PACIFIC
ENGINEERING TECHNOLOGIES, INC.

<u>20088</u>

<u>Doug Rosen Residence – Pier & Pile Design</u>

5995 SE 30th Street
Mercer Island, Washington 98040

KYLE J. BOZICK
STATE OF WASHINGTON
41970
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

12/2/2020

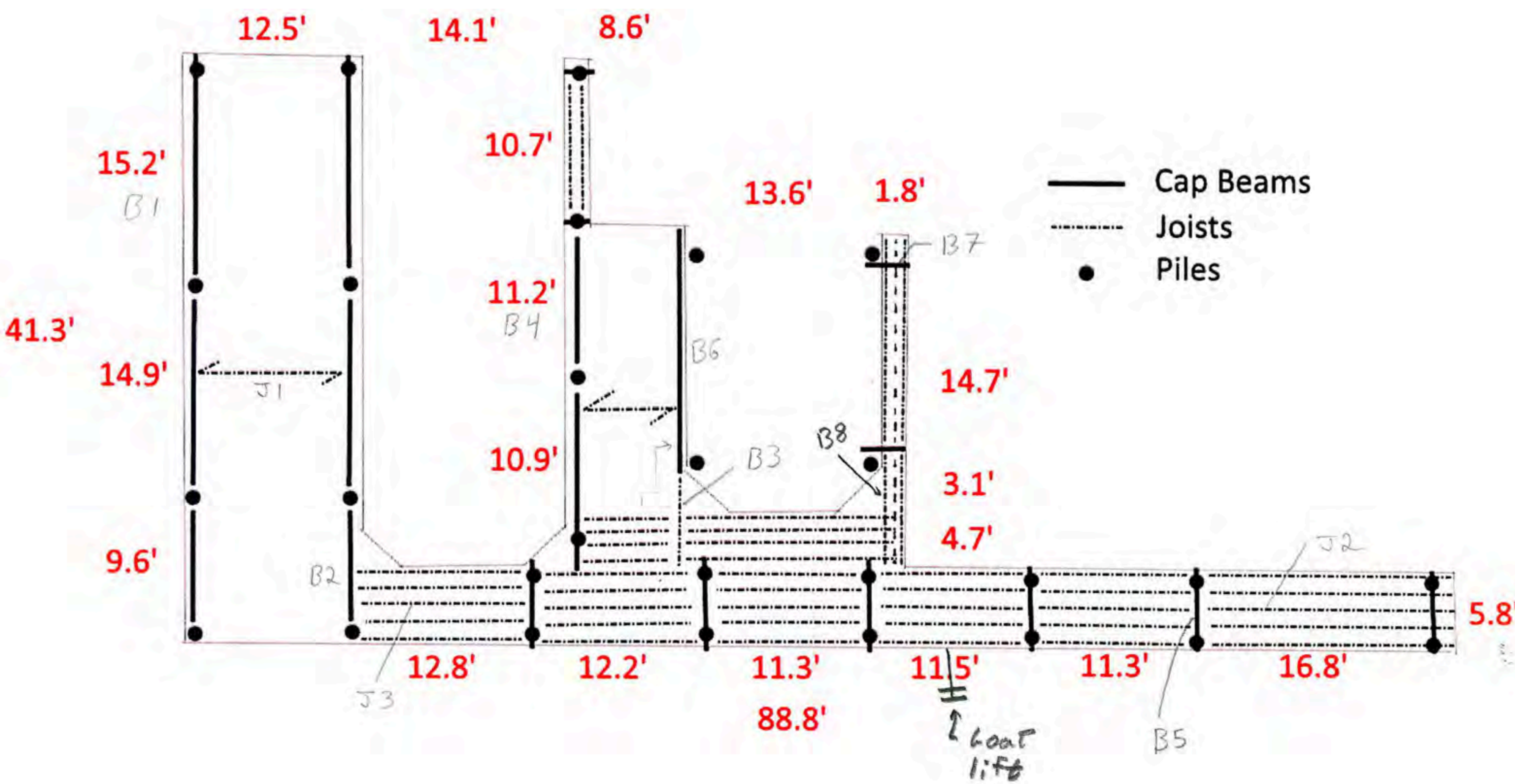
The engineering seal on these calculations are for the items listed below:

- **Design of pier framing members: cap beams and joists.**
- **Analysis of the proposed pile splice and timber riser connections.**
- **Design of the Jet Ski lift attachments.**
- **Design of the steel piles supporting the moorage roof.**

Design is in accordance with the 2015 International Building Code and 2015 International Existing Building Code. Our scope of work does not include analysis and design of the grating, bulkhead, connection to grade, moorage cover roof and/or as associated connections.

The site information, dimensions and plan layout, has been provided to us by Waterfront Construction, Inc.

20088 Rosem Pier
Framing Layout



Project No: 20088 Date: Jun 1 2020 Sheet: ____ Of: ____

Project Name: Rosem Pier

Comp. By: C.S. Chk.By: ____

Contents: Unit weight of pier

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Grating	2.0 psf
4x8 at 16" o.c.	4.6 psf
MISC	1.5 psf
	~8 psf

'roject No: 20088 Date: Jun 1 2020 Sheet: ___ Of: ___

Project Name: Rosen pier

Comp. By: G.S. Chk.By: ___

Contents: J1

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J1

Proposed size 2x8 at 16" o.c.

Assume DF No. 2 P.T.

Clear span = 10'

Simple Span

$M = w * L^2/8 = 90.7 * 10^2/8 = 1134$ #-ft

$w = [8 psf, D + 60 psf, L] * 1.33 ft = 90.7$ plf

$f_b = M/S_x = 1134$ #-ft $* (12$ in/ft$) / 13.14 in^3 = 1036$ psi

$S_x = 1.5 * 7.25^2/6 = 13.14 in^3$

$F'_b = (F_b = 900 psi) * (C_F = 1.15) * (C_r = 1.2) * (C_i = 0.8) = 994$ psi

$\Delta_{LL} = \frac{5\ w\ L^4}{384\ E'\ I} = \frac{5 * 90.7\frac{\#}{ft} * (\frac{ft}{12 in}) * (10' * 12\frac{in}{ft})^4}{384 * 1.52 * 10^6 psi * 47.6 in^4} = 0.282$ in

$E' = (E = 1.6 * 10^6 psi) * (C_i = 0.95) = 1.52 * 10^6$ psi

$I = 1.5 * 7.25^3/12 = 47.6 in^4$

$\Delta_{LL, Allow} = \frac{L}{360} = \frac{10' * 12''/ft}{360} = 0.333$ in

2x8 DF No. 2 at 16" o.c. $f_b > F_b$ 4.2%

Project No: 20088 Date: Jun 1, 2020 Sheet: ___ Of: ___

Project Name: Rosen pier

Comp. By: G. S. Chk.By: ___

Contents: J2

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J2

Proposed size 4x8 at 16" o.c.

Assume DF No. 1 P.T.

Clear span = 16.8' − 5.5" = 16.4'

Simple span

$M = w * L^2/8 = 90.7\,plf * (16.4ft)^2/8 = 3049\,\#ft$

$w = [60psf_{,L} + 8psf_{,D}] * 1.33ft = 90.7\,plf = 80\,plf_{,L} + 10.7\,plf_{,D}$

$f_b = M/S_x = 3049*12/30.7 = 1192\,psi$

$S_x = 3.5 * 7.25^2/6 = 30.7\,in^3$

$F_b' = (F_b = 1000psi) * (C_r = 1.15) * (C_F = 1.3) * (C_i = 0.8) = 1196\,psi$

$\Delta_{LL} = \dfrac{5\, w\, L^4}{384 * E * I} = \dfrac{5 * \left(80\, \frac{\#}{ft} * \frac{ft}{12in}\right) * \left(16.4ft * \frac{12in}{ft}\right)^4}{384 * 1.62*10^6 psi * 111.1\, in^4} = 0.723\,in$

$E' = (E = 1.7*10^6\, psi) * (C_i = 0.95) = 1.62*10^6\, psi$

$I = 3.5 * 7.25^3/12 = 111.1\, in^4$

$\Delta_{LL,ALLOW} = \dfrac{L}{360} = \dfrac{16.4ft * 12\, in/ft}{360} = 0.547\,in$

4 X 8 DF No. 2 PT at 16" o.c. $f_b < F_b'$ o.k.

$\Delta_{LL} > \Delta_{LL,ALLOW}$ 32.2% over

4X8 DF No. 2 PT at 12" o.c. req'd Δ_{LL} = 5% over N.G.

Use 4X8 DF No. 1 PT at 12" o.c.

