**J-2 EQUIPMENT PLATFORM BRACING**

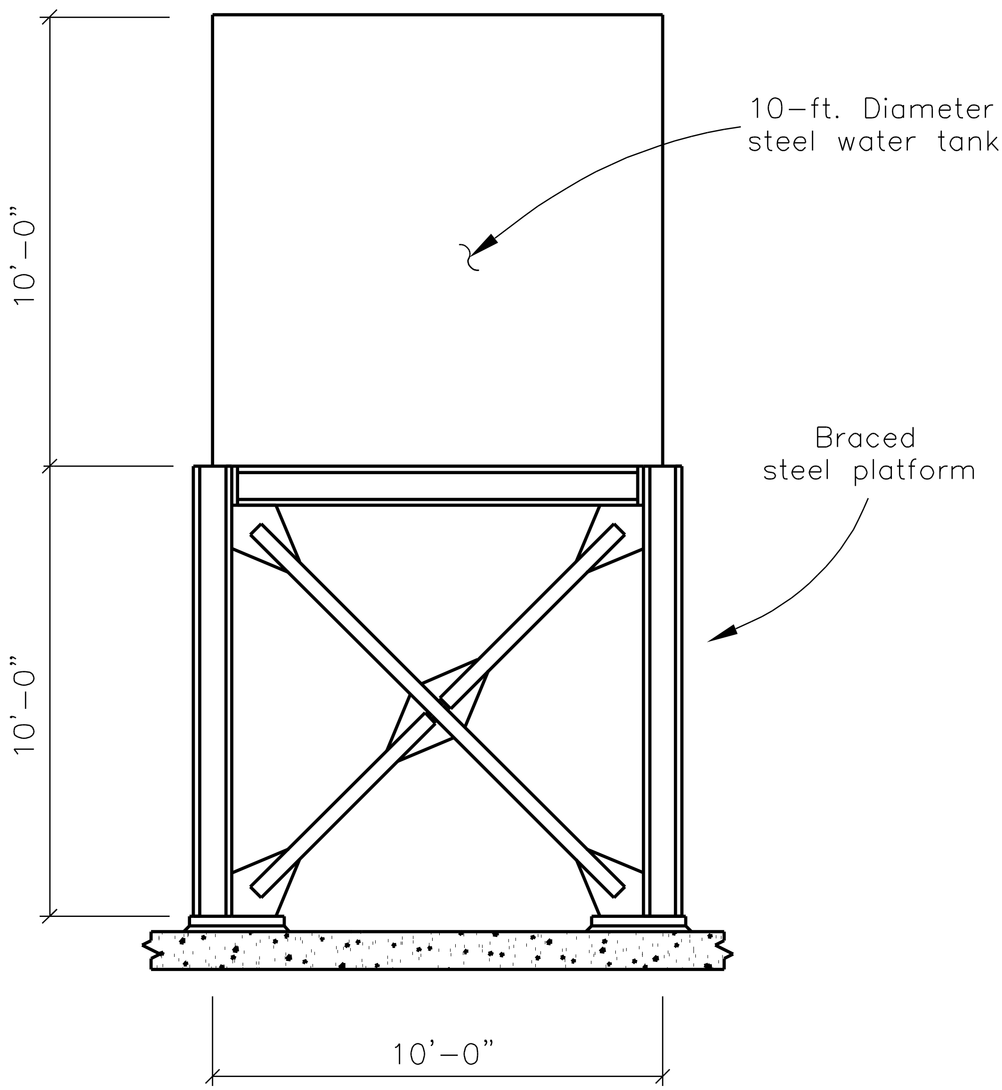
*a. Introduction.*

(1) Purpose. The purpose of this example problem is to illustrate the design of a braced steel platform supporting heavy equipment.

(2) Scope. The problem generally follows the steps in Table 4-5 and the procedures in Chapter 10 of this document and Chapter 6 of FEMA 302.

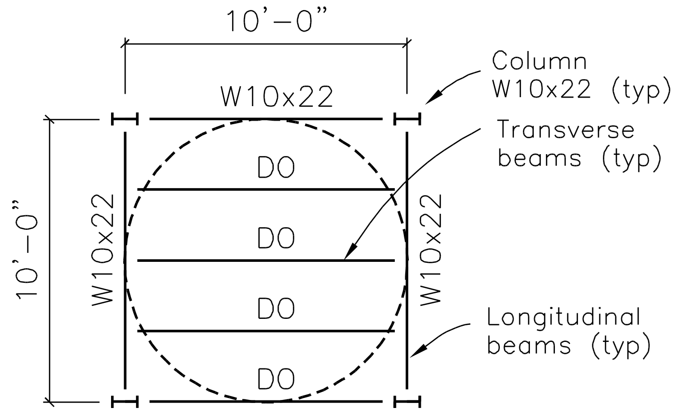
*b. Component description.*

The equipment in this example problem is an elevated steel water tank on a braced steel platform located on the roof of a two story building. An elevation of the tank and platform is shown in Figure J2-1 and a framing plan of the platform is shown in Figure J2-2.




Note: For metric equivalents; 1-ft = 0.30m

Figure J2-1. Elevation of tank and platform support structure




1-in = 25.4mm
1-ft = 0.30m
1plf = 14.58KN/m

Figure J2-2. Platform framing plan

*c. Component design.*

A.1 *Determine appropriate Seismic Use Group*

It is decided that the building supporting the tank be functional after an earthquake, therefore the tank is given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

| | | |
|---|---|---|
| Performance Level: | IO | (per problem statement) |
| Seismic Use Group: | IIIE | (Table 4-4) |
| Ground Motion: | 3/4 MCE (B) | (Table 4-4) |
| Performance Objective: | 3B | (Table 4-4) |

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 1.20g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.02$ (interpolated) (Table 3.2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_aS_S = 1.02(1.20)g = 1.22g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(1.22) = 0.92g$ (EQ. 3-3)

A.7 *Lateral load reisiting system.*

Steel tank to be welded to platform and platform to be laterally supported by cross bracing in both directions.

