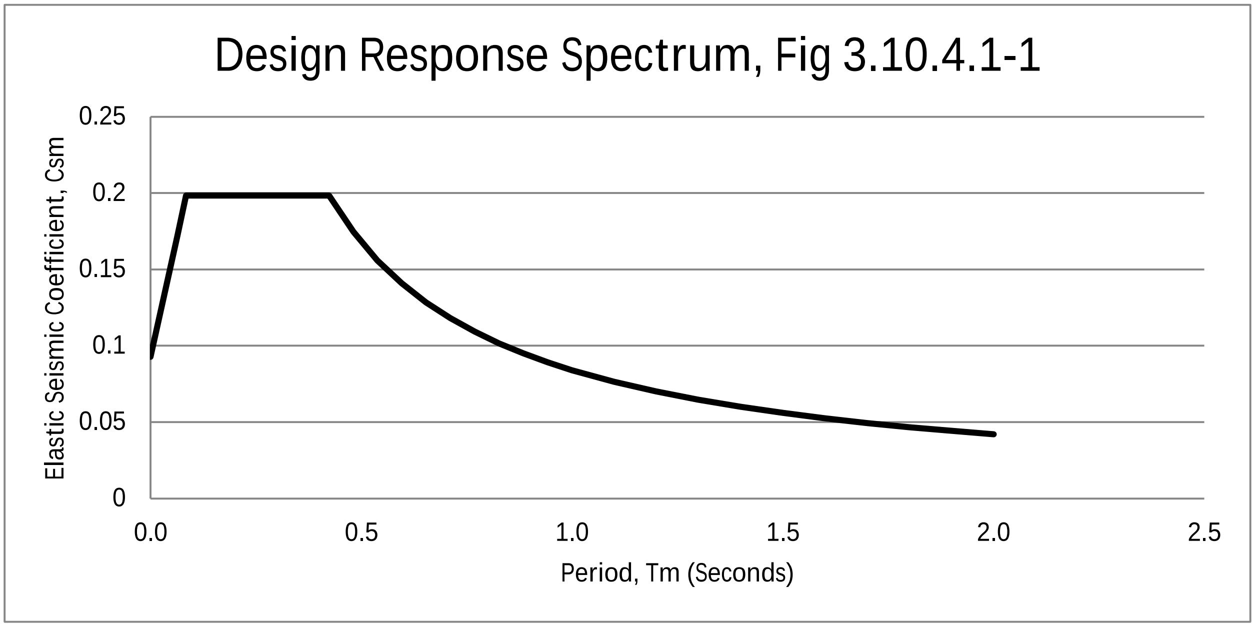
Miscellaneous (Int) = 0 lb/ft

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} =	0.7641 in	Deflection = Δ_{DWE} =	0.525 in	at midpoint
Rotation = θ_{DWI} =	0.0021 rad	Rotation = θ_{DWE} =	0.0014 rad	at support

DL Camber = 6.2 in

Residual Camber = 0 in   **Total Camber = 6.325 in   use 6.375**

Reference



Site Class = D

$A_s = F_{PGA}PGA$ =	0.0928	ABDS (3.10.4.2-2)			
$S_{DS} = F_aS_S$ =	0.1984	ABDS(3.10.4.2-3)	Seismic Zone =	2	Table 3.10.6-1
$S_{D1} = F_vS_1$ =	0.084	ABDS (3.10.4.2-6)	PGA =	0.058	Figure 3.10.2.1-1
$T_o = 0.2T_s$ =	0.0847	sec	S_S =	0.124	Figure 3.10.2.1-2
$T_s = S_{D1}/S_{DS}$ =	0.4234	sec	S_1 =	0.035	Figure 3.10.2.1-3
			F_{PGA} =	1.6	Table 3.10.3.2-1
Use C_{sm} = 0.1893	at T_m = 0.0774	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 = $C_{sm}W/R$ =	33.593	k (for Abutment Connection)	
Minimum Seismic Load = A_sW =	13.175	lbs (for Abutment Connection)	3.10.9.1
Seismic Load = $C_{sm}W$ =	26.875	k (for Bridge Reaction)	

Tributary Permanent Load = W = 141.98 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

γ_{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; a = 0.0984; b = 7.35; c = 478.4	ABDS (3.8.1.2.1-2, -3, -4)
K_z (IBC) = $2.01(Z/z_g)^{2/\alpha}$ = N/A; α = N/A; z_g = N/A; 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
Vertical Wind Pressure = P_V = ***0.020*** ksf (Strength III)	
*Strength V Load Factor = γ_V = 0.4839	ABDS Tbl 3.8.1.1.2-1
*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	0.3667	0.5411	0	0	0	0	0	0	0	0	
	All	0.5158	0.435	0	0	0	0	0	0	0	0	ABDS 4.6.2.2.2
	Use	0.5158	0.5411	0	0	0	0	0	0	0	0	
Shear	1	0.7989	0.7102	0	0	0	0	0	0	0	0	
	All	0.9331	0.6221	0	0	0	0	0	0	0	0	ABDS 4.6.2.2.3
	Use	0.9331	0.7102	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 Number of Lanes = N_L = 2

f = 1 ABDS C3.6.1.1.2

ADTT<= 1000

	Truck (T)	Tandem (A)	Lane (L)	Controlling			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
$\Delta_{LL\ (total)}$ =	2.4968	1.8237	1.4495	3.3208	in	(Max(IM_f(T) or IM_f(0.25T)) + L)	ABDS 3.6.1.3.2
$\theta_{LL\ (total)}$ =	0.0065	0.0046	0.0039	0.0125	rad	(Max(T or A)IM + L)	ABDS 3.6.1.3.1

	Interior Girder	Exterior Girder		
V_{LL+I} =	110.51 k	84.114 k	at support	(fgV_{LL})
M_{LL+I} =	1438.8 ft-k	1509.4 ft-k	near midpoint	(fgM_{LL})

Δ_{LL+I} = 1.3283 in near midpoint ($N_L\Delta_{LL}/N_S$) <= L/800 = 1.4859 **OK**