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Project Name: Rosen Pier

Comp. By: G.S. Chk.By: ___

Contents: J3

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J3

Proposed size 4x8 at 16" o.c.
Assume DF No. 2 PT

Clear span = 12.8' − 5.5" = 12.4'

Simple span

$M = w * L^2/8 = 90.7\,plf * (12.4ft)^2/8 = 1743\#ft$

$w = [60psf_{,L} + 8psf_{,D}] * 1.33ft = 90.7\,plf \quad (80plf_{,L})$

$f_b = M/S_x = 1743*12/30.7 = 681\,psi$

$S_x = 3.5*7.25^2/6 = 30.7\,in^3$

$F_b' = (F_b = 900psi) * (C_r = 1.15) * (C_F = 1.3) * (C_i = 0.8) = 1076\,psi$

$$\Delta_{LL} = \frac{5\,w\,L^4}{384*E'*I} = \frac{5 * (80plf_{,L} * \frac{ft}{12in}) * (12.4ft * \frac{12in}{ft})^4}{384 * 1.52*10^6 psi * 111.1\,in^4} = 0.252\,in$$

$E' = (E = 1.6*10^6\,psi) * (C_i = 0.95) = 1.52*10^6\,psi$

$I = 3.5 * 7.25^3/12 = 111.1\,in^4$

$$\Delta_{LL,ALLOW} = \frac{L}{360} = \frac{12.4ft * 12in/ft}{360} = 0.413\,in$$

4x8 DF No. 2 at 16" o.c. or better $\quad f_b < F_b'$, $\Delta_{LL} < \Delta_{LL,ALLOW}$

Joists are adequate.

Joist Reaction = 90.7 plf * 12.4 ft/2 = 562#

(use Simpson LUS46 Face Mount hangers, cap = 885#)

Project No: 20088 Date: Jan 1 2020 Sheet: ___ Of: ___

Project Name: Kosem pier

Comp. By: G. S. Chk.By: ___

Contents: B1

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B1
Proposed size 6 X 8
Assume DF No. 2 P.T.

Center to Center span = 15.2'
Clear span ≈ 15.2' - 1' = 14.2'

Simple span

$M = w * L^2/8 = 425 plf * (14.2 ft)^2/8 = 10,712 \#ft$

$w = [60 psf_{LL} + 8 psf_{DL}] * \left(\frac{12.5'}{2}\right) = 425 plf$

$f_b = M/S_x = 10.712 \#ft * (12 in/ft) / 48.2 in^3 = 2667 psi$

$S_x = 5.5 * 7.25^2/6 = 48.2 in^3$

$F_b' = (F_b = 875 psi) * (C_i = 0.8) = 700 psi$

$\Delta_{LL} = \frac{5 \; w L^4}{384 \; E' I} = \frac{5 * (425 \frac{\#}{ft} * \frac{ft}{12 in}) * (14.2' * (12 in/ft))^4}{384 * 1.235 * 10^6 psi * 175 in^4} = 1.8 in$

$E' = 1.3 * 10^6 psi * (C_i = 0.95) = 1.235 * 10^6 psi$

$I = 5.5 * 7.25^3/12 = 175 in^4$

$\Delta_{LL,ALLOW} = \frac{L}{360} = \frac{14.2 ft * 12 in/ft}{360} = 0.473 in$

6 X 8 DF No. 2 P.T. $f_b > F_b'$ 281%

$\Delta_{LL} > \Delta_{LL,ALLOW}$ 281%

$\boxed{\text{6 X 12 DF No. 1 P.T. Req'd}}$

Project No: 20088 Date: Jun 1 2020 Sheet: ___ Of: ___

Project Name: Rosen pier

Comp. By: G.S. Chk.By: ___

Contents: B2

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B2

Proposed size 6x8

Assume DF No.2 P.T.

Clear span = 9.6ft − 1ft = 8.6ft

w_1 = [60psf$_{LL}$ + 8psf$_{D}$] * (12.5ft/2) = 425 plf

w_2 = [60psf$_{LL}$ + 8psf$_{D}$] * (12.8ft/2) = 435 plf

Beam Size Properties

A_n = 39.9 in^2

S_x = 48.2 in^3

I_x = 174.7 in^4

Material Properties

F_b' = (F_b = 875psi) * (C_d = 0.8) = 700psi

E' = (E = 1.3*10^{6}psi) * (C_i = 0.95) = 1.235*10^{6}psi

M = 6811 #ft → f_b = 1696 psi > F_b'

Δ_{TOT} = 0.412 in → L/250

Δ_{LL} = 0.364 in → L/283 > L/360

$\boxed{\text{6 x 12 DF No.1 P.T. Req'd}}$

Project No: 20088 Date: Jun 1 2020 Sheet: Of:

Project Name: Rosen Pier

Comp. By: G.S. Chk.By:

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B4

Proposed size 6 x 8

Assume DF No. 2 P.T.

Clear span = 11.2' - 1' = 10.2'

Simple Span

$M = w * L^2/8 = 292 \text{ Plf} * (10.2 \text{ ft})^{\wedge 2}/8 = 3797 \text{ \#ft}$

$w = [60 \text{ psf}_{,L} + 8 \text{ psf}_{,D}] * (8.6'/2) = 292 \text{ Plf} \ (258 \text{ Plf}_{,L})$

$f_b = M/S_x = 3797 * 12/48.2 = 945 \text{ psi}$

$S_x = 5.5 * 7.25^{\wedge 2}/6 = 48.2 \text{ in}^3$

$F'_b = (F_b = 875 \text{ psi}) * (C_i = 0.8) = 700 \text{ psi}$

$\Delta_{LL} = \dfrac{5 w L^4}{384 E' I} = \dfrac{5 * (258 \text{ plf} * \frac{ft}{12 in}) * (10.2 ft * \frac{12 in}{ft})^{\wedge 4}}{384 * 1.235 * 10^{\wedge 6} \text{ psi} * 175 \text{ in}^4} = 0.291 \text{ in}$

$E' = (1.3 \times 10^6 \text{ psi}) * (C_i = 0.95) = 1.235 * 10^6 \text{ psi}$

$I = 5.5 * 7.25^{\wedge 3}/12 = 175 \text{ in}^4$

$\Delta_{LL, ALLOW} = \dfrac{L}{360} = \dfrac{10.2' * 12 in/ft}{360} = 0.34 \text{ in}$

6 x 8 DF No. 2 P.T. $f_b > F'_b$ 35%

$\Delta_{LL} < \Delta_{LL, ALLOW}$

6 x 10 DF No. 2 P.T. is Req'd

Project No: 20088 Date: Jun 1 2020 Sheet: Of:

Project Name: Rosen Pier

Comp. By: G. S. Chk.By:

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B5

Proposed size 6x8

O.K by inspection

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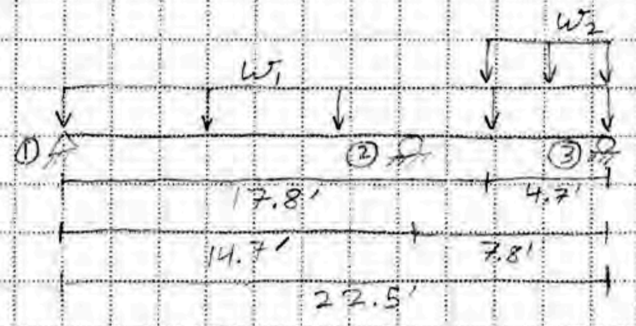
B6
Proposed size Exist 6x8
Inadequate by inspection
Steel member is required for connection to steel Pile



w_1 = [60psf,L + 8psf,D] * (8.6'/2) = 292 PLF
w_2 = [60psf,L + 8psf,D] * (13.6'/2) = 462 PLF

M_{max} = 6647 # ft
V_{max} = 2645 #
Reaction 1 = 1694 #
Reaction 2 = 5244 #
Reaction 3 = 1804 #

Assume full lateral support at top flange of steel beam

Try MC 10 x 22
Z = 23.9 in^3
I = 102 in^4
E = 29×10^6 psi
F_y = 50,000 psi
Ω = 1.67

$M_a = M_n/\Omega = M_p/\Omega = F_y Z/\Omega$
$Z_{req'd} = \Omega M_a/F_y$ = 2.66 in^3 < 23.9 in^3 o.k.
Δ_{TOT} = 0.052 in o.k.
Δ_{LL} = 0.046 in o.k.

