$$\therefore B_2 = \frac{1}{1 - 15.6^k\left(\frac{0.2"}{0.95^k(180")}\right)} = 1.02 \quad \text{Therefore, } B_2 = 1.02$$

Therefore; $M_u = B_1 M_{nt} + B_2 M_{lt} = 1.0(28.74^{ft-kips}) + 1.02(6.83^{ft-kips}) = 35.7^{ft-kips}$ (48.4KN-m)

Note: Since both $M_{nt}$ and $M_{lt}$ carry the same sign, only the magnitude is shown.

Check compact section criteria (per AISC seismic provisions);

Web local buckling;

$$P_y = A_g F_y = 10.0 - in^2(50ksi) = 500^k \ (2.22MN)$$

$$\frac{P_u}{\phi_b P_y} = \frac{8.24^k}{0.9(500^k)} = 0.018 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{8.24^k}{0.9(500^k)}\right] = 71.5$$

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 71.5 = \lambda_p$$ **O.K.**

Flange local buckling;

$$\lambda_{W10X30} = \frac{b_f}{2t_f} = 7.4 \approx 7.35 = \frac{52}{\sqrt{50ksi}} = \frac{52}{\sqrt{F_y}} = \lambda_p$$ **O.K.**

Determine which interaction equation to use;

Maximum $\frac{KL}{r}$, $K_y = 1.0$ due to the braced frames

$K_x$ is determined using AISC alignment charts as follows;

$$\sum I_c / L_c = 340 - in^4 / 15' = 22.67$$

$$\sum I_g / L_g = 245 - in^4 / 30' = 8.17$$

$$\frac{\sum I_c / L_c}{\sum I_g / L_g} = \frac{22.67}{8.17} = 2.77 = G_A$$

At column base; $G_B = 10$ due to the pinned condition

Therefore, from the charts; $K_x = 2.3$

$$\frac{K_x L_x}{r_x} = \frac{2.3(15'(12"/1'))}{5.83"} = 71$$

$$\frac{K_y L_y}{r_y} = \frac{1.0(15'(12"/1'))}{1.53"} = 117.6$$ (governs)

From the AISC table 3-50, $\phi_c F_{cr} = 15.43ksi$ (106.4MPa) (interpolated)

$$\phi_c P_n = A_g(\phi_c F_{cr}) = 10.0 - in^2(15.43ksi) = 154^k \ (685.0KN)$$

$$\frac{P_u}{\phi_c P_n} = \frac{8.24^k}{154^k} = 0.054 < 0.2 \therefore \text{ use AISC LRFD equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$ (EQ. H1-1b AISC LRFD)

$\phi_b M_{nx} = 112^{ft-kips}$ per AISC LRFD 2nd ed. beam design charts (using an unbraced length of 15', $C_b = 1.0$, and $\phi_b = 0.9$)

Therefore; $$\frac{8.24^k}{2(154^k)} + \left(\frac{35.7^{ft-kips}}{112^{ft-kips}} + 0\right) = 0.35 < 1.0$$ **O.K.**

Check Shear; From the analysis output; $V_{u,column} = 2.37^k$ (10.5KN)

$f_v V_n = 77.5^k$ (344.7KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore V_{u,column} = 2.37^k < 77.5^k = f_v V_n$ (10.5KN < 344.7KN) **O.K.**

(1) Moment Frames without truss (High Roof).

There will be one design used for the worst case situation and this design will be used throughout the high roof. The worst case situation is the interior moment frame because it supports the largest tributary area.

Determine Design Loads:
Roof:

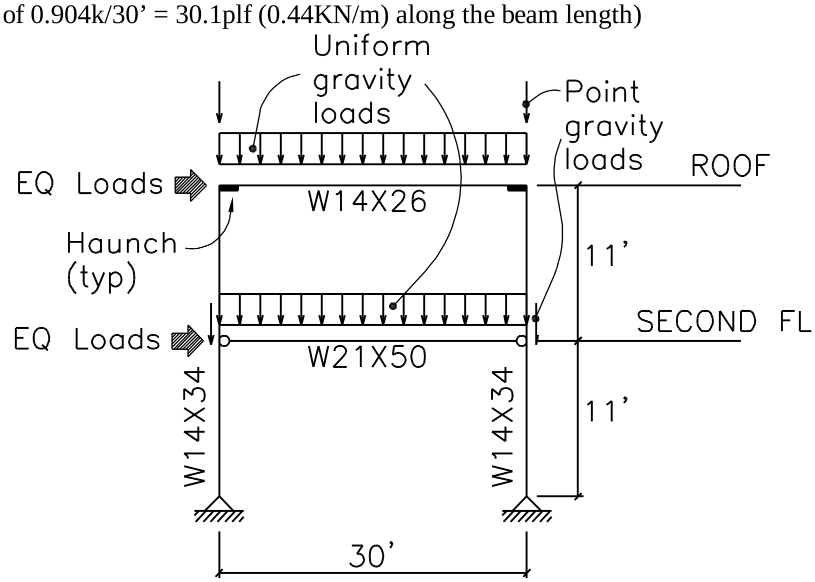
$w_{LR}$:
Live load reduction per ANSI/ASCE 7-95;
$A_T$ = 15'(30') = 450-ft$^2$ (41.8m$^2$)
$R_1 = 1.2-0.001(A_T)$
$= 1.2-0.001(450\text{-ft}^2)$
$= 0.75$
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore w_{LR}$ = 15'(20psf)0.75 = 225plf (3.28KN/m)
$w_{DR}$ = 15'(18.7psf)
= 280plf (4.08KN/m)
$P_{DR}$ = 5.9psf(11')(15') = 974-lb (4.33KN) (point load due to wt. of side walls)

Note: Weight of story panel is conservatively placed at the top of the column.

$E_R = 1.0Q_E = 1.17^k$ (5.20KN) (applied as a uniform load of $1.17^k$/30' = 39plf (0.57KN/m) along the beam length)

Second Floor:

Note: there is no live load reduction at this level because the floor consists of a one way slab.
$w_{LF}$ = 15'(40psf) = 600plf (8.75KN/m)
$w_{DF}$ = 15'(84.8psf)
= 1,272plf (18.55KN/m)
$P_{DF}$ = 5.9psf(5.5')(15') = 487-lb (2.17KN) (point load due to wt. of side walls)
$E_F = 1.0Q_E = 1.0(0.90^k+0.004^k) = 0.904^k$ (4.02KN)(includes torsion effects, and is applied as a uniform load of 0.904k/30' = 30.1plf (0.44KN/m) along the beam length)




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

Design Members:

Note; As for the low roof area haunch properties $I_x$, $I_y$, $S_x$, and A as well as the length of the haunch were previously calculated as; $I_x$ = 640-in$^4$ (266.4X10$^6$ mm$^4$), $I_y$ = 13.3-in$^4$ (5.54X10$^6$ mm$^4$), $S_x$ = 58-in$^3$ (950.4X10$^3$ mm$^3$), A = 11.48-in$^2$ (7.41X10$^3$ mm$^2$), and 'L' length from centerline of column to toe of haunch is 1.85' (22.25")(0.56m).

General;

As in the case of the low roof, the moment frame was analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0). All load combinations were investigated to determine the worst case loading for each element and the worst case deflection for the frame. In all cases, the controlling load combination was again equation 4a; 1.314D+$Q_E$+0.5L. After comparing the frame deflection to the allowable story drift, a check on the strength requirements of the frame were completed. Since the same members being used are the same as for the frame elements as at the low roof, a check to ensure the location of the plastic hinge was determined to be unnecessary.

Drift requirements;

Calculated drift; $d_{calc} = 0.565"$ (14.4mm) (worst case at first story level)

Allowable story drift; $\Delta_{allow} = 0.025 h_{sx}$ (Table 6.1)

$\Delta_{allow} = 0.025(11'(12"/1')) = 3.3"$ (83.8mm)

Therefore; $C_d \times d_{calc} = 5.5(0.565") = 3.11" < \Delta_{allow} = 3.3"$ (79.0mm < 83.8mm) **O.K.**

Strength requirements;

Roof Beam:

The following maximum loads were obtained from the analysis output at the toe of the haunch;

$M_{u,beam} = 30.79^{ft-kips}, V_{u,beam} = 5.99^{k}$ (26.6KN)

$f_b M_n = 109^{ft-kips}$ (147.8KN-m) per AISC LRFD 2$^{nd}$ ed. load factor design selection table (using an unbraced length 'Lb' of the compression flange of 5'(1.53m))

$f_v V_n = 69^{k}$ (306.9KN) per AISC LRFD 2$^{nd}$ ed. maximum uniform load tables

$\therefore M_{u,beam} = 30.79^{ft-kips} < 109^{ft-kips} = f_b M_n$ (41.8KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 5.99^{k} < 69^{k} = f_v V_n$ (26.6KN < 306.9KN) **O.K.**

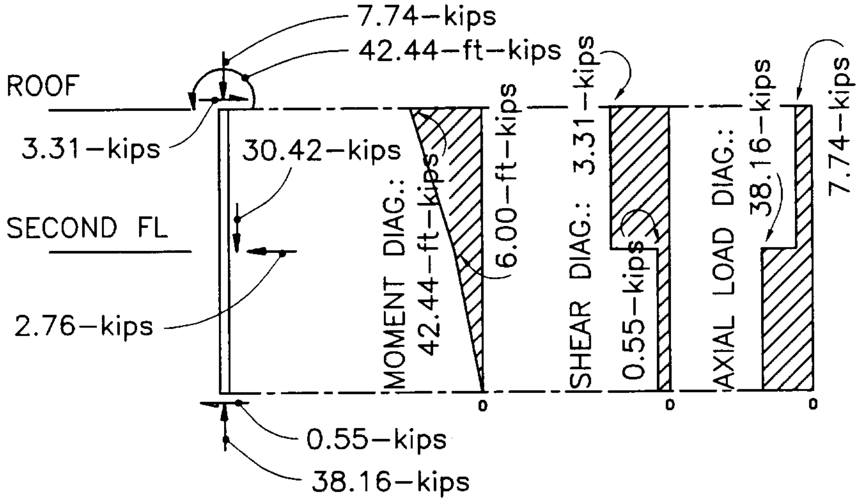
Column:

Note: The columns will be analyzed as if it were a single story column spanning from the first floor level to the roof and subjected to transverse loading (caused by the second floor diaphragm). Therefore, the interstory drift in the moment magnification calculation will be taken as the displacement of the roof level relative to the first floor level.