A.8 *Select $R_p$, $a_p$, and $I_p$ factors.*

$a_p = 2.5$ (Table 10-1)
$R_p = 2.5$ (Table 10-1)
$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

Determine structural weights;
Weight above platform;
Weight of water;
Note: Assume tank is normally full

$$V = \pi R^2 h = \pi(5')^2 10' = 785\text{-ft}^3 \quad (22.23\text{m}^3) \qquad \text{density of water} = 62.4\text{-lb/ft}^3 \quad (9800\text{KN/m}^3)$$

$$W_{water} = 785\text{-ft}^3(62.4^{\text{lbs/ft}^3})(1^k/1000\text{-lb}) = 49.0^k \quad (218.0\text{KN})$$

Weight of tank shell (1/4-in. (6.4mm) plate);

$$\text{Area} = \pi Dh = \pi(10')10' = 314.2\text{-ft}^2 \quad (29.19\text{m}^2)$$

$$w_{plate} = 0.25''(1'/12'')490\text{-lb/in}^3 = 10.2\text{psf} \quad (0.49\text{KN/m}^2)$$

$$W_{shell} = 314.2\text{-ft}^2(10.2\text{psf})(1^k/1000\text{-lb}) = 3.2^k \quad (14.2\text{KN})$$

Weight of tank top and bottom (3/8-in. (9.53mm) plate);

$$\text{Area} = 2\pi r^2 = 2\pi(5')^2 = 157\text{-ft}^2 \quad (14.59\text{m}^2)$$

$$w_{plate} = 0.375''(1'/12'')490\text{-lb/in}^3 = 15.3\text{psf} \quad (0.73\text{KN/m}^2)$$

$$W_{top\&bot} = 157\text{-ft}^2(15.3\text{psf})(1^k/1000\text{-lb}) = 2.4^k \quad (10.7\text{KN})$$

Therefore, the weight above the platform is;

$$W_{p1} = W_{water} + W_{shell} + W_{top\&bot} = 49.0^k + 3.2^k + 2.4^k = 54.6^k \quad (242.9\text{KN})$$

Weight of platform;
Weight of platform beams (conservatively assume beam weight at 30psf over platform plan area);

$$W_{beams} = 30\text{psf}(10')10'(1^k/1000\text{-lb}) = 3.0^k \quad (13.3\text{KN})$$

Weight of legs and braces (assume legs and braces at 10psf projected horizontally);
Note: Tributary height of 5-ft. (1.53m) is taken in lumping load at platform level

$$W_{legs\&braces} = 4\text{sides}(10\text{psf})5'(10')(1^k/1000\text{-lb}) = 2^k \quad (8.9\text{KN})$$

Therefore, the weight to lump at the platform level is;

$$W_{p2} = W_{beams} + W_{legs\&braces} = 3^k + 2^k = 5^k \quad (22.2\text{KN})$$

Design transverse platform beams (see Figure J2-2, and J2-3);
Note: It is conservatively assumed that the middle transverse beam supports half of $W_{p1}$ with the distribution as shown in Figure J2-3. One design will be made for this beam and used throughout for all transverse beams.

Using load combination U = 1.4D;

$$W_u = 1.4(0.5(54.6^k)) = 38.2^k \quad (169.9\text{KN})$$

$$M_u = \frac{W_u L}{6} = \frac{38.2^k(10')(12''/1')}{6} = 764^{\text{in-k}} \quad (86.3\text{KN})$$

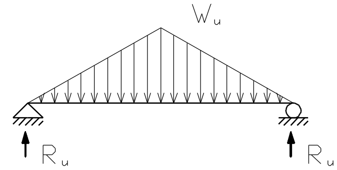



Figure J2-3. Loading on middle transverse beam

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{\phi F_y} = \frac{764^{in-k}}{0.9(36ksi)} = 23.6-in^3 < 26.0\text{-}in^3 = Z_{W10x22} \quad (386.7X10^3\ mm^3 < 426.1X10^3\ mm^3)$$

**Use W10x22 (254mmX1.05KN/m) for transverse beams**

Design longitudinal platform beams (see Figures J2-2, and J2-4);

$P_{1u} = (1/2)38.2^k = 19.1^k$ (89.96KN)

$P_{2u} = (1/2)P_{1u} = 9.6^k$ (42.70KN)

Therefore, reactions are; $R_u = 19.1^k$ (89.96KN)

The maximum moment occurs at mid span as;

$M_u = [19.1^k(5') – 9.6^k(2.5')](12"/1') = 858^{in\text{-}k}$ (96.95KN-m)

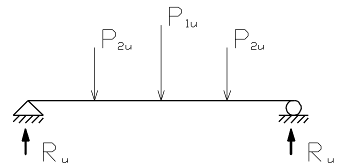



Figure J2-4. Loading on longitudinal beams

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{\phi F_y} = \frac{858^{in-k}}{0.9(36ksi)} = 26.5-in^3 > 26.0 = Z_{W10x22} \quad (434.3X10^3\ mm^3 > 426.1X10^3\ mm^3)$$

(okay because of conservative assumptions)

**Use W10x22 (254mmX1.05KN/m) for longitudinal beams**

Design columns;

Using load combination U = 1.4D;

$$P_u\ /\ column = 1.4(W_{p1} + W_{p2})/\#columns = 1.4(54.6^k + 5^k)/4 = 20.9^{k/column} \quad (93.0KN/column)$$

Try W10x22;

Relevant properties of a W10x22 are as follows;

$A = 6.49\text{-}in^2$ ($4.19X10^3\ mm^2$) $r_y = 1.33$-in (33.8mm) $r_x = 4.27$-in (108.5mm)

Check capacity of W10x22;

Note: Elements are in a braced frame with K = 1.0

Since $KL_y = KL_x$;

$$\frac{KL_y}{r_y} = \frac{1.0(10')(12"/1')}{1.33"} = 90.2$$

From AISC LRFD Table 3-36; $\phi_c F_{cr} = 19.94ksi$ (137.5MPa) (interpolated)

$$\phi_c P_n = \phi_c F_{cr} A_g = 19.94ksi(6.49-in^2) = 129.4^k \quad (575.6KN)$$

$$\phi_c P_n = 129.4^k > 20.9^k = P_u \quad (575.6KN > 93.0KN)$$

**Use W10x22 (W254mmX1.05KN/m) for columns**

Design transverse beam to longitudinal beam connection (see Figures J2-2, and J2-5);

Use; $F_y$ = 36ksi (248.2MPa), and $F_u$ = 58ksi (399.9MPa)

Note: It was previously determined that the middle transverse beam supports half of $W_{p1}$ with the distribution as shown in Figure J2-3. Therefore, the reaction can be taken as $R_u = (1/2)38.2^k = 19.1^k$ (89.96KN). One design will be made for this beam and used throughout for all transverse beam connections. Formulas are taken from Part 9 of AISC LRFD volume II, 2nd edition.

