Western Wood Structures, Inc.
PO Box 130
Tualatin, Oregon 97062
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Structural Design Calculations

For:

BRIDGES

Designer: ZF

Harper Park Bridge

Port Orford, Washington
WWSI Job No. 194017

June 29, 2020

06/29/20

BRADLEY J. BENNETT
STATE OF WASHINGTON
49029
REGISTERED
PROFESSIONAL ENGINEER

The attached calculations (14 Sheets) show the span and loading conditions and product design/selection for the above referenced project.

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Harper Park Bridge - Port Orford, Washington - WWSI# Job #194017

June 10, 2020

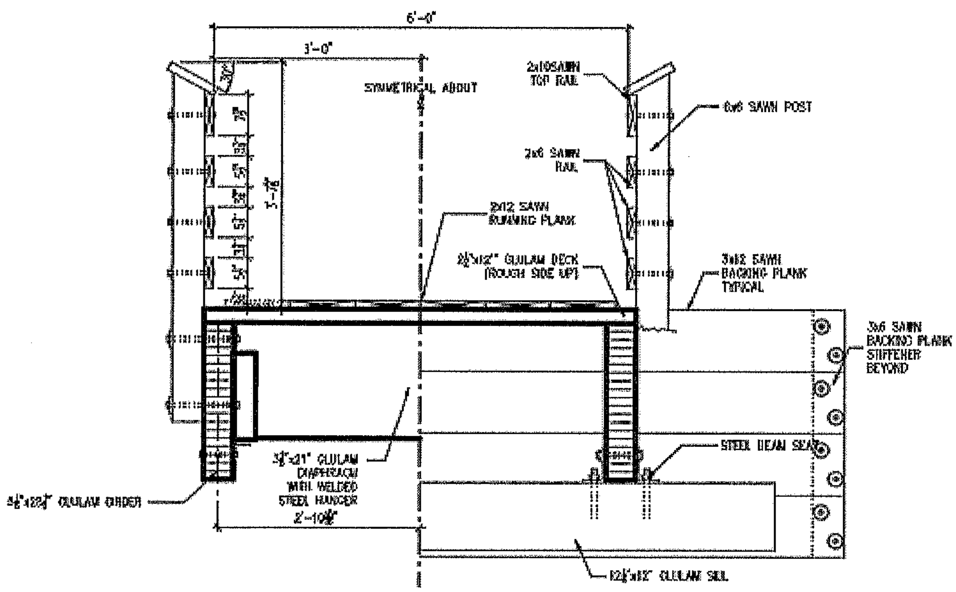
LRFD Pedestrian Timber Bridge Design

This design conforms to the AASHTO LRFD Bridge Design Specification (AASHTO), 2015 and the AASHTO Guide Specifications for Pedestian Bridges, 2009 w/ 2015 Interims (Guide). Design for wind induced forces conforms to AASHTO Standard Specifications for the Structural Support of Highway Signs, Luminaires and Traffice Signals, 2009 (Signs)

Design Criteria:

Bridge Span:	40.00 ft.	
Design Span:	39.50 ft.	
Bridge Width:	6.00 ft. -	out to out girder
Pedestrian Live Load:	90 psf.	(Guide 3.1)
Snow Load	120 psf.	
Vehicle Load:	None	
Additional Loads:		
Exterior Girder	17.58 plf.	Description:
Interior Girder	plf.	Description:
Wind Speed, V_{3S}:	115.0 mph	(Signs Figure 3-2a)
Height of Bridge:	10.0 ft.	
Seismic Design:		
Ground Acceleration, PGA	0.420 g	(AASHTO Figure3.10.2.1-1)
S_s	1.000 g	(AASHTO Figure3.10.2.1-2)
S_1	0.350 g	(AASHTO Figure3.10.2.1-3)
Seismic Zone:	4	(AASHTO Table 3.10.6-1)
Importance Classification:	Other	
Site Class:	D	
Response Coefficient, R	0.80	(AASHTO 3.10.7.1-2)
Rail Type:	None	
Number of Girders:	2	
Girder Spacing:	5.57 ft.	
Block Width	0.00 in.	
Deck Thickness:	2.500 in.	
Allowable Deflection = L/	425	
Number of Diaphragm Spaces:	3	