Per FEMA 302 paragraph 5.2.6.4.1, 30% of the seismic load effects from the orthogonal direction will be included. Since in the orthogonal direction brace frames are acting, this results in only an additional axial load. However, in this case the load resulting from the braced frames is zero.

$P_{u,total} = 38.16^{k}$ (169.7KN)

A free body diagram showing loading as well as moment, shear, and axial load diagrams for the most highly loaded column are shown below.




1-kip = 4.48KN
1-ft-kip = 1.356KN-m

Determine $M_u = B_1 M_{nt} + B_2 M_{lt}$;

$M_{nt} = -25.39^{\text{ft-kips}}$ (34.4KN-m) (calculated reaction at the restraints; @ roof = $-1.01^k$ (4.49KN), @ floor = $-1.09^k$ (4.85KN))

$M_{lt} = -17.11^{\text{ft-kips}}$ (23.2KN-m)

$B_1$ is determined as follows;

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \leq 1.0$$ (EQ. C1-2 AISC LRFD)

where; $C_m = 0.85$ (since member end is restrained at the roof)

$P_u = 38.16^k$ (169.7KN) (as calculated)

$$P_{e1} = \frac{\pi^2 E A_g}{(KL/r)^2}$$ with; $A_g = 10.0\text{-in}^2$ ($6.45\text{X}10^3$ mm$^2$)

$E = 29{,}000$ksi ($200\text{X}10^3$ MPa)

$K_x = 1.0$ (sidesway inhibited)

$$\frac{K_x L}{r_x} = \frac{2.3(11')(12''/1')}{5.83''} = 22.7$$

$$P_{e1} = \frac{\pi^2 (29{,}000\text{ksi})10.0-\text{in}^2}{(22.7)^2} = 5{,}555^k \text{ (27.71MN)}$$

$$\therefore B_1 = \frac{0.85}{\left(1 - \frac{38.16^k}{5{,}555^k}\right)} = 0.856 < 1.0$$ Therefore, $B_1 = 1.0$

$B_2$ is determined as follows;

$$B_2 = \frac{1}{1 - \sum P_u \left(\frac{\Delta_{oh}}{\sum HL}\right)}$$ (EQ. C1-4 AISC LRFD)

where; $\Delta_{oh} = 0.910''$ (23.1mm) (from RISA-2D analysis)

$$\sum H = 1.17^k + 0.904^k = 2.07^k \text{ (9.20KN)(lateral loading, "}Q_E\text{")}$$
$$L = 22'(12''/1') = 264'' \text{ (6.71m)} \quad \text{(column height)}$$
$$\sum P_u = 38.16^k + 35.85^k = 74.01^k \text{ (329.2KN)}$$

Note: The $35.85^k$ load in other column (from RISA-2D analysis)

$$\therefore B_2 = \frac{1}{1 - 74.01^k\left(\dfrac{0.910''}{2.07^k(264'')}\right)} = 1.14 \qquad \text{Therefore, } B_2 = 1.14$$

Therefore; $\quad M_u = B_1 M_{nt} + B_2 M_{lt} = 1.0(25.39^{ft-kips}) + 1.14(17.11^{ft-kips}) = 45^{ft-kips} \text{ (61.0KN-m)}$

Check compact section criteria (per AISC seismic provisions);
Web local buckling;

$$P_y = A_g F_y = 10.0 - in^2(50ksi) = 500^k \text{ (2.22MN)}$$

$$\frac{P_u}{\phi_b P_y} = \frac{38.3^k}{0.9(500^k)} = 0.085 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{38.16^k}{0.9(500^k)}\right] = 63.9$$

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 63.9 = \lambda_p \qquad \textbf{O.K.}$$

Note: Flange local buckling has been checked previously.
Determine which interaction equation to use;

Maximum $\frac{KL}{r}$, $\quad K_y = 1.0$ due to the braced frames

$K_x$ is determined using AISC alignment charts as follows;

$$\sum I_c / L_c = 340 - in^4 / 22' = 15.5$$

$$\sum I_g / L_g = 245 - in^4 / 30' = 8.17$$

$$\frac{\sum I_c / L_c}{\sum I_g / L_g} = \frac{15.5}{8.17} = 1.90 = G_A$$

At column base; $G_B = 10$ due to the pinned condition
Therefore, from the charts; $K_x = 2.1$

$$\frac{K_x L_x}{r_x} = \frac{2.1(22'(12''/1'))}{5.83''} = 95.1 \qquad \text{(governs)}$$

$$\frac{K_y L_y}{r_y} = \frac{1.0(11'(12''/1'))}{1.53''} = 86.3$$

From the AISC table 3-50, $\phi_c F_{cr} = 21.94ksi$ (151.3MPa) (interpolated)

$$\phi_c P_n = A_g(\phi_c F_{cr}) = 10.0 - in^2(21.94ksi) = 219^k \text{ (974.1KN)}$$

$$\frac{P_u}{\phi_c P_n} = \frac{38.16^k}{219^k} = 0.17 < 0.2 \therefore \text{ use AISC equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n}+\left(\frac{M_{ux}}{\phi_b M_{nx}}+\frac{M_{uy}}{\phi_b M_{ny}}\right)\le 1.0$$ (EQ. H1-1b AISC LRFD)

$\phi_b M_{nx} = 128^{ft-kips}$ (173.6KN-m) per AISC LRFD 2nd ed. beam design charts (using an unbraced length of 11' (3.36m), $C_b = 1.0$, and $\phi_b = 0.9$)

Therefore; $\frac{38.16^k}{2(219^k)}+\left(\frac{45^{ft-kips}}{128^{ft-kips}}+0\right)=0.44<1.0$ **O.K.**

Check Shear; From the analysis output; $V_{u,column} = 3.31^k$ (14.7KN)

$\phi_v V_n = 77.5^k$ (344.7KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore V_{u,column} = 3.31^k < 77.5^k = \phi_v V_n$ (14.7KN < 344.7KN) **O.K.**

(7) Moment Frame with truss (High Roof).

Determine Design Loads:

Roof:

$w_{LR}$:

Live load reduction per ANSI/ASCE 7-95;

$A_T = 7.5'(30') = 225\text{-ft}^2$ (20.96m$^2$)

$R_1 = 1.2-0.001(A_T)$

$= 1.2-0.001(225\text{-ft}^2)$

$= 0.975$

Roof slope is flat $\therefore R_2 = 1.0$

$\therefore w_{LR} = 7.5'(20psf)0.975 = 146plf$ (2.13KN/m)

$w_{DR} = 7.5'(18.7psf)$

$= 140plf$ (2.04KN/m)

$P_{DR} = 5.9psf(11')(7.5') = 490\text{-lb}$ (2.18KN) (point load due to wt. of side walls)

Note: Weight of story panel is conservatively placed at the top of the column.

$E_R = 1.0Q_E = 0.585^k$ (2.60KN)(applied as a uniform load of $0.585^k/30' = 19.5plf$ (0.28KN/m) along the beam length)

Second Floor:

Note: there is no live load reduction at this level because the floor consists of a one way slab.

$w_{LF} = 7.5'(40psf) = 300plf$ (4.38KN/m)

$w_{DR} = 7.5'(84.8psf)$

$= 636plf$ (9.28KN/m)

$P_{D2} = 5.9psf[(7.5')(5.5')+(5.5')(10')] = 568\text{-lb}$ (2.53KN) (point load due to wt. of side walls)

$E_F = 1.0Q_E = 3.05^k+0.003^k=3.053^k$ (13.58KN) (applied as a uniform load of $3.053^k/30' = 102plf$ (1.49KN/m) along the beam length)

Loads from adjacent low roof:

$w_{LR}$:

Live load reduction per ANSI/ASCE 7-95;

$A_T = 10'(30') = 300\text{-ft}^2$ (27.9m$^2$)

$R_1 = 1.2-0.001(A_T)$

$= 1.2-0.001(300\text{-ft}^2)$

$= 0.90$

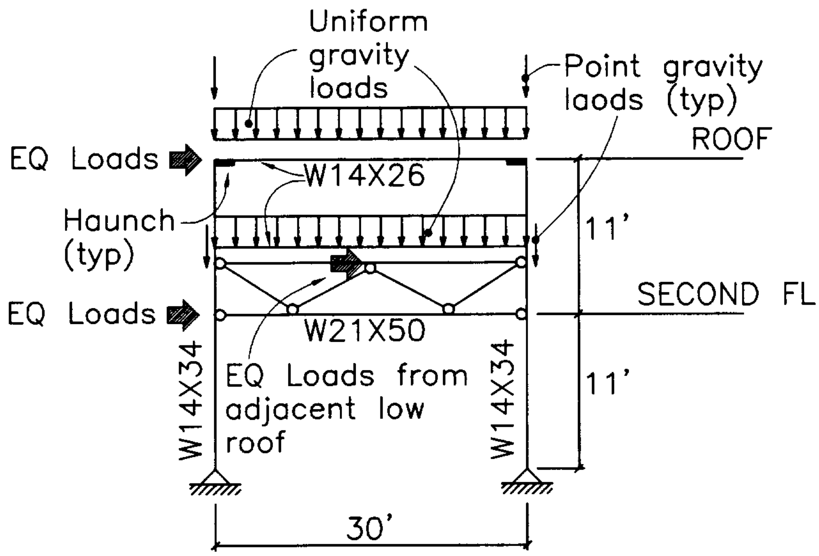
Roof slope is flat $\therefore R_2 = 1.0$

$\therefore w_{LR} = 10'(20psf)0.90 = 180plf$ (2.63KN/m)

$w_{DR} = 10'(17.1psf)$

$= 171plf$ (2.49KN/m)

$E_R = 1.0Q_E = 0.475^k$ (2.11KN) (applied as a uniform load of $0.475^k/30' = 15.8plf$ (0.23KN/m) along the beam length)




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

Design Members:

Note; As for the low roof area haunch properties $I_x$, $I_y$, $S_x$, and A as well as the length of the haunch were previously calculated as; $I_x$ = 640-in$^4$ (266.4X10$^6$ mm$^4$), $I_y$ = 13.3-in$^4$ (5.54X10$^6$ mm$^4$), $S_x$ = 58-in$^3$ (950.4X10$^3$ mm$^3$), A = 11.48-in$^2$ (7.41X10$^3$ mm$^2$), and 'L' length from centerline of column to toe of haunch is 1.85' (22.25") (0.56m).