Relevant properties of a W10x22 are as follows;

$t_w$ = 0.240-in (6.1mm) $b_f$ = 5.750-in (146.1mm)

$t_f$ = 0.360-in (9.1mm) d = 10.17-in (258.3mm)

Determine coping dimensions c and $d_c$;

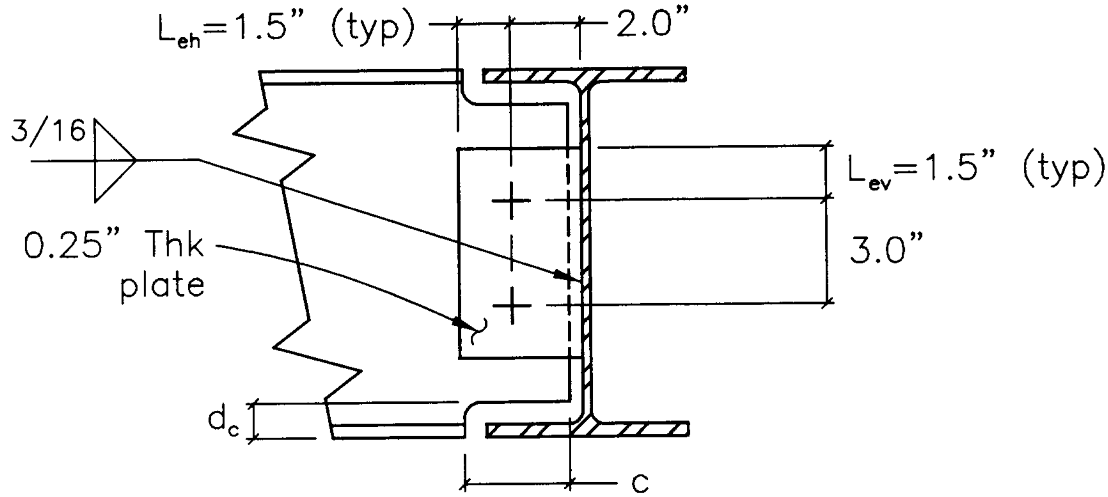
$$c = \frac{5.750"-0.240"}{2} = 2.76-in \quad (70.1mm) \qquad \text{Say } c = 3.0\text{-in} \quad (76.2mm)$$

$d_c$ = 0.360" + 0.500" = 0.86" (21.8mm) Say $d_c$ = 1.0-in (25.4mm)

Check flexural yielding of coped section assuming two 7/8-in. (22.2mm) $\phi$ bolts;

$$R_u \le \frac{\phi_b M_n}{e} \qquad \text{where; } \phi = 0.9$$

$$M_n = F_y S_{net}$$




Note: For metric equivalents; 1-in = 25.4mm
Figure J2-5. Transverse beam connection, elevation view

$$S_{net} = \frac{1}{12}(0.240")(10.17"-2(1.0"))^3\left(\frac{2}{(10.17"-2(1.0"))}\right) = 2.67-\text{in}^3 \quad (43.8X10^3 \text{ mm}^3)$$

$$\therefore M_n = 36\text{ksi}(2.67-\text{in}^3) = 96.1^{\text{in-k}} \quad (10.86\text{KN-m})$$

$$e = c + 0.5" = 3.5" \quad (88.9\text{mm})$$

$$\frac{\phi_b M_n}{e} = \frac{0.9(96.1^{\text{in-k}})}{3.5"} = 24.7^k > 19.1^k = R_u \quad (109.9\text{KN} > 85.0\text{KN})$$ **O.K.**

Check local web buckling of coped section;

$$R_u \le \frac{\phi_b M_n}{e} \quad \text{where;} \quad \phi = 0.9$$

$$M_n = F_{bc} S_{net}$$

$$F_{bc} = 0.62\pi E \frac{t_w^2}{ch_0} f_d$$

$$f_d = 3.5 - 7.5\left(\frac{d_c}{d}\right) = 3.5 - 7.5\left(\frac{1.0"}{10.17"}\right) = 2.76$$

$$h_0 = 10.17" - 2(1") = 8.17" \quad (207.5\text{mm})$$

$$\therefore F_{bc} = \frac{0.62\pi(29{,}000\text{ksi})(0.240")^2}{3.0"(8.17")}(2.76") = 366\text{ksi} \quad (2.52X10^3 \text{ MPa})$$

$$\therefore M_n = 366\text{ksi}(2.67-\text{in}^3) = 977^{\text{in-k}} \quad (110.4\text{KN-m})$$

$$\frac{\phi_b M_n}{e} = \frac{0.9(977^{\text{in-k}})}{3.5"} = 251^k > 19.1^k = R_u \quad (1.12\text{MN} > 0.085\text{MN})$$ **O.K.**

Design bolts, single plate, and welds;
Note: The tank being welded along the top of the beam prevents the beam from rotating thereby making the support for the connection relatively rigid.
From Table 9-10 of volume 2 of AISC LRFD 2nd edition, assuming the girder provides a rigid support, for 7/8-in. (22.2mm) diameter A325-N bolts and single plate material with $F_y$ = 36ksi (248.2MPa) and $F_u$ = 58ksi (400MPa), select two rows of bolts, 1/4-in. (6.4mm) single plate thickness, and 3/16-in. (4.8mm) fillet weld size.

$$\phi R_n = 25.5^k > 19.1^k = R_u \quad (113.4\text{KN} > 85\text{KN})$$ **O.K.**

Check supported beam web;
From table 9-2 of volume 2 of AISC LRFD 2nd edition, for two rows of 7/8-in. (22.2mm) diameter bolts, beam material with $F_y$ = 36ksi (248.2MPa) and $F_u$ = 58ksi (400MPa), and $L_{ev}$ = 1-1/2-in. (38.1mm) minimum and $L_{eh}$ = 1-1/2-in. (38.1mm) minimum;

$$\phi R_n = 104^{k/in}(0.240") = 25^k > 19.1^k = R_u \quad (11.2KN > 85.0KN)$$

**O.K.**

**Use two 7/8-in. (22.2mm) ϕ bolts with 1/4-in. (6.4mm) single plate and 3/16-in. (4.8mm) welds as shown in Figure J2-5**

Design longitudinal beam to column connection (see Figure J2-2);
By inspection, since the end reactions are the same, and the connection to a column is also a rigid connection, the same single plate connection design connecting the transverse beams to the longitudinal beams can be used for this connection.