Definition Sketch



BRIDGE SECTION @ MIDSPAN/ABUTMENT

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Deck Design

Using a 2 1/2 in. glulam deck
Use West Coast Douglas Fir, Comb. 2 Use Wet Stresses

F_{bo} =	1800 psi	C_{KF} =	2.5/ϕ	C_{mb} =	0.8
F_{vo} =	230 psi	c_{fu} =	1.19	C_{mv} =	0.875
E =	1.60E+06 psi	c_λ =	0.80	C_{mE} =	0.8333
		$C_I = C_d$ =	1.00		

Minimum Panel Width = 12.00 in.
Center Design Span = 69.88 in.

Check Moment Capacity

w_{DC} = 2.5 * 12 * 50/144 = 10.42 plf.
w_{LL} = 90 * 12/12 = 90.00 plf.
$M_{DC} = w_{DC} * L^2/8$ = 44.15 ft. lbs.
$M_{LL} = w_{LL} * L^2/8$ = 381.45 ft. lbs.
$M_U = 1.25M_{DC} + 1.75\ M_{LL}$ = 722.72 ft. lbs. (Strength I)

$\phi M_n = \phi * F'_b * S$
ϕ = 0.85
$F'_b = F_{bo} * C_{KF} * C_m * C_{fu} * C_I * C_d * C_\lambda$
= 1800 * 2.5/0.85 * 0.8 * 1.19 * 1 * 1 * 0.8
= 4,032.00 psi
B_{eff} = 12.00 in.
$S = B_{eff} * h^2/6$ = 12 * 2.5^2/6 = 12.50 in^3

$\phi M_n = \phi * Fb'_{yy} * S/12$ = 0.85 * 4032 * 12.50/12 = 3,570.00 ft. lbs. **OK**

Check Shear Capacity

V_{DC} = 10.42 * ((69.88/2)/12 - 2.5/12) = 28.16 lbs.
V_{LL} = 90.00 * ((69.88/2)/12 - 2.5/12) = 243.28 lbs. **Controls**

$Vu = 1.25V_{DC} + 1.75V_{LL}$ = 460.94 lbs. (Strength I)

$\phi V_n = \phi * F'_v * (2/3) * B_{eff} * h$
ϕ = 0.75
$F'_v = F_{vo} * C_{KF} * C_m * C_I * C_\lambda$
= 230 * 2.5/0.75 * 0.875 * 1 * 0.8
= 536.67 psi

ϕV_n = 0.75 * 536.67 * (2/3) * 12 * 2.5 = 8,050.00 lbs. **OK**

Check Deflection (Service 1)

$I = b_{eff} * h^3/12$ = 12 * 2.5^3/12 = 15.63 in^4
$\Delta_{LL} = 5 * w_{EQ} * L^4/(384 * E * I)$ = 5 * 90.00/12 * 69.88^4/(384 * 1,600,000.00 * 0.83 * 15.63)
= 0.11 in = L/625.28 **OK**

Permit Number: 19-00892

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Rail Design

Use West Coast Douglas Fir, No. 1 Use Wet Stresses
Rail Width: 1.500 in.
Rail Depth: 7.500 in.

F_{boxx} =	1000 psi	C_{KF} =	2.5/ϕ	C_{mb} =	0.85	C_i =	0.8
F_{boyy} =	1000 psi	c_{fu} =	1.15	C_{mv} =	0.97	C_F =	1.3
F_{voxx} =	180 psi	c_λ =	0.80	C_{mE} =	0.9		
F_{voyy} =	180 psi	$C_l = C_d$ =	1.00				
E =	1.70E+06 psi						

Max. Post Spacing: 6.5 ft.

Check Moment Capacity

w_{DC} = 1.5 * 7.5 * 50/144 = 3.91 plf.
w_{xx} = wyy = 50 plf.
$M_{DC} = w_{DC} * L^2/10$ = 3.91 *6.5^2/10 = 16.50 ft. lbs.
Pxx = Pyy = 200 lbs.
Myy = $(w_{yy} * L^2/10) + (Pyy * L/6)$ = (50 *6.5^2/10) + (200 * 6.5/6) = 427.92 ft. lbs.
Mxx = $(w_{xx} * L^2/10)$ = (50 *6.5^2/10) = 211.25 ft. lbs.

