Bridge Design Calculations

for

Sulphur Creek Bridge

54.75' Long X 22.6667' Wide

Vehicular Modular Rolled Girder Bridge

With Cast In Place Concrete Deck

Napa County, CA

CONTECH Project No. 830834-010

Design Specifications:

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

Structural Materials:

Structural Steel:	**ASTM A709 GR 50W Weathering Steel**
Deck Concrete:	**f'c = 4.5 ksi, maximum unit weight = 145 pcf**
Elastomeric Pads:	**Grade 4, 55 Durometer**
Anchor Bolts:	**ASTM F1554 Grade 105 or Better**

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3/5/25

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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

P15 Owner-Specified Vehicle

Maximum ADTT is 31

35 psf Future Wearing Surface

TL-2 Design Rated Bridge Rail Non Crash Tested

Grade Load Adjustment Factor = 1.0029

Bridge Layout: Clear Width = 22.667 ft

Deck Out to Out Width = 22.667 ft

Horizontal Length @ CL Bridge = 54.75 ft

Skew at End 1 = 0 °

Skew at End 2 = 0 °

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}
54.75	54.75	54.75	54.75	0	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}
6.8048	6.8048	6.8048	0	0	0	0	0	0	0	0	0	0

Grade = 7.56%

DL Camber = 1.6875 in Use 1 11/16 in

Beam Splice Location = N/A

Number of Diaphragm Lines = 4 on Exterior Bays, 4 on Interior Bays

Number of Bridge Rail Post = 18 per side

Bridge Rail Post Spaced 1 at 3.125 ft, 15 at 3.125 ft, 1 at 3.125 ft and 0.891 ft from each end

Deck Layout: Use a Cast In Place Concrete Deck with Superelevation

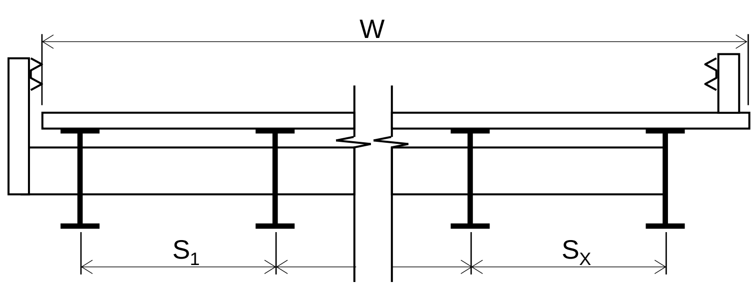
8 in Thick at Edge and 9 in at Edge with 6 inch by 0.667 feet Concrete Sidewalk

with #5 top transverse bars at 6 in spacing and 2 in of top cover

with #4 top longitudinal bars at 12 in spacing

with #5 bottom transverse bars at 6 in spacing and 1.5 in of bottom cover

with #5 bottom longitudinal bars at 8 in spacing



Unfactored Bearing Reactions in Kips		Max at Interior Girder			Max at Exterior Girder			Total @ Abutment		
		P	T	L	P	T	L	P	T	L
DC		27.22			20.64			95.72		
DW		5.44			5.44			21.78		
HL-93	LL	55.89			48.67			153.29		
	LL+IM	70.21			61.14			192.54		
P15	LL	78.51			68.36			107.65		
	LL+IM	98.13			85.46			134.56		
WS		-12.41	2.05		0.00	2.05		-12.41	8.21	
TU				4.14			4.14			16.56
BR				9.00			9.00			36.00
EQ			24.89	38.19		24.89	38.19		99.57	152.75

Notes: DC = Dead Load
DW = Wearing Surface Load
PL = Pedestrian Live Load
SL = Snow 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
T 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:

	Design Vehicle	Owner Specified Vehicle			
Vehicle	HL-93	P15	None	None	None
P	16 k	27 k	0 k	0 k	0 k
IM	1.33	1.25	1	1	1

Future Wearing Surface (FWS) = 35 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 =	10.4 in	
Total Slab Thickness =	9 in	Girder Web Thickness =	0.47 in	
Top Clearance =	2 in	Girder Spacing =	6.8048 ft	
Bottom Clearance =	1.5 in	Effective Span = S =	6.3519 ft	ABDS 9.7.3.2

	Size	Bar Diameter	Bar Area	Bar Spacing
Top Bar:	5	0.625 in	0.3068 in^2	6 in
Bottom Bar:	5	0.625 in	0.3068 in^2	6 in
Add. Overhang Top Bar:	N/A	0 in	0 in^2	N/A in

w = SIP + Slab = 127.5 psf

w = FWS = 35 psf

w = PL = 0 psf

Deck Moments

	(+)k-ft/ft	(-)k-ft/ft		
$M_{SIP+Slab}$ =	0.4311	-0.5742	Calculated using moment distribution	
M_{FWS} =	0.1183	-0.1576	Calculated using moment distribution	
M_{PL} =	0	0	Calculated using moment distribution	
M_{LL+I} =	4.9467	-4.5596	(Design Vehicle)	ABDS Table A4-1
M_{LL+I} =	7.8454	-7.2316	(Owner Specified Vehicle)	

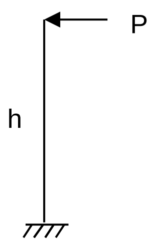
		(+)k-ft/ft	(-)k-ft/ft	
Strength I M_u =	$1.25\ M_{SIP+Slab} + 1.5\ M_{FWS} + 1.75\ M_{PL}$ =	0.7163	-0.9542	
Strength I M_u =	$1.25\ M_{SIP+Slab} + 1.5\ M_{FWS} + 1.75\ M_{LL+I}$ =	9.373	-8.9336	ABDS Tbl 3.4.1-1
Strength II M_u =	$1.25\ M_{SIP+Slab} + 1.5\ M_{FWS} + 1.35\ M_{LL+I}$ =	14.446	-10.717	
	Use M_u =	14.446	-10.717	

Reference

Deck Cantilever Overhang Moments

Deck Overhang = O =	1.1261	ft	Overhang Deck Thickness =	8	in	
Curb/Rail Moment Arm = X_c =	1.1261	ft	Overhang Design Thickness =	8	in	
$w_{stripover}$ = 45.0+10.0X =	46.261	in	Overhang Deck Load = w_o =	100	psf	
$M_{slab+Rail} = w_oO^2/2+P_cX_c$ =	0.0634	k-ft/ft	Overhang inside the Rail = O_i =	1.1261	ft	
$M_{FWS} = wO_i^2/2$ =	0.0222	k-ft/ft	Curb/Rail Load = P_c =	0	plf	
$M_{PL} = wO_i^2/2$ =	0	k-ft/ft	LL Moment Arm = X = O_i-1 =	0.1261	ft	ABDS Tbl 4.6.2.1.3-1
$M_{LL+I} = (PI/w_{stripover})X$ =	0.6963	k-ft/ft	(Design Vehicle)			
$M_{LL+I} = (PI/w_{stripover})X$ =	1.1044	k-ft/ft	(Owner Specified Design Vehicle)			

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



γF_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
γ =	1.2					ABDS A13.4.2

$h = H_e + a$ = 0 in

Post Spacing = L_P = 3.125 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+2c =	56.261	in	LL Moment Arm = X = O_i =	1.1261	ft	ABDS Tbl 4.6.2.1.3-1
			Curb Depth = c =	0	in	
$M_{IM} = (P_th+P_vX)/w_{striprail}$ =	0	ft-k				

Strength I	$M_u(-)$ =	1.25	$M_{slab+Rail}$	+ 1.5	M_{FWS}	+ 1.75	M_{PL} =	0.1126	k-ft/ft	
Strength I	$M_u(-)$ =	1.25	$M_{slab+Rail}$	+ 1.5	M_{FWS}	+ 1.75	M_{LL+I} =	1.3311	k-ft/ft	ABDS Tbl 3.4.1-1
Strength II	$M_u(-)$ =	1.25	$M_{slab+Rail}$	+ 1.5	M_{FWS}	+ 1.35	M_{LL+I} =	1.6034	k-ft/ft	
Extreme Event	$M_u(-)$ =	1.0	$M_{slab+Rail}$	+ 1.0	M_{FWS}	+ 1.0	M_{IM} =	0.0856	k-ft/ft	
							Use $M_u(-)$ =	1.6034	k-ft/ft	

Deck Capacity (+):

d_s =	6.1875	in	$c = A_sf_s/(\alpha_1 f'_c\beta_1 b)$ =	0.9722				
A_s =	0.6136	in^2	c/d_s =	0.1571	<=	$0.003/(0.003 + \varepsilon_{cl})$	**OK**	ABDS (5.6.2.1-1)
$a = \beta_1 c$ =	0.8021	in	ε_t =	0.0161	ϕ =	0.9		ABDS 5.6.2.1 & ABDS 5.5.4.2

$\phi M_n = \phi A_sf_s(d_s-a/2)$ = 15.977 k-ft/ft > $M_u(+/-)$ **OK** ABDS 5.6.3.2.3

Reference

Deck Capacity (-): d_s = 5.6875 in $c = A_sf_s/(\alpha_1 f'_c\beta_1 b)$ = 0.9722

A_s = 0.6136 in^2 c/d_s = 0.1709 <= $0.003/(0.003 + \varepsilon_{cl})$ **OK** ABDS (5.6.2.1-1)

$a = \beta_1 c$ = 0.8021 in ε_t = 0.0146 ϕ = 0.9 ABDS 5.6.2.1 & ABDS 5.5.4.2

$\phi M_n = \phi A_sf_s(d_s\text{-}a/2)$ = 14.597 k-ft/ft > M_u(+/-) **OK** ABDS 5.6.3.2.3

Overhang Deck Cap (-): d_s = 5.6875 in $c = A_sf_s/(\alpha_1 f'_c\beta_1 b)$ = 0.9722

A_s = 0.6136 in^2 c/d_s = 0.1709 <= $0.003/(0.003 + \varepsilon_{cl})$ **OK** ABDS (5.6.2.1-1)

$a = \beta_1 c$ = 0.8021 in ε_t = 0.0146 ϕ = 0.9 ABDS 5.6.2.1 & ABDS 5.5.4.2

$\phi M_n = \phi A_sf_s(d_s\text{-}a/2)$ = 14.597 k-ft/ft > M_u(-) **OK** ABDS 5.6.3.2.3

Longitudinal Steel

Bottom

% of Bottom Steel = $220/S^{1/2}$ = 87.291 % > 67%, use 67 % ABDS 9.7.3.2

$As_{long\ bottom\ required}$ = 0.4111 in^2/ft

Bar Size = ***5***

Bar Area = 0.3068 in^2

Bar Spacing = 8.9552 in max use 8 in

Top Y

Use # ***4*** bars at 12 in spacing (min of 1.5 x total thickness or 12 in)

Crack Control

$s <= 700\gamma_e/\beta_s f_{ss} - 2d_c$ = 14.168 in >= s **OK** ABDS 5.6.7

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

γ_e = 0.75 Class 2 exposure conditon

$f_{ss} = M_{Ser}(-)/A_s(d-a/2)$ = 19.576 ksi <= $0.6f_y$

M_{Ser} = 5.2915 ft-k

Reference

Development and Splice Length

Splice Class = ***B*** (A or B) — ABDS 5.10.8.4.3a

Epoxy Coating Used = ***Y*** (Y for Yes or N for No)