**Use two 7/8-in. (22.2mm) ϕ bolts with 1/4-in. (6.4mm)single plate and 3/16-in. (4.8mm) welds as shown in Figure J2-5**

Note: The following steps do not have a one to one correspondence to steps listed in table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces ($F_p$) shall be determined in accordance with chapter 10 as follows:

$$F_p = \frac{0.4a_pS_{DS}W_p}{R_p/I_p}\left(1+2\frac{z}{h}\right) \qquad \text{(EQ. 10-1)}$$

where; z/h = 1.0 (Equipment attached at roof of building)

$F_p$ is not required to be taken greater than:

$$F_p = 1.6S_{DS}I_pW_p \qquad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3S_{DS}I_pW_p \qquad \text{(EQ. 10-3)}$$

Seismic load at the tank;

$$\therefore F_{p1} = \frac{0.4(2.5)0.92(54.6^k)}{2.5/1.5}(1+2(1)) = 1.66(54.6^k) = 90.4^k \quad (402.1KN)$$

$$(F_p)_{max} = 1.6(0.92)1.5(54.6^k) = 121^k > 90.4^k = F_p \quad (538.2KN > 402.1KN)$$ **O.K.**

$$(F_p)_{min} = 0.3(0.92)1.5(54.6^k) = 22.6^k < 90.4^k = F_p \quad (100.5KN < 402.1KN)$$ **O.K.**

Seismic load at the platform level;

$$\therefore F_{p2} = \frac{0.4(2.5)0.92(5.0^k)}{2.5/1.5}(1+2(1)) = 1.66(5.0^k) = 8.3^k \quad (36.9KN)$$

$$(F_p)_{max} = 1.6(0.92)1.5(5.0^k) = 11^k > 8.3^k = F_p \quad (48.9KN > 36.9KN)$$ **O.K.**

$$(F_p)_{min} = 0.3(0.92)1.5(5.0^k) = 2.1^k < 8.3^k = F_p \quad (9.3KN < 36.9KN)$$ **O.K.**

F.2 *Design members.*

Design tank connections to support frame (see Figure J2-6);
Connections, to resist overturning and shear, will consist of welds at four opposite faces of the tank and located at the mid center of the perimeter beams. Loading is eccentric to the plane of the welds.

Try 3/16-in. (4.8mm) welds;
Capacity for a 3/16-in. (4.8mm) weld;

$$\phi R_n = 0.75 t_e (0.6 F_{exx})$$

$$\text{where;} \quad t_e = \frac{3''}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133-\text{in} \quad (3.38\text{mm})$$

$$F_{exx} = 70\text{ksi} \quad (482.7\text{MPa}) \quad (\text{E70XX electrodes})$$

$$\therefore \phi R_n = 0.75(0.133'')(0.6(70\text{ksi})) = 4.19^{k/in} \quad (0.73\text{KN/mm})$$

By inspection, load combination 'U = 0.9D + E' governs, and the overturning moment is calculated as;

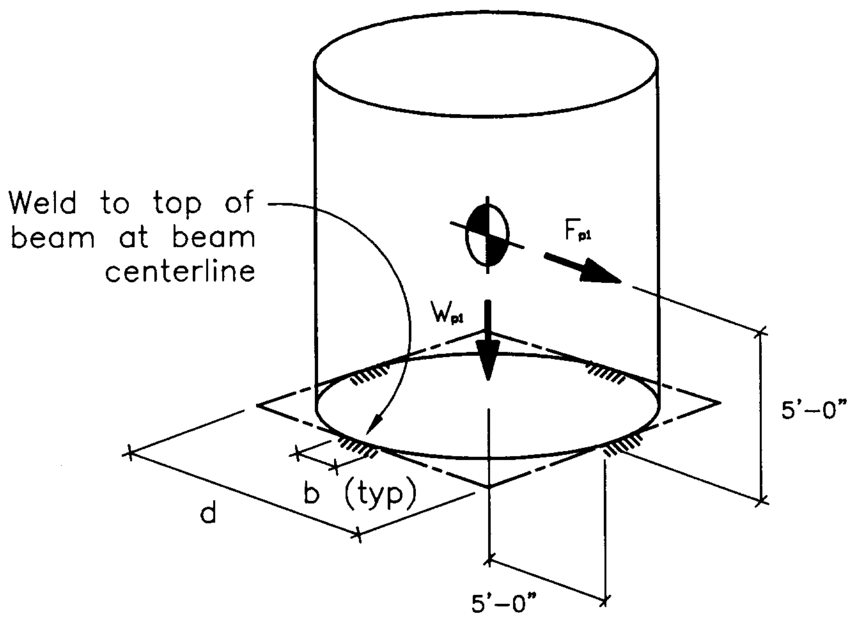
$$\therefore M_u = \left[90.4^k(5') - 0.9(54.6^k)5'\right](12''/1') = 2{,}476^{in-k} \quad (433.6\text{KN/mm})$$

The section modulus for the weld group is calculated as follows;

$$S_{welds} = (I_o + Ad^2)/(d/2)$$

$$S_{welds} \approx 2x\left[(1'')b\left(\frac{d}{2}\right)^2 + \frac{1}{12}(1'')b^3\right] \div \left(\frac{d}{2}\right) = bd + \frac{b^3}{3d}$$

Try b = 8-in. (203.2mm) (note that d = 120-in. (3.05m))




Note: For metric equivalents; 1-ft = 0.30m
Figure J2-6. Force diagram for tank to platform welds

$$\therefore S_{welds} = 8''(120'') + \frac{(8'')^3}{3(120'')} = 961-\text{in}^2 \quad (620.0\text{X}10^3\ \text{mm}^2)$$

$$f_s = \frac{P_u}{A} = \frac{90.4^k}{4(8'')} = 2.83^{k/in} \quad (0.50\text{KN/mm})$$

$$f_b = \frac{M_u}{S_{welds}} = \frac{2{,}476^{in/k}}{961-\text{in}^2} = 2.58^{k/in} \quad (0.45\text{KN/mm})$$

$$R_u = f_r = \sqrt{f_s^2 + f_b^2} = \sqrt{(2.83^{k/in})^2 + (2.58^{k/in})^2} = 3.83^{k/in} < 4.19^{k/in}$$

(0.67KN/mm < 0.73KN/mm) **O.K.**

**Use four 3/16-in. (4.8mm) welds to secure tank to perimeter beams**

Check transverse perimeter beams;

The transverse perimeter beams support relatively little of the tanks dead weight, but resist the overturning reaction. This reaction was previously calculated to be $P_u = 20.7^k$ (92.1KN). These beams were sized in the gravity load design as W10x22's and will be checked here. It is conservatively assumed that the beams support only the overturning seismic reaction.

The maximum moment occurs at mid span as;

$$M_u = \frac{P_u L}{4} = \frac{20.7^k(10')(12''/1)}{4} = 621^{in-k} \quad (70.17\text{KN-m})$$

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{fF_y} = \frac{621^{in-k}}{0.9(36ksi)} = 19.2 - in^3 < 26.6 = Z_{W10x22}$$

$(314.6X10^3\ mm^3 < 435.9X10^3\ mm^3)$ **O.K.**

**Keep W10x22 (W254mmX1.05KN/m) for transverse beams**

Check longitudinal beams;

The longitudinal beams support all of the tanks dead weight (transferred to it from the interior perimeter beams), and also resist the overturning reaction of $P_u = 20.7^k$ (92.1KN). These beams were sized in the gravity load design as W10x22's and will be checked here.