$Mu_{xx} = 1.25*M_{DC} + 1.75*M_{XX}$ = 1.25 * 16.50 + 1.75 *211.25 = 390.31 ft. lbs.
$Mu_{yy} = 1.75*M_{yy}$ = 1.75 *427.92 = 748.85 ft. lbs.
$\phi M_{nxx} = \phi * F'_{bxx} * S_{xx}$
ϕ = 0.85
$F'_{bxx} = F_{boxx} * C_{KF} * C_m * C_l * C_d * C_\lambda * C_i * C_F$
= 1000 * 2.5/0.85 * 0.85 * 1 * 1 * 0.8 * 0.8 * 2,080.00 psi
$F'_{byy} = F_{boyy} * C_{KF} * C_m * C_{fu} * C_l * C_d * C_i * C_F$
= 1000 * 2.5/0.85 * 0.85 * 1.15 * 1 * 1 * 0.8 * 0.8 * 1.3 2,392.00 psi
Sxx = B * h2/6 = 1.5 * 7.5^2/6 = 14.06 in^3
Syy = h * B2/6 = 7.5 * 1.5^2/6 = 2.81 in^3
$\phi M_{nxx} = F'b_{xx} * S_{xx}/12$ = 2,071.88 ft. lbs.
$\phi M_{nyy} = F'b_{yy} * S_{yy}/12$ = 476.53 ft. lbs.
$M_{uxx}/\phi M_{nxx} + M_{uyy}/\phi M_{nyy}$ = 390.31/2,071.88 + 748.85/476.53 = 0.19 + 1.57 = 1.76 **>1.0, NG** SEE HAND CALCS

Check Shear Capacity

V_{DC} = 3.91 * (6.50/2 - 7.5/12) = 10.25 lbs.
V_{xx} = 50 * (L/2-d/12) + Pxx * (L - d/12)/L = 50 * (6.5/2 - 7.5/12) + 200 * (6.5 - 7.5/12)/6. 312.02 lbs.
V_{yy} = 50 * (L/2-b/12) + Pyy * (L-b/12)/L = 50 * (6.5/2 - 1.5/12) + 200 * (6.5 - 1.5/12)/6.5 = 352.40 lbs.
Vuxx = 1.25VDC + 1.75Vxx = 1.25 * 10.25 + 1.75 * 312.02 = 558.85 lbs. (Strength I)
Vuyy = 1.75Vyy = 1.75 * 352.40 = 616.71 lbs.
$\phi V_n = \phi * F'_v * (2/3) * B_{eff} * h$
ϕ = 0.75
$F'_{vxx} = F_{voxx} * C_{KF} * C_m * C_l * C_\lambda$
= 180 * 2.5/0.75 * 0.97 * 1 * 0.8 = 465.60 psi
$\phi V_{nxx} = \phi V_{nyy}$ = 0.75 * 465.60 * (2/3) * 1.5 * 7.5 = 2,619.00 lbs. **OK**
$F'_{vyy} = F_{voyy} * C_{KF} * C_m * C_l * C_\lambda$
= 180 * 2.5/0.75 * 0.97 * 1 * 0.8 = 465.60 psi
$\phi V_{nxx} = \phi V_{nyy}$ = 0.75 * 465.60 * (2/3) * 1.5 * 7.5 = 2,619.00 lbs. **OK**

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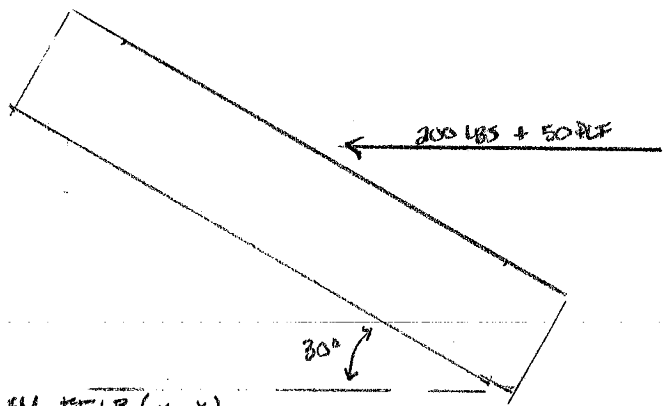
RAIL DESIGN, USE 2x10 DF#1 TOP RAIL, POST SPACING = 6.5 FT MAX