General;

As in the case of the low roof, the moment frame was analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0). All load combinations were investigated to determine the worst case loading for each element and the worst case deflection for the frame. In all cases, the controlling load combination was again equation 4a; 1.314D+$Q_E$. After comparing the frame deflection to the allowable story drift, a check on the strength requirements of the frame were completed. Since the same members are being used for the moment frame elements as at the low roof, a check to ensure the location of the plastic hinge formation was determined to be unnecessary.

Drift requirements;

Calculated drift; $\delta_{calc}$ = 0.267" (6.8mm)

Allowable story drift; $\Delta_{allow} = 0.025h_{sx}$ (Table 6.1)

$\Delta_{allow}$ = 0.025(11'(12"/1')) = 3.3" (83.8mm)

Therefore; $C_d x \delta_{calc}$ = 5.5(0.267") = 1.47" < $\Delta_{allow}$ = 3.3" (37.3mm < 83.8mm) **O.K.**

Strength requirements;

Roof Beam:
The following maximum loads were obtained from the analysis output at the toe of the haunch;

$$M_{u,beam} = 10.14^{ft-kips}(13.74KN-m), V_{u,beam} = 2.49^{k}(11.08KN)$$

$\phi_b M_n = 109^{ft-kips}$ (147.8KN-m) per AISC LRFD 2$^{nd}$ ed. load factor design selection table (using an unbraced length 'Lb' of the compression flange of 5' (1.53m))

$\phi_v V_n = 69^k$ (306.9KN) per AISC LRFD 2nd ed. maximum uniform load tables

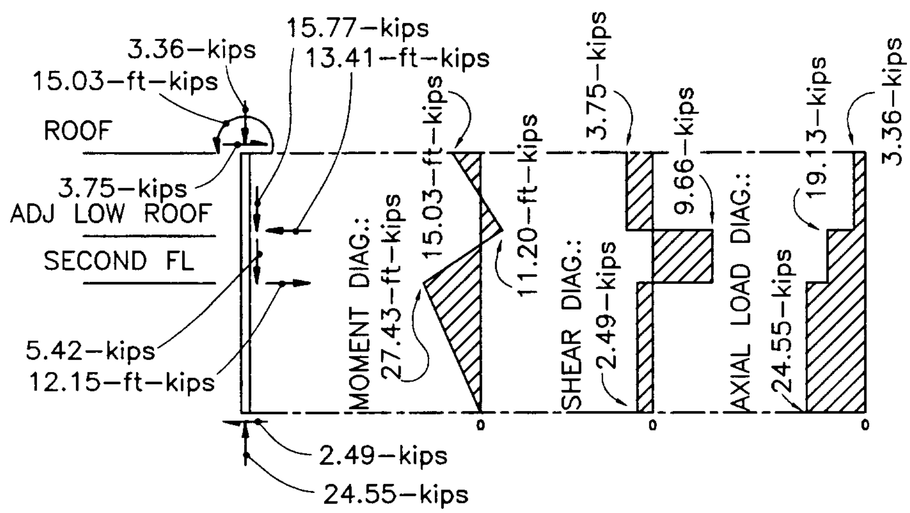
$\therefore M_{u,beam} = 10.14^{ft-kips} < 109^{ft-kips} = \phi_b M_n$ (13.74KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 2.49^k < 69^k = \phi_v V_n$ (11.08KN < 306.9KN) **O.K.**

Column:

Note: The columns will be analyzed as if it were a single story column spanning from the first floor level to the roof and subjected to transverse loading (caused by the second floor diaphragm). Therefore, the interstory drift in the moment magnification calculation will be taken as the displacement of the roof level relative to the first floor level.

A free body diagram with loads as well as moment, shear, and axial load diagrams for the most highly loaded column are shown below.




1-kip = 4.448KN
1-ft-kip = 1.356KN-m

Per FEMA 302 paragraph 5.2.6.4.1, 30% of the seismic load effects from the orthogonal direction will be included. Since in the orthogonal direction brace frames are acting this results in only an additional axial load. The axial load due to the brace frame is calculated as follows;

$$P_{axial} = \frac{P_{horiz}}{2\cos\phi} = \frac{(3.15^k/2)}{2(0.806)} = 0.98^k \text{ (4.36KN)}$$

$$P_{u,total} = 24.55^k + 0.3(0.98^k \sin\phi) = 24.55^k + 0.3\left(0.98^k\left(\frac{11'}{18.6'}\right)\right) = 24.7^k \text{ (109.9KN)}$$

Determine $M_u = B_1 M_{nt} + B_2 M_{lt}$;

$M_{nt} = -12.60^{ft-kips}$ (17.09KN-m) (calculated reaction at the restraints; @ roof = -1.92$^k$ (8.54KN), @ floor = -20.97$^k$ (93.3KN), and @ low roof = +18.39$^k$ (81.8KN))

$M_{lt} = -27.37^{ft-kips}$ (37.11KN-m)

$B_1$ is determined as follows;

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \le 1.0 \qquad \text{(EQ. C1-2 AISC LRFD)}$$

where; $C_m = 0.85$ (since member end is restrained at the roof)

$P_u = 24.7^k\,(109.9KN)$ (as calculated)

$P_{e1} = 5{,}555^k$ (24.71MN) (as previously calculated)

$$\therefore B_1 = \frac{0.85}{\left(1-\dfrac{24.7^k}{5{,}555^k}\right)} = 0.854 < 1.0$$ Therefore, $B_1 = 1.0$

$B_2$ is determined as follows;

$$B_2 = \frac{1}{1-\sum P_u\left(\dfrac{\Delta_{oh}}{\sum HL}\right)}$$ (EQ. C1-4 AISC LRFD)

where; $\Delta_{oh} = 0.279"$ (7.1mm) (from RISA-2D analysis)

$\sum H = 5.85^k + 0.475^k + 3.053^k = 9.38^k$ (41.7KN) (lateral loading, "$Q_E$")

L = 22'(12"/1') = 264" (6.71m) (column height)

$\sum P_u = 24.7^k + 21.00^k + 0.3\left(0.98\left(\dfrac{11'}{18.6'}\right)\right) = 45.9^k$ (204.2KN)

load in other column (from RISA-2D analysis)

$$\therefore B_2 = \frac{1}{1-45.9^k\left(\dfrac{0.279"}{9.38^k(264")}\right)} = 1.01$$ Therefore, $B_2 = 1.01$

Therefore; $M_u = B_1M_{nt} + B_2M_{lt} = 1.0(12.60^{ft-kips}) + 1.01(27.37^{ft-kips}) = 40.2^{ft-kips}$ (54.51KN-m)

Note: Since both $M_{nt}$ and $M_{lt}$ carry the same sign, only the magnitude is shown.

Note: Compact section criteria have been checked previously.

Determine which interaction equation to use;
Note: By inspection, $K_yL/r_y$ governs;

$$\frac{K_yL_y}{r_y} = \frac{1.0(11'(12"/1'))}{1.53"} = 86.3$$

From the AISC table 3-50, $\phi_cF_{cr} = 24.7\text{ksi} \Rightarrow \phi_cP_n = A_g(\phi_cF_{cr}) = 10-\text{in}^2(24.7\text{ksi}) = 247^k$ (1.10MN)

$$\frac{P_u}{\phi_cP_n} = \frac{24.7^k}{247^k} = 0.05 < 0.2 \therefore \text{ use AISC equation H1-1b}$$

$$\frac{P_u}{2\phi_cP_n} + \left(\frac{M_{ux}}{\phi_bM_{nx}} + \frac{M_{uy}}{\phi_bM_{ny}}\right) \le 1.0$$ (EQ. H1-1b AISC LRFD)

$\phi_bM_{nx} = 128^{ft-kips}$ (173.6KN-m) (calculated previously)

Therefore; $$\frac{24.7^k}{2(247^k)} + \left(\frac{40.2^{ft-kips}}{128^{ft-kips}} + 0\right) = 0.36 < 1.0$$ **O.K.**

Check Shear; From the analysis output; $V_{u,column} = 9.66^k$ (42.97KN)

$f_v V_n = 77.5^k$ (344.7KN) (calculated previously)

$\therefore V_{u,column} = 9.66^k < 77.5^k = f_v V_n$ (42.97KN < 344.7KN) **O.K.**

B-12 *Check allowable drift and P $\Delta$ effect.*

Transverse direction;

Drift;

In the transverse direction, allowable drift was checked during the moment frame design.

P $\Delta$ effect;

$$q = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(EQ. 5.3.7.2-1 FEMA 302)}$$

By inspection, the worst case condition occurs at the first story of the interior high roof moment frame because it has the largest total vertical load "$P_x$" and the largest design story drift "$\Delta$".