θ_{LL} = 0.005 rad at support ($N_L\theta_{LL}/N_S$)

Fatigue Truck Interior | Exterior IM_f = ***1.15*** ABDS 3.6.2.1

Reference

$V_{LL\ (total)}$ = 58.76 k	V_{LL+I} =	44.983	39.99 k	at support	$(IM_f gV_{LL}/1.2)$	
$M_{LL\ (total)}$ = 1272.23 k-ft	M_{LL+I} =	447.06	659.76 ft-k	near midpoint	$(IM_f gM_{LL}/1.2)$	

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} = 0 lb/lf

Deck Machine Load (CLL) = R_{CLL} = 0 lbs

	Int. Girders		Ext. Girders	
	M (k-ft)	V (k)	M (k-ft)	V (k)
CEL =	0	0	0	0
CLL =	0	0	0	0

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 Presence 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***

$K_g = n(I+Ae_g^2)$ = 353534 — ABDS (4.6.2.2.1-1)
$n = E_B/E_D$ = 7.1308 — 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	$g = 0.06+(S/14)^{0.4}(S/L)^{0.3}(K_g/(12Lt_s^3))^{0.1} =$	0.3895	ABDS Tbl 4.6.2.2.2b-1
	2+	$g = 0.075+(S/9.5)^{0.6}(S/L)^{0.2}(K_g/(12Lt_s^3))^{0.1} =$	0.548	
Concrete Deck on Steel Beams, Exterior Moment:	2+	$g = (0.77+d_e/9.1)g_{interior} =$	0.4621	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
$1-(0.25(K_g/(12.0Lt_s^3)^{0.25}(S/L)^{0.5})(\tan\theta)^{1.5} =$ 0.9413

Skew Shear Correction Factor Concrete Deck on Steel Beams: — ABDS Tbl 4.6.2.2.3c-1
$1+0.20((12.0Lt_s^3)/K_g)^{0.3}\tan\theta =$ 1.2354

Reference

Lever Rule — ABDS C4.6.2.2.1

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

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

Interior Beams (All Lanes Loaded): $m_x(1+(S-X)/S+(S-T)/S)/2$ 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

$\Sigma x^2 =$ 513.61

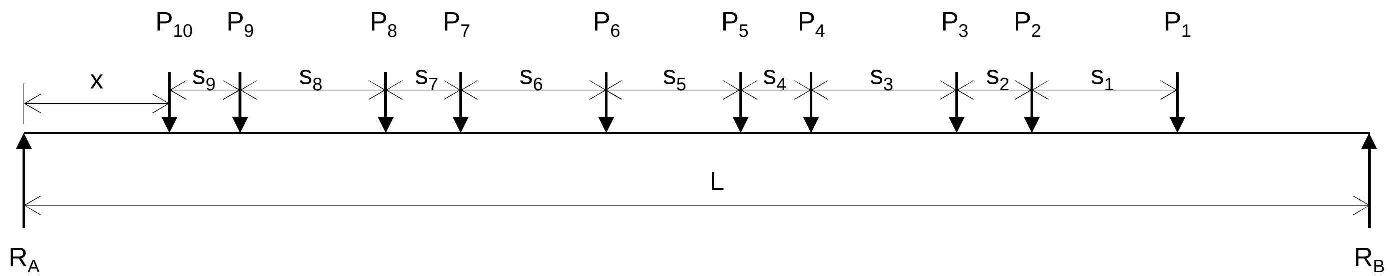
Girder	ΣS	x	x^2	Σx^2
1	0	14.333	205.44	205.44
2	7.1667	7.1667	51.361	256.81
3	14.333	0	0	256.81
4	21.5	7.1667	51.361	308.17
5	28.667	14.333	205.44	513.61
6	28.667	0	0	513.61
7	28.667	0	0	513.61
8	28.667	0	0	513.61
9	28.667	0	0	513.61
10	28.667	0	0	513.61

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		L =	99.057 ft
Alternate or Tandem Design Truck =	HL-93 Tandem			
Fatigue Design Truck =	HL-93 Fatigue		E =	29000 ksi
			I =	32982 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} =	1507.0 ft-k	with x =	37.71 ft	M_{Max} =	1464.3 ft-k	with x =	45.529 ft
	R_{Max} = V_{Max} =	65.22 k	with x =	0 ft	V_{Max} =	36.485 k	with x =	39.529 ft
	Δ_{Max} =	2.4968 in	with x =	37.71 ft				
	θ_A =	0.0065 rad	with x =	37.71 ft				
	θ_B =	0.0063 rad	with x =	37.71 ft				
Alt or Tandem	M_{Max} =	1188.7 ft-k	with x =	46.52 ft	M_{Max} =	1147.8 ft-k	with x =	39.529 ft
	R_{Max} = V_{Max} =	48.99 k	with x =	0 ft	V_{Max} =	29.04 k	with x =	39.529 ft
	Δ_{Max} =	1.8237 in	with x =	46.52 ft				
	θ_A =	0.0046 rad	with x =	46.52 ft				
	θ_B =	0.0046 rad	with x =	46.52 ft				
Fatigue Truck	M_{Max} =	1272.2 ft-k	with x =	25.34 ft	M_{Max} =	1260.0 ft-k	with x =	29.529 ft
	R_{Max} = V_{Max} =	58.76 k	with x =	0 ft	V_{Max} =	30.02 k	with x =	39.529 ft
	Δ_{Max} =	2.1993 in	with x =	25.34 ft				
	θ_A =	0.0060 rad	with x =	25.34 ft				
	θ_B =	0.0056 rad	with x =	25.34 ft				

Reference

Lateral Stress:

Girder Depth = d =	37.1 in	L =	99.057 ft
Flange Thickness = t_f =	1.57 in	L_b =	22.708 ft
Flange Width = b_f =	12.1 in	O_D =	0.6667 ft

Dead Loading

$f_{lDC} = 72M_{DC}/(t_f b_f^2)$ = 0.0967 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

$f_{lWS} = 72M_{WS}/(t_f b_f^2)$ = 6.3358 ksi

$M_{WS} = WL_b^2/10 + WL^2/8N_b$ = 20.227 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)