TOP RAIL DESIGNED FOR HORIZONTAL LOAD



$W_{DC} = \frac{1.5(9.25)(50)}{144} = 4.82$ PLF

$W_{yy} = 50 \sin 30 = 25$ PLF

$W_{xx} = 50 \cos 30 = 43.30$ PLF

$P_{yy} = 200 \sin 30 = 100$ LBS

$P_{xx} = 200 \cos 30 = 173.21$ LBS

$M_{DC} = \frac{(4.84 \sin 30)(6.5)^2}{10} = 10.18$ FT-LB (Y-Y) AND 17.64 FT-LB (x-x)

$M_{yy} = \frac{25(6.5)^2}{10} + \frac{100(6.5)}{6} = 213.96$ FT-LB

$M_{xx} = \frac{43.30(6.5)^2}{10} + \frac{173.21(6.5)}{6} = 370.59$ FT-LB

$M_{uyy} = 1.25(10.18) + 1.75(213.96) = 387.16$ FT-LB

$M_{uxx} = 1.25(17.64) + 1.75(370.59) = 670.58$ FT-LB

$F'_{byy} = F_b C_{KF} C_M C_\lambda C_i C_F C_{fu} = 1000\left(\frac{2.5}{0.85}\right)(0.85)(0.8)(0.8)(1.3)(1.15) = 2{,}392$ PSI

$F'_{bxx} = 1000\left(\frac{2.5}{0.85}\right)(0.85)(0.8)(0.8)(1.3) = 2{,}080$ PSI

$S_{yy} = \frac{9.25(1.5)^2}{6} = 3.47$ IN^3

$S_{xx} = \frac{1.5(9.25)^2}{6} = 21.39$ IN^3

$\phi M_{nyy} = 0.85(2{,}392)3.47/12 = 587.93$ FT LBS

$\phi M_{nxx} = 0.85(2080)\, 21.39/12 = 3{,}151.46$ FT LBS

$\frac{M_{uxx}}{\phi M_{nxx}} + \frac{M_{uyy}}{\phi M_{nyy}} = \frac{670.58}{3151.46} + \frac{387.16}{587.93} = 0.87 < 1.0$ ✓ OK

$\boxed{\text{CAP DESIGN GOVERNED BY BENDING}}$

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PROJECT HARPER PARK BRIDGE
LOCATION PORT ORFORD, WA
JOB NO. 194017 BY ZF

DATE 6.8.20
SHEET 5 OF

RAIL DESIGN (CONTINUED), USE 2x8 DF#1 RAIL, POST SPACING = 6.5 FT MAX

RAIL DESIGNED FOR VERTICAL LOAD

$W_{DL} = \frac{1.5(5.5)50}{144} = 2.86$ PLF

$W_{xx} = 50$ PLF

$M_{DL} = \frac{2.86(6.5)^2}{16} = 12.08$ FT-LB

$M_{xx} = \frac{50(6.5)^2}{10} = 211.25$ FT-LB

$M_{uxx} = 1.25(12.08) + 1.75(211.25) = 384.79$ FT-LB

$F'_{bxx} = 2{,}080$ PSI

$S_{xx} = \frac{1.5(5.5)^2}{6} = 7.5625$ IN^3

$\phi M_{nxx} = \frac{0.85(2080)7.5625}{12} = 1{,}114.21$ FT-LB > 384.79 FT-LB ✓ OK

$\boxed{\text{RAIL DESIGN GOVERNED BY BENDING}}$

6

Post Design

Use West Coast Douglas Fir, No. 1 Use Wet Stresses

Post Depth: 5.500 in. (Weak Axis)

Post Width: 5.500 in.

F_{boxx} =	1000 psi	C_{KF} =	2.5/ϕ	C_{mb} =	0.85
F_{boyy} =	1000 psi	c_{fu} =	1.10	C_{mv} =	0.97
F_{voxx} =	180 psi	c_{λ} =	0.80	C_{mE} =	0.9
F_{voyy} =	180 psi	$C_l = C_d$ =	1.00		
E =	1.70E+06 psi				

Height of rail above deck: 43.625 in.