$0.75 <= \lambda = 7.5w_c <= 1.0 =$ 1 — ABDS 5.4.2.8

	Top Bars			Bottom Bars		Reference
	Trans	Long	Over	Trans	Long	
λ_{rl}	1	1	N/A	1	1	ABDS 5.10.8.2.1b
λ_{cf}	1.2	1.2	N/A	1.5	1.2	ABDS 5.10.8.2.1b
λ_{rl} x λ_{cf} (1.7 Max)	1.2	1.2	N/A	1.5	1.2	ABDS 5.10.8.2.1b
$0.4 <= \lambda_{rc} = d_b/(c_b+k_{tr}) <= 1.0$	0.4	0.4	N/A	0.4	0.4	ABDS 5.10.8.2.1c
k_{tr}	**0**	**0**	**N/A**	**0**	**0**	ABDS C5.10.8.2.1c
c_b	2.3125	2.875	N/A	1.8125	2.4375	ABDS 5.10.8.2.1c
$\lambda_{er} = A_{s\ req}/A_{s\ prov}$	1	1	N/A	1	1	ABDS 5.10.8.2.1c
$\ell_{db} = 2.4d_bf_y/f'_c{}^{1/2}$ (in)	42.426	33.941	N/A	42.426	42.426	ABDS 5.10.8.2.1a
$\ell_d = \ell_{db}(\lambda_{rl}\lambda_{cf}\lambda_{rc}\lambda_{er}/\lambda)$ (in)	20.365	16.292	N/A	25.456	20.365	ABDS 5.10.8.2.1a
Minimum Lap (in)	27	22		34	27	

Deck Slab Summary

Use a 8 in thick slab with a 1 in crown

with #5 top transverse bars at 6 in spacing & 27 in minimum lap lengths
with #5 bottom transverse bars at 6 in spacing & 34 in minimum lap lengths

with #4 top longitudinal bars at 12 in spacing & 22 in minimum lap lengths
with #5 bottom longitudinal bars at 8 in spacing & 27 in minimum lap lengths

Reference

Girder Spacing = S = 6.8048 ft — Number of Girders = N_S = 4

Overhang Width = O = 1.1261 ft — Girder Length = L = 53.5 ft

Traveled Way Overhang Width = O_T = 1.1261 ft

Dead Loads (DC_1)

Weight of Decking =	15 psf	Girder Self Wt. =	90 lb/ft
Concrete Deck =	112.5 psf	Diaphragm Wt & Misc. (Int) =	18.319 lb/ft
	127.5 psf	Diaphragm Wt & Misc. (Ext) =	9.1597 lb/ft
		Bridge Rail, Curb & Sidewalk (Int) =	0 lb/ft
		Bridge Rail, Curb & Sidewalk (Ext) =	60 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DC_I =	978.71 lb/ft	Dead Loads = DC_E =	738.65 lb/ft	
Shear = V_{DCI} =	26.181 k	Shear = V_{DCE} =	19.759 k	at support
Moment = M_{DCI} =	350.17 ft-k	Moment = M_{DCE} =	264.27 ft-k	at midpoint
Deflection = Δ_{DCI} =	1.7232 in	Deflection = Δ_{DCE} =	1.3006 in	at midpoint
Rotation = θ_{DCI} =	0.0086 rad	Rotation = θ_{DCE} =	0.0065 rad	at support

Dead Loads (DC_2)

Bridge Rail and Curb (Int) =	0 lb/ft
Bridge Rail and Curb (Ext) =	0 lb/ft
Sidewalk & Miscellaneous (Int) =	13.167 lb/ft
Sidewalk & Miscellaneous (Ext) =	13.167 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DL_I =	13.204 lb/ft	Dead Loads = DL_E =	13.204 lb/ft	
Shear = V_{DLI} =	0.3532 k	Shear = V_{DLE} =	0.3532 k	at support
Moment = M_{DLI} =	4.7242 ft-k	Moment = M_{DLE} =	4.7242 ft-k	at midpoint
Deflection = Δ_{DLI} =	0.0095 in	Deflection = Δ_{DLE} =	0.0107 in	at midpoint
Rotation = θ_{DLI} =	5E-05 rad	Rotation = θ_{DLE} =	5E-05 rad	at support

Wearing Surface (DW) — Total = 35 psf

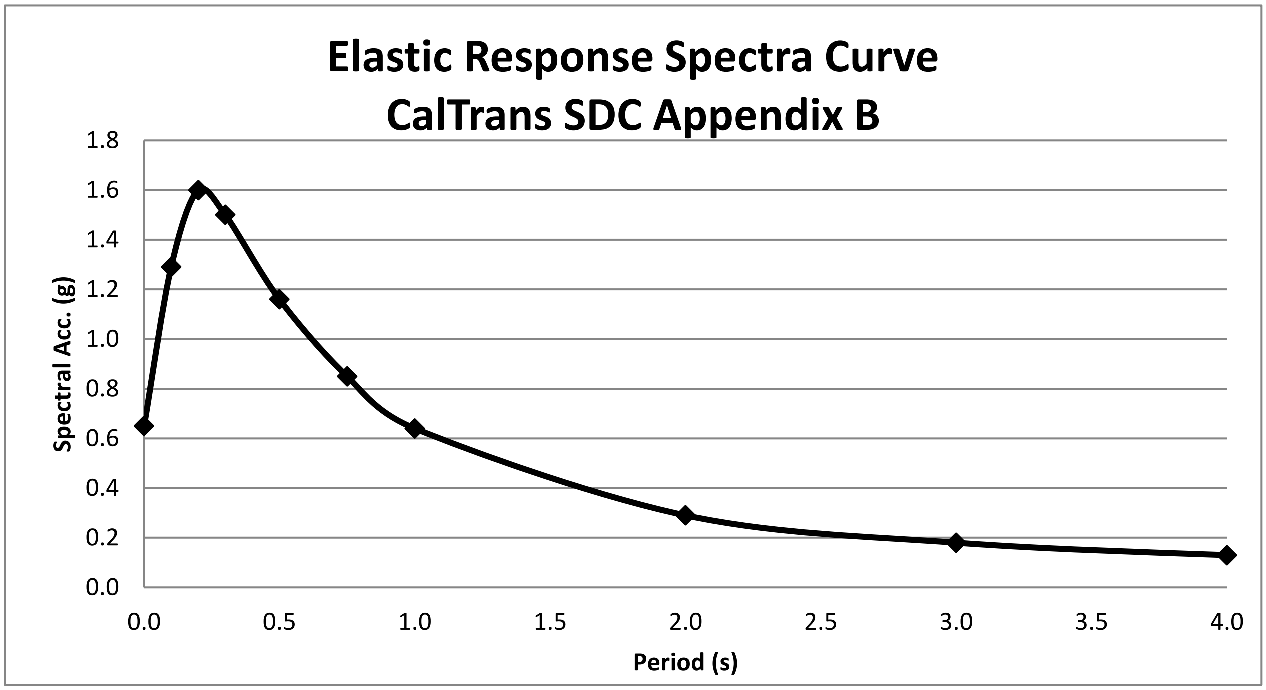
Miscellaneous (Int) =	0 lb/ft
Miscellaneous (Ext) =	0 lb/ft

Interior Girders		Exterior Girders		
Dead Loads = DW_I =	198.9 lb/ft	Dead Loads = DW_E =	198.9 lb/ft	
Shear = V_{DWI} =	5.3205 k	Shear = V_{DWE} =	5.3205 k	at support
Moment = M_{DWI} =	71.162 ft-k	Moment = M_{DWE} =	71.162 ft-k	at midpoint
Deflection = Δ_{DWI} =	0.1425 in	Deflection = Δ_{DWE} =	0.1609 in	at midpoint
Rotation = θ_{DWI} =	0.0007 rad	Rotation = θ_{DWE} =	0.0008 rad	at support

DL Camber = 1.6737 in

Residual Camber = 0 in — **Total Camber = 1.6737 in** — **use** **1.6875**

Reference



V_{s30} = 486 m/s

Location: Latitude = *38.488193*

Longitude = *-122.486377*

Use Sa = 0.8473 at T_m = 0.0308 sec for Horizontal Loading

Use Sa = 0.65 at T_m = 0 sec for Longitudinal Loading

Tributary Permanent Load = W = 58.751 k ((Total Bridge Weight + gPL(Total PL)) / Number of Girders)

γ_{SL} = 0.00 — ASCE 12.7.2

Connection Between Superstructure and Abutment

	Transverse	Longitudinal	
Seismic Load = SaW/R =	62.228 k	47.735 k	(for Abutment Connection)
Seismic Load = SaW =	49.783 k	38.188 k	(for Bridge Reaction)

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

		Reference
3-Second Gust Wind Speed = V =	110 mph	ABDS Fig. 3.8.1.1.2-1
Wind Exposure Category =	C	ABDS 3.8.1.1.5
Wind Pressure = $P_Z = 2.56\text{x}10^{-6}V^2K_ZGC_D$ =	0.0403 ksf	ABDS (3.8.1.2.1-1)
Wind Pressure = $P_Z = 2.56\text{x}10^{-6}V^2K_ZK_{zt}K_dK_eGC_f$ =	N/A ksf	

K_z (AASHTO) = $(2.5\ln(Z/a)+b)^2/c$ = 1.0014 — ABDS (3.8.1.2.1-2, -3, -4)

a = 0.0984

b = 7.35

c = 478.4

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

K_e = N/A

		Reference
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 =	0.5282	ABDS Tbl 3.8.1.1.2-1
*Service I Load Factor = γ_{SerI} =	0.4044	

*Load combinations are adjusted for 80 mph and 70 mph wind speeds for Strength V and Service I load combinations respectively and Kz adjusted to 1.0

Reference

Pedestrian 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/500 = 1.284 **OK**

Snow Load (SL)

Total SL = 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

Vehicle LL Distribution Factors (g)

Concrete Deck on Steel Beams

	Design Lanes	HL-93		P15		None		None		None		
		Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	
Moment	1	0.4159	0.6351	0.4159	0.6351	0	0	0	0	0	0	
	All	0.5529	0.4941	0.5529	0.4941	0	0	0	0	0	0	ABDS 4.6.2.2.2
	Use	0.5529	0.6351	0.5529	0.6351	0	0	0	0	0	0	
Shear	1	0.6322	0.6351	0.6322	0.6351	0	0	0	0	0	0	
	All	0.7293	0.5197	0.7293	0.5197	0	0	0	0	0	0	ABDS 4.6.2.2.3
	Use	0.7293	0.6351	0.7293	0.6351	0	0	0	0	0	0	

Reference

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
$V_{LL\ (total)}$ =	59.47	48.14	17.169	96.27	k	(Max(T or A)IM + L)	ABDS 3.6.1.3.1
$M_{LL\ (total)}$ =	691.08	619.82	229.63	1148.77	k-ft	(Max(T or A)IM + L)	ABDS 3.6.1.3.1
$\Delta_{LL\ (total)}$ =	1.0401	0.8536	0.3701	1.3834	in	(Max(IM_f(T) or IM_f(0.25T)) + L)	ABDS 3.6.1.3.2
$\theta_{LL\ (total)}$ =	0.0052	0.0041	0.0018	0.0088	rad	(Max(T or A)IM + L)	ABDS 3.6.1.3.1