$\boxed{\text{Use MC10 x 22 For B6}}$
Refer to calc ahead for support connection

Project No: 20088 Date: Jun 1, 2020 Sheet: ___ Of: ___
Project Name: Rosen Pier
Comp. By: G.S. Chk.By: ___
Contents: B6 connections

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B6 connections:

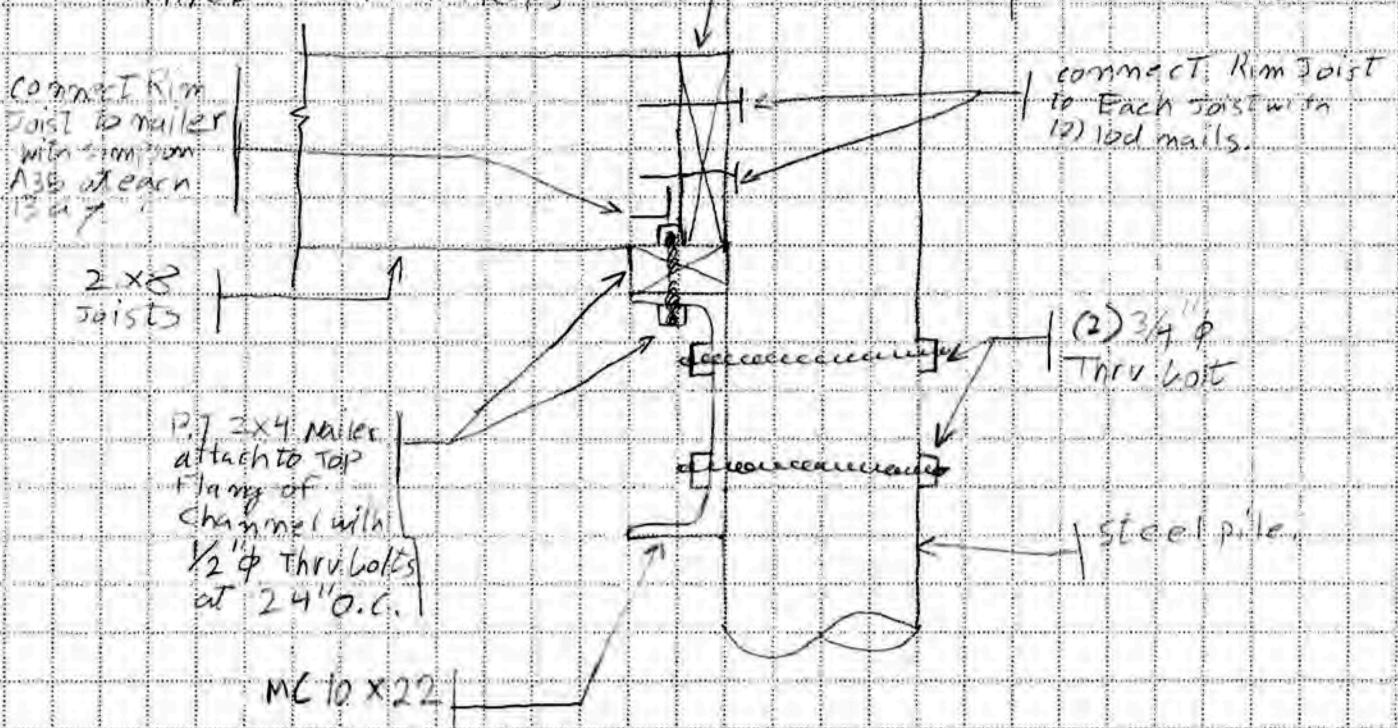
Total demand on Mid support, ② = 5244#

$\boxed{\text{Use (2) } 3/4''\phi \text{ A 307 bolt in single shear}}$

AISC Table 7-1

$r_n/\Omega = 5.97$ kips

$R_n/\Omega = 11.94$ kips



Project No: 20088 Date: Jun 16, 2020 Sheet: ___ Of: ___

Project Name: Rosem Pier

Comp. By: G.S. Chk.By: ___

Contents: B6 connections

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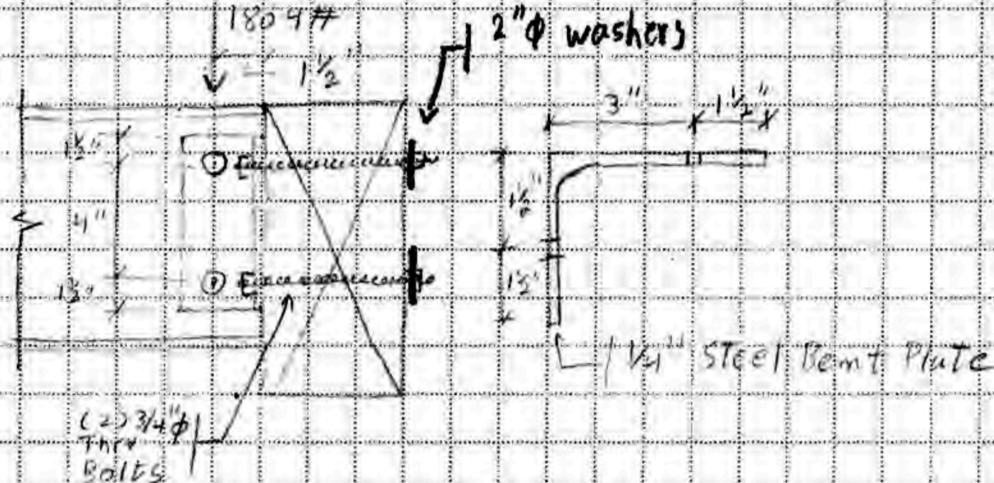
B6 connections

Total demand on end support ③ = 1804#



$\frac{2}{3}(4" + 1\frac{1}{2}") = 3.7"$

Moment = 1804# * 1½" = 2706 #-in

T/C = 2706#in / 3.7" = 731#

Shear per bolt = 1804# / 2 bolt = 902#

Tension resisted by top washer

2"ϕ washer

$F'_{c\perp} = 625\ psi * (C_i = 1.0) = 625\ psi$

$F_{c\perp} = 625\ psi * 2.54\ in^2 = 1588\#$ o.k.

Bearing area $= \pi/4\,(2^2 - (3/4 - 1/8)^2) = 2.54\ in^2$

Shear resisted by bolts

$Z_\perp = 940\#$ o.k.

$\boxed{\text{use the connection shown above}}$

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Project Name: Rosem Pier

Comp. By: G. S. Chk.By:

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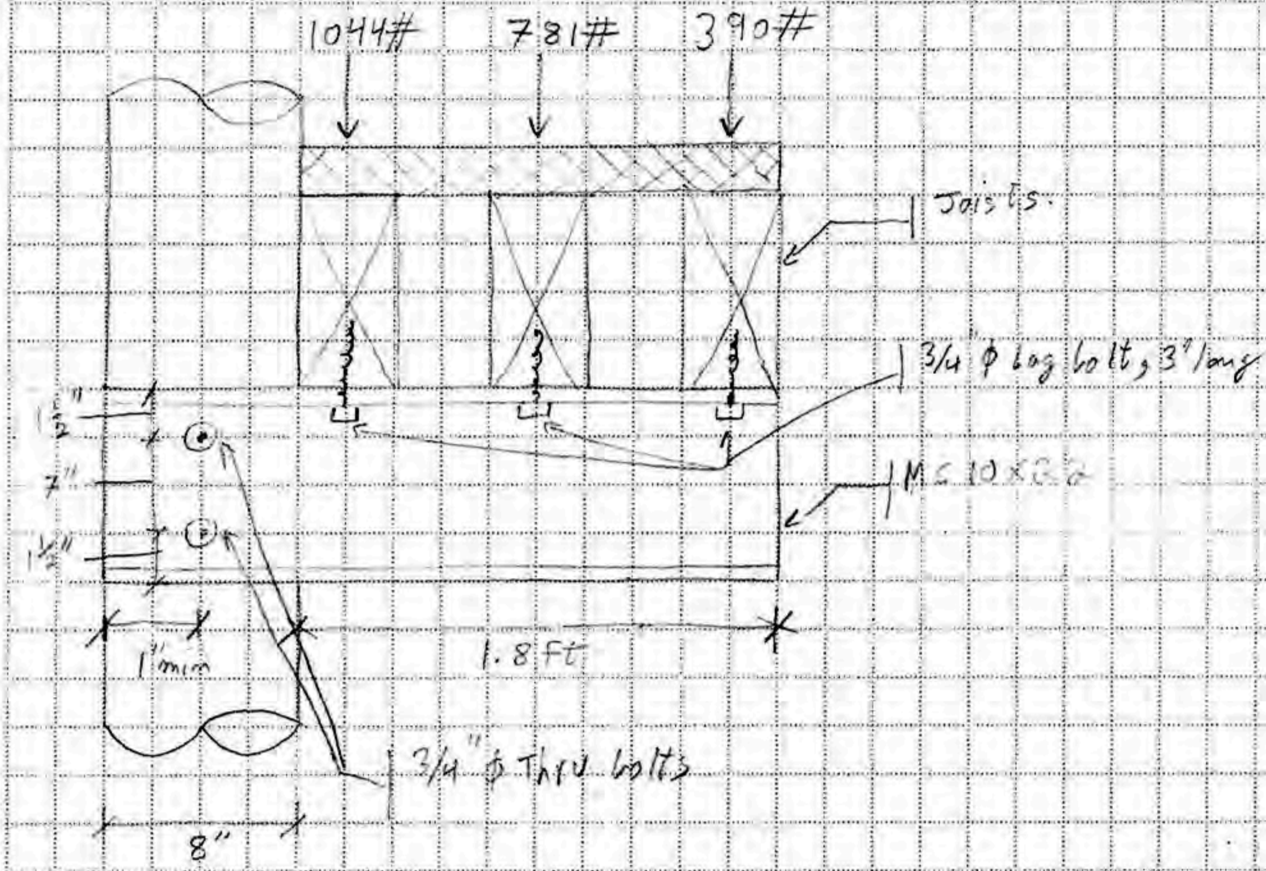
B7



M = 2163 #ft
V = 2215 #

$$\text{shear load per bolt} = \sqrt{\left(\frac{2163\#ft}{7''}\right)^2 + \left(\frac{2215\#}{2}\right)^2} = 1150\#$$

$\boxed{\text{USE (2) 3/4" } \phi \text{ A307 bolts as illustrated above}}$

AISC Table 7-1
$r_n/\Omega = 5.97$ Kips

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Project Name: Rosem Pier

Comp. By: G.S. Chk.By: ____

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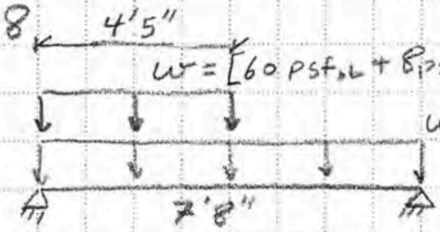
B8



w = [60 psf, L + 8 psf, D] * 6.8 ft = 462 plf

w = [60 psf, L + 8 psf, D] * 0.5 ft = 34 plf

TRY 4X8 DF No.1 P.T.