γ = ***1*** — ABDS Tbl 3.4.1-1

η_i = ***1*** — ABDS 1.3.2.1

P_D = 0.0441 ksf

Construction Loading

$f_{lCEL} = 72M_{CEL}/(t_f b_f^2)$ = 0 ksi

$M_{CEL} = F_{CEL}L_b^2/12$ = 0 k-ft — ABDS (C6.10.3.4.1-2)

$F_{CEL} = W_{CEL}O_D/d$ = 0 k/ft

$f_{lCLL} = 72M_{CLL}/(t_f b_f^2)$ = 0 ksi

$M_{CLL} = P_{CLL}L_b/8$ = 0 k-ft — ABDS (C6.10.3.4.1-3)

$P_{CLL} = R_{CLL}O_D/d$ = 0 k

Stream Overtopping Loading

Stream Velocity = V = 0 fps

$P_{max} = C_D V^2$ = 0 psf — ABDS (3.7.3.1-1)

C_D = 1.4

Reference

Lateral Deflections:

$I = I_{y\text{-Girders}} + I_{y\text{-Deck}} =$ 9E+06 in^4 L = 1188.7 in (c-c brg)

$E = E_b =$ 29000 ksi

Iy Girders:

Girder	S	y	y^2	A	Ay^2	I_y
1	0	172	29584	68	2E+06	468
2	86	86	7396	68	502928	468
3	86	0	0	68	0	468
4	86	-86	7396	68	502928	468
5	86	-172	29584	68	2E+06	468
6	0	0	0	0	0	0
7	0	0	0	0	0	0
8	0	0	0	0	0	0
9	0	0	0	0	0	0
10	0	0	0	0	0	0
	344				5E+06	2340

$I_{y\text{-Girders}} = \Sigma A_y^2 + \Sigma I_y =$ 5E+06 in^4

I_y Deck:

Width = d = 360 in

Equivalent Thickness = b = 8 in

$E_d =$ 4066.8 ksi

$I_{y\text{-Deck}} = bd^3/12(E_d/E_b) =$ 4E+06 in^4

$\Delta_{DL\ lat} = 5WL^3/(384EI) =$ 0.0586 in W = 729.84 k/ft

$\Delta_{WL\ lat} = 5W_{WL}L^3/(384EI) =$ 0.0024 in $W_{WL} =$ 30.00 k

$T_m = 2\pi(\Delta_{DL\ lat}/g)^{1/2} =$ 0.0774 sec (Lateral Period of the Bridge) g = ***32.2*** ft/sec^2

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)	$f_l S_x$ (k-ft)
Exterior	DC_1	994.26	40.149	809	14.748	809	14.748	0.0967	6.5225
	DC_2	422.77	17.072	1666	3.0451	965.36	5.2553	***0***	0
	DW	195.48	7.8936	1666	1.408	965.36	2.4299	***0***	0
	PL	0	0	3772.2	0	1078	0	***0***	0
	LL	1509.4	84.114	3772.2	4.8016	1078	16.801	***0***	0
	LL_O	0	0	3772.2	0	1078	0	***0***	0
	WS	***0***	***0***	3772.2	0	1078	0	6.3358	569.18
	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
Interior	DC_1	1319.7	53.288	809	19.575	809	19.575	***0***	0
	DC_2	178.04	7.1895	2323.2	0.9196	1018.8	2.0972	***0***	0
	DW	329.63	13.311	2323.2	1.7026	1018.8	3.8828	***0***	0
	PL	0	0	6725.6	0	1126.5	0	***0***	0
	LL	1438.8	110.51	6725.6	2.5671	1126.5	15.326	***0***	0
	LL_O	0	0	6725.6	0	1126.5	0	***0***	0
	WS	***0***	0	6725.6	0	1126.5	0	6.3358	594.78
	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 ($1.25DC_1+1.5(CEL+CLL)+1.75LL_C$):	18.435	0.1209	18.556	18.475
Strength III ($1.25DC_1+1.5CEL+1.0WS$):	18.435	6.4567	24.892	20.587
Strength IV ($1.5DC_1+1.5(CEL+CLL)$):	22.122	0.1451	22.267	22.17
Strength V ($1.25DC_1+1.5CEL+1.35(CLL+LL_C)+1.0\gamma_V WS$):	18.435	3.187	21.622	19.497
	22.122	6.4567	24.892	22.17

Interior Girder

	f_{bu} (ksi)	f_l (ksi)	$f_{bu}+f_l$ (ksi)	$f_{bu}+1/3f_l$ (ksi)
Strength I ($1.25DC_1+1.75LL_C$):	24.468	0	24.468	24.468
Strength III ($1.25DC_1+1.0WS$):	24.468	6.3358	30.804	26.58
Strength IV ($1.5DC_1$):	29.362	0	29.362	29.362
Strength V ($1.25DC_1+1.35LL+1.0\gamma_V WS$):	24.468	3.0661	27.534	25.49
	29.362	6.3358	30.804	29.362

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 ($1.00DC_1+1.00DC_2+1.0DW+1.00PL+1.00\gamma_{SerI}WS$):	19.201	22.433	2.4442	20.423	23.655
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	24.003	39.235	2.4442	25.225	40.457
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	25.443	44.275	0.0967	25.492	44.324
	25.443	44.275	2.4442	25.492	44.324

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 ($1.00DC_1+1.00DC_2+1.0DW+1.00PL+1.00\gamma_{SerI}WS$):	22.197	25.555	2.3475	23.371	26.728
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	24.764	40.881	2.3475	25.938	42.054
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	25.534	45.479	0	25.534	45.479
	25.534	45.479	2.3475	25.938	45.479

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 ($1.25(DC_1+DC_2)+1.5DW+1.75PL$):	2064.5	83.366	0.1209	8.1531	2067.2
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	4705.9	230.57	0.1209	8.1531	4708.6
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	2064.5	83.366	0.1209	8.1531	2067.2
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	2064.5	83.366	6.4567	577.33	2257
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	2418.8	97.672	0.1451	9.7837	2422
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	4102.2	196.92	3.187	283.6	4196.7
	4705.9	230.57	6.4567	577.33	4708.6