	Interior Girder	Exterior Girder		
V_{LL+IM} =	70.207 k	61.137 k	at support	(fgV_{LL})
M_{LL+IM} =	635.13 ft-k	729.53 ft-k	near midpoint	(fgM_{LL})

Δ_{LL+I} =	0.6917 in	near midpoint	($N_L\Delta_{LL}/N_S$)	<= L/500 = 1.284	**OK**
θ_{LL} =	0.0044 rad	at support	($N_L\theta_{LL}/N_S$)		

Fatigue I Truck: HL-93 Fatigue — IM_f = ***1.15*** (ABDS 3.6.2.1)

				Interior	Exterior		
$V_{LL\ (total)}$ =	47.55 k		V_{LL+IM} =	28.806	28.937 k	at support	(IM_fgV_{LL}/1.2)
$M_{LL\ (total)}$ =	480.61 k-ft		M_{LL+IM} =	191.58	292.5 ft-k	near midpoint	(IM_fgM_{LL}/1.2)

Fatigue II Truck: P9 — IM_f = ***1.15*** (ABDS 3.6.2.1)

				Interior	Exterior		
$V_{LL\ (total)}$ =	100.86 k		V_{LL+IM} =	61.104	61.381 k	at support	(IM_fgV_{LL}/1.2)
$M_{LL\ (total)}$ =	1140.75 k-ft		M_{LL+IM} =	454.72	694.26 ft-k	near midpoint	(IM_fgM_{LL}/1.2)

Owner Specified Vehicle (LL_O): P15

	Truck
$V_{LL\ (total)}$ =	107.65 k
$M_{LL\ (total)}$ =	1197.51 k-ft

IM = 1.25 (ABDS 3.6.2.1)

N_L = 1

	Interior Girder	Exterior Girder		
V_{LL+IM} =	98.132 k	85.455 k	at support	($IMgV_{LL}$)
M_{LL+IM} =	827.61 ft-k	950.61 ft-k	near midpoint	($IMgM_{LL}$)

Construction Loading:

Conventional Deck Forming (CEL) = 0 psf

Overhang Forming Load (CEL) = W_{CEL} = 250.71 lb/lf

Deck Machine Load (CLL) = R_{CLL} = 3008.6 lbs

	Int. Girders		Ext. Girders	
	M (k-ft)	V (k)	M (k-ft)	V (k)
CEL =	0	0	89.7	6.7066
CLL =	0	0	40.239	3.0086

Reference

Deck Clear Width = W = 22.667 ft
Girder Spacing = S = 6.8048 ft
LL Overhang = d_e = 1.1261 ft
Span Length = L = 53.5 ft
Deck Type = Cast-In-Place Concrete

Number of Girders = N_S = 4
Number of Lanes = N_L = 2
Skew End 1 = θ_1 = 0
Skew End 2 = θ_2 = 0
Use Skew = θ = 0

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***

$K_g = n(I+Ae_g^2)$ = 93445 — 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)/2+h = 19 in
t_s or t_g = 8 in
Haunch = h = 0.25 in

Beam = W30x90 — A = 26.3 in^2
I = 3610 in^4 — d = 29.5 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 =	1.0156	ABDS Tbl 4.6.2.2.2a-1
	2+	g = S/7.5 =	0.9073	
Steel Bridge Plank Deck on Steel Beams, Interior Moment:	1	g = S/9.2 =	0.7397	ABDS Tbl 4.6.2.2.2c-1
	2+	g = S/9.0 =	0.7561	
Open Steel Grid or Steel Ties on Steel Beams, Interior Moment:	1	g = S/10.0 =	0.6805	ABDS Tbl 4.6.2.2.2b-1
	2+	g = S/10.0 =	0.6805	
Concrete Deck on Steel Beams, Interior Moment:	1	$g = 0.06+(S/14)^{0.4}(S/L)^{0.3}(K_g/(12Lt_s^3))^{0.1}$ =	0.4159	ABDS Tbl 4.6.2.2.2b-1
	2+	$g = 0.075+(S/9.5)^{0.6}(S/L)^{0.2}(K_g/(12Lt_s^3))^{0.1}$ =	0.5529	
Concrete Deck on Steel Beams, Exterior Moment:	2+	$g = (0.77+d_e/9.1)g_{interior}$ =	0.4941	ABDS Tbl 4.6.2.2.2d-1
Concrete Deck on Steel Beams, Interior Shear:	1	g = 0.36+S/25.0 =	0.6322	ABDS Tbl 4.6.2.2.3a-1
	2+	$g = 0.2+(S/12)-(S/35)^{2.0}$ =	0.7293	
Concrete Deck on Steel Beams, Exterior Shear:	2+	$g = (0.6+d_e/10)g_{interior}$ =	0.5197	ABDS Tbl 4.6.2.2.3b-1

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

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

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): $m_1(1+(S-T)/S)/2$

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

Note: (S-T) or (S-X) minimum is 0 except on 3 girder bridges.

	HL-93	P15	None	None	None
Lane Width (ft)	12	12	22.667	22.667	22.667
Number of Lanes	2	1	1	1	1
m_x	1	***1***	***1***	***1***	***1***
Wheel to Edge of Lane = C (ft)	2	2	0	0	0
Track Width = T (ft)	6	6	0	0	0
Track Width Between Trucks = X (ft)	4	6	0	0	0

Rigid Cross-Section (C4.6.2.2.2d): — ABDS C4.6.2.2.2d

$R = N_L/N_S+X_{ext}\Sigma e/\Sigma x^2$

X_{ext} = 10.207

Σx^2 = 231.53

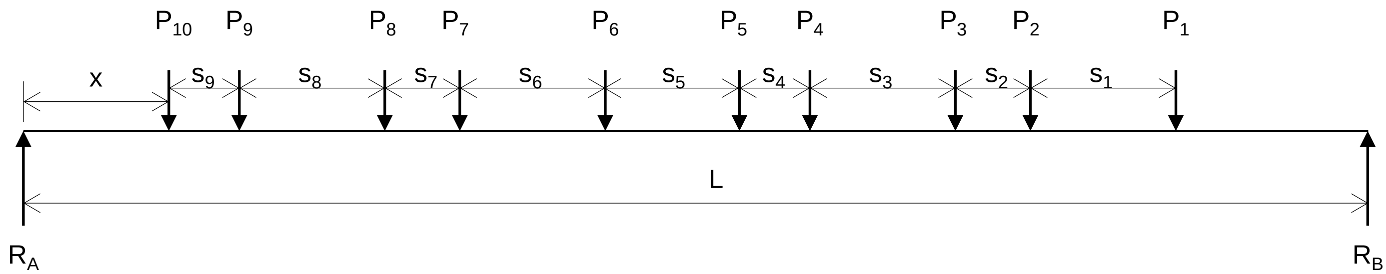
Girder	ΣS	x	x^2	Σx^2
1	0	10.207	104.19	104.19
2	6.8048	3.4024	11.576	115.76
3	13.61	3.4024	11.576	127.34
4	20.414	10.207	104.19	231.53
5	20.414	0	0	231.53
6	20.414	0	0	231.53
7	20.414	0	0	231.53
8	20.414	0	0	231.53
9	20.414	0	0	231.53
10	20.414	0	0	231.53

N_L	m	HL-93			
		e	Σe	R	mR
1	1.2	6.3333	6.3333	0.5292	0.6351
2	1	-5	1.3333	0.5588	0.5588
3	0.85	0	1.3333	0.8088	0.6875
4	0.65	0	1.3333	1.0588	0.6882
5	0.65	0	1.3333	1.3088	0.8507
6	0.65	0	1.3333	1.5588	1.0132
7	0.65	0	1.3333	1.8088	1.1757

N_L	P15			None			None			None		
	e	Σe	R	e	Σe	R	e	Σe	R	e	Σe	R
1	6.3333	6.3333	0.5292	11.333	11.333	0.7496	11.333	11.333	0.7496	11.333	11.333	0.7496
2	0	6.3333	0.7792	0	11.333	0.9996	0	11.333	0.9996	0	11.333	0.9996
3	0	6.3333	1.0292	0	11.333	1.2496	0	11.333	1.2496	0	11.333	1.2496
4	0	6.3333	1.2792	0	11.333	1.4996	0	11.333	1.4996	0	11.333	1.4996
5	0	6.3333	1.5292	0	11.333	1.7496	0	11.333	1.7496	0	11.333	1.7496
6	0	6.3333	1.7792	0	11.333	1.9996	0	11.333	1.9996	0	11.333	1.9996
7	0	6.3333	2.0292	0	11.333	2.2496	0	11.333	2.2496	0	11.333	2.2496

Summary of Distribution Factors	Design Lanes	HL-93		P15		None		None		None	
		Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
Lever Rule	1	0.671	0.5229	0.5591	0.4358	0	0	0	0	0	0
	All	0.7652	0.4358	0.6183	0.4358	0	0	0	0	0	0
Minimum Using Rigid Cross-Section	1		0.6351		0.5292		0		0		0
	All		0.5588		0.5292		0		0		0

Design Truck =	HL-93	L =	53.5	ft
Alternate or Tandem Design Truck =	HL-93 Tandem			
Fatigue I Design Truck =	HL-93 Fatigue	E =	29000	ksi
Fatigue II Design Truck =	P9	I =	11022	in^4

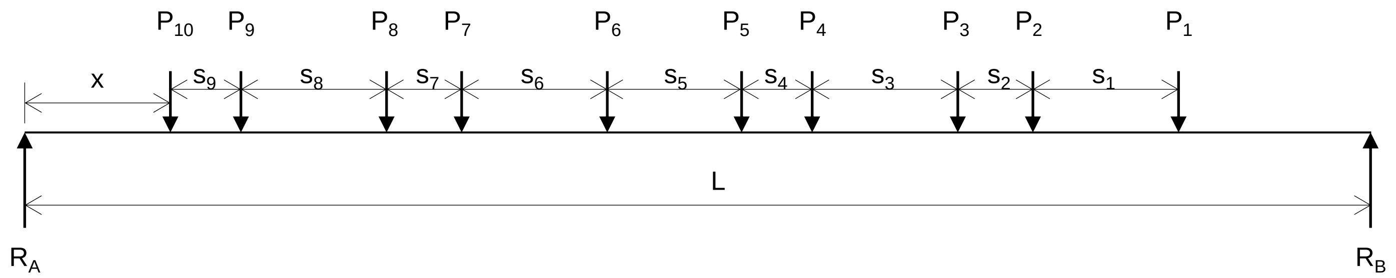


	Design Truck	Alt or Tandem	Fatigue I Truck	Fatigue II Truck	
s_1 =	13.96	3.9886	13.96	15.705	ft
s_2 =	13.96	0	29.915	4.4872	ft
s_3 =	0	0	0	13.462	ft
s_4 =	0	0	0	4.4872	ft
s_5 =	0	0	0	13.462	ft
s_6 =	0	0	0	4.4872	ft
s_7 =	0	0	0	13.462	ft
s_8 =	0	0	0	4.4872	ft
s_9 =	0	0	0	0	ft
Total Length =	27.92	3.9886	43.875	74.039	ft