M = 2526 #ft

V = 1583 #

Δ_{TOT} = 0.140 in

P.T. 4X8 DF No.1 is adequate, Refer to calc for J2

Project No: 20088 Date: Jun 1 2020 Sheet: Of:

Project Name: Rosem Pier

Comp. By: G. S. Chk.By:

Contents: Moorage Cover Piles

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Moorage Cover Piles
Wind loads per pile
Assume 10ft height of boat above water under moorage cover.
Assume 15ft length of boat centered between piles.
Boat can be analyzed as low rise enclose building

ASCE 7-10 Ch-28 Part 2

Risk category I/II
Baise wind speed 110 mph
$K_{zt} = 1.0$
Exposure category = C
$\lambda = 1.21$

Design for Zome C $\rightarrow P_{s30} = 12.7$ psf
$P_s = \lambda K_{zt} P_{s30}$
$= 1.21 * 1.0 * 12.7$ psf
$= 15.4$ psf on boat
0 psf on Roof

Code Min
$P_s = 16$ psf on boat
$P_s = 8$ psf on Roof
ignore K_{zt} + λ

wind loads from boat on pile. = 16 psf * 10ft * 15ft / 2 = 1200 #

Wind loads from Roof cover

Total projected Area = 20' * 1.7' = 34 sq ft
Trib to each pile = 34 sq ft / 4 = 8.5 sq ft

Wind loads from Roof cover = 8.5 sq ft * 8 psf = 68#

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Project Name: Rosen Pier

Comp. By: G. S. Chk.By: ___

Contents: Moorage Cover Piles.

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Moorage Cover Piles

Seismic load per Pile

Analyze max loaded pile

Dead load trib to pile

Moorage cover

$(10\,psf,D) * \frac{14ft * 20ft}{4} = 700\#,D$

- Pier load

Load from B6 interior support connection = 3944# Total

= 464#,D

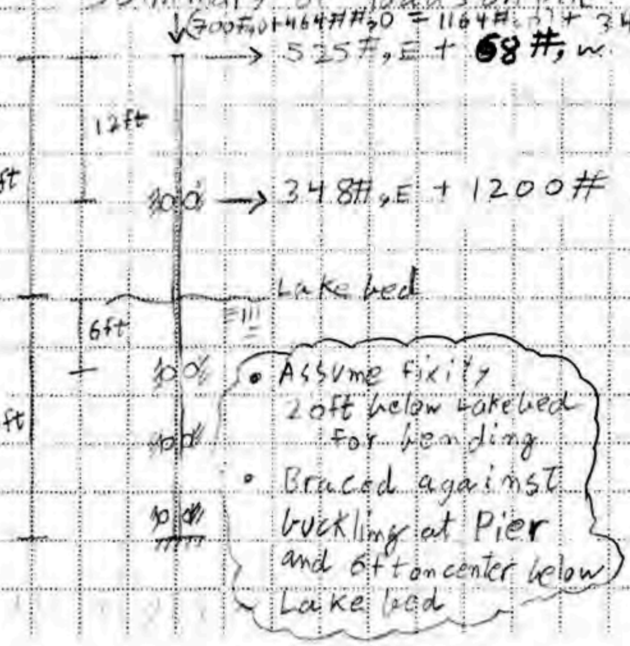
$C_s = 0.75$

Seismic loads on pile

Moorage cover = 525#,E

Pier = 348#,E

Summary of loads on pile



$M_E = 525\# * 42ft + 348\# * 30ft$

$= 32,490\#ft$

$M_w = 68\# * 42ft + 1200\# * 30ft$

$= 38,856\#ft$

D = 1164#

L = 3480#

Max unbraced length, $L_u = 16ft$

Pile Size	D (LBS)	L (LBS)	M_E (#ft)	M_W (#ft)
8" dia. STD	1164	3480	32490	38856

Fy (ksi)	Max unbraced length, Lb (ft)	r	k	4.71*sqrt(E/Fy)	kL/r	Fe (ksi)	Fcr (ksi)	Pn/Ωc (K)	Mn/Ωb (KFT)
45	16.0	3.0	1.2	119.6	78.1	46.9	30.1	141.6	46.7

15% Check		(1.0 + 0.14 Sds)D + 0.7*Ω*Qe			(1.0 + 0.105 Sds)D + 0.525*Ω*Qe + 0.75L			1.0D + 0.6W		
P	Pdem/Pcap ≤ 15%	P	M	Pr/Pc less than 0.2, Pr/(2*Pc) + Mdem/Mcap ≤ 1.0	P	M	Pr/Pc less than 0.2, Pr/(2*Pc) + Mdem/Mcap ≤ 1.0	P	M	Pr/Pc less than 0.2, Pr/(2*Pc) + Mdem/Mcap ≤ 1.0
3.889	2.7%	1.317	28.429	0.61	3.889	21.322	0.47	1.164	23.3136	0.50
	ok			ok			ok			ok

	Wt	r	Ag (sqin)	I (in^4)	Z (in^3)		Fy (psi)	Mn/Ωb (KFT)	
4" dia. X	15 PLF	1.48 in	4.14	9.12		5.53	35	9.658183633	
~~6" dia. STD~~	~~19 PLF~~	~~2.25 in~~	~~5.22~~	~~26.50~~		~~10.6~~	~~35~~	~~18.51297405~~	(unable to be installed)
6" dia. X	28.6 PLF	2.2 in	7.88	38.30		15.6	35	27.24550898	
~~8" dia. STD~~	~~28.6 PLF~~	~~2.95 in~~	~~7.85~~	~~68.10~~		~~20.8~~	~~45~~	~~46.70658683~~	(unable to be installed)
8" dia. X	43.4 PLF	2.89 in	11.9	100.00		31	45	69.61077844	
10" dia. STD	40.5 PLF	3.68 in	11.5	151.00		36.9	45	82.85928144	
12" dia. STD	49.6 PLF	4.39 in	13.7	262.00		53.7	45	120.5838323	
14" dia 1/2 " Wall	73.6 PLF	in							
16" dia. 1/2 " Wall	84.5 PLF	in							

Steel Density	500 PCF
E, steel	29000000 psi
Ωb	1.67

				ASCE 7-10
Site Class	D		Site class definitions (Soil type)	*Table 20.3-1 pg. 204*
	I		Occupancy Category	*Table 1.5-1 pg 2*
R =	1.25		Response Modification Factor	*Table 12.2-1 pgs. 73-76*
Seismic Force-Resist. System	Steel Ordinary Cantilever Column Systems			*Table 12.2-1 pgs. 73-76*
I =	1.00		Importance Factor	*Table 1.5-2 pg. 5*
Spectral Response Spectra:				
S_S =	140.7	%	Spectral Response Acceleration	*Figure 22-1 pg. 212*
S_1 =	49	%	Spectral Response Acceleration	*Figure 22-2 pg. 214*
F_a =	1.00		Site Coefficient Adjustment for S_s	*Table 11.4-1 pg. 66*
F_v =	1.50		Site Coefficient Adjustment for S_1	*Table 11.4-2 pg. 66*
TL =	6.00		Long Period	*Fig 22-12, pg. 224*
S_{MS} =	1.407		$S_{MS} = F_a*S_s$ Maximum Spectral Response Short Periods	*Section 11.4-1 pg. 65*
S_{M1} =	0.735		$S_{M1} = F_v*S_1$ Maximum Spectral Response 1 Sec. Periods	*Section 11.4-2 pg. 65*
S_{DS} =	0.938		Maximum Design Spectral Response Short Periods	*Section 11.4-3 pg. 65*
S_{D1} =	0.490		Maximum Design Spectral Response 1 Sec. Periods	*Section 11.4-4 pg. 65*
S_a =	0.938		Design response spectrum.	*Section 11.4-5 pg. 66*
	D		Design Category	*Table 11.6-1&2 pg. 67*
C_s =	0.750		$C_s = S_{DS}/(R/I)$	
C_{smin} =	0.041		$C_{smin} = 0.044*S_{DS}*I$	
C_{smax} =	1.293		$C_{smax} = S_{D1}/(T*(R/I))$	
Seismic Response coeff., C_s =	**0.750**			
Base Shear, **V** =	**C_s x (W)**			
Building Period:				
	OTHER		Structure Type	
	37.5	ft.	Structure Height	
T_a =	0.30	sec.	Fundamental period $T_a = C_t*(h_n)^x$	*Section 12.8.2.1 pg. 90*
T_0 =	0.10		$T_0 = 0.2*(S_{D1}/S_{DS})$	
T_S =	0.52		$T_S = S_{D1}/S_{DS}$	
C_t =	0.02			
x =	0.75			
Ω =	1.25			*Table 12.2-1 pgs. 73-77*
Cd =	1.25			*Table 12.2-1 pgs. 73-77*
ρ =	1.00		Redundancy factor	*Section 12.3.4, pg. 83*
Eh =	0.750	x DL	#REF!	*Section 12.4.2.1 pg. 84*
Ev =	0.188	x DL	#REF!	*Section 12.4.2.2 pg. 86*
Dead Load, W =	#REF!	kips		