$P_x$ = Total vertical design load without load factors
= 15'(0.225+0.280+0.600+1.272)klf+2(0.974$^k$+0.649$^k$)
= 38.9$^k$ (73.0KN)
$\Delta$ = 0.565" (14.4mm)
$V_x$ =1.17$^k$ + 0.90$^k$ + 0.004= 2.074$^k$ (9.23KN)
$h_{sx}$ = 11'(12"/1') = 132" (3.36m)
$C_d$ = 5.5

$$\therefore q = \frac{38.9^k (0.565")}{2.074^k (132")5.5} = 0.015 < 0.10$$

Therefore, P $\Delta$ effects need not be considered in the transverse direction.

Longitudinal direction;

Check Drift;

Low Roof;
The stiffness for a single 'x' brace (one per building side) consisting of two braces in the horizontal direction is;

$$K_{horiz} = \frac{2AE}{L}\cos^2 f = \frac{2(3.59 - in^2)29{,}000ksi}{25'(12"/1')}\cos^2(37^\circ) = 444 - k/in \ \ (77.8KN/mm)$$

Note: All load combinations reduce to $P_u = E = Q_E = 1.25^k$ (5.56KN).

$$\therefore d_{calc} = \frac{P_u}{K_{horiz}} = \frac{1.25^k}{444 - k/in} = 0.003" \ \ (0.08mm)$$

$$\Delta_{allow} = 0.025h_{sx} = 0.025(15'(12"/1')) = 4.5" \ \ (114.3mm)$$

$C_d \, x \, d_{calc} = 5.0(0.003") = 0.015" < 4.5" = \Delta_{allow}$ (0.38mm < 114.3mm) **O.K.**

High Roof;
The stiffness of two diagonal braces (two per building side per story) in the horizontal direction is;

$$K_{horiz} = \frac{2AE}{L}\cos^2\phi = \frac{2(3.59-in^2)29{,}000ksi}{18.6'(12"/1')}\cos^2(36^\circ) = 611-k/in\ \ (107.0KN/mm)$$

Note: The worst case condition occurs at the first story because the stiffness is the same while the loads are larger. The worst case condition is checked s follows:

$$\therefore \delta_{calc} = \frac{P_u}{K_{horiz}} = \frac{1.575^k + 3.28^k + 0.323^k}{611-k/in} = 0.008"\ \ (0.20mm)$$

$$\Delta_{allow} = 0.025h_{sx} = 0.025(11'(12"/1')) = 3.3"\ \ (83.8mm)$$

$$C_d x \delta_{calc} = 5.0(0.008") = 0.04" < 3.3" = \Delta_{allow}\ \ (1.02mm < 83.8mm)$$ **O.K.**

$P\Delta$ effect;

By inspection, $P\Delta$ effect will not be an issue because it is not as issue for the more flexible transverse moment frames.

e. Enhanced Performance Objective (following steps in table 4-6 for Immediate Occupancy for an Essential Facility)

Note: Ground Motion B (3/4 MCE) is to be used. Seismic effects are to be scaled by the factor R x 0.75/0.67, and 'm' values for Immediate Occupancy are applicable.

F-1 through F-3 *Determine seismic effects.*

Note: Step F of Table 4-6 for performance objective 3B (Immediate Occupancy for an Essential Facility) specifies that seismic effects for the enhanced performance objective may be determined by scaling the values used for performance objective 1A as follows;

$$\text{Base Shear} = V = Rx\left(\frac{0.75}{0.67}\right)xC_1xC_2xC_3x(C_sxW)$$

where; $C_s$ = Previously calculated seismic response coefficient determined in performance objective 1A (step B-3)

W = Previously calculated dead load and applicable live load

R = Response modification coefficient used in performance objective 1A

Coefficients $C_1$, $C_2$, and $C_3$ are defined in paragraph 5-4f(2).

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.37}{0.57} = 0.65$ (Note: $T_s$ is calculated by equating equations 3-11 and 3-12)

$T_{transverse} = 0.36\sec$, $T_{longitudinal} = 0.2\sec$

$T_{transverse} = 0.36\sec < 0.65\sec = T_s$, $T_{longitudinal} = 0.2\sec < 0.65 = T_s$

$C_1$ is interpolated between 1.5 for $T \le 0.10\sec$, and 1.0 for $T \ge T_s$ in accordance with the alternate procedure listed in section 3.3.1.3 of FEMA 273 (dated October 1997).

$C_{1,transverse} = 1.26$, $C_{1,longitudinal} = 1.41$

$C_2 = 1.0$ per table 5-2

$C_3 = 1.0$ for the building has a positive post yield stiffness

Therefore, the scale factors in each of the buildings principle directions are calculated as follows;

$$SF_{trans} = 8\left(\frac{0.75}{0.67}\right)1.26(1.0)1.0 = 11.3$$

$$SF_{long} = 6\left(\frac{0.75}{0.67}\right)1.41(1.0)1.0 = 9.47$$

**Therefore use $SF_{trans} = 11.3$, and $SF_{long} = 9.47$**

F-4 *Determine combined load effects.*

Load combinations were previously determined in step B-10.

F-5 *Identify force controlled and deformation controlled structural components.*

Note: Table 7-10 for concentric braced frames, and Table 7-12 for fully restrained moment frames identify deformation controlled components for these lateral load resisting systems and supply m values for them. Components of concentric braced frame or fully restrained moment frame lateral load resisting systems not identified in these tables are force-controlled components. Therefore, connections for concentric braced frames, and Chord/Collector elements and their connections are force-controlled components. Additionally, braces in the moment frame with a truss are considered force-controlled because yielding of the deformation-controlled components controls the forces that can be delivered to them.

Deformation controlled components and associated m factors;

Moment frames;

Beams at plastic hinge location in flexure;

$F_{ye} = R_y F_y = 1.5(36\text{ksi}) = 54\text{ksi}\ \ (372.3\text{MPa})$, with $R_y = 1.5$ per AISC seismic provisions sec. 6.2 for ASTM A36

$$\therefore \left(\frac{b}{2t_f}\right)_{W14X26} = 6.0 < 7.08 = \frac{52}{\sqrt{F_{ye}}}, \text{ therefore;} \qquad m = 2.0$$

Columns in flexure with $P/P_{ye} < 0.20$;

Consider the most highly loaded column which is the interior first story column at the high roof area;

$P = [1.2(280 + 1{,}272)\text{plf} + 1.6(600\text{plf}) + 0.5(225\text{plf})]15' + 1.2(980 + 653)\text{lbs} = 46^k$ (204.6KN)
$P_{ye} = R_y F_y A_g = 1.1(50\text{ksi})10\text{-in}^2 = 550^k$ (2.47MN), with $R_y = 1.1$ per AISC seismic provisions sec. 6.2 for Gr. 50
$\therefore P/P_{ye} = 46^k/550^k = 0.084 < 0.2$

$F_{ye} = R_y F_y = 1.1(50\text{ksi}) = 55\text{ksi}\ \ (379.2\text{MPa})$

$$\frac{52}{\sqrt{F_{ye}}} = 7.01 < \left(\frac{b}{2t_f}\right)_{W14X34} = 7.4 < 12.8 = \frac{95}{\sqrt{F_{ye}}}, \text{ therefore, by interpolation;} \qquad m = 1.93$$

Panel zones in shear; m = 1.5

Fully restrained moment connections;
For full penetration flange welds and bolted or welded web connections with no panel zone yielding;

m = 1.0

Braced Frames;

Columns in tension; $m = 1.0$

Braces in compression; $m = 0.8$

Braces in tension; $m = 1.0$

F-6 *Determine $Q_{CE}$ for deformation-controlled components.*

Note: $Q_{UD}$ for each component will be determined in step F-7.

Note: Per paragraph 6-3a(3)(a), $Q_{CE}$ is defined as the nominal strength multiplied by 1.25.

W14X34 (W355.6mmX0.50KN/m) Column in flexure;

$Q_{CE} = 1.25M_n = 1.25(187^{ft\text{-}kips}) = 234^{ft\text{-}kips}$ (317.3KN-m)

where; $M_n$ is calculated using AISC load factor design selection table as follows;
For an unbraced length of 11-ft (3.36m);

$$\phi_b M_n = C_b[\phi_b M_n - BF(L_b - L_p)]$$

$$\phi_b M_n = 1.0[205^{ft-kips} - 6.58(11'-5.4')] = 168^{ft-kips}(227.8KN - m)$$

$$\therefore M_n = 168^{ft-kips} / 0.9 = 187^{ft-kips}(253.6KN - m)$$

W14X34 (W355.6mmX0.50KN/m) Column in tension;

$Q_{CE} = 1.25P_n = 1.25A_gF_y = 1.25(10.0\text{-in}^2)50ksi = 625^k$ (2.78MN)

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$Q_{CE} = 1.25\ R_v$

Per AISC seismic provisions; $R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]$

where; $b_{cf} = 6.745"$, $d_b = 20.91"$, $d_c = 13.98"$, $t_p = 0.285"$, $t_{cf} = 0.455"$ (11.6mm)

$$\therefore R_v = 0.6(50ksi)13.97"(0.285")\left[1 + \frac{3(6.745")(0.455")^2}{20.91"(13.98")0.285"}\right] = 126^k \ (560.4KN)$$

$Q_{CE} = 1.25(126^k) = 158^k$ (702.8KN)

W14X26 (W355.6mm X 0.38KN/m) Beam in flexure;

$Q_{CE} = 1.25M_n = 1.25M_p = 1.25(119^{ft\text{-}kips}) = 149^{ft\text{-}kips}$ (202.0KN-m)

where; $M_n$ is calculated using AISC LRFD 2nd ed. load factor design selection table as follows;
For an unbraced length of 5-ft (1.53m);

$$\phi_b M_n = C_b[\phi_b M_n - BF(L_b - L_p)]$$

$$\phi_b M_n = 1.0[109^{ft-kips} - 4.44(5'-4.5')] = 107^{ft-kips}(145.1KN - m)$$

$$\therefore M_n = 107^{ft-kips} / 0.9 = 119^{ft-kips}(161.4KN-m)$$

TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm) Brace in compression;

$Q_{CE} = 1.25P_n = 1.25(22.4^k) = 28^k$ (124.5KN)

where; $P_n$ is calculated using AISC LRFD 2nd ed. column load tables as follows;
From the tables with KL = 1.0(25') = 25' (7.63m)
$P_n = 19^k/0.85 = 22.4^k$ (99.6KN)

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Brace in tension;

$Q_{CE} = 1.25P_n = 1.25F_yA_g = 1.25(46ksi)3.59\text{-in}^2 = 206^k$ (916.3KN)

F-7 *Determine DCR's for deformation-controlled components and compare with allowable m values for Immediate Occupancy.*

Low Roof Structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D + Q_E$..