Interior Girders

	M_u (k-ft)	V_u (k)	f_{lbf} (ksi)	f_lS_x (k-ft)	Mu + $1/3f_lS_{xt}$
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75PL$):	2366.6	95.564	0	0	2366.6
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	4884.4	288.96	0	0	4884.4
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	2366.6	95.564	0	0	2366.6
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	2366.6	95.564	6.3358	594.78	2564.8
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	2741	110.68	0	0	2741
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	4308.9	244.75	3.0661	287.83	4404.8
	4884.4	288.96	6.3358	594.78	4884.4

Max f_l = 6.4567 ksi <= $0.6F_y$ = 30 ksi **OK** ABDS (6.10.1.6-1)

Reference

Beam Size = W36x232

Z_x =	936	in^3				$F_y = F_{yc} = F_{yt} = F_{yw}$ =	50	ksi	
S_x =	809	in^3				E =	29000	ksi	
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 <= $1.12(Ek/F_{yw})^{1/2}$, C = 1.0	C = 1	ABDS (6.10.9.3.2-4)
If D/t_w > $1.12(Ek/F_{yw})^{1/2}$ and <= $1.40(Ek/F_{yw})^{1/2}$, $C = (1.12/(D/t_w))(Ek/F_{yw})^{1/2}$ =	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	<	λ_{rw} =	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	**Section is Compact**	ABDS (A6.2.1-1)
$\lambda_w = 2D_c/t_w$ =	39.034	<	λ_{rw} =	137.27		ABDS (A6.2.2-1)

$\lambda_{pw(Dcp)} = (E/F_{yc})^{1/2}/(0.54(M_p/(R_hM_y))-.09)^2$ =	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

λ_{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,	$M_{nc} = R_{pc}M_{yc} =$	N/A	N/A	k-in	ABDS (A6.3.3-1)
If L_p < L_b <= L_r,	$M_{nc} =C_b[1-(1-F_{yr}S_{xc}/(R_{pc}M_{yc}))((L_b-L_p)/(L_r-L_p))]R_{pc}M_{yc} =$	33150	33150	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)
	$F_{cr} = (C_b\pi^2E/(L_b/r_t)^2)(1+0.078J(L_b/r_t)^2/(S_xh))^{0.5} =$	49.422	49.422	ksi	ABDS (A6.3.3-8)
	$L_b =$	272.5	272.5	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} = 2762.5 k-ft Constructability Condition (S1)

Use M_{nc} = 2762.5 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

Interior Beam Composite Section Flexural Capacity

Slab Thickness = t_s = 8 in
M_p = 6113.1 ft-k
Y' = 36.476 in from bottom of beam
D_p = 8.6239 in <= $0.42D_t$ **OK** ABDS (6.10.7.3-1)
D_t = 45.1 in $0.42D_t$ = 18.942 in

Check for Compact Section ABDS 6.10.6.2.2

F_y = 50 ksi < 70 ksi **OK**
D/t_w = 39.034 < 150 **OK** ABDS (6.10.2.1.1-1)
$2D_{cp}/t_w$ = 0 < $3.76(E/F_{yc})^{0.5}$ = 90.553 **OK** ABDS (6.10.6.2.2-1)

D_{cp} = d-Y'-tf = 0 in ABDS D6.3.2

Section is Compact

If D_p <= 0.1 D_t $M_n = M_p$ = 6113.1 ft-k ABDS (6.10.7.1.2-1)
Otherwise $M_n = M_p(1.07-0.7(D_p/D_t))$ = 5722.7 ft-k ABDS (6.10.7.1.2-2)

Use $M_{n(Int)}$ = 5722.7 ft-k

Exterior Beam Composite Section Flexural Capacity

Slab Thickness = t_s = 8 in
M_p = 5661.6 ft-k
Y' = 35.606 in from bottom of beam
D_p = 9.4935 in <= $0.42D_t$ **OK** ABDS (6.10.7.3-1)

Check for Compact Section ABDS 6.10.6.2.2

$2D_{cp}/t_w$ = 0 < $3.76(E/F_{yc})^{0.5}$ = 90.553 **OK**

D_{cp} = d-Y'-tf = 0 in ABDS D6.3.2

Section is Compact

If D_p <= 0.1 D_t $M_n = M_p$ = 5661.6 ft-k ABDS (6.10.7.1.2-1)
Otherwise $M_n = M_p(1.07-0.7(D_p/D_t))$ = 5223.6 ft-k ABDS (6.10.7.1.2-2)

Use $M_{n(Ext)}$ = 5223.6 ft-k ABDS 6.10.7.1.2

Reference

Effective Width of Concrete Slab — ABDS 4.6.2.6.1

Span = 99.057 ft — Concrete Slab Width = 86 in

Girder Spacing = 7.1667 ft — Concrete Slab Thickness= 8 in

Use Effective Width = 7.1667 ft — f'_c = 4.5 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		

$n = E/E_c$ = 7.1308 $E_c = 33000K_1w_c^{1.5}(f'_c)^{1/2}$ = 4066.8 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:	32.161	41.1	1321.8	-15.309	7537.8	171.52
W-Beam:	68	18.55	1261.4	7.2406	3565	15000
	100.16		2583.2		11103	15172

Long Term

y_b = 25.791 in

I = 26274 in^4

S top slab = 1360.7 in^3

S top beam = 2323.2 in^3

S bot beam = 1018.8 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:	96.482	41.1	3965.4	-9.3226	8385.3	514.57
W-Beam:	68	18.55	1261.4	13.227	11898	15000
	164.48		5226.8		20283	15515

Short Term

y_b = 31.777 in

I = 35797 in^4

S top slab = 2687 in^3

S top beam = 6725.6 in^3

S bot beam = 1126.5 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	3.825	41.1	2631.6	37.1	3400	-2.3359	0	-4.6239	12168
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	0.9461	36.476	N-Axis	395.45
Web	2	0.8636	33.96	50	18.55	1466.4	1.57	966.79	47.452	0	17.926	26287
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	4.8974	0	35.691	34506