Load Combinations

ASCE 7-10 Section 2.4.1

2	D + L	=	1.000	**D**	+	1.000	**L**				
4	D + 0.75L + 0.75S	=	1.000	**D**	+	0.750	**L**	+	0.750	**S**	

ASCE 7-10 Section 12.14.3.1

Cs	0.750
Sds	0.938
Ω	1.25

5	(1.0 + 0.14 Sds)D + 0.7Qe *Ω	=	1.131	**D**	+	0.875	**Qe**			
6b	(1.0 + 0.105 Sds)D + 0.525Qe*Ω + 0.75L	=	1.098	**D**	+	0.656	**Qe**	+	0.750	**L**

ASCE 7-10 Section 2.4

5	D + 0.6W + H	=	1.000	**D**	+	0.600	**W**	+	1.000	**H**

OSHPD

20088

5995 SE 30th St, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.5837898, -122.2519332

Date	6/3/2020, 1:40:18 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.407	MCE_R ground motion. (for 0.2 second period)
S_1	0.49	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.407	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.938	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.602	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.662	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.407	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.56	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.287	Factored deterministic acceleration value. (0.2 second)
S1RT	0.49	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.547	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.335	Factored deterministic acceleration value. (1.0 second)

Type	Value	Description
PGAd	1.132	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.896	Mapped value of the risk coefficient at a period of 1 s

Project No:______ Date:______ Sheet:____ Of:____

Project Name: PILE SPLICE

Comp. By: KJB   Chk.By:______

Contents:______

Pacific Engineering Technologies INC.

1700 Westlake Avenue North – Suite 100 Seattle, Washington 98109-6212
Telephone: [206] 281-7500 Facsimile: [206] 281-4611
[800] 621-7300

CHECK PILE SPLICE 10"ϕ PILE (12" ϕ SIMILAR BY INSPECTION)

WOOD PILE BENDING

10"ϕ

$S = \pi r^3/4 = \pi 5^3/4 = 98.2 \text{ in}^3$

$F_b = 1260\text{PSI}$ DF

$F_b S = 10.3\text{K}'$

12"ϕ

$S = 170 \text{ in}^3$

$F_b = 1260 \text{PSI}$

$F_b S = 17.8\text{K}'$ (73% INCREASE)

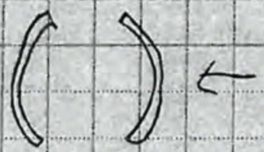
CHECK SPLICE TO HAVE EQUIVULENT BENDING STRENGTH AS WOOD

CHECK PARALLEL TO BOLTS : PLATE BENDING

$L_{PLATE} = 1/3$ CIRCUM $= 2\pi r/3 = 10.5"$

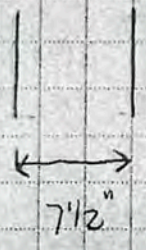
$t = 3/8"$

$d = 75\% \times \phi = 7.5"$



$I = .375 \times 10.5 \times (10.5/2)^2 \times 2 = 217 \text{ in}^4$

$S = I/y = 217/7.5/2 = 57.9 \text{ in}^3$



MODEL

$F_y S = 45\text{KSI} \times 57.9/1.67 = 1560\text{K}'' = 130\text{K}' > 10.3\text{K}'$

OK

$\therefore$ 3/8" PLATE SUFFICIENT

Project No: Date: Sheet: Of:
Project Name: PILE SPLICE
Comp. By: KJB Chk.By:
Contents:

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[800] 621-7300

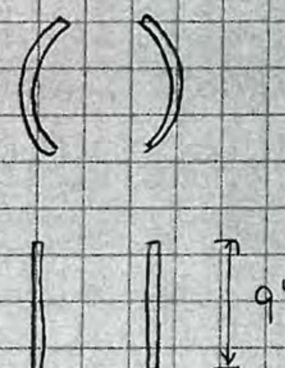
CHECK PILE SPLICE (CONT)

CHECK PERPENDICULAR TO BOLTS

$I = .375 \times 9^3/12 \times 2 = 45.6 \text{ in}^4$

$S = I/y = 45.6/(9/2) = 10.1 \text{ in}^3$

$Z = .375 \times 9^2/4 \times 2 = 15.2 \text{ in}^3$



$M = F_y Z = 45 \text{ KSI} \times 15.2 = 683 K'' = 56.9 K'$

$M/\Omega = 34 K' > 10.3 K'$ OK

SEE p2

Project No: ______ Date: ______ Sheet: ______ Of: ______

Project Name: PILE SPLICE

Comp. By: KJB Chk.By: ______

Contents: ______

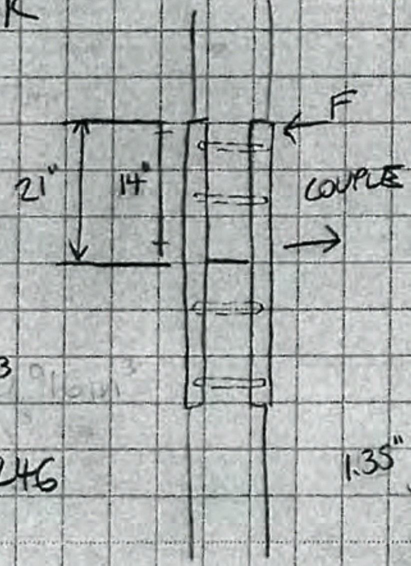
Pacific Engineering Technologies INC.
1700 Westlake Avenue North – Suite 100 Seattle, Washington 98109-6212
Telephone: [206] 281-7500 Facsimile: [206] 281-4611
[800] 621-7300

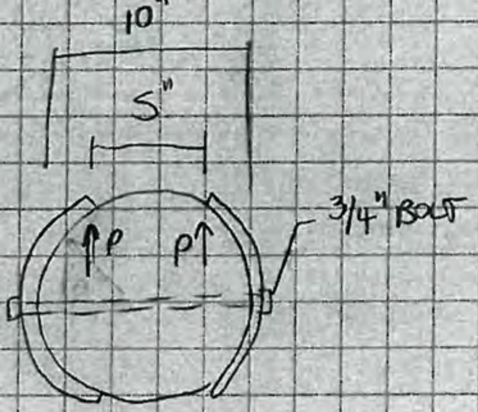
CHECK PILE SPLICE (CONT)

CHECK PERPENDICULAR TO BOLTS: PLATE LOCAL BENDING*

COUPLE FORCE FOR FULL STRENGTH WOOD
$= 10.3K' / (14/12) = 8.8K$

$M = 8.8 \times 1.35'' / 2$ SIDES
$= 5.9K''$

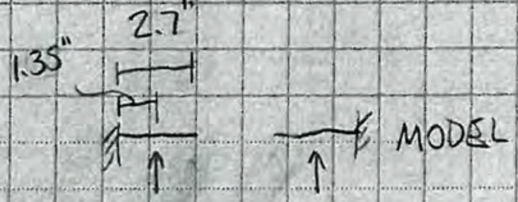




FLAT PLATE BENDING
AT ANGLE $\theta = 21°$

$Z = .375^2 \times 7/4 = 0.246 in^3$

$M_N = F_y Z = 45 KSI \times 0.246$
$= 11.1K''$



$M_N/\Omega = 6.6K'' > 5.9K''$ OK

*CHECK TO SEE
IF C-SHAPE WILL
PRY OPEN AT END

COUPLE FORCE ALSO RESISTED BY BOLTS

$Z_\perp = 1890^\#$
$C_M = 0.7$
$Z_\perp' = 1320^\#$

WOOD BEARING LENGTH
$F_{c\perp}' = 230$ PSI NDS '05

$M_N = 1320 \times 12 = 15.9K'' > 5.9K''$ OK

$L = 8.8/2/230/7'' = 2.7''$

$\therefore$ 3/4" BOLTS SUFFICIENT

Project No: 20088 Date: 6/22/20 Sheet: ____ Of: ____

Project Name: Rosem Pier

Comp. By: G. S. Chk.By: ____

Contents: Jet ski attachment

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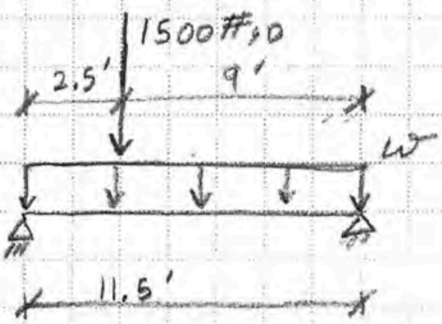
per waterfront, 'Boat lifts international' Products will be used.

per manufactures recomendation, the Framing should be designed for gravity loads only



$w = [60psf,_L + 8psf,_D] * 0.72ft = 43.2 PlF,_L + 5.8 PlF,_D$
$= 49 PlF$

$M = \frac{w * L^2}{8} + \frac{P*a*b}{L} = \frac{49 * 11.5^2}{8} + \frac{1500 * 2.5 * 9}{11.5} = 2935\#ft$

$\boxed{\text{P.T. 4X8 DF No.1 o.k., Refer to J2 calc.}}$

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<u>Supplementry Calculation Package 1</u>
<u>20088</u>
<u>Doug Rosen Residence – Revised Pier Framing Design</u>

5995 SE 30th Street
Mercer Island, Washington 98040

KYLE J. BOZICK
STATE OF WASHINGTON
41970
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

12/1/2020

The following calculations are revisions to the original calculation package titled 'Doug Rosen Residence – Pier & Pile Design' dated July 7th, 2020.