Distance from top bolt to top of girder: 3.00 in.

Distance from top bolt to bottom of post: 15 in.

Post bolt diameter: 3/4 in.

P= wxx * L + Pxx = 50 * 6.50 + 200 = 525 lbs.

a = 43.63 - 7.50/2 + 2.50 + 3.00 = 45.375 in.

M_{xx} = P * a = 525.00 * 45.38 = 23,821.88 in lbs.

M_u = 1.75 * Mxx = 1.75 * 23,821.88 = 41,688.28 in lbs.

$(F'_{byy} = F_{boyy} * C_{KF} * C_m * C_{fu} * C_l * C_d * C_\lambda)$; $(F'_{bxx} = F_{boxx} * C_{KF} * C_m * C_l * C_d * C_l)$

F'byy = 1000 * 2.5/0.85 * 0.85 * 1.1 * 1 * 1 * 0.8 = 2,200.00 psi

Snet = (5.5 -(0.75 + 0.125)) * 5.5^2/6 = 23.32 in^3

$\phi M_{nxx} = \phi * F'b_{yy\ or\ xx} * S_{NET}$ = 43,604.11 in. lbs. **OK**

ϕ = 0.85

$\sum M_T$ = P * a - C * (2/3) * b = 0

= 525.00 * 45.38 - C * (2/3) *15.00 = 0

C = V = 2,382.19 lbs.

Vu = 1.75 * V = 1.75 * 2,382.19 = 4,168.83 lbs.

Anet = (5.5 -(0.75 + 0.125)) * 5.5 = 25.44 in^2

ϕ = 0.75

$(F'_{vyy} = F_{voyy} * C_{KF} * C_{mv} * C_l * C_d * C_\lambda)$; $(F'_{vxx} = F_{voxx} * C_{KF} * C_{mv} * C_l * C_d * C_l)$

= 180 * 2.5/0.75 * 0.97 * 1 * 1 * 0.8 = 465.60 psi

$\phi V_n = \phi * F'v_{yy\ or\ xx} * (2/3) * Anet$ = 5,921.85 lbs. **OK**

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PROJECT HARPER PARK BRIDGE
LOCATION PORT ORFORD, WA
JOB NO. 194017 BY ZF
DATE 6.8.20

POST DESIGN, USE 6x6 DF#1, WET USE

$\Sigma M_T = 0 = -525(45.375) + \left(\frac{2}{3}\right) 15C$

$\rightarrow C = 2{,}382.19$ LBS

$\Sigma F_x = 0 = P + C - T = 525 + 2{,}382.19 - T$

$\rightarrow T = 2{,}907.19$ LBS

$R_u = 1.75(2907.19) = 5{,}087.58$ LBS

$C_b = \frac{l_b + 0.375}{l_b} = \frac{3 + 0.375}{3} = 1.125$

$\phi R_n = \phi F_{c\perp} C_{KF} C_M C_i C_2 A_{NET}$

$A_{NET} = (3.0^2 - 0.815^2)\pi/4 = 6.47\ IN^2$

$\phi R_n = 0.9(625)\left(\frac{2.1}{0.9}\right)(0.91)(1.125)(0.8)\,6.47 = 6{,}951.85$ LBS $> 5{,}087.58$ LBS ✓ ok

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Exterior Girder Design

Girder Width: 5.125 in.
Girder Depth: 28.5 in.
Use West Coast Douglas Fir, 24F-V4 Use Wet Stresses

F_{bo} =	2400 psi	C_{mb} =	0.8000	C_{KF} =	2.5/f
F_{vo} =	265 psi	C_{mv} =	0.8750	c_l =	0.80 For Strength I Design
E =	1.80E+06 psi	C_{me} =	0.8333	$C_l = C_d$ =	1.00
Emin =	9.50E+05 psi	$C_{mFc(perp)}$ =	0.53		
$F_{C(perp)}$ =	650 psi				

$$C_V = \left(\frac{5.125}{b}\right)^{\frac{1}{10}}\left(\frac{12}{d}\right)^{\frac{1}{10}}\left(\frac{21}{L}\right)^{\frac{1}{10}} =$$ 0.8610 **Controls**

Dead Load

Rails & Cap	17.32 plf.
Posts	8.64 plf.
Balusters	0.00 plf.
Deck	31.25 plf.
Blocks	0.00 plf.
Curb	0.00 plf.
Other	17.58 plf.
Girder	50.72 plf.
W_{DC} =	125.50 plf.