Per load combination 'U = 0.9D + E' the center load $P_{1u}$ in figure J2-4 is reduced to an uplift load of;

$20.7^k - 19.1^k = 1.6^k$ (7.12KN) (uplift)

By inspection, this reduces the end reactions and the maximum moment acting within the beam. Therefore, the W10x22 is still adequate.

**Keep W10x22 for longitudinal beams**

Design transverse beam to column connection;

The worst case beam reaction is $R_u/2 = 20.7^k/2 = 10.4^k$ (46.3KN). By inspection, the same single plate connection used for the other beam to beam or beam to column connections is adequate.

**Use two 7/8-in. (22.2mm) f A325-N bolts with 1/4-in. (6.4mm) single plate and 3/16-in. (4.8mm) welds similar to Figure J2-5**

Check column for combined loading (see Figure J2-7);

Determine design loads;

Calculate reactions;

From symmetry;

$R_{1H} = R_{2H} = (1/2)(45.2^k + 4.15^k) = 24.7^k$ (109.9KN)

$$\sum M_2 = 0;$$

$$(R_{1V})10' - 45.2^k(15') - 4.15^k(10') = 0$$

$R_{1V} = 72^k$ (320.3KN) (tension)

$$\sum F_y = 0;$$

$R_{2V} = 72^k$ (compression)

Calculate compressive force in column;

Summation of loads at point 2;

Due to the 45 degree inclination of the brace;

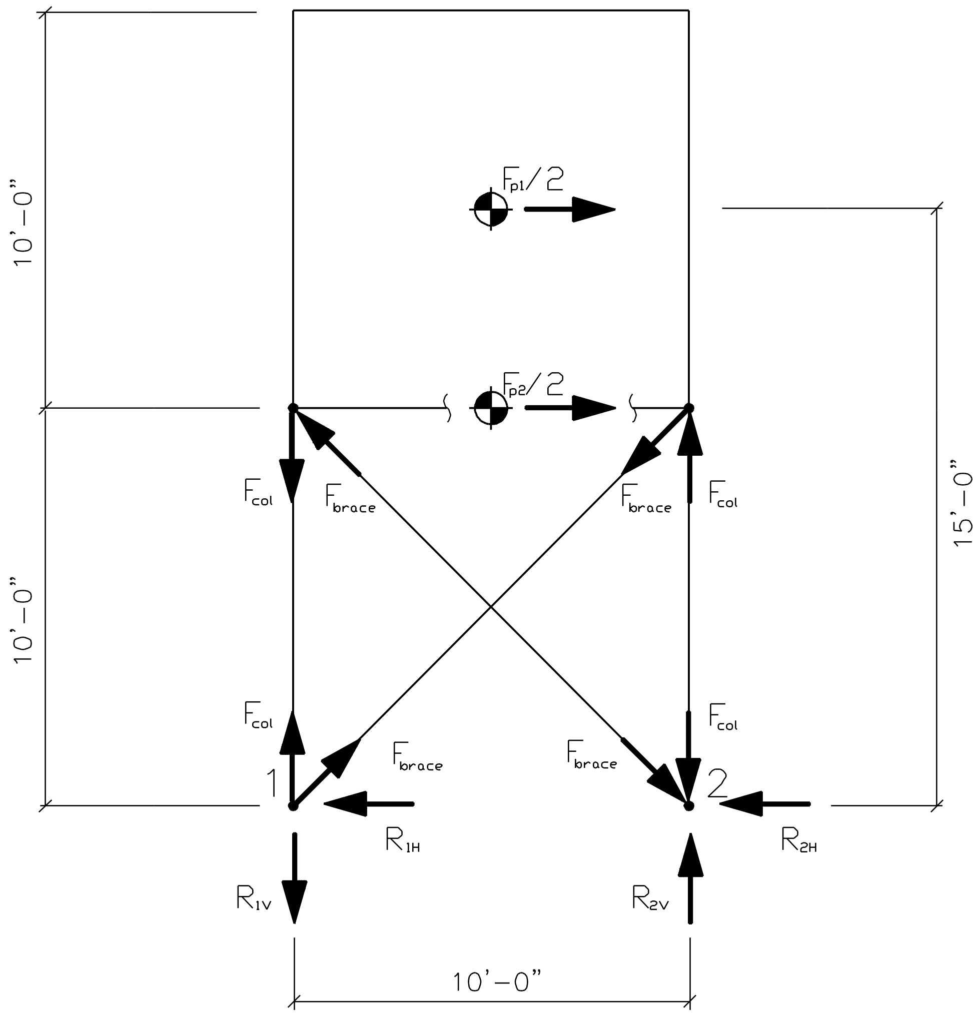
$(F_{brace})_{horz} = (F_{brace})_{vert} = 24.7^k$ (109.9KN)

$$\sum F_y = 0;$$

$$(F_{col})_{vert} = R_{2V} - (F_{brace})_{vert} = 72^k - 24.7^k = 47.3^k \quad (210.4\text{KN})$$

Superimposing the dead load;

From load combination 'U = 1.2D + E';




Note: For metric equivalents; 1-ft = 0.30

Figure J2-7. Seismic force diagram for supporting legs and braces

Dead load = $(W_{p1} + W_{p2})/4 = (54.6^k + 5^k)/4 = 14.9^k$ (66.3KN)

$P_u = 1.2(14.9^k) + 47.3^k = 65.2^k$ (290.0KN)

Check column capacity;

$$f_c P_n = f_c F_{cr} A_g = 129.4^k \quad \text{(575.6KN) (calculated previously)}$$

$$f_c P_n = 129.4^k > 65.2^k = P_u \quad \text{(575.6KN > 290.0KN)}$$ **O.K.**

Design brace;

Determine deign loads (see Figure J2-7);

Note: Brace does not support gravity loads. Therefore, all load combinations reduce to $P_u = E$.

$$F_{brace} = \pm\sqrt{(F_{brace})^2_{horz} + (F_{brace})^2_{vert}} = \pm\sqrt{2(24.7^k)^2} = \pm 34.9^k \quad \text{(155.2KN)}$$

Compression;

A doubly symmetric section will be choosen. Therefore, per Figure 7-21, the out-of-plane buckling will govern capacity. Assume K = 0.67;

$$KL_b = 0.67(10')\sqrt{2} = 9.5' \quad (2.90m)$$

Try a 3-in. (76.2mm) ϕ standard weight pipe;
From the columns tables in Part 3 of the AISC LRFD manual;

$$\phi_c P_n = 41^k > 34.9^k = P_u \quad (182.4KN > 155.2KN)$$ **O.K.**

Tension;
Okay by inspection.