The engineering seal on these calculations are for the design of pier steel cap beams to match the existing height of the pier.

The calculations ahead replace the same sections of the original calculation package.

Design is in accordance with the 2015 International Building Code and 2015 International Existing Building Code.

The site information, dimensions, and plan layout, has been provided to us by Waterfront Construction, Inc.

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Project No: 20088 Date: NOV 20, 2020 Sheet: ____ Of: ____

Project Name: Rosen Pier, Steel Framing Rev.

Comp. By: G.S. Chk.By: ____

Contents: Scope, B1 & B6 design summary

Scope

Client wants to use steel cap beam framing to match the existing height of the pier.

Re design beams, B1, B6, & B7 connection

B1

clear span = 14.2'

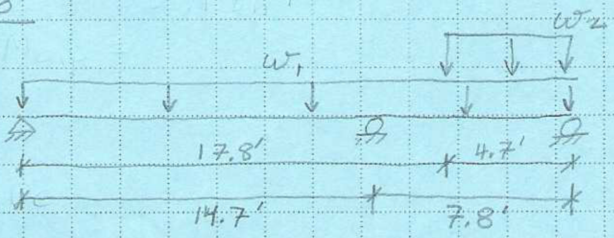
Demand:

$w = 375 \, plf_{,L} + 50 \, plf_{,D}$

$M = 10{,}712$ #ft

use W 6" x 16, Refer to calc attached

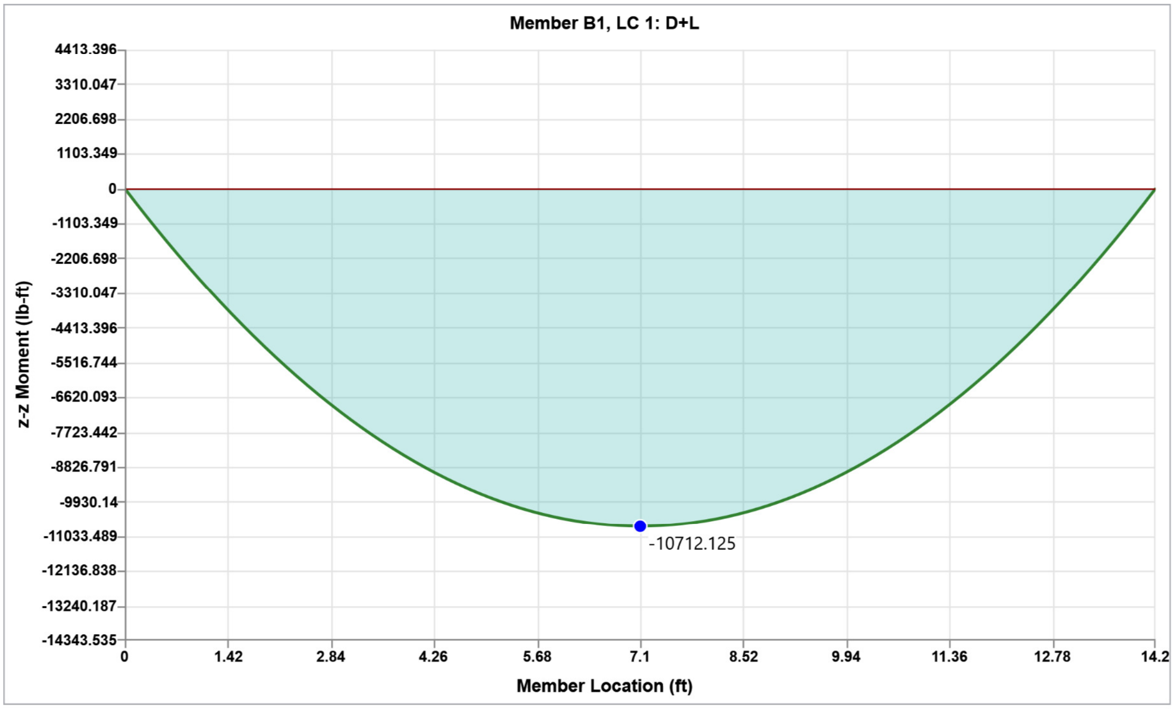
B6



$w_1 = 259 \, plf_{,L} + 34 \, plf_{,D}$

$w_2 = 408 \, plf_{,L} + 54 \, plf_{,D}$

Use MC6 x 15.3, Refer to calc attached


Member B1, LC 1: D+L
4413.396
3310.047
2206.698
1103.349
0
-1103.349
-2206.698
-3310.047
-4413.396
-5516.744
-6620.093
-7723.442
-8826.791
-9930.14
-11033.489
-12136.838
-13240.187
-14343.535
z-z Moment (lb-ft)
-10712.125
0
1.42
2.84
4.26
5.68
7.1
8.52
9.94
11.36
12.78
14.2
Member Location (ft)

Job Number:	20088		
Member I.D.:	B1		

AISC 360-10 - section F2
Double symmetric compact I-shaped members and channels bent about their major axis

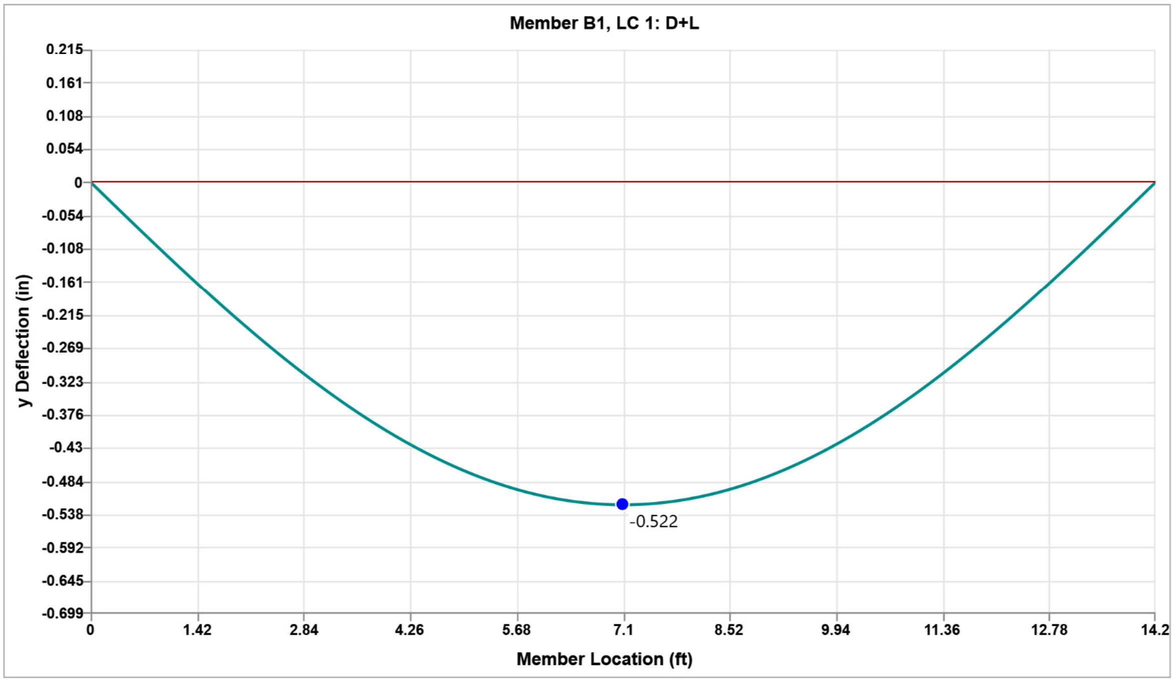
Member	W6X16		must be a compact member
Fy	50	ksi	
Type of Member	Doubly symmetric I-shapes		
E	29000	KSI	
M_max	10712.1	lb-ft	absolute value of maximun moment in the unbraced segment
M_A	8034.1	lb-ft	absolute value of moment at quarter point of the unbraced segment
M_B	10712.1	lb-ft	absolute value of moment at centerline of unbraced segment
M_C	8034.1	lb-ft	absolute value of moment at three-quarter point of the unbraced segment
Lb	170.4	in	length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section