P_{total} = 6031.6

$P_{total}/2$ = 3015.8

Y' = 36.476 in

M_p = 6113.1 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 =	4.5	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		

$n = E/E_c =$ 7.1308 $E_c = 33000K_1w_c^{1.5}(f'_c)^{1/2} =$ 4066.8 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:	19.072	41.1	783.86	-17.611	5915	101.72
W-Beam:	68	18.55	1261.4	4.9393	1659	15000
	87.072		2045.3		7573.9	15102

Long Term

y_b =	23.489	in
I =	22676	in^4
S top slab =	1049.3	in^3
S top beam =	1666	in^3
S bot beam =	965.36	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:	57.216	41.1	2351.6	-12.246	8580.4	305.15
W-Beam:	68	18.55	1261.4	10.304	7219.7	15000
	125.22		3613		15800	15305

Short Term

y_b =	28.854	in
I =	31105	in^4
S top slab =	1914.6	in^3
S top beam =	3772.2	in^3
S bot beam =	1078	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

-335544 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	3.825	41.1	1560.6	37.1	3400	-9.4292	0	-5.4935	8573.2
Top Flange	3	12.316	1.57	50	36.315	966.79	35.53	2433.2	0.0765	35.606	N-Axis	688.6
Web	2	0.8636	33.96	50	18.55	1466.4	1.57	966.79	35.051	0	17.056	25012
Bottom Flange	1	12.316	1.57	50	0.785	966.79	0	0	4.0278	0	34.821	33665

P_{total} = 4960.6

$P_{total}/2$ = 2480.3 Y' = 35.606 in

M_p = 5661.6 ft-k

CONTECH ENGINEERED SOLUTIONS / BIG R BRIDGE		
Project Name:	Mesa Top Drive Bridge	By: CDA
Project No.:	621715	Date: 7/13/2021
Subject:	SHEAR CONNECTORS	Page: 25 of 39

Reference

Fatigue

$d =$ 0.875 in	Spacing = $p =$ 18 in	$Z_{rI} = 5.5d^2 =$	4.2109 k	ABDS (6.10.10.2-1)
$\alpha = Z_r/d^2 =$ 8.155		$V_{srI} = V_{fI}Q/I =$	2.2763 k	
$n =$ 3 /group		$Q = yA =$	899.46 in^3	
Min Spacing = 5.25 in		$p_{max} = nZ_{rI}/V_{srI} =$	**5.5496 N.G. for Infinite Life**	ABDS (6.10.10.1.2-1)
$V_{fI} = \gamma_I V_{LL(fatigue)} =$ 78.72 k				
$V_{fII} = \gamma_{II} V_{LL(fatigue)} =$ 35.986 k		$Z_{rII} = pV_{srII}/n =$	6.244 k	
$\gamma_I =$ ***1.75***	Fatigue I - Infinite Life	$V_{srII} = V_{fII}Q/I =$	1.0406 k	ABDS (6.10.10.1.2-3)
$\gamma_{II} =$ ***0.8***	Fatigue II - Finite Life	$N = 10^{(34.5-\alpha)/4.28} =$	1E+06 cycles	ABDS (6.10.10.2-3)
$V_{LL(fatigue)} =$ 44.983 k		Stress Cycles per Truck, $n =$	***1***	ABDS Tbl 6.6.1.2.5-2
$I =$ 31105 in^4		$(ADTT)_{SL\ Max} = N/((365)(75)n) =$	52.242	ABDS (6.6.1.2.5-3)
$y =$ 9.3226 in		$p =$	0.85	ABDS Tbl 3.6.1.4.2-1
$A =$ 96.482 in^2		$ADTT_{SL}/p =$	61	ABDS (3.6.1.4.2-1)

Maximum ADTT = 61 OK

Ultimate Strength

$F_y =$ 50 ksi	$b =$ 86 in	
$f'_c =$ 4.5 ksi	$t_s =$ 8 in	
$A_s =$ 68 in^2	$H =$ 4 in	
$E_c =$ 4066.8 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

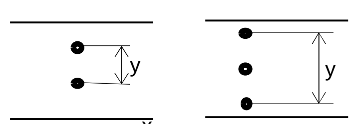
$\phi_{sc} =$ ***0.85*** — ABDS 6.5.4.2

$Q_n = 0.5A_{sc}(f'_cE_c)^{0.5} <$ $A_{sc}F_u =$ 40.673 k — Use $Q_n =$ 36.079 k — ABDS 6.10.10.4.3

$A_{sc} =$ 0.6013 in^2

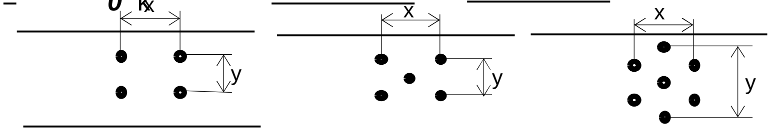
$F_u =$ ***60*** ksi — ABDS 6.4.4

$n_{min} = P/Qr =$	85.811 studs < Total Studs = 99 **OK**	ABDS (6.10.10.4.1-2)
$P = (P_P^2 + F_P^2)^{0.5} =$	2631.6 k	ABDS (6.10.10.4.2-1)
$P_{1P} = 0.85f'_cb_st_s =$	2631.6 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	

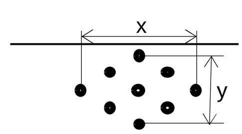


$x =$ 0 in

$y =$ 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)	$\eta_D =$	***1***	ABDS 1.3.3
$\eta_{iE} = \eta_D \eta_I =$	1 (Extreme Limit State)	$\eta_R =$	1	ABDS 1.3.4
		$\eta_I =$	***1***	ABDS 1.3.5

Resistance Factors — ABDS 6.5.4.2

$\phi_f =$ ***1*** $\phi_v =$ ***1***

Constructability Checks — ABDS 6.10.3

Compression Flange

$f_{bu}+f_l =$	30.804	ksi	<=	$\phi_f R_h F_{yc} =$	50	ksi	**OK**	ABDS (6.10.3.2.1-1)
$f_{bu}+1/3f_l =$	29.362	ksi	<=	$\phi_f F_{nc} = \phi_f M_{nc}/S_x =$	40.976	ksi	**OK**	ABDS (6.10.3.2.1-2)
$f_{bu} =$	29.362	ksi	<=	$\phi_f F_{crw} =$	616.66	ksi	**OK**	ABDS (6.10.3.2.1-3)