**Use 3-in. (76.2mm) std wt pipe for braces**

Design brace connection to column and beam (see Figure J2-8);
One design for the worst case situation will be used throughout the structure. Worst case occurs where the gusset plate attaches to the column web (this condition has the smallest buckling capacity due to the increased length of plate). Per the AISC Seismic provisions (dated April 15, 1997) section 13.2b, the required compressive strength is $\phi_c P_n$, and per section 13.3.a, the required tensile strength is $R_y F y A_g$.

Therefore;
Compressive design load = $\phi_c P_n = 41^k$ (182.4KN)
Tensile design load = $R_y F_y A_g = 1.5(46ksi)2.23\text{-in}^2 = 154^k$ (685.0KN)

Try a 3/8-in. (9.5mm) thick gusset plate ($F_y$ = 36ksi (248.2MPa));
Design welds to gusset plate using E70XX electrodes
Minimum weld size = 3/16-in. (4.8mm) (per AISC LRFD 2nd edition Table J2.4, based on gusset plate thickness)
Maximum weld size = 0.216-in. (5.49mm) (per AISC LRFD 2nd edition section J2.2b.(b), based on brace thickness)
Required length of four 3/16-in. fillet welds;

$$L_w = \frac{R_u}{\phi R_n} = \frac{154^k / 4-\text{sides}}{4.19^{k/in}} = 9.19'' \quad \text{Say } 9.25'' \ (235.0mm)$$

Check base metal;

$$\phi R_n = \phi(0.6F_y)A_g = 0.9(0.6(36^{ksi}))0.216''(9.25'')4 = 155^k \quad (689.4KN)$$

$$\phi R_n = 155^k > 154^k = P_u \quad (689.4KN > 685.0KN)$$ **O.K.**

Check shear tension rupture (refer to Figure J2-9 for nomenclature);

$$\phi R_n = \phi t_{brace}\left[0.6F_y\left(\frac{2l_w}{\cos 30^\circ}\right) + F_u w_l\right]$$

$$\text{where;} \quad w_l = \phi_{brace} + 2l_w(\tan 30^\circ) = 3.5''+2(9.25'')0.577 = 14.2'' \quad (360.7mm)$$

$$\therefore \phi R_n = 0.75(0.375'')\left[0.6(36ksi)\left(\frac{2(9.25'')}{0.866}\right) + 58ksi(14.2'')\right] = 361^k > 154^k = P_u$$

(1.61MN > 0.68MN) **O.K.**

or, $\phi R_n = \phi t(0.6F_u(2l_w) + F_y w_b)$

$$\phi R_n = 0.75(0.375'')(0.6(58ksi)(2(9.25'') + 36ksi(3.5'')) = 217^k > 154^k = P_u$$

(0.97MN > 0.68MN) **O.K.**

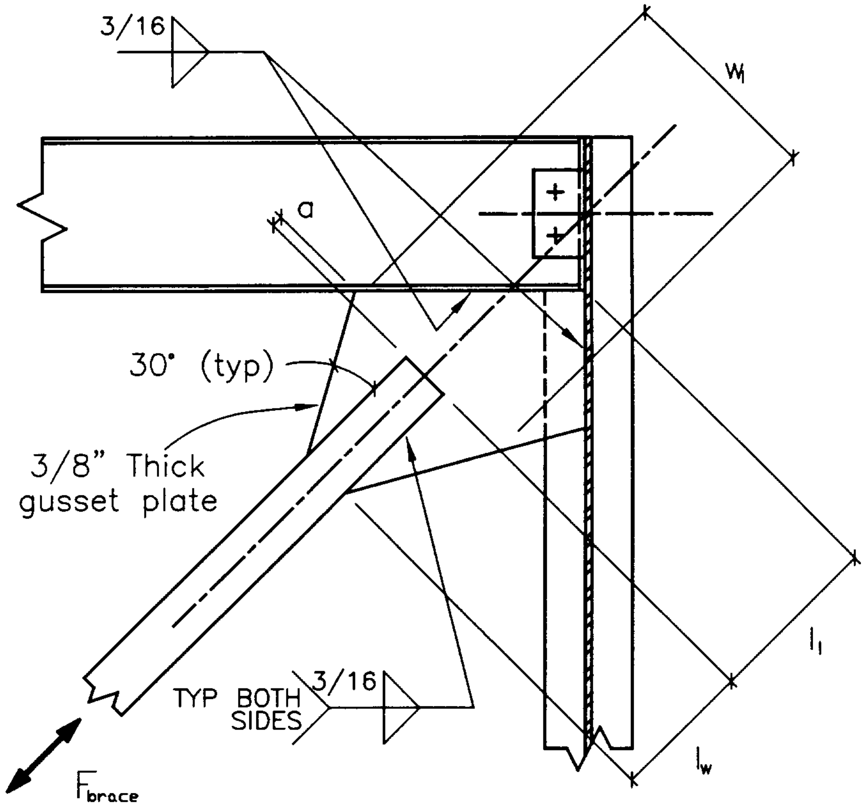
Check tensile capacity;

$$\phi R_n = \phi w_l F_u t = 0.75(14.2'')58ksi(0.375'') = 232^k > 154^k = P_u$$

(1.03KN > 0.68KN) **O.K.**

Check compressive capacity;

Note: The critical buckling strength for the brace is $A_g F_{cr} = \frac{\phi_c P_n}{\phi_c} = \frac{41^k}{0.85} = 48.2^k$ (214.4KN)




Note: For metric equivalents; 1-in = 25.4mm
Figure J2-8. Brace connection details and nomenclature

$$\phi R_n = \phi F_y w_l t = 0.9(36\text{ksi})14.2"(0.375") = 173^k > 48.2^k = A_g F_{cr}$$

(769.5KN > 214.4KN) **O.K.**

Check buckling of gusset plate;

$$\phi R_n = \phi\left(\frac{4{,}000t^3\sqrt{F_y}}{l_l}\right) \geq A_g F_{cr}$$

where; $l_l = \frac{w_l}{2} + a(\tan 30°) + \frac{d_b}{2}(\sin 45°)$ (for this bracing configuration only)

$a = 2t = 2(0.375") = 0.75"$ (19.1mm)

$$l_l = \frac{14.2"}{2} + 0.75"(0.577) + \frac{10.17"}{2}(0.707) = 11.1" \quad (281.9\text{mm})$$

$$\phi R_n = 0.90\left(\frac{4{,}000(0.375")^3\sqrt{36\text{ksi}}}{11.1"}\right) = 103^k > 48.2^k = A_g F_{cr}$$