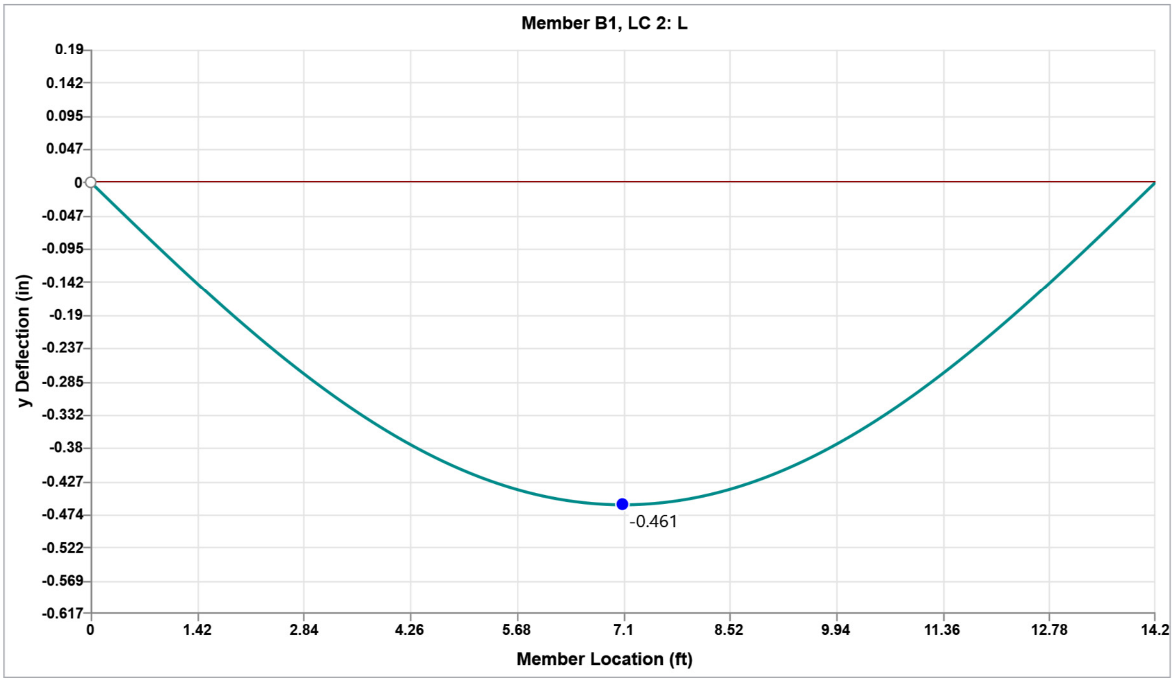
Mn/Ω_b	20056.7	lbs-ft	**Pass: M_capacity <= M_demand**

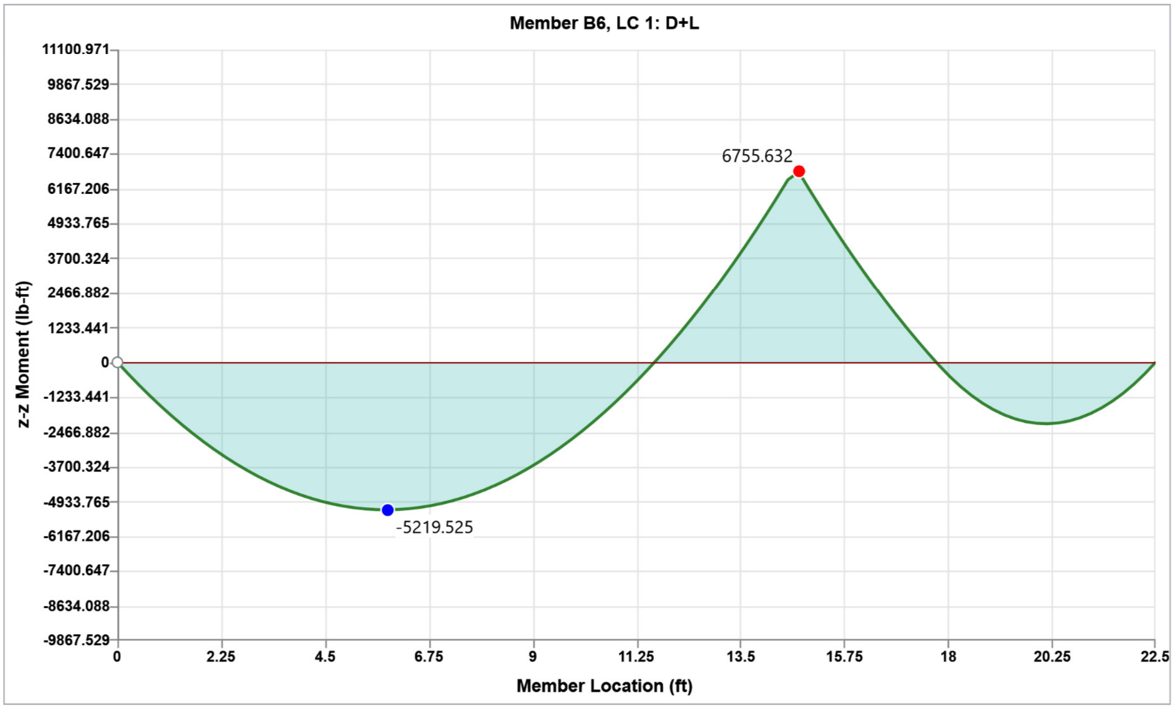
Lp	40.988	in
Lr	169.104	in
Cb	1.136	
Zx	11.700	in^3
ry	0.967	in
Cw	38.200	in^6
Iy	4.430	in^4
Sx	10.200	in^3
r_ts	1.129	in
d	6.280	in
tf	0.405	in
h_0	5.875	in
c	1.000	
J	0.223	in^4

c, Inelastic lateral torsional buckling coefficient	
	c
Doubly symmetric I-shapes	1
Channels	1.00

F_cr	39.41	ksi		
Ω_b	1.67			
Lb<=Lp				
Mn = Mp	585.0	K-in	48750.0	lbs-ft
Lp<Lb<=Lr				
Mn <= Mp	403.1	K-in		
min(Mn,Mp)	403.1	K-in	33588.4	lbs-ft
Lb>Lr				
Mn <= Mp	401.9	K-in		
min(Mn,Mp)	401.9	K-in	33494.6	lbs-ft
Mn	401.9	K-in	33494.6	lbs-ft
Mn/Ω_b	240.7	K-in	20056.7	lbs-ft


Member B1, LC 1: D+L
0.215
0.161
0.108
0.054
0
-0.054
-0.108
-0.161
-0.215
-0.269
-0.323
-0.376
-0.43
-0.484
-0.538
-0.592
-0.645
-0.699
y Deflection (in)
-0.522
0
1.42
2.84
4.26
5.68
7.1
8.52
9.94
11.36
12.78
14.2
Member Location (ft)


Member B1, LC 2: L
0.19
0.142
0.095
0.047
0
-0.047
-0.095
-0.142
-0.19
-0.237
-0.285
-0.332
-0.38
-0.427
-0.474
-0.522
-0.569
-0.617
y Deflection (in)
-0.461
0
1.42
2.84
4.26
5.68
7.1
8.52
9.94
11.36
12.78
14.2
Member Location (ft)


Member B6, LC 1: D+L
z-z Moment (lb-ft)
Member Location (ft)
11100.971
9867.529
8634.088
7400.647
6167.206
4933.765
3700.324
2466.882
1233.441
0
-1233.441
-2466.882
-3700.324
-4933.765
-6167.206
-7400.647
-8634.088
-9867.529
0
2.25
4.5
6.75
9
11.25
13.5
15.75
18
20.25
22.5
6755.632
-5219.525