$(Q_E)_{roof} = SF_{trans} x F_{roof} = 11.3 \text{ x } 0.95^k = 10.7^k$ (47.6KN)

W14X26 (W355.6mm X 0.38KN/m) Beam at plastic hinge location in flexure;

$(Q_{UD})_{worst\ case} = 84.96^{ft-kips}$ (115.2KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{84.96^{ft-kips}}{149^{ft-kips}} = 0.57 < 2.0 = m$$ **O.K.**

W14X34 (W355.6mm X 0.50KN/m) Column in flexure;

$(Q_{UD})_{worst\ case} = 106.14^{ft-kips}$ (143.9KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{106.71^{ft-kips}}{234^{ft-kips}} = 0.46 < 1.93 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$$Q_{UD} = \frac{M_u}{d_{beam} + d_{haunch} - t_f}$$

$$(Q_{UD})_{worst\ case} = \frac{99.63^{ft-kips}(12''/1')}{13.91''+7''-0.420''} = 58.2^{ft-kips} \text{ (78.9KN-m)}$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{58.2^{ft-kips}}{158^{ft-kips}} = 0.37 < 1.5 = m$$ **O.K.**

Check Deflection;

$$\Delta_{allow} = 0.015h_{sx} = 0.015(15'(12''/1')) = 2.7'' \text{ (68.6mm)}$$

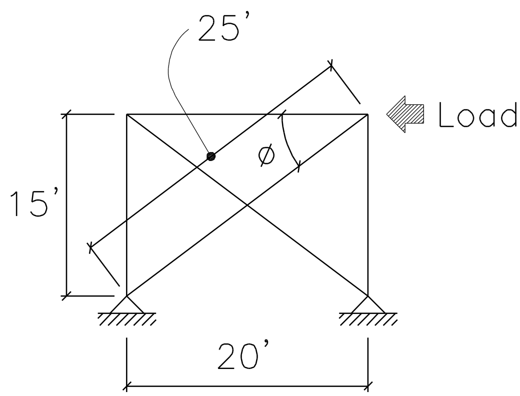
$d_{calc} = 2.14'' < 2.7'' = \Delta_{allow}$ (54.5mm < 68.6mm) **O.K.**

Longitudinal direction;

$(Q_E)_{roof} = SF_{long} x F_{roof} = 9.47(2.5^k) = 23.7^k$ (105.4KN)

W14X34 (W355.6mmX0.50KN/m) Column in tension;

Note: Axial load in column is conservatively calcuated
Ignoring the gravity loads as follows;




1-ft = 0.30m

$$(Q_{UD})_{col\ axial} = \frac{23.7^k}{2(2)}(\sin \phi) = 5.93^k \left(\frac{15'}{25'}\right) = 3.56^k \text{ (15.8KN)}$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{3.56^k}{625^k} = 0.006 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{23.7^k}{2(2)}\left(\frac{1}{\cos \phi}\right) = 5.93^k \left(\frac{25'}{20'}\right) = 7.41^k \text{ (33.0KN)}$$

Compression; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{28^k} = 0.27 < 0.8 = m$ **O.K.**

Tension; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{206^k} = 0.04 < 1.0 = m$ **O.K.**

High roof structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D+Q_E+0.5L$.

For the moment frame without a truss;

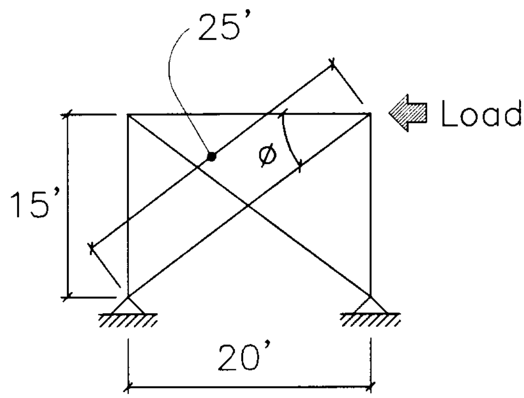
$\delta_{calc} = 2.14" < 2.7" = \Delta_{allow}$ (54.5mm < 68.6mm) **O.K.**

Longitudinal direction;

$(Q_E)_{roof} = SF_{long} x F_{roof} = 9.47(2.5^k) = 23.7^k$ (105.4KN)

W14X34 (W355.6mmX0.50KN/m) Column in tension;

Note: Axial load in column is conservatively calcuated
Ignoring the gravity loads as follows;




1-ft = 0.30m

$$(Q_{UD})_{col\ axial} = \frac{23.7^k}{2(2)}(\sin\phi) = 5.93^k\left(\frac{15'}{25'}\right) = 3.56^k \text{ (15.8KN)}$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{3.56^k}{625^k} = 0.006 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{23.7^k}{2(2)}\left(\frac{1}{\cos\phi}\right) = 5.93^k\left(\frac{25'}{20'}\right) = 7.41^k \text{ (33.0KN)}$$

Compression; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{28^k} = 0.27 < 0.8 = m$ **O.K.**

Tension; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{206^k} = 0.04 < 1.0 = m$ **O.K.**

High roof structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D+Q_E+0.5L$.

For the moment frame without a truss;

$(Q_E)_{roof} = SF_{trans}xF_{roof} = 11.3 \text{ x } 1.17^k = 13.2^k$ (58.7KN)

$(Q_E)_{floor} = SF_{trans}xF_{floor} = 11.3 \text{ x } 0.904^k = 10.2^k$ (45.4KN)

For the moment frames with a truss;

$(Q_E)_{roof} = SF_{trans}xF_{roof} = 11.3 \text{ x } 0.585^k = 6.42^k$ (28.6KN)

$(Q_E)_{floor} = SF_{trans}xF_{floor} = 11.3 \text{ x } 3.053^k = 34.3^k$ (152.6KN)

$(Q_E)_{floor} = SF_{trans}xF_{roof\ adj} = 11.3 \text{ x } 0.475^k = 5.37^k$ (23.9KN)

Note: In all of the following checks, the moment frame without a truss governed.

W14X26 (W355.6mmX0.38KN/m) Beam at plastic hinge location in flexure;

$(Q_{UD})_{worst\ case} = 187.19^{ft\text{-}kips}$ (253.8KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{187.19^{ft-kips}}{149^{ft-kips}} = 1.26 < 2.0 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Column in flexure;

$(Q_{UD})_{worst\ case} = 220.84^{ft\text{-}kips}$ (299.5KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{220.84^{ft-kips}}{234^{ft-kips}} = 0.94 < 1.93 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$$Q_{UD} = \frac{M_u}{d_{beam} + d_{haunch} - t_f}$$

$$(Q_{UD})_{worst\ case} = \frac{210.15^{ft-kips}}{13.91''+7''-0.420''} = 123^{ft-kips} \quad (166.8\text{KN-m})$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{123^{ft-kips}}{158^{ft-kips}} = 0.80 < 1.5 = m$$ **O.K.**

Check Deflection;

$$\Delta_{allow} = 0.015h_{sx} = 0.015(11'(12''/1')) = 1.98'' \quad (50.3\text{mm})$$

$$d_{calc} = 6.375'' > 1.98'' = \Delta_{allow} \quad (161.9\text{mm} > 50.33\text{mm})$$ **N.G.**

Note: High roof moment frames will have to be redesigned.

Longitudinal direction;

$Q_E = SF_{long}x\{F_{roof}+F_{floor}+F_{torsion}\} = 9.47(1.575^k+3.28^k+0.323^k) = 9.47(5.18^k) = 49.0^k$ (218.0KN)

W14X34 (W355.6mmX0.50KN/m) Column in tension;

$$(Q_{UD})_{col\ axial} = \frac{49.0^k}{2}(\sin\phi) = \frac{49.0^k}{2}\left(\frac{11'}{18.6'}\right) = 14.5^k \ (64.5KN)$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{14.5^k}{625^k} = 0.023 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{49.0^k}{2}\left(\frac{1}{\cos\phi}\right) = \frac{49.0^k}{2}\left(\frac{18.6'}{15'}\right) = 30.4^k \ (135.2KN)$$

Compression; $\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{28^k} = 1.09 < 0.8 = m$ **N.G.**

Note: Braces will have to be redesigned.

Tension; $\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{206^k} = 0.15 < 1.0 = m$ **O.K.**

F-8 *Determine QUF and QCL for force-controlled components and compare QUF with QCL.*

Note: $Q_{CL}$ contains the appropriate strength reduction factor per paragraph 6-3a(3)(b).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3J} \qquad \text{(EQ. 6-4a)}$$

where; $J = 1.0 + S_{DS} \le 2.0$ (EQ. 6-5)

$= 1.0+0.57 = 1.57 < 2.0$

$C_{1,trans} = 1.26$, $C_{1,long} = 1.41$ (as previously calculated)

$C_2 = C_3 = 1.0$ (as previously calculated)

Therefore, the scale factor for $Q_E$ is;

Transverse; $SF_{trans} = \frac{1}{C_1C_2C_3J} = \frac{1}{1.26(1.0)1.0(1.57)} = 0.51$

Longitudinal; $SF_{long} = \frac{1}{C_1C_2C_3J} = \frac{1}{1.41(1.0)1.0(1.57)} = 0.45$

Chord/Collector elements;

Worst case condition is used.