$F_{crw} = 0.9Ek/(D/t_w)^2 =$	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

$f_{bu}+f_l =$	30.804	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

Top Flange

f_{ftf} + 0 or $f_l/2 =$	25.534	ksi	<=	0.95	$R_h F_{ytf} =$	47.5	ksi	**OK**	ABDS (6.10.4.2.2-1 or 3)

Bottom Flange

$f_{fbf}+f_l/2 =$	45.479	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

Compression Flange:

Exterior Girder

$\eta_{iS}(M_u) =$	4705.9	ft-k	<	$\phi_f M_n =$	5223.6	ft-k	**OK**	ABDS 6.10.7.1

Interior Girder

$\eta_{iS}(M_u) =$	4884.4	ft-k	<	$\phi_f M_n =$	5722.7	ft-k	**OK**	ABDS 6.10.7.1

Tension Flange:

Exterior Girder

$\eta_{iS}(M_u+(1/3)f_l S_{xt}) =$	4708.6	ft-k	<	$\phi_f M_n =$	5223.6	ft-k	**OK**	ABDS 6.10.7.1

Interior Girder

$\eta_{iS}(M_u+(1/3)f_l S_{xt}) =$	4884.4	ft-k	<	$\phi_f M_n =$	5722.7	ft-k	**OK**	ABDS 6.10.7.1

Shear Capacity Checks — ABDS 6.10.9

$V_u =$	288.96	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
Fraction of Traffic Lane = p = 0.85 — ABDS Tbl 3.6.1.4.2-1

M_{LL} = 659.76 ft-k
I_x = 35797 in^4 (Interior Girder)
I_x = 31105 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 $ADTT_{SL}$
Interior Girder	Top of TF	C	10	4400000000	-11.309	-4.3772	-2.001	OK for Infinite Life
	Top of BF	C'	12	4400000000	24.221	9.3743	4.2854	OK for Infinite Life
	Top of Diaphragm	E	4.5	1100000000	-6.3094	-2.442	-1.1163	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	14.391	5.5697	2.5462	2434.333207
Exterior Girder	Top of TF	C	10	4400000000	-13.611	-6.0625	-2.7714	OK for Infinite Life
	Top of BF	C'	12	4400000000	21.919	9.7634	4.4633	OK for Infinite Life
	Top of Post Block	E	4.5	1100000000	-8.6107	-3.8354	-1.7533	OK for Infinite Life
	Bottom of Post Block	E	4.5	1100000000	5.0893	2.2669	1.0363	OK for Infinite Life
	Top of Diaphragm	E	4.5	1100000000	-8.6107	-3.8354	-1.7533	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	12.089	5.3849	2.4617	2693.758612

Use Maximum ADTT: Finite Life **Use ADTT = 2863** **OK** — ABDS (3.6.1.4.2-1)

								Reference
Diaphragm Spacing = L_b =	22.708 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	r_o = –	in	
$r_s = r_{y\text{-}y}$ =	1.26 in	h =	18.76 in	H = –		
I_x =	843 in^4	I_y =	20.7 in^4			

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

Member Design Check

P_u = 20.00 k <= $\phi_c P_n$ = 458.88 k OK

ϕ_c = 0.95 — ABDS 6.5.4.2

Total Load on Diaphragms

W = 0.0681 k/ft

$P_w = WL_b$ = 1.5472 k P_u = 1.4 P_w = 2.1661 k — ABDS (C4.6.2.7.1-4)

P_u = 20 k — ABDS 6.6.1.3.1

Member Capacity

KL_c/r = 50.673 <= 140 **OK** — ABDS 6.9.3

K = ***0.75*** — ABDS 4.6.2.5

$P_n = F_{cr}A_{eff}$ = 483.03 k — ABDS (6.9.4.2.2-1)

$F_{cr} = P_{cr}/A_g$ = 41.441 ksi — ABDS (6.9.4.2.2-2)

$P_{cr} = (0.658^{(Po/Pe)})P_o$ = 538.74 k, or $P_{cr} = 0.877P_e$ = 1275.2 k — ABDS (6.9.4.1.1-1) & (6.9.4.1.1-2)

P_o/P_e = 0.4486 <= 2.25   Use P_{cr} = 538.74 k

$P_o = F_yA_g$ = 650 k — ABDS 6.9.4.1.1

Reference

Use P_e = 1449.1 k

Elastic Flexural Buckling Resistance:

$P_e = (\pi^2E/(KL_c/r_s)^2)A_g$ = 1449.1 k — ABDS (6.9.4.1.2-1)

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

Use P_e = 5146.5 k

For open-sections doubly symmetric members:

$P_e = (\pi^2EC_w/(K_zL_z)^2+GJ)A_g/(I_x+I_y)$ = 5146.5 k — ABDS (6.9.4.1-3-1)

$K_zL_z = 0.5L_c$ = 42.565 in — ABDS C6.9.4.1.3

$G = 0.385E$ = 11165 ksi — ABDS 6.9.4.1.3

For open-sections singly symmetric members:

$P_e = ((P_{ey}+P_{ez})/(2H))(1-(1-4P_{ey}P_{ez}H/(P_{ey}+P_{ez})^2)^{1/2})$ = N/A k — ABDS (6.9.4.1-3-2)

$P_{ey} = (\pi^2E/(K_yL_{cy}/r_y)^2)A_g$ = N/A k — ABDS (6.9.4.1.3-4)

$P_{ez} = (\pi^2EC_w/(K_zL_z)^2+GJ)/r_o^2$ = N/A k — ABDS (6.9.4.1.3-5)