W_{LL} = 90.00 * 3.0000 = 270.00 plf

Check Moment Capacity

M_{DC} = 125.50 * 39.5^2/8 = 24,476.21 ft. lbs.
M_{LL} = 270.00 * 39.5^2/8 = 52,658.44 ft. lbs. **Controls**
$M_U = 1.25 * M_{DC} + 1.75 * M_{LL}$ = 122,747.53 ft. lbs.
Lu = 13.16666667 ft.
Le = 1.63 * Lu + 3 * d = 1.63 * 13.17 +(3 * 28.5/12) = 28.5867 ft.
$Rb = (Le * d / b^2)^{1/2}$ = 19.29
$F'_{BE} = 1.2 * E'_{min}/Rb^2$ = 2,552.14 psi.
$F^*_b = F_b * C_{mb}$ = 2400 * 0.8 = 1,920.00 psi.

$$C_L = \frac{1+\left(\frac{F_{bE}}{F^*_b}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{F_{bE}}{F^*_b}\right)}{1.9}\right] - \frac{\frac{F_{bE}}{F^*_b}}{0.95}} =$$ 0.9039

$\phi M_n = \phi * F'_b * S$
ϕ = 0.85
$F'_b = F_{bo} * C_{KF} * C_m * C_l * C_v * C_l$
= 2400 * 2.5/0.85 * 0.8 * 1 * 1 * 0.8610 * 0.8
= 3,889.63 psi
$S = B * h^2/6$ = 693.796875 in^3
ϕM_n = 0.85 * 3,889.63 * 693.796875/12 = 191,151.71 ft. lbs. OK

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Exterior Girder Design

Check Shear Capacity

V_{DC} = 125.50 * (39.5/2 - 28.5/12) = 2,180.54 lbs.
V_{LL} = 270.00 * (39.5/2 - 28.5/12) = 4,691.25 lbs.
$V_U = 1.25*V_{DC} + 1.75*V_{LL}$ = 10,935.37 lbs.

$\phi V_n = \phi * F'_v * (2/3) * b * h$
ϕ = 0.75
$F'_v = F_{vo} * C_{KF} * C_m * C_i * C_l$
= 265 * 2.5/0.75 * 0.88 * 1.00 * 0.8
= 618.33 psi

ϕV_n = 0.75 * 618.33 * (2/3) * 5.125 * 28.50 = 45,157.66 lbs. **OK**

Check Deflection Service I limit state

$\delta_{LL} = 5 * W_{LL} * L^4/(384*E*C_{ME}*I)$ = 5 * 270.00 * 39.5^4 * 1728/(384 * 1800000 * 0.833 * 9,886.61)
= 1.00 = L/475.30 **OK**

Check Bearing

R_U = (1.25 * 125.50 + 1.75 * 270.00) * 40/2 = 12,587.47 lbs.
ϕ = 0.9
$F'_{c(perp)} = F_{c(perp)} * C_{KF} * C_{mFc(perp)} * C_i$ = 650.00 * 2.1/0.90 * 0.53 * 0.80 = 643.07 psi
ϕRn = $\phi * F'_{c(perp)} * b * L$ = 0.90 * 643.07 * 5.125 * 6.00 = 17,796.87 lbs. **OK**

Harper Park Bridge
Port Orford, Washington
Job Number: 194017

LATERAL LOAD DESIGN:

Lateral load design is based on the AASHTO Guide Specifications for Pedestrian Bridges (2009), AASHTO Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, (2015) 1st Edition

Wind Speed $V_{3S,\,ult}$ = 115 mph (Signs figure 3.8.1a)

K_z = 0.84 (Signs 3.8.4)
K_d = 1.00 (Signs 3.8.5)
G = 1.14 (Signs 3.8.6)
C_d = 1.22 (Signs 3.8.7)

Design Wind Pressure: $P_z = 0.00256 * K_z * K_d * G * V^2 * Cd$
Design Wind Pressure: P_z = 39.55 psf

Projected Height: H = 60.68 in.