$Q_E$;

Maximum Chord Force = 234-lb (1.04KN)

Maximum Collector Force = 788-lb (3.51KN) (governs)

$Q_G$;

$w_u = 1.314D = 1.314(2.5')18.7psf + 0.5(50plf) = 86.4plf$ (1.26KN/m)

$M_u = w_uL^2/8 = 86.4plf(15')^2/8 = 2.43^{ft\text{-}kips}$ (3.30KN/m)

$\therefore Q_{UF} = SF_{long}(Q_E) = 0.45(788\text{-lb}) = 355\text{-lb}$ (1.58KN) compressive seismic axial load with $2.43^{ft\text{-}kips}$ (3.30KN-m) in flexure due to gravity loads.

Determine $Q_{CL}$;

$(Q_{CL})_{flexure} = \phi_b M_n = 47^{ft-kips}$ (63.7KN-m) per AISC LRFD 2nd ed. load factor design selection table for a beam with continuous lateral support of the compression flange.

$(Q_{CL})_{compression} = \phi_c P_n = 118^k$ (0.52KN) using AISC LRFD 2nd ed. table 3-36 as follows;

$$\frac{KL}{r} = \frac{1.0(15')(12''/1')}{4.62''} = 39 \Rightarrow \phi_c F_{cr} = 28.25\text{ksi}\ (194.4\text{MPa})$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 28.25\text{ksi}(4.16 - \text{in}^2) = 118^k\ (524.9\text{KN})$$

Check interaction equation;

$$\frac{Q_{UF}}{Q_{CL}} \geq 1.0$$

$$\frac{P_u}{\phi_c P_n} = \frac{0.355^k}{118^k} = 0.003 < 0.2$$

Therefore, use AISC LRFD equation H1-1b

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(EQ. H1-1b AISC LRFD)}$$

$$\frac{0.355^k}{2(118^k)} + \left(\frac{2.43^{ft-kips}}{47^{'k}} + 0\right) = 0.053 < 1.0$$

**O.K.**

Note: By inspection, the collector element at the low roof is acceptable.

Braces in moment frame with a truss;

The Risa-2D analysis for this frame was rerun with the scale factor $SF_{trans} = 0.51$ applied to the seismic loading. The following load combination governed;

$$Q_{UF} = Q_G + \frac{Q_E}{C_1 C_2 C_3 J} = 1.314D + 0.5L + 0.51E$$

The governing loads are as follows;

$(P_u)_{max} = 29.50^k$ (131.2KN) Tension

$(P_u)_{max} = 7.10^k$ (31.6KN) Compression

Determine $Q_{CL}$;

$$(Q_{CL})_{tension} = \phi_t A_g F_y = 0.9(3.59 - \text{in}^2)46\text{ksi} = 149^k\ (662.8\text{KN})$$

$(Q_{CL})_{compression} = 103.5^k$ (460.4KN) per AISC LRFD 2nd ed. column tables using KL = 8.5' (2.60m) and $\phi_c = 0.85$.

Tension; $\frac{Q_{UF}}{Q_{CL}} = \frac{29.5^k}{149^k} = 0.20 < 1.0$ **O.K.**

Compresion; $\frac{Q_{UF}}{Q_{CL}} = \frac{7.10^k}{103.5^k} = 0.07 < 1.0$ **O.K.**

F-9 *Revise member sizes as necessary and repeat analysis.*

Moment frames of the two-story structure (high roof) do not meet deflection requirements and must be resized. The beam size will be chosen first and then the column size will be chosen to comply with the strong column/weak beam requirements of the AISC seismic provisions. The beam size (required moment of inertia '$I_x$') is scaled up in inverse proportion to the drift as follows;

$$\frac{I_{req'd}}{I_{W14X26}} = \frac{\delta_{calc}}{\Delta_{allow}} = \frac{6.375"}{1.98"} = 3.22\text{ , Say } 3.5$$

$$\therefore (I_{req'd})_{beam} \geq 3.5(I_{W14X26}) = 3.5(245-in^4) = 858-in^4\ (359.1X10^6\ mm^4)$$

Check compact section criteria (per AISC seismic provisions):
Try W14X82 (W355.6mm X 1.20KN/m);

$$\frac{b_f}{2t_f} = 5.9 < 8.7 = \frac{52}{\sqrt{36ksi}} = \frac{52}{\sqrt{F_y}}$$ **O.K.**

**Choose W14X82 (W355.6mm X 1.20KN/m) I = 882-in$^4$ (367.1X10$^6$ mm$^4$) > 858-in$^4$ (357.1X10$^6$ mm$^4$), Z = 139-in$^3$ (2.28X10$^6$ mm$^3$)**

Determine column size;

Assume a W14 section for the column providing a column depth '$d_c$' approximately equal to the beam depth '$d_b$'. Therefore, the location of the plastic hinge from column centerline 'x' is calculated as follows;

$$x = \frac{d_c}{2} + \frac{3d_b}{4} + \frac{d_b}{3} \approx 1.6d_b = 1.6(14.31") = 22.9"\ (581.7mm),$$ Say 24" or 2' (0.6m)

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geq 1.0$$ (EQ. 9-3 AISC Seismic Provisions)

$$\sum M^*_{pb} = \sum(1.1R_y M_p + M_v)$$

where; $R_y$ = 1.5
$M_p$ = Plastic Moment = $ZF_y$ = 139-in$^3$(36ksi)
= 5,004$^{in-kips}$ (565.5KN-m)

$M_v$;

$$V_p = \frac{2M_p}{(L-2x)} = \frac{2(5{,}004^{in-kips})}{(30'-2(2'))(12"/1')} = 32.1^k\ (142.8KN)$$

$$\therefore M_v = V_p(2') = 32.1^k(24") = 770^{in-kips}\ (87.0KN\text{-}m)$$

Therefore;

$$\sum M^*_{pb} = 1.1(1.5)5{,}004^{in-kips} + 770^{in-kips} = 9{,}027^{in-kips}\ (1.02MN\text{-}m)$$

and;

$$\sum M^*_{pc} = \sum Z_c(F_{yc} - P_{uc}/A_g)$$

where; $P_{uc} \approx 1.2(20psf)15'(15')(1^k/1000^{lb}) = 5.4^k$ (24.0KN)
$A_g \approx$ 10-in$^2$ (6.45X10$^3$ mm$^2$) (assumed)
$\therefore P_{uc}/A_g \approx$ 0.5ksi (3.44MPa)

Therefore;

$$\sum M^*_{pc} = Z_{req'd}(50ksi - 0.5ksi) = 49.5Z_{req'd}$$

and;
$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \Rightarrow Z_{req'd} \geq \frac{9{,}027^{"k}}{49.5ksi} = 182 - in^3 \ (2.98X10^6 \ mm^3)$$

Check compact section criteria (per AISC seismic provisions):
Try W14X132 (W355.6mmX1.93KN/m);

Web local buckling;

$$P_y = A_g F_y = 32.0 - in^2(50ksi) = 1{,}600^k \ (7.12MN)$$

$$\frac{P_u}{\phi_b P_y} = \frac{38.3^k}{0.9(1{,}600^k)} = 0.027 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{38.3^k}{0.9(1{,}600^k)}\right] = 70.5$$

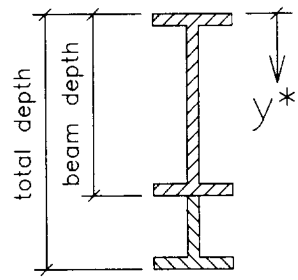
$$\lambda_{W14X132} = \frac{h}{t_w} = 17.7 < 70.5 = \lambda_p$$ **O.K.**

Flange local buckling;

$$\lambda_{W14X132} = \frac{b_f}{2t_f} = 7.1 < 7.35 = \frac{52}{\sqrt{50ksi}} = \frac{52}{\sqrt{F_y}} = \lambda_p$$ **O.K.**

**Choose; W14X132 (W355.6mm X 1.93KN/m), Z=234-in$^3$ (3.83X10$^6$ mm$^3$), I = 1530-in$^4$ (636.8X10$^6$mm$^4$)**

Haunch properties are calculated on spread sheet as follows;




Beam = W14X82

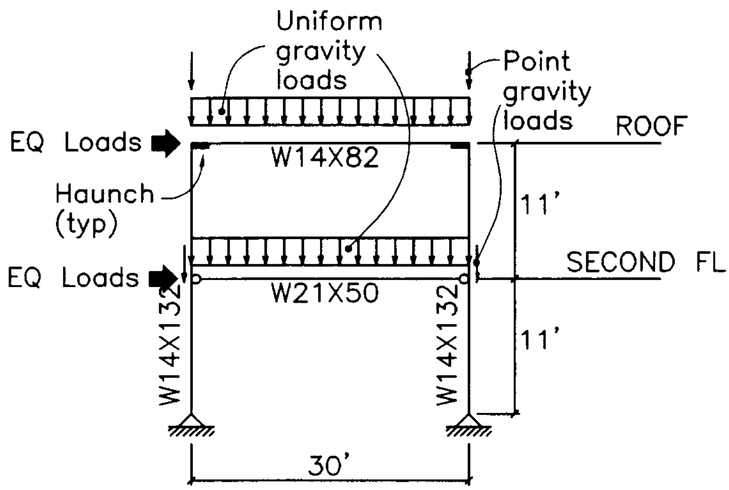
| | | | |
|---|---|---|---|
| Beam I (in) = | 882.00 | y* (in) = | 11.36 |
| Beam A (in$^2$) = | 24.10 | Haunch $I_x$ (in$^4$) = | 2218 |
| Beam depth (in) = | 14.31 | Total A (in$^2$) = | 35.90 |
| Flange Thickness (in) = | 0.855 | Haunch $I_y$ (in$^4$) = | 222.2 |
| Web Thickness (in) = | 0.510 | Haunch $S_x$ (in$^3$) = | 195 |
| Flange Width (in) = | 10.130 | Haunch $Z_x$ (in$^3$) = | 177.16 |
| Total Depth (in) = | 21.31 | | |
| Haunch Depth (in) = | 7.00 | | |