	Design Truck	Alt or Tandem	Fatigue ITruck	Fatigue II Truck	
P_1 =	8	25	8	26	k
P_2 =	32	25	32	27	k
P_3 =	32	0	32	27	k
P_4 =	0	0	0	27	k
P_5 =	0	0	0	27	k
P_6 =	0	0	0	27	k
P_7 =	0	0	0	27	k
P_8 =	0	0	0	27	k
P_9 =	0	0	0	27	k
P_{10} =	0	0	0	0	k
Total Weight =	72	50	72	242	k

	Design Loading						Splice Loading:				
Design Truck	M_{Max} =	691.08	ft-k	with x =	15.058	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	R_{Max} = V_{Max} =	59.475	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	1.0401	in	with x =	15.058	ft					
	θ_A =	0.0052	rad	with x =	15.058	ft					
	θ_B =	0.0049	rad	with x =	15.058	ft					
Alt or Tandem	M_{Max} =	619.82	ft-k	with x =	25.793	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	R_{Max} = V_{Max} =	48.136	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	0.8536	in	with x =	25.793	ft					
	θ_A =	0.004	rad	with x =	25.793	ft					
	θ_B =	0.0041	rad	with x =	25.793	ft					
Fatigue I Truck	M_{Max} =	480.61	ft-k	with x =	-4.6415	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	R_{Max} = V_{Max} =	47.546	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
Fatigue II Truck	M_{Max} =	1140.7	ft-k	with x =	-12.514	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	R_{Max} = V_{Max} =	100.86	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft

Owner-Specified Vehicle 1 =	P15	L =	53.5	ft
Owner-Specified Vehicle 2 =	None			
Owner-Specified Vehicle 3 =	None	E =	29000	ksi
Owner-Specified Vehicle 4 =	None	I =	11022	in^4



	Owner Truck 1	Owner Truck 2	Owner Truck 3	Owner Truck 4	
s_1 =	17.949	0	0	0	ft
s_2 =	17.949	0	0	0	ft
s_3 =	17.949	0	0	0	ft
s_4 =	17.949	0	0	0	ft
s_5 =	17.949	0	0	0	ft
s_6 =	17.949	0	0	0	ft
s_7 =	17.949	0	0	0	ft
s_8 =	0	0	0	0	ft
s_9 =	0	0	0	0	ft
Total Length =	125.64	0	0	0	ft

	Owner Truck 1	Owner Truck 2	Owner Truck 3	Owner Truck 4	
P_1 =	26	0	0	0	k
P_2 =	54	0	0	0	k
P_3 =	54	0	0	0	k
P_4 =	54	0	0	0	k
P_5 =	54	0	0	0	k
P_6 =	54	0	0	0	k
P_7 =	54	0	0	0	k
P_8 =	54	0	0	0	k
P_9 =	0	0	0	0	k
P_{10} =	0	0	0	0	k
Total Weight =	404	0	0	0	k

	Design Loading						Splice Loading:				
Owner Truck 1	M_{Max} =	1197.5	ft-k	with x =	-45.035	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	$R_{Max} = V_{Max}$ =	107.65	k	with x =	-36.244	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	1.8175	in	with x =	-45.035	ft					
Owner Truck 2	M_{Max} =	0	ft-k	with x =	0	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	$R_{Max} = V_{Max}$ =	0	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	0	in	with x =	0	ft					
Owner Truck 3	M_{Max} =	0	ft-k	with x =	0	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	$R_{Max} = V_{Max}$ =	0	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	0	in	with x =	0	ft					
Owner Truck 4	M_{Max} =	0	ft-k	with x =	0	ft	M_{Max} = N/A	ft-k	with x =	N/A	ft
	$R_{Max} = V_{Max}$ =	0	k	with x =	0	ft	V_{Max} = N/A	k	with x =	N/A	ft
	Δ_{Max} =	0	in	with x =	0	ft					

Reference

Lateral Stress:

Girder Depth = d =	29.5 in	L =	53.653 ft
Flange Thickness = t_f =	0.61 in	L_b =	17.885 ft
Flange Width = b_f =	10.4 in	O_D =	1.1261 ft

Dead Loading

$f_{lDC} = 72M_{DC}/(t_f b_f^2)$ = 0.8442 ksi

$M_{DC} = W_{DC}L_b^2/12$ = 0.7735 k-ft

$W_{DC} = (w_d O_D^2/2)/d$ = 0.029 k/ft

Overhang Wet Concrete Load = w_d = 112.5 psf

Wind Loading

$f_{lWS} = 72M_{WS}/(t_f b_f^2)$ = 6.5963 ksi

$M_{WS} = WL_b^2/10 + WL^2/8N_b$ = 6.0446 k-ft — ABDS (C4.6.2.7.1-3)

N_b = 4 (Number of Beams that load is distributed to)

$W = \eta_i \gamma P_D d/2$ = 0.0496 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 = 0.0403 ksf

Construction Loading

$f_{lCEL} = 72M_{CEL}/(t_f b_f^2)$ = 3.341 ksi

$M_{CEL} = F_{CEL}L_b^2/12$ = 3.0616 k-ft — ABDS (C6.10.3.4.1-2)

$F_{CEL} = W_{CEL}O_D/d$ = 0.1148 k/ft

$f_{lCLL} = 72M_{CLL}/(t_f b_f^2)$ = 3.3625 ksi

$M_{CLL} = P_{CLL}L_b/8$ = 3.0812 k-ft — ABDS (C6.10.3.4.1-3)

$P_{CLL} = R_{CLL}O_D/d$ = 1.3782 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:

$I = I_{y\text{-Girders}} + I_{y\text{-Deck}} =$ 3E+06 in^4 $L =$ 642 in (c-c brg)

$E = E_b =$ 29000 ksi

Iy Girders:

Girder	S	y	y^2	A	Ay^2	I_y
1	0	122.49	15003	26.3	394576	115
2	81.658	40.829	1667	26.3	43842	115
3	81.658	-40.829	1667	26.3	43842	115
4	81.658	-122.49	15003	26.3	394576	115
5	0	0	0	0	0	0
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
	244.97				876835	460

$I_{y\text{-Girders}} = \Sigma A_y^2 + \Sigma I_y =$ 877295 in^4

I_y Deck:

Width = d = 272 in

Equivalent Thickness = b = 9 in

$E_d =$ 4066.8 ksi

$I_{y\text{-Deck}} = bd^3/12(E_d/E_b) =$ 2E+06 in^4

$\Delta_{DL\ lat} = 5WL^3/(384EI) =$ 0.0093 in $W =$ 234.5 k/ft

$\Delta_{WL\ lat} = 5W_{WL}L^3/(384EI) =$ 0.0007 in $W_{WL} =$ 16.43 k

$T_m = 2\pi(\Delta_{DL\ lat}/g)^{1/2} =$ 0.0308 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} (in^3)	f_{ftf} (ksi)	S_{xbf} (in^3)	f_{fbf} (ksi)	f_l (ksi)	f_lS_x (k-ft)
Exterior	DC_1	264.27	19.759	245	12.944	245	12.944	0.8442	17.235
	DC_2	4.7242	0.3532	1214.7	0.0467	341.13	0.1662	***0***	0
	DW	71.162	5.3205	1214.7	0.703	341.13	2.5033	***0***	0
	PL	0	0	7159.2	0	377.11	0	***0***	0
	SL	0	0	7159.2	0	377.11	0	***0***	0
	LL	729.53	61.137	7159.2	1.2228	377.11	23.214	***0***	0
	LL_O	950.61	85.455	7159.2	1.5934	377.11	30.249	***0***	0
	WS	***0***	***0***	7159.2	0	377.11	0	6.5963	207.3
	CEL	89.7	6.7066	245	4.3935	245	4.3935	3.341	68.213
	CLL	40.239	3.0086	245	1.9709	245	1.9709	3.3625	68.65
	LL_C	0	0	245	0	245	0	***0***	0
Interior	DC_1	350.17	26.181	245	17.151	245	17.151	***0***	0
	DC_2	4.7242	0.3532	1953.8	0.029	355.55	0.1594	***0***	0
	DW	71.162	5.3205	1953.8	0.4371	355.55	2.4018	***0***	0
	PL	0	0	953159	0	388.83	0	***0***	0
	SL	0	0	953159	0	388.83	0	***0***	0
	LL	635.13	70.207	953159	0.008	388.83	19.601	***0***	0
	LL_O	827.61	98.132	953159	0.0104	388.83	25.541	***0***	0
	WS	***0***	***0***	953159	0	388.83	0	6.5963	213.74
	CEL	0	0	245	0	245	0	***0***	0
	CLL	0	0	245	0	245	0	***0***	0
	LL_C	0	0	245	0	245	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 ($1.25DC_1+1.5(CEL+CLL)+1.75LL_C$):	25.727	11.11	36.837	29.43
Strength III ($1.25DC_1+1.5CEL+1.0WS$):	22.77	12.663	35.433	26.991
Strength IV ($1.5DC_1+1.5(CEL+CLL)$):	28.963	11.321	40.284	32.737
Strength V ($1.25DC_1+1.5CEL+1.35(CLL+LL_C)+1.0\gamma_V WS$):	22.77	9.5507	32.321	25.954
	28.963	12.663	40.284	32.737

Interior Girder

	f_{bu} (ksi)	f_l (ksi)	$f_{bu}+f_l$ (ksi)	$f_{bu}+1/3f_l$ (ksi)
Strength I ($1.25DC_1+1.75LL_C$):	21.439	0	21.439	21.439
Strength III ($1.25DC_1+1.0WS$):	21.439	6.5963	28.035	23.637
Strength IV ($1.5DC_1$):	25.726	0	25.726	25.726
Strength V ($1.25DC_1+1.35LL+1.0\gamma_V WS$):	21.439	3.484	24.923	22.6
	25.726	6.5963	28.035	25.726

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 ($1.00DC_1+1.00DC_2+1.0DW+1.00Max(PLorSL)+1.00\gamma_{SerI}WS$):	13.694	15.614	3.5116	15.45	17.369
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	14.917	38.828	3.5116	16.672	40.584
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	15.283	45.792	0.8442	15.706	46.214
	15.283	45.792	3.5116	16.672	46.214

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 ($1.00DC_1+1.00DC_2+1.0DW+1.00Max(PL\ or\ SL)+1.00\gamma_{SerI}WS$):	17.617	19.712	2.6674	18.951	21.046
Service I ($1.00DC_1+1.00DC_2+1.0DW+1.00LL+1.00\gamma_{SerI}WS$):	17.625	39.314	2.6674	18.959	40.647
Service II ($1.00DC_1+1.00DC_2+1.0DW+1.30LL$):	17.627	45.194	0	17.627	45.194
	17.627	45.194	2.6674	18.959	45.194