Wind Load, ult = 200.01 plf (strength level)
Wind Load = 142.86 plf (service level)

Width of Bridge = 6.0 ft.
Height of Bridge = 10.0 ft.
Weight of Bridge (plf): W = 126 PLF
Length of Bridge = 40.0 ft.
Span of Bridge: L = 39.50 ft.
Girder Spacing: s = 69.88 in.

Ext. Girder Width: be = 5.125 in.
Ext. Girder Depth: de = 28.5 in.
Int. Girder Width: bi =
Int. Girder Depth: bi =

E_y = 1600000 psi
C_m = 0.833
E'_y = 1332800 psi
(for deflection calcs.)

I_{eff} = 2 * de * be^3 / 12
I_{eff} = 639 in.^4
Lateral Load Deflection = 9.182 in. = L / 52

Use Rod Bracing System To Reduce Deflection.

Use Diaphragms at 13.17 ft o/c With (1) Bay of Rod Bracing Each End.
Center Span Length = 13.167 ft.
Center Span Deflection = 0.113 in. = L / 1394 **< L/360 OK**

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Harper Park Bridge
Port Orford, Washington
Job Number: 0194017

LATERAL LOAD DESIGN: (Continued)

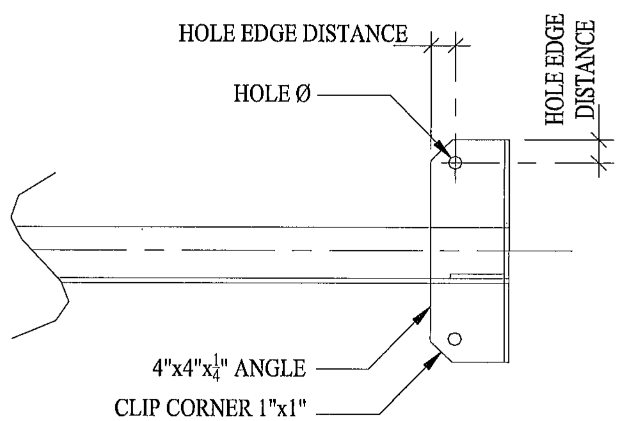
Rod Brace Design:

Use A36 Steel Rods F_y = 36000 psi
F_u = 58000 psi

Use Diaphragms at 13.17 ft o/c With (1) Bay of Rod Bracing Each End.

Angle of Rod to Beam = 22.29°

Location	Lateral Load Reaction	Rod Force	Rod Description	Tension Capacity	Check	Hole Edge Distance	Hole Ø
End of bridge:	4000						
		7032	Use (1) 5/8 in. Ø Rod	8750	OK	1.25"	11/16"
First Diaphragm:	2667						



ANGLE TAB AT DIAPHRAGM BASE
(NO SCALE)

Location	Lateral Load Reaction	Force Parallel to Girder	Connection to Girder	Conn. Capacity	Check
End of bridge:	4000	6547	Use (1) 1" Ø Machine Bolt	8647.13	OK
First Diaphragm:	2683	6547	Use (3) 3/4" Ø Machine Bolts @3" o/c	7484	OK

Western Wood Structures, Inc.
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(503) 692-6900

June 10, 2020

Harper Park Bridge
Port Orford, Washington
Job Number: 194017

Seismic Design

Seismic Design is based on AASHTO LRFD Bridge Design Specifications (AASHTO)

AASHTO 4.7.4.2 states, "Seismic analysis is not required for single span bridges, regardless of Seismic zone. Connections between the Bridge Superstructure and the abutments shall be shall be designed for the minimum force requirements as specified in Article 3.10.9. Minimum Support length requirements shall be satisfied at each abutment as specified in Article 4.7.4.4.

From Article 3.10.9

PGA = 0.42 g (AASHTO Figure 3.10.2.1-1)
F_{pga} = 1.08 (AASHTO table 3.10.3.2-1)

Weight of Bridge: W = 125.5 * 40 = 5,020.00 lbs.