1-in = 25.4mm
1-in$^2$ = 645.2 mm$^2$
1-in$^3$ = 16.37X10$^3$ mm$^3$
1-in$^4$ = 416.2X10$^3$ mm$^4$

This frame was analyzed using RISA-2D. All load combinations were investigated to determine the worst case deflection. The controlling load combination was equation 4a; 1.314D+$Q_E$+0.5L. Because the members of the frame have changed, a check to ensure the location of the plastic hinge was made. The following frame and loading was analyzed;

Design Loads:
Roof:
$w_{LR}$ = 225plf (3.28KN/m)
$w_{DR}$ = 280plf (4.08KN/m)
$P_{RD}$ = 974-lb (4.33KN)
$E_R$ = 13.2$^k$ (58.7KN) (applied as a uniform load of 13.2$^k$/30' = 440plf (6.42KN/m) along the beam length)

Floor:
$w_{LF}$ = 600plf (8.75KN/m)
$w_{DF}$ = 1,272plf (18.55KN/m)
$P_{FD}$ = 649-lb (2.89KN)
$E_R$ = 10.2$^k$ (45.4KN) (applied as a uniform load of 10.2k/30' = 340plf (4.96KN/m) along the beam length)



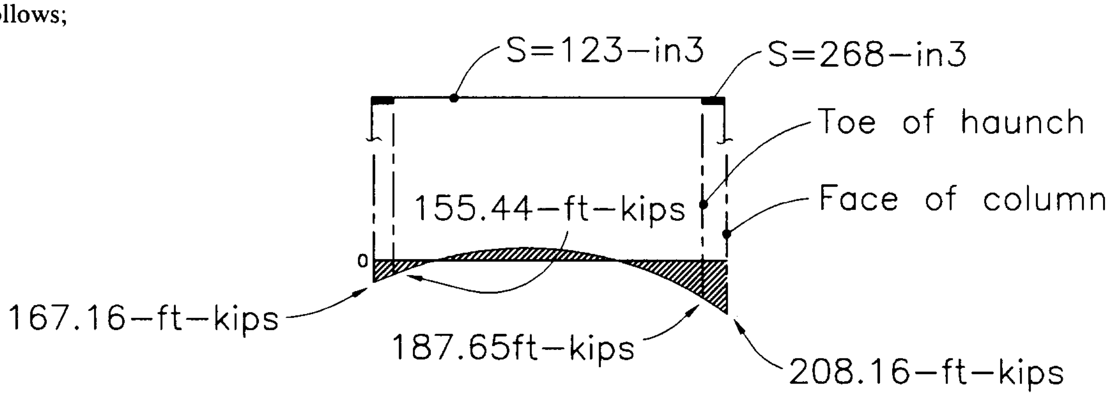

1-in = 25.4mm
1-ft = 0.3m
1plf = 14.58N/m

Drift requirements;

Calculated drift; $\delta_{calc}$ = 1.539" (39.1mm)

Therefore; $\delta_{calc}$ = 1.539" < $\Delta_{allow}$ = 1.98" (39.1mm < 50.3mm) **O.K.**

Check plastic hinge location;
The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;




1-in$^3$ = 16.4X10$^3$ mm$^3$
1-ft-kip = 1.356KN-m

By inspection of this diagram, it is clear that a plastic hinge will form on the right side of the beam where the moments are greatest (with increased loading, the moments will increase proportionately until yielding occurs).

The stress ratio on the right side is;

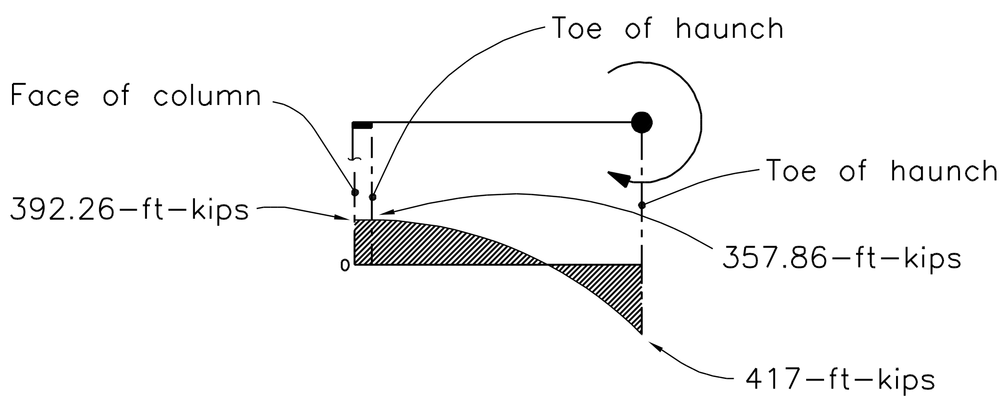
$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{\text{toe}} / S_{x,\text{toe}}}{M_{\text{face}} / S_{x,\text{haunch}}} = \frac{187.65^{\text{ft-kips}}(12''/1')/123-\text{in}^3}{208.16^{\text{ft-kips}}(12''/1')/268-\text{in}^3} = 1.96 > 1.2 \qquad \textbf{O.K.}$$

The left side was investigated by placing a plastic hinge at the assumed hinge location on the right side and analyzing the resulting configuration. The lateral load was increased until yielding occurred at the toe of the haunch on the left side.

$$M_p = Z_x F_y = 139-\text{in}^3(36\text{ksi}) = 417^{\text{ft-kips}} \text{ (565.5KN-m)}$$

$$M_y = S_x F_y = 123-\text{in}^3(36\text{ksi}) = 369^{\text{ft-kips}} \text{ (500.4KN-m)}$$

The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;




1-ft-kip = 1.356KN-m

The resulting stress ratio on the left side is;

$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{\text{toe}} / S_{x,\text{toe}}}{M_{\text{face}} / S_{x,\text{haunch}}} = \frac{357.86^{\text{ft-kips}}(12''/1')/123-\text{in}^3}{392.26^{\text{ft-kips}}(12''/1')/268-\text{in}^3} = 1.99 > 1.2 \qquad \textbf{O.K.}$$

Check unbraced length of the compression flanges (per AISC seismic requirements);

Try 5' on center (same spacing as the perpendicular floor joists);

$$L_b = 5' < 14.4' = \frac{2500(2.48'')(1'/12'')}{36\text{ksi}} = \frac{2500 r_y}{F_y} \quad (1.53\text{m} < 4.39\text{m}) \qquad \textbf{O.K.}$$

Note: By inspection, the high roof moment frame with a truss will be acceptable.

Braces:
Try TS 4.5x4.5x1/4 (TS 114.3mmX114.3mmX6.4mm)
Check DCR in compression;

$Q_{CE} = 1.25P_n = 1.25(34.1^k) = 42.6^k$ (189.5KN)

where; $P_n$ is calculated using AISC LRFD 2nd ed. column load tables as follows;
From the tables with KL = 1.0(25') = 25' (7.62m)
$P_n = 29^k/0.85 = 34.1^k$ (151.7KN)

Check DCR ratio in compression;

$$\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{42.6^k} = 0.71 < 0.8 = m$$ **O.K.**

**Use TS 4.5x4.5x1/4 (TS 114.3mmX114.3mmX6.4mm)**

*e. Design connections.*

General;

Governing design loads can be from either of ground motions A or B. Ground motion A consists of two thirds of the Maximum Considered Earthquake (MCE), includes R values, and was used in performance objective 1A. Ground motion B consists of three-quarters of the MCE, does not include R-values, and was used in performance objective 3B. Further, for force controlled components, the seismic load from ground motion B is divided by the modification factors $C_1$, $C_2$, $C_3$, and J to restore it to a force-controlled action. However, some elements of the connections may derive their design loads from the expected strength of other connection elements. For example, panel zone shear used to determine the need for doubler plates is limited by expected strength of the beams framing to the column flanges, and the design load for continuity plates is taken from the expected strength of the beam flanges. Additionally, brace frame connections may be designed for the expected strength of the brace.

Two example connection designs will be shown; a moment connection and a braced frame connection. Both connections are from the low roof portion of the building.

Low Roof;

Transverse direction;

Moment frames;

In designing the main elements of the moment frame (i.e., the beams and columns) most of the requirements of the AISC seismic provisions for moment connections had been met. What remains is to determine the need for doubler plates, the need for continuity plates, and the design of the bolted gravity load connection at the beam web.

Determine if doubler plates are required;

Determine Demand;

Note: Loading from performance objective 1A including the structural overstrength factor ($\Omega_\circ = 3$) could have been used to determine a smaller demand '$R_u$'. However, it is more convenient to apply the expected strength requirement of 0.8 times $\sum R_y M_p$ as follows;

$$R_u = \frac{0.8\sum R_y M_p}{d_b + d_{haunch} - t_f}$$ (per AISC seismic provisions dated April 15, 1997)

where; $R_y = 1.5$ (for ASTM A36)
$M_p = Z_x F_y = 43.24\text{-in}^3(36\text{ksi}) = 1{,}557^{\text{in-kips}}$ (175.9KN-m)
$d_b = 13.91"$ (353.3mm)

$d_{haunch}$ = 7.00” (177.8mm)
$t_f$ = 0.420” (10.7mm)

$$\therefore R_u = \frac{0.8(1.5(1{,}557^{"k}))}{13.91"+7.00"-0.42"} = 91.1^k \quad (405.2KN)$$

Determine Capacity;

$(R_v)_{W14X34} = 158^k$ (702.8KN) (calculated previously)

Check;

$f_v R_v = 0.75(158^k) = 119^k > 91.1^k = R_u$ (529.3KN > 405.2KN) **O.K.**

Check panel-zone thickness;