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 ($1.25(DC_1+DC_2)+1.5DW+1.75Max(PL\ or\ SL)$):	442.99	33.121	1.0552	21.544	450.17
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	1719.7	140.11	1.0552	21.544	1726.9
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	1726.3	148.49	1.0552	21.544	1733.5
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	442.99	33.121	7.6515	228.84	519.27
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	510.24	38.149	1.2662	25.852	518.86
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	1427.9	115.66	4.5392	131.03	1471.5
	1726.3	148.49	7.6515	228.84	1733.5

Interior Girders

	M_u (k-ft)	V_u (k)	f_{lbf} (ksi)	f_lS_x (k-ft)	Mu + $1/3f_lS_{xt}$
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75Max(PL\ or\ SL)$):	550.36	41.148	0	0	550.36
Strength I ($1.25(DC_1+DC_2)+1.5DW+1.75LL$):	1661.8	164.01	0	0	1661.8
Strength II ($1.25(DC_1+DC_2)+1.5DW+1.35LL_O$):	1667.6	173.63	0	0	1667.6
Strength III ($1.25(DC_1+DC_2)+1.5DW+1.0WS$):	550.36	41.148	6.5963	213.74	621.6
Strength IV ($1.5(DC_1+DC_2)+1.5DW$):	639.08	47.782	0	0	639.08
Strength V ($1.25(DC_1+DC_2)+1.5DW+1.35LL+1.0\gamma_VWS$):	1407.8	135.93	3.484	112.89	1445.4
	1667.6	173.63	6.5963	213.74	1667.6

Max f_l = 12.663 ksi <= $0.6F_y$ = 30 ksi **OK** ABDS (6.10.1.6-1)

								Reference
Beam Size = W30x90				$F_y = F_{yc} = F_{yt} = F_{yw} =$ 50 ksi				
$Z_x =$ 283 in^3				E = 29000 ksi				
$S_x =$ 245 in^3								
$b_f =$ 10.4 in	$D = d\text{-}2t_f =$ 28.28 in			$M_p = F_yZ_x =$ 14150 k-in				
$t_f =$ 0.61 in	$D_c = D_{cp} = D/2 =$ 14.14 in			$M_y = F_yS_x =$ 12250 k-in				
d = 29.5 in	$h = d\text{-}t_f =$ 28.89 in			$F_{yr} = 0.7F_{yc} =$ 35 ksi				
$t_w =$ 0.47 in								
$I_x =$ 3610 in^4	$D/t_w =$	60.17	<=	150			OK	ABDS (6.10.2.1.1-1)
	$b_f/2t_f =$	8.5246	<=	12			OK	ABDS (6.10.2.2-1)
	$b_f =$	10.4	>=	D/6 = 4.7133			OK	ABDS (6.10.2.2-2)
	$t_f =$	0.61	>=	$1.1t_w =$ 0.517			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

$V_p = 0.58F_yDt_w =$ 385.46 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 $<= 1.40(Ek/F_{yw})^{1/2}$, $C = (1.12/(D/t_w))(Ek/F_{yw})^{1/2} =$		1.0024	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 =	1.2576	ABDS (6.10.9.3.2-6)
k = ***5***	Use C =	1	

$V_n = V_{cr} = CV_p =$ 385.46 k — ABDS (6.10.9.2-1)

Non-Composite Section Flexural Capacity

Moment Gradient Modifier: $C_b =$ ***1*** — ABDS (A6.3.3-6)

Slenderness Ratios:	$\lambda_{rw} = 5.7\ (E/F_{yc})^{1/2} =$	137.27	ABDS (A6.2.1-3)
	$\lambda_f = b_{fc}/(2t_{fc}) =$	8.5246	ABDS (A6.3.2-3)
	$\lambda_{pf} = 0.38(E/F_{yc})^{1/2} =$	9.1516	ABDS (A6.3.2-4)
	$\lambda_{rf} = 0.95(Ek_c/F_{yr})^{0.5} =$	23.839	ABDS (A6.3.2-5)
	$\lambda_{rf} = 0.56(E/F_{yr})^{0.5} =$	16.12	ABDS (6.10.8.2.2-5)

Web Hybrid Factor $R_h =$ ***1*** — ABDS 6.10.1.10.1

Web Load-Shedding and Factor $2D_c/t_w =$ 60.17 < λ_{rw} — ABDS (6.10.1.10.2-1)

$R_b =$ 1 — ABDS 6.10.1.10.2

Use the Provisions of Appendix A6

F_y	<=	70 ksi		**OK**	ABDS A6.1
$2D_c/t_w$ = 60.17	<	λ_{rw} = 137.27		**OK**	ABDS (A6.1-1)
I_{yc}/I_{yt} = ***1***	>=	0.3		**OK**	ABDS (A6.1-2)

Web Plastification Factors

$2D_{cp}/t_w$ = 60.17	<=	$\lambda_{pw(Dcp)}$ = 84.534	**Section is Compact**	ABDS (A6.2.1-1)
$\lambda_w = 2D_c/t_w$ = 60.17	<	λ_{rw} = 137.27		ABDS (A6.2.2-1)

$\lambda_{pw(Dcp)} = (E/F_{yc})^{1/2}/(0.54(M_p/(R_hM_y))-.09)^2 =$ 84.534 <= $\lambda_{rw}(D_{cp}/D_c)$ Use $\lambda_{pw(Dcp)}$ = 84.534 ABDS (A6.2.1-2)

$\lambda_{pw(Dc)} = \lambda_{pw(Dcp)}(D_c/D_{cp}) =$ 84.534 <= λ_{rw} Use $\lambda_{pw(Dc)}$ = 84.534 ABDS (A6.2.2-6)

$R_{pc} = R_{pt} =$ 1.1551 (A6.2.1-4 & A6.2.1-5)

Compression Flange ABDS A6.3

Local Buckling Resistance

λ_f <= λ_{pf} $M_{nc} = R_{pc}M_{yc} =$ 14150 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)

$k_c =$ ***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	14150	k-in	ABDS (A6.3.3-1)
If $L_p < L_b <= L_r$,	$M_{nc} = C_b[1-(1-F_{yr}S_{xc}/(R_{pc}M_{yc}))((L_b-L_p)/(L_r-L_p))]R_{pc}M_{yc} =$	9562.3	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)
	$L_b =$	214.62	12	in	
	$L_p = 1.0r_t(E/F_y)^{1/2} =$	62.247 in			ABDS (A6.3.3-4)
	$L_r = 1.95r_t(E/F_{yr})(J/(S_{xc}h))^{.5}(1+(1+6.76(F_{yr}S_{xc}h/(EJ))^2)^{1/2})^{1/2} =$	247.42 in			ABDS (A6.3.3-5)
	$F_{cr} = (C_b\pi^2E/(L_b/r_t)^2)(1+0.078J(L_b/r_t)^2/(S_xh))^{0.5} =$	45.273 ksi			ABDS (A6.3.3-8)
	$r_t = b_{fc}/(12(1+D_ct_w/(3b_{fc}t_{fc})))^{1/2} =$	2.5847 in			ABDS (A6.3.3-10)
	$J = Dt_w^3/3+b_{fc}t_{fc}^3(1-0.63t_{fc}/b_{fc})/3+b_{ft}t_{ft}^3(1-0.63t_{ft}/b_{ft})/3 =$	2.4943 in^4			ABDS (A6.3.3-9)

Use M_{nc} = 9562.3 k-in Constructability Condition (S1)

Use M_{nc} = 14150 k-in Final Condition (S2 & S3 without composite deck)

Use M_{nc} = 796.86 k-ft Constructability Condition (S1)

Use M_{nc} = 1179.2 k-ft Final Condition (S2 & S3 without composite deck)

Tension Flange

$M_{nt} = R_{pt}M_{yt} =$ 14150 k-in ABDS (A6.4-1)

Use M_{nt} = 1179.2 k-ft

Interior Beam Composite Section Flexural Capacity

Slab Thickness = t_s = 8 in

M_p = 2262.3 ft-k M_y = 1620.1 ft-k

Y' = 33.29 in from bottom of beam

D_p = 4.2102 in <= $0.42D_t$ **OK** ABDS (6.10.7.3-1)

D_t = 37.5 in $0.42D_t$ = 15.75 in

Check for Compact Section ABDS 6.10.6.2.2

F_y = 50 ksi < 70 ksi **OK**

D/t_w = 60.17 < 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 $M_n = M_p$ = 2262.3 ft-k ABDS (6.10.7.1.2-1)

Otherwise $M_n = M_p(1.07-0.7(D_p/D_t))$ = 2242.9 ft-k ABDS (6.10.7.1.2-2)

$M_n = (1-(1-M_y/M_p)((D_p/D_t-0.1)/0.32)M_p$ = 2237.7 ft-k use M_n = 2237.7 ft-k

(Second Equation is from Caltrans Amendments)

Use $M_{n(Int)}$ = 2237.7 ft-k

Exterior Beam Composite Section Flexural Capacity

Slab Thickness = t_s = 8 in

M_p = 2146.4 ft-k M_y = 1571.3 ft-k

Y' = 31.174 in from bottom of beam

D_p = 6.3264 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 $M_n = M_p$ = 2146.4 ft-k ABDS (6.10.7.1.2-1)

Otherwise $M_n = M_p(1.07-0.7(D_p/D_t))$ = 2043.2 ft-k ABDS (6.10.7.1.2-2)

$M_n = (1-(1-M_y/M_p)((D_p/D_t-0.1)/0.32)M_p$ = 2022.9 ft-k use M_n = 2022.9 ft-k

(Second Equation is from Caltrans Amendments)

Use $M_{n(Ext)}$ = 2022.9 ft-k ABDS 6.10.7.1.2

Reference

Effective Width of Concrete Slab — ABDS 4.6.2.6.1

Span = 53.5 ft
Girder Spacing = 6.8048 ft
Use Effective Width = 6.8048 ft

Concrete Slab Width = 81.658 in
Concrete Slab Thickness= 8 in
Haunch = 0.25 in
f'_c = 4.5 ksi

Beam = W30x90		S_x =	245 in^3	A =	26.3 in^2	b_f =	10.4 in
F_y =	50 ksi	Z_x =	283 in^3	d =	29.5 in	t_f =	0.61 in
E =	29000 ksi	I_x =	3610 in^4	t_w =	0.47 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:	30.537	33.75	1030.6	-8.7918	2360.4	162.86
W-Beam:	26.3	14.75	387.93	10.208	2740.6	3610
	56.837		1418.5		5101	3772.9

Long Term

y_b = 24.958 in
I = 8873.9 in^4
S top slab = 707.54 in^3
S top beam = 1953.8 in^3
S bot beam = 355.55 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:	91.61	33.75	3091.9	-4.238	1645.4	488.59
W-Beam:	26.3	14.75	387.93	14.762	5731.2	3610
	117.91		3479.8		7376.6	4098.6

Short Term

y_b = 29.512 in
I = 11475 in^4
S top slab = 1436.6 in^3
S top beam = 953159 in^3
S bot beam = 388.83 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	10.787	0.61	50	29.195	329.01	28.89	985.99	-0.609	0	-14.445	4752.6
Web	2	***0.4646***	28.28	50	14.75	656.97	0.61	329.01	14.14	14.75	N-Axis	4644.8
Bottom Flange	1	10.787	0.61	50	0.305	329.01	0	0	1.219	0	14.445	4752.6