$A_S = F_{pga}$ * PGA = 1.08 * 0.42 = 0.453600 (AASHTO Eq. 3.10.4.2-2)
$P_{EQ} = A_S$ * W = 0.4536 * 5,020.00 = 2,277.07 lbs.
P_{EQ} /0.8= 2,277.07 / 0.8 = 2846.3 lbs.

From Article 4.7.4.4

H= 0.00 ft.
S= 0.00 degrees

$N = (8 + 0.02L + 0.08H)(1+0.000125S^2)$ = 8.80 in. → 9 IN + 1 IN CLEAR = 10 IN

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Harper Park Bridge -Port Orford, WA - WWSI# 194017
June 10, 2020

Mudsill Design

Soil Bearing Pressure: 2000 psf

Mudsill width	12 in.
Mudsill depth:	6.75 in.
Mudsill Length:	8 ft.
Girder Spacing:	5.82 ft.

Use West Coast Douglas Fir, Comb. 2 — Use Wet Stresses

F_b =	1800 psi	C_{KF} =	2.5/ϕ	C_{mb} =	0.8
F_v =	230 psi	c_{fu} =	1.10	C_{mv} =	0.875
E =	1.60E+06 psi	c_λ =	0.80	C_{mE} =	0.8333
		$C_l = C_d$ =	1.00		

Dead Load:	5020 lb
Live Load:	10800 lb
Total =	15820

Bearing Capacity

R= 12/12*8*2000 = 16000 lb **ok**

Check Moment Capacity

W_{DC} = 5020/8 = 627.50 plf

W_{LL} = 10800/8 = 1350.00 plf

M_{DC} = -(627.50*8/2)*(5.82/2)+(627.50*(8^2)/8)	-2284.1	ft. lbs.	**<---Controls**
M_{DC} = (627.50*((8-5.82)/2)^2)	372.766	ft. lbs.	
M_{LL} = -(1,350.00*8/2)*(5.82/2)+(1,350.00*(8^2)/8)	-4914.0	ft. lbs.	**<---Controls**
M_{LL} = (1,350.00*((8-5.82)/2)^2)	801.968	ft. lbs.	

$M_U = 1.25 * M_{DC} + 1.75 * M_{LL}$ = 11,454.63 ft. lbs.

$\phi M_n = \phi * F'_b * S$

ϕ = 0.85

$F'_b = F_{bo} * C_{KF} * C_m * C_{fu} * C_l * C_\lambda$

= 1800 * 2.5/0.85 * 0.8 * 1.1 * 1 * 0.8000

= 3,727.06 psi

$S = B * h^2/6$ = 91.125 in^3

ϕM_n = 0.85 * 3,727.06 * 91.125/12 = 24,057.00 ft. lbs. **OK**

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Backwall Design Active Pressure: 35 psf

Backwall width	2.5 in.	
Member depth	11.25 in.	
Backwall depth:	45.3125 in.	
Backwall Length:	13 ft.	MAX

Use West Coast Douglas Fir, No. 1 Use Wet Stresses

F_b =	1000 psi	C_{KF} =	2.5/ϕ	C_{mb} =	0.85	C_F =	1
F_v =	180 psi	C_{fu} =	1.00	C_{mv} =	0.97	C_i =	0.8
E =	1.70E+06 psi	C_λ =	0.80	C_{mE} =	0.9		
		$C_i = C_d$ =	1.00				

R= (1/288)*35*(45.3125^2-(45.3125-11.25)^2) = 108.52 plf

M = -(108.52*13/2)*(5.823/2)+(108.52*(13^2)/8) 238.77 ft. lbs.
M = (108.52*((13-5.823)/2)^2) 698.72 ft. lbs. <---Control:
M_U = 1.5 * M = 1,048.09 ft. lbs.

$\phi M_n = \phi * F'_b * S$
ϕ = 0.85
$F'_b = F_{bo} * C_{KF} * C_m * C_{fu} * C_\lambda * C_F * C_i$
= 1000 * 2.5/0.85 * 0.85 * 1 * 0.8 * 1 * 0.85 * 1 * 0.8
= 1,600.00 psi
$S = B * h^2/6 =$ 11.71875 in^3
ϕM_n = 0.85 * 1,600.00 * 11.71875/12 = 1,328.13 ft. lbs. **OK**

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