$A_{eff} = A_g-((b_f-b_{ef})t_f+(h-h_e)t_w)$ = 11.656 in^2 — ABDS 6.9.4.2.2

b/t_f =	7.2222	<=	$\lambda_{rf} = 0.56(E/F_y)^{1/2}$ =	13.487	Flanges are Nonslender	ABDS (6.9.4.2.1-1) & Tbl 6.9.4.2.1-1
h/t_w =	53.6	>	$\lambda_{rw} = 1.49(E/F_y)^{1/2}$ =	35.884	Web is Slender	

b/t_f =	7.2222	<=	$\lambda_{rf}(F_y/F_{cr})^{1/2}$ =	14.814	ABDS 6.9.4.2.2a
h/t_w =	53.6	>	$\lambda_{rw}(F_y/F_{cr})^{1/2}$ =	39.416	

b = 3.25 in

$b_{ef} = b_f(1-c_{1f}(F_{elf}/F_{cr})^{1/2})(F_{elf}/F_{cr})^{1/2}$ = 6.5294 in, use 6.5 in

$h_{ew} = h(1-c_{1w}(F_{elw}/F_{cr})^{1/2})(F_{elw}/F_{cr})^{1/2}$ = 14.92 in, use 14.92 in

ABDS (6.9.4.2.2a-1) or (6.9.4.2.2a-2)

c_{1f} = ***0.22*** c_{1w} = ***0.18*** c_{2f} = 1.4854 c_{2w} = 1.3079 — ABDS Tbl 6.9.4.2.2a-1

$F_{elf} = (c_{2f}\lambda_{rf}/(b/t_f))^2F_y$ = 384.71 ksi $F_{elw} = (c_{2w}\lambda_{rw}/(h/t_w))^2F_y$ = 38.335 ksi — ABDS (6.9.4.2.2a-4)

Reference

Loading Data

$R_u = V_u =$ 288.96 k (Total Factored Load)

$R_{LL} =$ 90.43 k (Unfactored w/o IM)
$R_{DL} =$ 74.49 k (Unfactored)
$R =$ 164.92 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
$\Delta_s = 1.2(Max(\Delta_{Rise}$ 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
$\theta_L =$	0.0050	0.0050	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
$\theta_s = \theta_D + \theta_L + \theta_R + \theta_T =$	-0.0009	0.0191	rad, use	0.0191 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' =	N	$A' = L_EW_E' - n_h\pi d^2/4$ =	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

$\sigma_s = R/(L_EW_E)$ = 1.2216 ksi
$\sigma_s' = R/(L_EW_E')$ = 1.2216 ksi < 1.25 ksi **OK**
1.25 $G_{Min}S_i'$ = 1.5987 > σ_s' **OK**

Check Deflection: — ABDS 14.7.6.3.3

$\delta = Max(\varepsilon_i \text{ or } \varepsilon_i')h_{ri}$ = 0.0052 in < $.09h_{ri}$ = 0.0231 **OK**
$\varepsilon_i = \sigma_s/(4.8G_{Min}S_i^2)$ = 2.0226 % — ABDS (C14.7.5.3.6-1)
$\varepsilon_i' = \sigma_s'/(4.8G_{Min}S_i'^2)$ = 2.0226 %

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

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)

Steel Friction Resistance = μV_{DL} = 13.32 k < H_m **Use Restraint System**
Coefficient of Friction = μ = ***0.2***

Concrete Friction Resistance = μV_{DL} = 26.64 k
Coefficient of Friction = μ = ***0.4***

Reference

Check Bearing Plates:

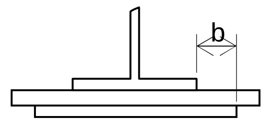


Plate Dimensions:	N_P =	8	in
	W_P =	22	in
	t_{Max} =	1.25	in
	t_{Min} =	1.25	in
	Taper =	0	in
	t =	1.25	in

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

Bottom Flange Width = b_f = 17.112 in

Pad Width Outside Flange = $b = (W_E' - b_f)/2 =$ 1.444 in

$V_{u(plate)} = (V_u/W_E)b =$ 20.863 k

$M_{u(plate)} = (V_u/W_E)b^2/2 =$ 15.063 k-in

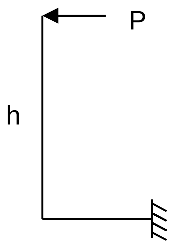
$\phi_f M_n = \phi_f F_y S_P =$ 104.17 k-in >= $M_{u(plate)}$ **OK**

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

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 =	3.04	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	$M_{yL} = F_y S_y$ =	152	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**	**<=**	**$\phi_f M_{yL}$ =**	**152**	**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

Reference

Check Post Block to Post Connection

Post Block Section = W14x22

b_f = 5 in

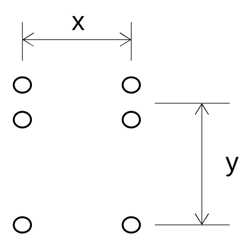
t_f = 0.335 in

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) — AISC Tbl 2-6

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 = 10 Ga Thrie-Beam

S = 2.87 in^3

F_y = ***50*** ksi

F_u = 65 ksi

γM_u = 106.91 in-k

FS = $\phi_f F_y S/\gamma M_u$ = 1.3423 (For Steel Yielding)

FS = $\phi_f F_u S/\gamma M_u$ = 1.7449 (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

$D = d\text{-}2t_f =$ 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 Tbl 6.13.2.6.6-1

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 = $w_{(ofp)} =$ 11.5 in — Min Inside Plate Width = 2.5 in **OK**

Inside Plate Width = $w_{(ifp)} =$ 3.875 in — Max Width = $b_f/2\text{-}k_1 =$ 4.425 in **OK**

Outside Plate Thickness = $t_{(ofp)} =$ 1 in — 17.1% 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 = $b_f/2\text{-}k_1 =$ 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	8.11	1265.86	953.73
DC_2	1.45	3.45	170.78	405.54
DW	2.69	1.59	316.20	187.51
$LL_{Des\ Veh}$	51.25	39.01	1392.95	1461.31
$LL_{Owner\ 1}$	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	108.98	85.10	4707.75	4537.65
	19.29	16.83	2270.09	1980.35
Strength II	19.29	16.83	2270.09	1980.35
	19.29	16.83	2270.09	1980.35
Service II	81.52	63.86	3563.67	3446.48
	14.90	13.15	1752.84	1546.78