P_{total} = 1315
$P_{total}/2$ = 657.5

Y' = 14.75 in
M_p = 1179.2 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	81.658	8	3.825	33.5	2498.7	29.5	1315	3.7898	33.29	N-Axis	2768.2
Top Flange	3	10.787	0.61	50	29.195	329.01	28.89	985.99	1.7073	0	4.0948	1347.3
Web	2	0.4646	28.28	50	14.75	656.97	0.61	329.01	67.92	0	18.54	12180
Bottom Flange	1	10.787	0.61	50	0.305	329.01	0	0	3.5354	0	32.985	10852

P_{total} = 3813.7
$P_{total}/2$ = 1906.9

Y' = 33.29 in
M_p = 2262.3 ft-k

Reference

Effective Width of Concrete Slab — ABDS 4.6.2.6.1

Span = 53.5 ft
Overhang = 1.1261 ft
Use Effective Width = 4.5285 ft

Concrete Slab Width = 54.343 in
Concrete Slab Thickness= 8 in
Haunch = 0.25 in
f'_c = 4.5 ksi

Beam = W30x90
F_y = 50 ksi
E = 29000 ksi
S_x = 245 in^3
Z_x = 283 in^3
I_x = 3610 in^4
A = 26.3 in^2
d = 29.5 in
t_w = 0.47 in
b_f = 10.4 in
t_f = 0.61 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:	20.322	33.75	685.87	-10.718	2334.6	108.38
W-Beam:	26.3	14.75	387.93	8.2819	1803.9	3610
	46.622		1073.8		4138.5	3718.4

Long Term
y_b = 23.032 in
I = 7856.8 in^4
S top slab = 543.05 in^3
S top beam = 1214.7 in^3
S bot beam = 341.13 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:	60.966	33.75	2057.6	-5.7262	1999	325.15
W-Beam:	26.3	14.75	387.93	13.274	4633.9	3610
	87.266		2445.5		6632.9	3935.2

Short Term
y_b = 28.024 in
I = 10568 in^4
S top slab = 1115.2 in^3
S top beam = 7159.2 in^3
S bot beam = 377.11 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	10.787	0.61	50	29.195	329.01	28.89	985.99	-0.609	0	-14.445	4752.6
Web	2	***0.4646***	28.28	50	14.75	656.97	0.61	329.01	14.14	14.75	N-Axis	4644.8
Bottom Flange	1	10.787	0.61	50	0.305	329.01	0	0	1.219	0	14.445	4752.6

P_{total} = 1315
$P_{total}/2$ = 657.5
Y' = 14.75 in
-335544
M_p = 1179.2 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	54.343	8	3.825	33.75	1662.9	29.5	1315	1.6736	31.174	N-Axis	4159.6
Top Flange	3	10.787	0.61	50	29.195	329.01	28.89	985.99	0.9325	0	1.9786	651
Web	2	0.4646	28.28	50	14.75	656.97	0.61	329.01	49.93	0	16.424	10790
Bottom Flange	1	10.787	0.61	50	0.305	329.01	0	0	2.7605	0	30.869	10156

P_{total} = 2977.9
$P_{total}/2$ = 1488.9
Y' = 31.174 in
M_p = 2146.4 ft-k

Reference

Ultimate Strength

F_y = 50 ksi | b = 81.658 in | Stud Dia. = d = 0.875 in

f'_c = 4.5 ksi | t_s = 8 in | Spacing = p = 12 in

A_s = 26.3 in^2 | H = 4 in

E_c = 4066.8 ksi — ABDS 5.4.2.4

D = 26.75 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

ϕ_{sc} = ***0.85*** — ABDS 6.5.4.2

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

R_g = 1 | A_{sc} = 0.6013 in^2

R_p = 1 | F_u = ***60*** ksi — ABDS 6.4.4

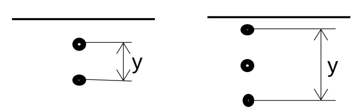
n_{min} = P/Qr = 42.357 studs < Total Studs = 81 **OK** — ABDS (6.10.10.4.1-2)

$P = (P_P^2 + F_P^2)^{0.5}$ = 1299 k — ABDS (6.10.10.4.2-1)

$P_{1P} = 0.85f'_cb_st_s$ = 2498.7 k — ABDS (6.10.10.4.2-2)

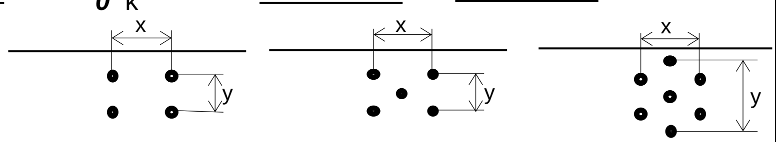
$P_{2P} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc}$ = 1299 k — ABDS (6.10.10.4.2-3)

F_p = ***0*** k

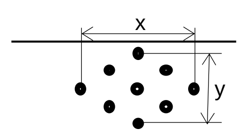


x = 0 in

y = 7 in



Use 0.875 in diameter by 4 in high studs
With 3 studs per Grouping
At 12 in between groups



Fatigue

$Z_{rI} = 5.5d^2$ = 4.2109 k — ABDS (6.10.10.2-1)

$\alpha = Z_r/d^2$ = 9.4248 | $V_{srI} = V_{fI}Q/I$ = 1.8603 k

n = 3 /group | Q = yA = 388.24 in^3

Min Spacing = 5.25 in | $p_{max} = nZ_{rI}/V_{srI}$ = **3.2013 N.G. for Infinite Life** — ABDS (6.10.10.1.2-1)

$V_{fI} = \gamma_I V_{LL(fatigue)}$ = 50.639 k

$V_{fII} = \gamma_{II} V_{LL(fatigue)}$ = 49.105 k | $Z_{rII} = pV_{srII}/n$ = 7.216 k

γ_I = ***1.75*** Fatigue I - Infinite Life | $V_{srII} = V_{fII}Q/I$ = 1.804 k — ABDS (6.10.10.1.2-3)

γ_{II} = ***0.8*** Fatigue II - Finite Life | $N = 10^{(34.5-\alpha)/4.28}$ = 722241 cycles — ABDS (6.10.10.2-3)

$V_{LL(fatigue\ I)}$ = 28.937 k | Stress Cycles per Truck, n = ***1*** — ABDS Tbl 6.6.1.2.5-2

$V_{LL(fatigue\ II)}$ = 61.381 k | $(ADTT)_{SL\ Max} = N/((365)(75)n)$ = 26.383 — ABDS (6.6.1.2.5-3)

I = 10568 in^4 | p = 0.85 — ABDS Tbl 3.6.1.4.2-1

y = 4.238 in | $ADTT_{SL}/p$ = 31 — ABDS (3.6.1.4.2-1)

A = 91.61 in^2 | **Maximum ADTT = 31** **OK**

								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								
$f_{bu}+f_l =$	40.284 ksi	<=		$\phi_f R_h F_{yc} =$	50 ksi		**OK**	ABDS (6.10.3.2.1-1)
$f_{bu}+1/3f_l =$	32.737 ksi	<=		$\phi_f F_{nc} = \phi_f M_{nc}/S_x =$	39.03 ksi		**OK**	ABDS (6.10.3.2.1-2)
$f_{bu} =$	28.963 ksi	<=		$\phi_f F_{crw} =$	259.53 ksi		**OK**	ABDS (6.10.3.2.1-3)
			$F_{crw} = 0.9Ek/(D/t_w)^2 =$	259.53 ksi				ABDS (6.10.1.9.1-1)
			$k = 9/(D_c/D)^2 =$	36				ABDS (6.10.1.9.1-2)
Tension Flange								
$f_{bu}+f_l =$	40.284 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 $f_l/2 =$	17.627 ksi	<=	0.95 $R_h F_{ytf} =$	47.5 ksi			**OK**	ABDS (6.10.4.2.2-1 or 3)
Bottom Flange								
$f_{fbf}+f_l/2 =$	46.214 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								
$\eta_{iS}(M_u) =$	1726.32 ft-k	<	$\phi_f M_n =$	2022.9 ft-k			**OK**	ABDS 6.10.7.1
Interior Girder								
$\eta_{iS}(M_u) =$	1667.63 ft-k	<	$\phi_f M_n =$	2237.7 ft-k			**OK**	ABDS 6.10.7.1
Tension Flange:								
Exterior Girder								
$\eta_{iS}(M_u+(1/3)f_l S_{xt}) =$	1733.50 ft-k	<	$\phi_f M_n =$	2022.9 ft-k			**OK**	ABDS 6.10.7.1
Interior Girder								
$\eta_{iS}(M_u+(1/3)f_l S_{xt}) =$	1667.63 ft-k	<	$\phi_f M_n =$	2237.7 ft-k			**OK**	ABDS 6.10.7.1
Shear Capacity Checks								ABDS 6.10.9
$V_u =$	173.63 k	<=	$\phi_v V_n =$	385.46 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} (Fatigue I)= 292.50 ft-k

M_{LL} (Fatigue II)= 694.26 ft-k

I_x = 11475 in^4 (Interior Girder)

I_x = 10568 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	-4.5418	-2.4312	-2.6379	OK for Infinite Life
	Bottom of BF	B	16	12000000000	24.958	13.36	14.496	OK for Infinite Life
	Top of Diaphragm	E	4.5	1100000000	0.4582	0.2452	0.2661	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	18.158	9.7198	10.546	34.25536055
Exterior Girder	Top of TF	C	10	4400000000	-6.4681	-3.7594	-4.0792	OK for Infinite Life
	Bottom of BF	B	16	12000000000	23.032	13.387	14.525	OK for Infinite Life
	Top of Post Block	E	4.5	1100000000	-1.4681	-0.8533	-0.9259	OK for Infinite Life
	Bottom of Post Block	E	4.5	1100000000	12.232	7.1095	7.7142	87.53347463
	Top of Diaphragm	E	4.5	1100000000	-1.4681	-0.8533	-0.9259	OK for Infinite Life
	Bottom of Diaphragm	E	4.5	1100000000	16.232	9.4344	10.237	37.45827031

Use Maximum ADTT: Finite Life **Use ADTT = 40** **OK** — ABDS (3.6.1.4.2-1)

Reference

Diaphragm Spacing = L_b = 17.885 ft — Beam Depth = d_b = 29.5 in

Number of Diaphragms = N_b = 4 — F_y = 50 ksi

Diaphragm Length = L_c = 81.188 in — E = 29000 ksi

Diaphragm Section = W18x35 — t_f = 0.425 in — C_w = 1140 in^6

Diaphragm Depth = d_d = 17.7 in — b_f = 6 in — J = 0.506 in^4

Diaphragm Area = A_g = 10.3 in^2 — t_w = 0.3 in — r_o = – in

$r_s = r_{y\text{-}y}$ = 1.22 in — h = 16.05 in — H = –

I_x = 510 in^4 — I_y = 15.3 in^4

Minimum Diaphragm Depth = $0.5d_b$ = 14.75 in <= d_d **OK** — ABDS 6.7.4.2

Total Load on Diaphragms

W = 0.0496 k/ft

$P_w = WL_b$ = 0.8865 k — P_u = 1.4 P_w = 1.2412 k — ABDS (C4.6.2.7.1-4)