(458.1KN > 214.4KN) **O.K.**

Design weld of gusset plate to beam and column;

Note: Length of weld to be along the entire length of contact between gusset plate and beam or column;

$(F_{brace})_{horz} = (F_{brace})_{vert} = 24.7^k$ (109.9KN)

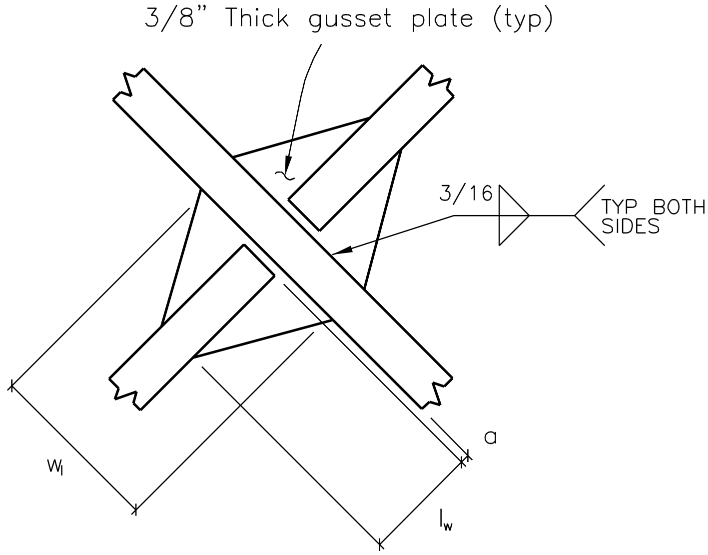
$L_{provided} \geq (w_l + 2a(\tan 30°))\cos 45°$ (for this bracing configuration only)

$L_{provided} \geq (14.2"+2(0.75")0.577)0.707 = 10.7"$ (271.8mm)

Required length of two 3/16-in. (4.8mm) fillet welds;

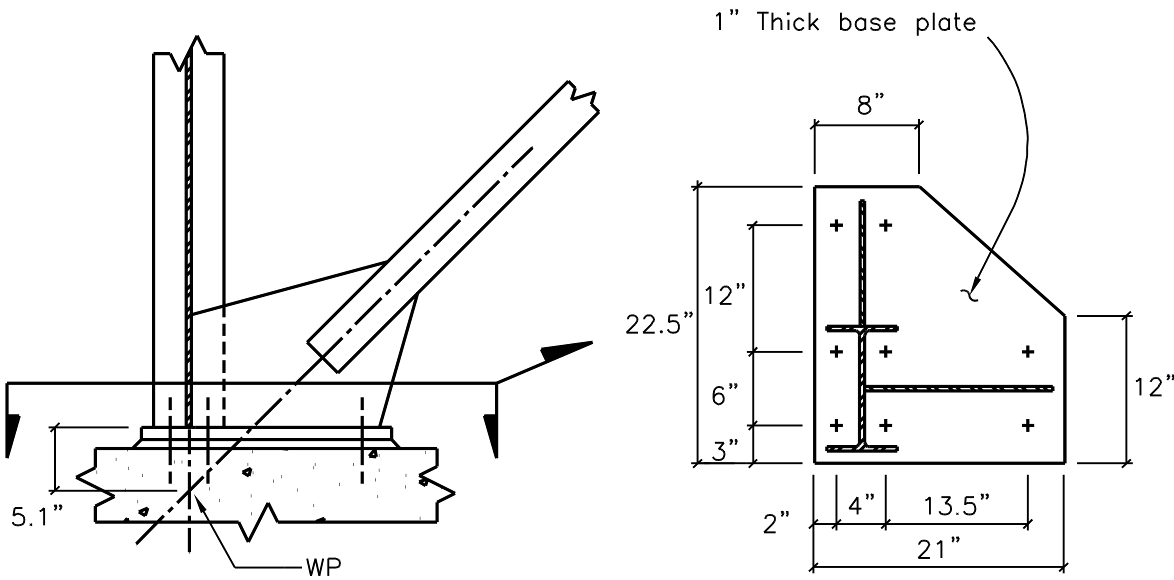
$$(L_w)_{req'd} = \frac{R_u}{\phi R_n} = \frac{24.7^k / 2-\text{sides}}{4.19^{k/in}} = 2.95" << 10.7" \leq L_{provided} \quad (74.9mm << 271.8mm) \quad \textbf{O.K.}$$

Design brace to brace connection (see Figure J2-9);
Use similar design as used at the brace connection to the column and beam. By inspection, the connection meets the requirements of shear/tension rupture, tensile capacity, and compressive capacity. Also, buckling of the gusset plate does not govern because $l_l$ is so much less.




Note: For metric equivalents; 1-in = 25.4mm
Figure J2-9. Brace to brace connection

Design gusset plate to column and column base plate (see Figure J2-10);
Use similar design as used at the column to beam connection. All parameters are the same except the gusset plate attaches to the column base plate instead of the beam.




Note: For metric equivalents; 1-in = 25.4mm
Figure J2-10. Brace to column and column base plate connection

Design column base plate;
For compressive loading, the base plate is approximated as a rectangular plate enclosing only the column cross section and having dimensions $b_f$ + 1" (25.4mm) by d + 1" (25.4mm). Part 11 of AISC LRFD 2nd edition outlines procedures for designing base plates that will be used here.
Determine design loads;

$R_u = 72^k$ (compression or tension)

Area of base plate = BN = (5.75" + 1")(10.17" + 1") = 75.4-in² (46.6X10³ mm²)

$$(A_1)_{req'd} = \frac{R_u}{\phi_c(0.85f_c')}$$

where; $\phi_c = 0.6$
$f_c'$ = 3,500psi (24.13MPa) (assumed)

$$\therefore (A_1)_{req'd} = \frac{72^k}{0.6(0.85)3.5\text{ksi}} = 40.3 - \text{in}^2 < 75.4 - \text{in}^2 = \text{BN} \;\; (26.2\text{X}10^3\ \text{mm}^2 < 46.6\text{X}10^3\ \text{mm}^2)\ \textbf{O.K.}$$

Determine required plate thickness for compressive loading;

$$t_{req'd} = l\sqrt{\frac{2R_u}{0.9F_y BN}}$$

where; $l = \lambda n'$ (small base plate)