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) =	4707.75	2270.09	3563.67
V (k) =	108.98	19.29	81.52

Bolt Capacity

		Reference
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) — AISC Tbl 2-6

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

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

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

$A_e = (\phi_uF_u/\phi_yF_y)A_n =$ 18.926 in^2 <= A_g, use $A_e =$ 18.926 in^2 — ABDS (6.13.6.1.3b-2)

$A_g = b_ft_f =$ 18.997 in^2

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

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

Minimum Number of Bolts Required = $n_t = P_{fy}/\phi_sR_n =$ 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

$M_{ubearing}$ = 4707.8 ft-k

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

Check Flange Splice Plates Capacities

Bottom Flange Plates (Tension)

$P_u = P_{fy} =$ 946.3 k

$A_{g(in)} = 2t_{(ifp)}w_{(ifp)} =$	9.6875	in^2	$A_{n(in)} = 2t_{(ifp)}(w_{(ifp)}\text{-}N_{vf}d_h/2) =$	7.3438	in^2	ABDS 6.8.3
$A_{g(o)} = t_{(ofp)}w_{(ofp)} =$	11.5	in^2	$A_{n(o)} = t_{(ofp)}(w_{(ofp)}\text{-}N_{fv}d_h) =$	9.625	in^2	
$A_{g(in+o)} =$	21.188	in^2	$A_{n(in+o)} =$	16.969	in^2	

$\gamma_{g(in)} = A_{g(in)}/A_{g(in+o)} =$	0.4572	$\gamma_{n(in)} = A_{n(in)}/A_{n(in+o)} =$	0.4328	ABDS C6.13.6.1.3b
$\gamma_{g(o)} = A_{g(o)}/A_{g(in+o)} =$	0.5428	$\gamma_{n(o)} = A_{n(o)}/A_{n(in+o)} =$	0.5672	ABDS C6.13.6.1.3b

$\phi_yP_n = \phi_yF_yA_{g(in)} =$	460.16 k	>=	$\gamma_{g(in)}P_u =$	432.67 k	**OK**	ABDS (6.8.2.1-1)
$\phi_uP_n = \phi_uF_uA_{n(in)}U =$	411.25 k	>=	$\gamma_{n(in)}P_u =$	409.54 k	**OK**	ABDS (6.8.2.1-2)
$\phi_yP_n = \phi_yF_yA_{g(o)} =$	546.25 k	>=	$\gamma_{g(o)}P_u =$	513.62 k	**OK**	ABDS (6.8.2.1-1)
$\phi_uP_n = \phi_uF_uA_{n(o)}U =$	539 k	>=	$\gamma_{n(o)}P_u =$	536.76 k	**OK**	ABDS (6.8.2.1-2)

U = ***1***

Bottom Flange Plates (Bearing)

$M_{ubearing(o)} = M_{ubearing}\gamma_{n(o)} =$ 2670.33 ft-k

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

$M_{ubearing(in)} = M_{ubearing}\gamma_{n(in)} =$ 2037.43 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

$M_u =$ 4707.75 ft-k

$H_{wshear} = (M_u - M_{nf})/A_w =$ 813.44 k

$R = (V_r^2 + H_{wshear}^2)^{1/2} =$ 1181.4 k

Minimum Number of Bolts Required = $n_t = R/\phi_s R_n =$ 22.74

Check Slip

$V_{uslip} =$ 81.52 k

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

$M_{uslip} =$ 3563.7 ft-k

$H_{wslip} = (M_u - M_{nf})/A_w =$ 501.92 k

$R = (V_{uuslip}^2 + H_{wslip}^2)^{1/2} =$ 508.5 k

Minimum Number of Bolts Required = $n_t = R/\phi_s R_n =$ 13.038

Check Bearing

$R_{tmax} =$ 1181.4 k

$N_b =$ 24

$R_u = R_{tmax}/N_b =$ 49.227 k <= $\phi_{bb}R_n$ (Web)   **OK**

$R_u = R_{tmax}/N_b =$ 49.227 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

Bearing Design Top Flange Splice **(This Section is Not Applicable)**

w_{DC} = 1.0759 k/ft L = 39.529 ft

$M_u = 1.25w_{DC}L^2/2$ = 1050.70 ft-k

$P_u = M/(d-t_f)$ = 354.87 k

\# of Bolts, n = ***2*** d_b = ***0.875*** in

$f_t = P_u/n$ = 177.43 k A_b = 0.6013 in^2

F_{ub} = 120 ksi — ABDS 6.4.3.1

Check Bolts $f_t/(\phi_t T_n)$ = 4.0443 > 1 **N/A**

Bolt Capacity in Tension

$\phi_t T_n$ = 0.76 $\phi_t A_b F_{ub}$= 43.872 k/bolt — ABDS (6.13.2.10.2-1)

Check Plate t = ***1.5*** in (thickness of Plate)

d_p = 4.5 in (depth of Plate)

a = 1.5 in (CL Bolt to Edge of Plate)

b = 1.1175 in (CL Bolt to Toe of Fillet of Connected Part)

$Q_u = [3b/(8a)-t^3/20]P_u$ = 19.629 k < 30% **N/A** — ABDS 6.13.2.10.3 & 6.13.2.10.4

$f_t = f_t+Q_u$ = 197.06 k

$f_v = V/n$ = 0 k

$f_t/(\phi_t T_n)$ = 4.4917 > 1 **N/A**

Check Weld To Plate

Flange Weld Capacity (Moment) = $R_n = R_r A_w$ = 83.366 k < P **N/A**

Fillet Weld Size = t = ***0.3125*** in

$t_e = 0.707t$ = 0.2209 in

$A_w = (b_f-tw)t_e$ = 2.4811 in^2

$R_r = 0.6\phi_{e2}F_{EXX}$ = 33.6 ksi — ABDS 6.13.3.2.4

F_{EXX} = ***70*** ksi

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 11.5 in wide by 55 in long outside plates and 1.25 in thick by 3.875 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