P_u = **20** k — ABDS 6.6.1.3.1

Member Check

$\phi_c P_n = \phi_c F_{cr} A_{eff}$ = 368.65 k >= P_u = 20.00 k **OK** — ABDS (6.9.4.2.2-1)

ϕ_c = 0.95 — ABDS 6.5.4.2

KL_c/r = 49.91 <= 140 **OK** — ABDS 6.9.3

K = ***0.75*** — ABDS 4.6.2.5

$F_{cr} = P_{cr}/A_g$ = 41.674 ksi — ABDS (6.9.4.2.2-2)

$P_{cr} = (0.658^{(Po/Pe)})P_o$ = 429.25 k, or $P_{cr} = 0.877P_e$ = 1041.4 k — ABDS (6.9.4.1.1-1) & (6.9.4.1.1-2)

P_o/P_e = 0.4352 <= 2.25 — Use P_{cr} = 429.25 k

$P_o = F_y A_g$ = 515 k — ABDS 6.9.4.1.1

Use P_e = 1183.5 k

Elastic Flexural Buckling Resistance:

$P_e = (\pi^2 E/(KL_c/r_s)^2)A_g$ = 1183.5 k — ABDS (6.9.4.1.2-1)

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

Use P_e = 3993.3 k

For open-sections doubly symmetric members:

$P_e = (\pi^2 EC_w/(K_zL_z)^2+GJ)A_g/(I_x+I_y)$ = 3993.3 k — ABDS (6.9.4.1-3-1)

$K_zL_z = 0.5L_c$ = 40.594 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)

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

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

$A_{eff} = A_g-((b_f-b_{ef})t_f+(h-h_e)t_w)$ = 9.3115 in^2 — ABDS 6.9.4.2.2

					Reference
$b/t_f =$ 7.0588	<=	$\lambda_{rf} = 0.56(E/F_y)^{1/2} =$	13.487	Flanges are Nonslender	ABDS (6.9.4.2.1-1)
$h/t_w =$ 53.5	>	$\lambda_{rw} = 1.49(E/F_y)^{1/2} =$	35.884	Web is Slender	& Tbl 6.9.4.2.1-1
$b/t_f =$ 7.0588	<=	$\lambda_{rf}(F_y/F_{cr})^{1/2} =$	14.772		ABDS 6.9.4.2.2a
$h/t_w =$ 53.5	>	$\lambda_{rw}(F_y/F_{cr})^{1/2} =$	39.305		

b = 3 in

$b_{ef} = b_f(1-c_{1f}(F_{elf}/F_{cr})^{1/2})(F_{elf}/F_{cr})^{1/2} =$ 5.8958 in, use 6 in

$h_{ew} = h(1-c_{1w}(F_{elw}/F_{cr})^{1/2})(F_{elw}/F_{cr})^{1/2} =$ 12.755 in, use 12.755 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

$F_{elf} = (c_{2f}\lambda_{rf}/(b/t_f))^2F_y =$ 402.73 ksi $F_{elw} = (c_{2w}\lambda_{rw}/(h/t_w))^2F_y =$ 38.479 ksi — ABDS (6.9.4.2.2a-4)

Weld Check

$R_r = (0.6\phi_{e2}F_{EXX})(0.707t)l_w =$ 197.61 k >= $P_u =$ 20.00 k OK — ABDS 6.13.3.2.4

$\phi_{e2} =$ 0.8 — ABDS 6.5.4.2

Weld Length = $l_w = 2(d_d - 2.5t_f) =$ 33.275 in

Fillet Weld Size = t = ***0.25*** in

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

Reference

Diaphragm Spacing = L_b = 17.885 ft — Beam Depth = d_b = 29.5 in

Number of Diaphragms = N_b = 4 — F_y = 50 ksi

Diaphragm Length = L_c = 81.188 in — E = 29000 ksi

Diaphragm Section = C15x33.9 — t_f = 0.65 in — C_w = 358 in^6

Diaphragm Depth = d_d = 15 in — b_f = 3.4 in — J = 1.01 in^4

Diaphragm Area = A_g = 10 in^2 — t_w = 0.4 in — r_o = 5.94 in

$r_s = r_{y\text{-}y}$ = 0.901 in — h = 21.4 in — H = 0.92

I_x = 315 in^4 — I_y = 8.07 in^4

Minimum Diaphragm Depth = $0.5d_b$ = 14.75 in <= d_d **OK** — ABDS 6.7.4.2

Total Load on Diaphragms

W = 0.0496 k/ft

$P_w = WL_b$ = 0.8865 k — P_u = 1.4 P_w = 1.2412 k — ABDS (C4.6.2.7.1-4)

P_u = 20 k — ABDS 6.6.1.3.1

Member Check

$\phi_c P_n = \phi_c F_{cr} A_{eff}$ = 292.31 k >= P_u = 20.00 k OK — ABDS (6.9.4.2.2-1)

ϕ_c = 0.95 — ABDS 6.5.4.2

KL_c/r = 67.581 <= 140 **OK** — ABDS 6.9.3

K = ***0.75*** — ABDS 4.6.2.5

$F_{cr} = P_{cr}/A_g$ = 35.414 ksi — ABDS (6.9.4.2.2-2)

$P_{cr} = (0.658^{(Po/Pe)})P_o$ = 354.14 k, or $P_{cr} = 0.877P_e$ = 533.91 k — ABDS (6.9.4.1.1-1) & (6.9.4.1.1-2)

P_o/P_e = 0.8241 <= 2.25 — Use P_{cr} = 354.14 k

$P_o = F_y A_g$ = 500 k — ABDS 6.9.4.1.1

Use P_e = 606.72 k

Elastic Flexural Buckling Resistance:

$P_e = (\pi^2 E/(KL_c/r_s)^2)A_g$ = 626.68 k — ABDS (6.9.4.1.2-1)

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

Use P_e = 606.72 k

For open-sections doubly symmetric members:

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

$K_zL_z = 0.5L_c$ = 40.594 in — ABDS C6.9.4.1.3

G = 0.385E = 11165 ksi — ABDS 6.9.4.1.3

For open-sections singly symmetric members:

$P_e = ((P_{ey}+P_{ez})/(2H))(1-(1-4P_{ey}P_{ez}H/(P_{ey}+P_{ez})^2)^{1/2})$ = 606.72 k — ABDS (6.9.4.1-3-2)

$P_{ey} = (\pi^2 E/(K_yL_{cy}/r_y)^2)A_g$ = 626.68 k — ABDS (6.9.4.1.3-4)

$P_{ez} = (\pi^2 EC_w/(K_zL_z)^2+GJ)/r_o^2$ = 2081.9 k — ABDS (6.9.4.1.3-5)

$A_{eff} = A_g-((b_f-b_{ef})t_f+(h-h_e)t_w)$ = 8.6886 in^2 — ABDS 6.9.4.2.2

						Reference
b/t_f =	5.2308	<=	λ_{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 =	53.5	>	λ_{rw} =1.49$(E/F_y)^{1/2}$ =	35.884	Web is Slender	
b/t_f =	5.2308	<=	$\lambda_{rf}(F_y/F_{cr})^{1/2}$ =	16.025		ABDS 6.9.4.2.2a
h/t_w =	53.5	>	$\lambda_{rw}(F_y/F_{cr})^{1/2}$ =	42.638		

b = 3.4 in

$b_{ef} = b_f(1-c_{1f}(F_{elf}/F_{cr})^{1/2})(F_{elf}/F_{cr})^{1/2}$ = -0.0183 in, use 3.4 in — ABDS (6.9.4.2.2a-1)

$h_{ew} = h(1-c_{1w}(F_{elw}/F_{cr})^{1/2})(F_{elw}/F_{cr})^{1/2}$ = 18.122 in, use 18.122 in — 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

$F_{elf} = (c_{2f}\lambda_{rf}/(b/t_f))^2F_y$ = 733.41 ksi $F_{elw} = (c_{2w}\lambda_{rw}/(h/t_w))^2F_y$ = 38.479 ksi — ABDS (6.9.4.2.2a-4)

Weld Check

$R_r = (0.6\phi_{e2}F_{EXX})(0.707t)l_w$ = 158.86 k >= P_u = 20.00 k OK — ABDS 6.13.3.2.4

ϕ_{e2} = 0.8 — ABDS 6.5.4.2

Weld Length = $l_w = 2(d_d-2.5t_f)$ = 26.75 in

Fillet Weld Size = t = ***0.25*** in

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

Reference

Loading Data

$R_u = V_u =$ 173.63 k (Total Factored Load)

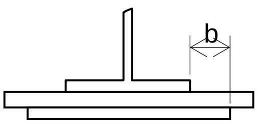
	Int. Girders	Ext. Girders			
$R_{LL} =$	55.89	48.67	k (Unfactored w/o IM)		
$R_{DL} =$	32.66	26.09	k (Unfactored)		
$R =$	88.56	74.76	use	88.56	k (Unfactored)

Check Bearing Plates:

Plate Dimensions:

$N_P =$ 8 in $t_{Max} =$ 1.25 in

$W_P =$ 22 in $t_{Min} =$ 0.6875 in



Taper = 0.5625 in

t = 0.9688 in

$S_P = N_P t^2/6 =$ 1.2513 in^3

Bottom Flange Width = $b_f =$ 10.4 in

Pad Width Outside Flange = $b = (W_E' - b_f)/2 =$ 2.3 in

$V_{u(plate)} = (V_u/W_E)b =$ 26.623 k

$M_{u(plate)} = (V_u/W_E)b^2/2 =$ 30.616 k-in

$\phi_f M_n = \phi_f F_y S_P =$ 62.565 k-in >= $M_{u(plate)}$ **OK**

Movements

Expansion Coefficient = $\alpha =$ 0.0000065 /°F

Expansion Length = L = 53.5 ft

Mean High Temperature = $T_H =$ 130 °F

Mean Low Temperature = $T_L =$ -10 °F

Max Base = $T_{BMax} =$ 60 °F Min Base = $T_{BMin} =$ 60 °F

Shear Movement:

$\Delta_{Rise} = \alpha(T_H - T_{BMin})L =$ 0.2921 in

$\Delta_{Fall} = \alpha(T_{BMax} - T_L)L =$ 0.2921 in

$\Delta_T = \Delta_{Rise} + \Delta_{Fall} =$ 0.5842 in

$\Delta_o = 0.65\Delta_T = \Delta_s =$ 0.3797 in ABDS 14.7.5.3.2

Reference

Check Elastomeric Bearing Pads

Use 55 Durometer Elastomer G_{Max} = 165 psi
G_{Min} = 112.5 psi

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

Pad Dimensions: L_E = 6.75 in — $A = L_E W_E - n_h \pi d^2/4$ = 101.25 in^2

W_E = 15 in — $S_i = A/(h_{ri}(2(L_E+W_E)+n_h \pi d)$ = 9.0751 — ABDS (C14.7.5.1-1)

W_E' (Effective Pad Width) = 15 in

Include Holes in W_E' = Y — $A' = L_E W_E' - n_h \pi d^2/4$ = 101.25 in^2

Hole Diameter = d = 0 in — $S_i' = A'/(h_{ri}(2(L_E+W_E')+n_h \pi d)$ = 9.0751 — ABDS (C14.7.5.1-1)