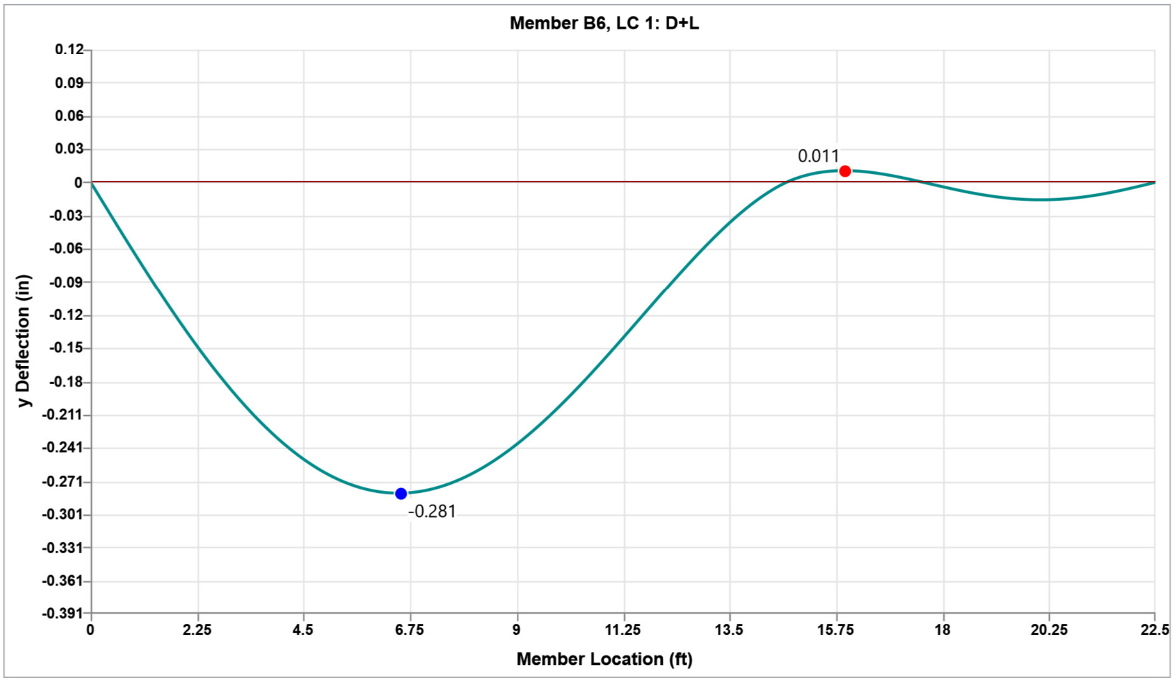
Job Number:	20088	
Member I.D.:	B6	

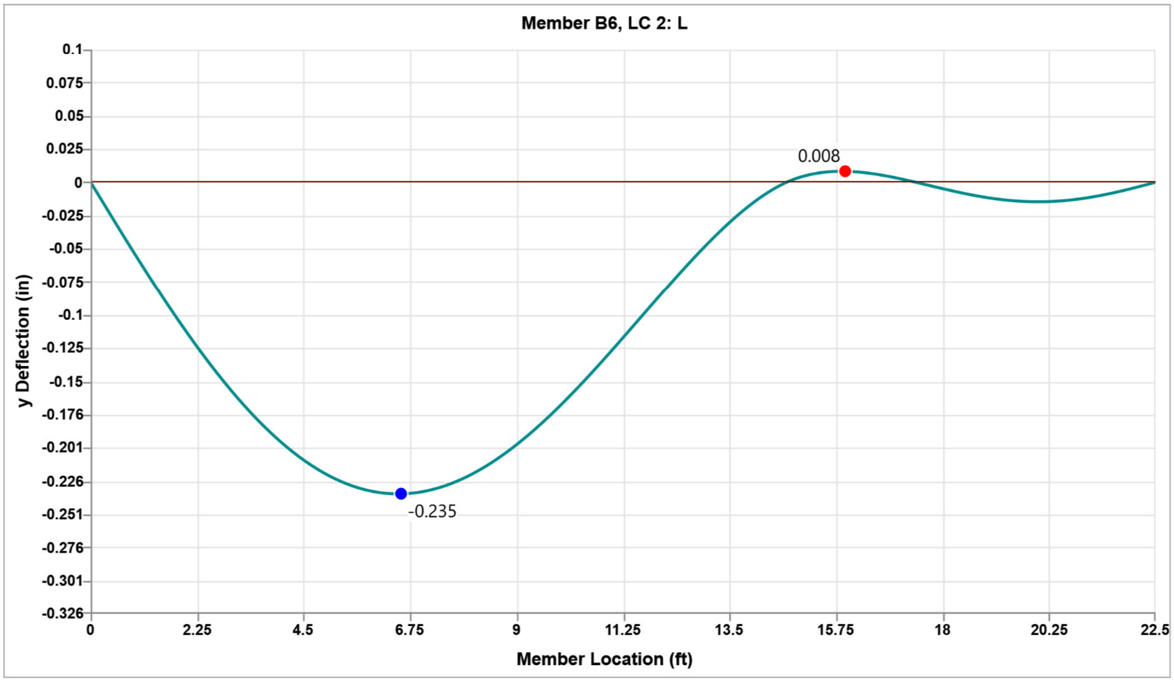
AISC 360-10 - section F2
Double symmetric compact I-shaped members and channels bent about their major axis

Member	MC6x15.3		must be a compact member
Fy	50	ksi	
Type of Member	Channels		
E	29000	KSI	
M_max	5704.4	lb-ft	absolute value of maximun moment in the unbraced segment
M_A	4368.7	lb-ft	absolute value of moment at quarter point of the unbraced segment
M_B	542.9	lb-ft	absolute value of moment at centerline of unbraced segment
M_C	1383.2	lb-ft	absolute value of moment at three-quarter point of the unbraced segment
Lb	270	in	length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section
Mn/Ω_b	23678.0	lbs-ft	**Pass: M_capacity <= M_demand**
Lp	44.506	in	
Lr	208.156	in	
Cb	2.117		
Zx	9.910	in^3	
ry	1.050	in	
Cw	30.000	in^6	
Ix	25.300	in^4	
Iy	4.910	in^4	
Sx	8.440	in^3	
r_ts	1.199	in	
h_0	5.620	in	
c	1.137		
J	0.223	in^4	

c, Inelastic lateral torsional buckling coefficient	
	c
Doubly symmetric I-shapes	1
Channels	1.14

F_cr	56.22	ksi		
Ω_b	1.67			
Lb<=Lp				
Mn = Mp	495.5	K-in	41291.7	lbs-ft
Lp<Lb<=Lr				
Mn <= Mp	465.2	K-in		
min(Mn,Mp)	465.2	K-in	38766.3	lbs-ft
Lb>Lr				
Mn <= Mp	474.5	K-in		
min(Mn,Mp)	474.5	K-in	39542.3	lbs-ft
Mn	474.5	K-in	39542.3	lbs-ft
Mn/Ω_b	284.1	K-in	23678.0	lbs-ft


Member B6, LC 1: D+L
0.12
0.09
0.06
0.03
0
-0.03
-0.06
-0.09
-0.12
-0.15
-0.18
-0.211
-0.241
-0.271
-0.301
-0.331
-0.361
-0.391
y Deflection (in)
0.011
-0.281
0
2.25
4.5
6.75
9
11.25
13.5
15.75
18
20.25
22.5
Member Location (ft)


Member B6, LC 2: L
y Deflection (in)
Member Location (ft)
0.1
0.075
0.05
0.025
0
-0.025
-0.05
-0.075
-0.1
-0.125
-0.15
-0.176
-0.201
-0.226
-0.251
-0.276
-0.301
-0.326
0
2.25
4.5
6.75
9
11.25
13.5
15.75
18
20.25
22.5
0.008
-0.235

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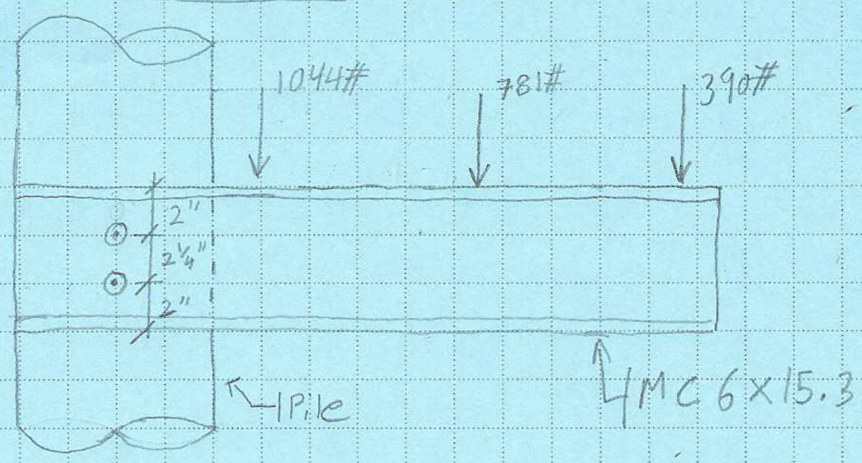
Project No: 20088 Date: NOV 20, 2020 Sheet: ____ Of: ____

Project Name: Rosen Pier, Steel Framing Rev

Comp. By: G.S. Chk.By: ____

Contents: B7 connection.

B7 connection



M = 2163# Ft

V = 2215#

$$\text{Shear load per bolt} = \sqrt{\left(\frac{2163\#\,\text{ft} * 12^{\text{in}/\text{ft}}}{2.25''}\right)^2 + \left(\frac{2215\#}{2}\right)^2}$$

= 11.6 Kip. per bolt

$\boxed{\text{Use (2) MC6 X15.3 on either side of Pile, connect with (2) 3/4"}\phi\text{ A307 Thru bolts as illustrated above}}$

AISC Table 7-1

capacity of (1) 3/4"ϕ A307 bolt in double shear $= \frac{r_n}{\Omega} = 11.9$ Kip

Refer to copy of table ahead

Table 7-1
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter, d, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
A307	–	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9

Nominal Bolt Diameter, d, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
A307	–	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9

ASD	LRFD	For end loaded connections greater than 38 in., see AISC *Specification* Table J3.2 footnote b.
$\Omega = 2.00$	$\phi = 0.75$	