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi_c P_p}$$

$\phi_c = 0.6$ (LRFD Specification section J9)
$P_p = 0.85f_c'A_1$ (load acts on full area of concrete support)
$P_p = 0.85(3.5\text{ksi})94.3\text{-in}^2 = 281^k$ (1.28MN)

$$\therefore X = \left(\frac{4(10.17")5.75"}{(10.17"+5.75")^2}\right)\frac{72^k}{0.6(281^k)} = 0.394$$

$$\lambda = \frac{2\sqrt{0.394}}{1+\sqrt{1-0.394}} = 0.706$$

$$n' = \frac{\sqrt{10.17"(5.75")}}{4} = 1.91" \;\; (48.5\text{mm})$$

$l = 0.706(1.91") = 1.35"$ (34.3mm)

$$\therefore t_{req'd} = \sqrt{\frac{2(72^k)}{0.9(36\text{ksi})94.3 - \text{in}^2}} = 0.22" \;\; (5.6\text{mm})$$

Determine required base plate thickness for tensile load;
Closest spacing of bolts is 4-in. (see Figure J2-10) and plate is assumed to span between these bolts;

$$M_u = \frac{P_u L}{4} = \frac{72^k(4")}{4} = 72^{\text{in-k}} \;\; (8.14\text{KN-m})$$

$$\phi_b M_n \ge M_u$$

where; $\phi_b = 0.9$
$M_n = Z_x F_y$

$$Z_x = \frac{Nt^2}{4}$$

$$\therefore \phi_b M_n = 0.9\left(\frac{Nt^2}{4}\right)F_y = M_u \Rightarrow t = \sqrt{\frac{4.44M_u}{NF_y}} = \sqrt{\frac{4.44(72^{in-k})}{(10.17"+2")36ksi}} = 0.85 \qquad \text{(governs)}$$

**Use 1.0-in. (25.4mm) thick base plate**

Design anchor bolts;
Note: The four bolts surrounding the column will be conservatively designed for the entire tension load, but all eight bolts will be assumed active in resisting shear.
Determine design loads;

$$V_u = \frac{24.7^k}{8-\text{bolts}} = 3.09^{k/bolt} \quad (13.7\text{KN/bolt})$$

From load combination 'U = 0.9D + E';

Dead load = $(W_{p1} + W_{p2})/4 = (54.6^k + 5^k)/4 = 14.9^k$ (66.3KN)

$P_u = 0.9(-14.9^k) + 72^k = 58.6^k$ (260.7KN) (tension)

Try 1.0-in. (25.4mm) $\phi$ A307 bolts;
Check capacity in shear;
Steel;

$$V_s = (0.75A_bF_u)n \qquad \text{(EQ. 9.2.4.2-1 FEMA 302)}$$

For 1.0" $\phi$ bolt (A 307)

where; $A_b = 0.785\text{-in}^2$ (506.5mm$^2$)
$F_u$ = 60ksi (413.7MPa)
n = 8-bolt

$$V_s = 0.75(0.785-\text{in}^2)60\text{ksi}(8-\text{bolts}) = 283^k > 24.7^k = V_u \quad (1.26\text{MN} > 0.11\text{MN})$$ **O.K.**

Concrete;

$$\phi V_c = (\phi 800A_b\lambda\sqrt{f_c'})n \qquad \text{(EQ. 9.2.4.2-2 FEMA 302)}$$

where; $\phi = 0.65$
$\lambda = 1.0$ (normal weight concrete)
$f_c'$ = 3,500psi
n = 8-bolts

$$\therefore \phi V_c = 0.65(800)0.785-\text{in}^2(8)\sqrt{3{,}500\text{psi}} = 0.65(297^k) = 193^k \quad (0.86\text{MN})$$

$$\phi V_c = 193^k > 24.7^k = V_u \quad (0.86\text{MN} > 0.11\text{MN})$$ **O.K.**

Check capacity in tension;
Steel;

$$P_s = (0.9A_bF_u)n \qquad \text{(EQ. 9.2.4.1-1 FEMA 302)}$$

where; $A_b = 0.785\text{-in}^2$ (506.5mm$^2$)
$F_u$ = 60ksi (413.7MPa)
n = 4-bolts

$$\therefore P_s = 0.9(0.785-\text{in}^2)60\text{ksi}(4-\text{bolts}) = 170^k > 58.6^k = P_u \quad (0.76\text{MN} > 0.26\text{MN})$$ **O.K.**

Concrete;

$$\phi P_c = \phi\lambda\sqrt{f_c'}(2.8A_p + 4A_T) \qquad \text{(EQ. 9.2.4.1-3 FEMA 302)}$$

where; $\phi$ =0.65
$\lambda = 1.0$ (normal weight concrete)
$f_c'$ = 3,500psi (24.13MPa)
$A_T$ = (4")6" = 24.0-in$^2$ (15.5X10$^3$ mm$^2$)

$$A_p = 2(8")\sqrt{2"}[4"+6"] + 4\left[8"\sqrt{2}\right]^2 = 738-\text{in}^2 \quad (476\text{X}10^3\ \text{mm}^4)$$

(for a 8-in. (203.2mm) embedment)

$$\therefore \phi P_c = 0.65(1.0)\sqrt{3{,}500\text{psi}}\left[2.8(738-\text{in}^2)+4(24-\text{in}^2)\right] = 0.65(128^k) = 83^k > 58.6^k = P_u$$

$(369.2\text{KN} > 260.7\text{KN})$ **O.K.**

Check combined tension and shear;
Per section 9.2.4.3 of FEMA 302 , all of the following conditions shall be met;

condition (a) $\frac{1}{\phi}\left(\frac{V_u}{V_c}\right) \le 1.0$ (EQ. 9.2.4.3-1a FEMA 302)

$$\frac{1}{0.65}\left(\frac{24.7^k}{297^k}\right) = 0.13 \le 1.0$$ **O.K.**

condition (b) $\frac{1}{\phi}\left(\frac{P_u}{P_c}\right) \le 1.0$ (EQ. 9.2.4.3-1b FEMA 302)

$$\frac{1}{0.65}\left(\frac{58.6^k}{128^k}\right) = 0.20 \le 1.0$$ **O.K.**

condition (c) $\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \le 1.0$ (EQ. 9.2.4.3-1c FEMA 302)

$$\frac{1}{0.65}\left[\left(\frac{57.1^k}{128^k}\right)^2 + \left(\frac{24.7^k}{297^k}\right)^2\right] = \frac{1}{0.65}(0.199+0.007) = 0.32 \le 1.0$$ **O.K.**

condition (d) $\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \le 1.0$ (EQ. 9.2.4.3-1d FEMA 302)

$$\left(\frac{58.6^k}{339^k}\right)^2 + \left(\frac{24.7^k}{283^k}\right)^2 = (0.030+0.008) = 0.038 \le 1.0$$ **O.K.**

**Use 1.0-in. (25.4mm) ϕ A307 anchor bolts**