Number of Holes = n_h = 0

Total Thickness = h = 2.25 in — S_i^2/n = 16.472 < 22 **OK** — ABDS 14.7.6.1

Number of Layers = n = 5

h_{ri} = 0.2565 in

Cover = 0.125 in

Shim Thickness = h_s = 0.1196 in — Shim F_y = **36** ksi

$\sigma_s = R/(L_E W_E)$ = 0.8746 ksi — $\sigma_s' = R/(L_E W_E')$ = 0.8746 ksi

h_{rt} = 1.5324 in

Check Compressive Stress: — ABDS 14.7.6.3.2

σ_s' = 0.8746 ksi < 1.25 ksi **OK**

1.25 $G_{Min} S_i'$ = 1.2762 > σ_s' **OK**

Check Deflection: — ABDS 14.7.6.3.3

$\delta = \text{Max}(\varepsilon_i \text{ or } \varepsilon_i')h_{ri}$ = 0.005 in < $.09h_{ri}$ = 0.0231 **OK**

$\varepsilon_i = \sigma_s/(4.8 G_{Min} S_i^2)$ = 1.9666 % — ABDS (C14.7.5.3.6-1)

$\varepsilon_i' = \sigma_s'/(4.8 G_{Min} S_i'^2)$ = 1.9666 %

Check Shear: h_{rt} = 1.5324 in >= $2\Delta_s$ = 0.7595 in **OK** — ABDS 14.7.6.3.4

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

$W_E'/3$ = 5 in >= h **OK**

Check Shear at Bearing

Shear Force on Pad = $H_u = GA\Delta_u/h_{rt}$ = 4.14 k — ABDS (14.6.3.1-2)

Steel Friction Resistance = μV_{DL} = 6.3002 k >= H_m **OK w/o Restraint System**

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

Concrete Friction Resistance = μV_{DL} = 12.6 k

Coefficient of Friction = μ = ***0.4***

Use 15 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

Check if Bearing Stiffeners are Needed

ϕ_b = ***0.8*** ϕ_w = ***0.8*** ϕ_c = ***0.95*** — ABDS 6.5.4.2

Girder = W30x90

d =	29.5	in	b_f =	10.4	in	E =	29000	ksi
t_w =	0.47	in	k_1 =	1.4375	in	F_y =	50	ksi
t_f =	0.61	in	$D = d-2t$ =	28.28	in			

Web Local Yielding — ABDS D6.5.2

$\phi_b R_n = \phi_b(2.5k+N)F_y t_w$ = 327.24 k >= R_u **OK**

Web Crippling — ABDS D6.5.3

$\phi_w R_n = \phi_w 0.4 t_w^2(1+3(N/d)(t_w/t_f)^{1.5})(EF_y t_f/t_w)^{1/2}$ = 150.33 k < R_u **N/A**

$\phi_w R_n = \phi_w 0.4 t_w^2(1+(4N/d-0.2)(t_w/t_f)^{1.5})(EF_y t_f/t_w)^{1/2}$ = 155 k < R_u **N.G.**

N/d = 0.2712 > 0.2

$k = b_f/2-k_1$ = 3.7625 — ABDS D6.5.2

N = 8 in **Bearing Stiffeners are Required**

Check Bearing Stiffeners

Stiffener Width = b_t = 4.875 in Number per Bearing = n = 1

Stiffener Thickness = t_p = 0.5 in

Cope Width = f = 1.25 in

Minimum Thickness:

$t_p >= b_t/(0.48(E/F_y)^{1/2})$ = 0.4217 in <= t_p **OK** — ABDS 6.10.11.2.2

Bearing Resistance: — ABDS 6.10.11.2.3

$(R_{sb})_r = \phi_b 1.4 A_{pn} F_y$ = 101.5 k

$A_{pn} = t_p(b_f-f-e)$ = 1.8125 in^2

$e = t_p-(b_f-t_w)/2$ = -0.09 in, use = 0 in

Total = Minimum ($\phi_b R_n$ or $\phi_w R_n$) + $n(R_{sb})_r$ = 256.5 k >= R_u **OK**

Axial Resistance (Check as Column): — ABDS 6.10.11.2.4

$P_r = \phi_c P_n$ = 303.21 k >= R_u **OK** — ABDS 6.9.2.1

Effective width of Web: $b_w = 9t_w + \min(9t_w \text{ or } b_b)$ = 8.46 in — ABDS 6.10.11.2.4a

Bearing Distance = b_b = 7.5 in $I_b = t_b(2b_t+t_w)^3/12+(b_w-t_p)t_w^3/12$ = 44.546 in^4

$A = nb_t t_p + b_w t_w$ = 6.4137 in^2 $r_s = (I_b/A)^{1/2}$ = 2.6354 in

If $P_o/P_e < 2.25$ then $P_n = (0.658^{(Po/Pe)})P_o$ = 319.17 k — ABDS 6.9.4.1.1

Otherwise $P_n = 0.877P_e$ = 24856 k, Use P_n = 319.17 k

$P_o = F_y A$ = 320.69 k

$P_e = \pi^2 EA/(0.75D/r_s)^2$ = 28342 k — ABDS (6.9.4.1.2-1)

P_o/P_e = 0.0113

Reference

Anchor Bolt Checks

Braking Force: BR = 9 k Max[0.25*Truck,0.05(Truck+Lane*L)]N_Lm/N_S — ABDS 3.6.4

Horizontal Loads:

WS_H = 2.0531 k	Strength III = $(1.0WS_H^2+1.2TU_L^2)^{1/2}$ =	5.38 k		ABDS Tbl 3.4.1-1
EQ_H = 31.11 k	Extreme Event I = $(EQ_L^2+(.3EQ_H)^2)^{1/2}$ =	48.639 k		ABDS 3.10.8
EQ_L = 47.735 k	Stream Overtopping = 1.0WA =	0.00 k		
TU_L = 4.14 k				
WA = 0.00 k	R_u = 48.64 k			

Uplift Load:

DC = 20.645 k	Strength III = 0.90DC+ 0.65DW+ 1.0WS_H =	9.7096 k	ABDS Tbl 3.4.1-1
DW = 5.4449 k	Stream Overtopping = 0.9DC+1.0WA =	18.58 k	
WS_H = -12.41 k			
WA = 0.00 k	P_u = 0 k		

Bearing Plate Thickness = t = 0.6875 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} = 125 ksi (F1554 Gr 105 Bolts) — AISC Tbl 2-6

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
$\phi_s R_n$ = 0.5 $\phi_s A_b F_{ub} N_s n$ = 115.05 k >= R_u **OK** — ABDS 6.13.2.12
N_s = ***1***

Bolt Capacity in Tension
$\phi_t T_n$ = 0.76 $\phi_t A_b F_{ub} n$ = 186.53 k >= P_u **OK** — ABDS 6.13.2.10

Bolt Capacity in Bearing
$\phi_{bb} R_{nb} = \phi_{bb} L_c t F_u n$ = 134.75 k >= R_u **OK** — ABDS 6.13.2.9

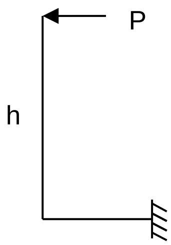
Capacity of Weld
$R_r = (0.6\phi_{e2}F_{EXX})(0.707t)l_w$ = 95.021 k >= γR_u **OK** — ABDS 6.13.3.2.4

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

Reference

Rail Test Level Loads = TL-2

Post and Post Block Design



				Reference
F_t =	27 k	$L_t = L_L$ =	4 ft	ABDS Tbl A13.2-1
F_L =	9 k	L_v =	18 ft	ABDS Tbl A13.2-1
F_v =	4.5 k	H_e (min) =	20 in	ABDS Tbl A13.2-1
		H (min) =	27 in	ABDS Tbl A13.2-1
$h = H_e + a$ =	35.85 in			
Post Spacing = L_P =	3.125 ft	a =	15.85 in	

$P_t = F_t(L_P - L_t/4)/L_P$ =	18.36 k	$M_t = P_t h$ =	658.21 k-in
$P_L = F_L/3$ =	3 k	$M_L = P_L h$ =	107.55 k-in
$P_v = F_v L_P / L_v$ =	0.7813 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 =	**658.21 k-in**	**<=**	**$\phi_f M_{yt}$ =**	**760 k-in**	**OK**
γM_L =	**107.55 k-in**	**<=**	**$\phi_f M_{yL}$ =**	**152 k-in**	**OK**

Check Shear (Unstiffened Webs)

γV_u =	**18.36 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

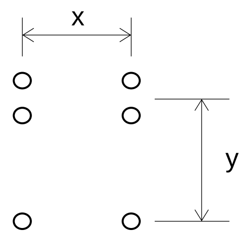
t_f = 0.335 in

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 $\phi_s A_b F_{ub} N_s$ = 25.977 k — ABDS (6.13.2.7-2)

N_s = ***1***

Bolt Capacity in Tension

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

$\gamma R_u = \gamma((P_L(h+y)/(yn_t/2))+P_V/(n_t+n_b))$ = 13.95 k <= $\phi_s R_n$ **OK**

$\gamma R_u = \gamma((P_L(h)/(yn_b/2))+P_V/(n_t+n_b))$ = 10.95 k <= $\phi_s R_n$ **OK**

$P_u = \gamma P_t(h+y)/(yn_t)$ = 42.09 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 k >= 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 = 61.414 in-k

FS = $\phi_f F_y S/\gamma M_u$ = 1.8074 (For Steel Yielding)

FS = $\phi_f F_u S/\gamma M_u$ = 2.3496 (For Steel Rupture)

Reference

Lifting Lug Checks

Lifting Lug Width = w = ***6*** in — F_y = 50 ksi

Lifting Lug Thickness = t = ***1*** in — F_u = 70 ksi

Lifting Lug Height = h = ***4*** in

Diameter of Hole = d = ***1.5*** in

Clearance of Hole to Top = L_c = 1.25 in

Resistance Factors — ABDS 6.5.4.2

ϕ_{bb}	ϕ_y	ϕ_u	ϕ_{e2}
0.8	***0.95***	***0.8***	***0.8***

Plate Capacity in Tension:

$P_r = \phi_y F_y A_g$ = 285 k — ABDS 6.8.3

$P_r = \phi_u F_u A_n R_p U$ = 252 k

A_g = tw = 6 in^2

A_n = tw-td = 4.5 in^2

R_p = ***1***

U = ***1***

Plate Capacity in Bearing

$R_n = \phi_{bb} L_c t F_u$ = 70 k — ABDS 6.13.2.9

Capacity of Weld:

$R_r = (0.6\phi_{e2}F_{EXX})t_e l_w$ = 258.1 k — ABDS 6.13.3.2.4

Weld Length = l_w = 12 in

Fillet Weld Size = t_f = ***0.375*** in

Goove Weld Size = t_g = ***0.375*** in

Effective Weld Size = $t_e = 0.707t_f + t_g$ = 0.6401 in

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

Controling Capacity:

P_{Min} = 70 k

Applied at 45° = P_{Min}Sin(45) = 49.497 k

Apply an additional FS of 2 = 24.749 k

Use = 24 k