$t \geq (d_z + w_z)/90$ (EQ. 9-2 AISC Seismic provisions)

where; $d_z$ = panel zone depth between continuity plates (which includes haunch depth)
$d_z$ = 13.91”+7.00”-2(0.42”) = 20.1” (510.5mm)
$w_z = d_c - 2t_f$ = 13.98”-2(0.455”) = 13.1” (332.7mm)

$$\therefore t = 0.285" < 0.369" = \frac{20.1"+13.1"}{90} = \frac{d_z + w_z}{90} \quad (7.24mm < 9.37mm)$$ **N.G.**

Try a 3/8” (9.53mm) thick doubler plate (using ASTM A36);

Provide weld to match shear strength of the required thickness of doubler plate (use E70XX electrode);

Shear capacity of doubler plate = $f F_{BM} A_g = f F_{BM} t_d b_d$

where; $F_{BM}$ = nominal shear capacity of base metal = $0.6F_y$
$A_g$ = gross area of the doubler plate
$t_d$ = thickness of doubler plate
$b_d$ = width of doubler plate

Shear capacity of welds = $f F_w (0.707s) b_d$

Where; $F_W$ = nominal shear capacity of welds = $0.6F_{EXX}$
s = weld leg length

Therefore; $f F_{BM} t_d b_d = f F_W (0.707s) b_d$

or $$s = \frac{f F_{BM} t_d}{0.707(f F_W)} = \frac{0.9(0.6(36ksi))0.369"}{0.707(31.5ksi)} = 0.322" \quad (8.18mm)$$

From AISC J2.2b, the maximum weld size is;

$$s = t_d - \frac{1"}{16} = \frac{3"}{8} - \frac{1"}{16} = 0.313" \quad (7.95mm)$$ **N.G.**

Try a 1/2” (12.7mm) thick doubler plate with a 3/8” (9.53mm) weld;

For a 1/2” (12.7mm) thick doubler plate; s = 0.438” (11.13mm) **O.K.**

From AISC table J2.4, the minimum weld size is; s = 3/16” < 3/8” (4.76mm < 9.53mm) **O.K.**

**Choose 1/2" (12.7mm) doubler plate secured with a 3/8" (9.53mm) fillet weld min**

Determine if continuity plates are required;

Determine demand;

$$P_{bf} = A_f F_{ye} = A_f R_y F_y = 0.420"(5.025")1.5(36ksi) = 114^k \quad (507.1KN)$$

Determine capacity;

$$\phi R_n = \phi[(2.5k + N)F_{yw} t_w + A_{st} F_{yst}]$$

where; $k = 1.0"$ (25.4mm) distance from outer surface to toe of fillet for the column

$N = t_{bf} = 0.420"$ (10.67mm) thickness of the beam flange

$F_{yw} = 50ksi$ (344.8MPa) yield strength of the column web

$t_w = 0.285"$ (7.24mm) thickness of column web

$A_{st} =$ area of the stiffner

$F_{yst} = 36ksi$ (248.2MPa) yield strength of the stiffener

$\phi = 1.0$

$$A_{st} = \frac{P_{bf} - (2.5k + N)F_{yw} t_w}{F_{yst}} = \frac{114^k - (2.5(1.0") + 0.420")50ksi(0.285")}{36ksi} = 2.01 - in^2 > 0$$

($A_{st} = 1.30X10^3\ mm^2$)

Therefore, stiffeners with a total area of at least 2.01-in$^2$ (1.30X10$^3$ mm$^2$) total are required.

Design stiffeners in accordance with AISC LRFD 2$^{nd}$ ed. section K.9;

$$b_{st} + \frac{t_{cw}}{2} \geq \frac{b_b}{3} \Rightarrow b_{st} \geq \frac{b_b}{3} - \frac{t_{cw}}{2}$$

where; $b_{st} =$ width of a single stiffener

$b_b =$ width of the beam flange

$t_{cw} =$ thickness of the column web

$$b_{st} = \frac{b_b}{2} - \frac{t_{cw}}{2} = \frac{5.025"}{2} - \frac{0.285"}{2} = 2.37" \quad (60.2mm),$$ Say two stiffeners with $b_{st} = 3.0"$ (76.2mm)

$$t_{st} \geq \frac{t_{bf}}{2}$$ where; $t_{st} =$ thickness of a single stiffener

$t_{bf} =$ thickness of the beam flange

use; $t_{st} = 0.375" > 0.210" = \frac{0.420"}{2} = \frac{t_b}{2}$, $(A_{st})_{total} = 0.375"(3.0")2 = 2.25\text{-}in^2 > 2.01\text{-}in^2$ **O.K.**

($t_{st} = 9.53mm > 5.33mm$) $(A_{st})total = 1.45X10^3\ mm^2 > 1.30X10^3\ mm^2$)

Check local buckling of the stiffeners;

$$\frac{b_{st}}{t_{st}} = \frac{3.0"}{0.375"} = 8.0 < 15.83 = \frac{95}{\sqrt{36ksi}} = \frac{95}{\sqrt{F_{y,st}}}$$ **O.K.**

where; $F_{y,st}$ = yield strength of the stiffener

Design welds for the stiffeners;

Stiffener to column web;

Minimum weld size = 3/16" (4.76mm) (per AISC LRFD 2$^{nd}$ ed. table J2.4, based on stiffener thickness)

Size required for strength;

Force to be resisted by a stiffener is;

$$F = P_{bf} - (2.5k + N)F_{yw}t_w = 114^k - (2.5(1.0") + 0.420")50ksi(0.285") = 72.4^k \quad \text{with } N = t_{bf}$$

(F = 322.0KN)

Length available for welding stiffeners to the column web is;

L = 12” (assuming a 1/2” chamfer) x 2 sides x 2 stiffeners = 48” (1.22m)

The required weld size is;

Note; For E70XX electrodes; $fF_w = 0.75[0.60(70ksi)] = 31.5ksi$ (265.0MPa)

$$s = \frac{R_u}{(0.707)L(fF_w)} = \frac{72.4^k}{0.707(48")31.5ksi} = 0.068" < \frac{3"}{16} \quad (1.72mm < 4.76mm)$$

Therefore, the minimum weld size governs

Check shear strength of the base metal;

$$fR_n = f(0.6t_{st}F_{st})x2 = 0.9(0.6)0.375"(36ksi)2 = 14.6 - kips/in \quad (2.56KN/mm)$$

$$fR_n = 14.6 - kips/in > 3.02 - kips/in = \frac{72.4^k}{48"/2} = \frac{F}{L/2} \quad (2.56KN/m > 0.53KN/m) \qquad \textbf{O.K.}$$

Stiffener to column flange; Use full penetration groove welds

**Choose two 3/8” (9.53mm) thick by 3.0” (76.2mm) wide stiffeners with a 3/16” (4.76mm) weld at the column web and a full pen-etration groove weld at the column flange**

Design the single-plate web connection;

Note: The governing load combination (1.2D+0.5L+1.6$L_r$) is based solely on gravity loads.

$w_u$ = 1.2D+0.5L+1.6$L_r$ = 1.2(343plf)+0.5(0)+1.6(240plf) = 796plf (11.6KN/m)

$$V_u = \frac{w_u(L - d_c)}{2} = \frac{796plf(30' - 13.98"(1'/12"))}{2} = 11.5^k \quad (51.2KN)$$

Try a 3/8” (9.53mm) plate;

Determine the number of 3/4” (19.05mm) diameter A325-N bolts required for shear;

From AISC LRFD 2nd ed. table 8-11;

$$n_{min} = \frac{R_u}{fr_n} = \frac{11.5^k}{15.9 - kips/bolt} = 0.72 \text{ bolts, Say 2 bolts}$$

Determine the number of 3/4” (19.05mm) diameter A325-N bolts required for bearing, assuming $L_e$ = 1-1/2” (38.1mm), and s = 3” (76.2mm). The .255” (6.48mm) beam web is more critical then the 3/8” (9.53mm).

From AISC LRFD 2nd ed. Table 8-13;

$$n_{min} = \frac{R_u}{\phi r_n} = \left(\frac{11.5^k}{78.3 - \text{kips/bolt} - \text{in}}\right)\frac{1}{0.255"} = 0.58 \text{ bolts} < 2 \text{ bolts}$$ **O.K.**

Check shear yielding of the plate;

$$\phi R_n = 0.9(0.6F_yA_g) = 0.9[0.6(36\text{ksi})6"(0.375")] = 43.7^k > 11.5^k$$ **O.K.**

(193.4KN > 51.2KN)

Check shear rupture of the plate;

$$\phi R_n = 0.75(0.6F_uA_n) = 0.75\left[0.6(58\text{ksi})\left(6"-2\left(\frac{3"}{4}+\frac{1"}{8}\right)\right)\frac{3"}{8}\right] = 41.6^k > 11.5^k$$ **O.K.**

(185.0KN > 51.2KN)

Check block shear rupture of the plate;

From AISC LRFD 2nd ed. table 8-47a, using $L_{eh}$ = 1.5" (38.1mm);

$$\frac{\phi[F_uA_{nt}]}{t} = 46.2 - \text{kips/in} \quad (8.09\text{KN/m})$$

From AISC LRFD 2nd ed. table 8-48a, using $L_{ev}$ = 1.5" (38.1mm);

$$\frac{\phi[0.6F_uA_{nv}]}{t} = 83 - \text{kips/in} \quad (14.53\text{KN/mm})$$

Therefore, $0.6F_uA_{nv} > F_uA_{nt}$. Thus, from AISC LRFD tables 8-48a and 8-48b;

$$\phi R_n = \phi[0.6F_uA_{nv} + F_yA_{gt}] = (83 - \text{kips/in} + 40.5 - \text{kips/in})0.375" = 46.3^k > 11.5^k$$ **O.K.**

(205.9KN > 51.2KN)

Determine required weld size for fillet welds to supporting column flange;

From AISC LRFD 2nd ed. table J2.4, since the column flange thickness $t_f$ = 0.455" > 1/4" (11.56mm > 6.35mm). The minimum fillet weld size is 3/16" (4.76mm). The length available for welding the plate to the column flange is;

L = 6" x 2 sides = 12" (304.8mm)

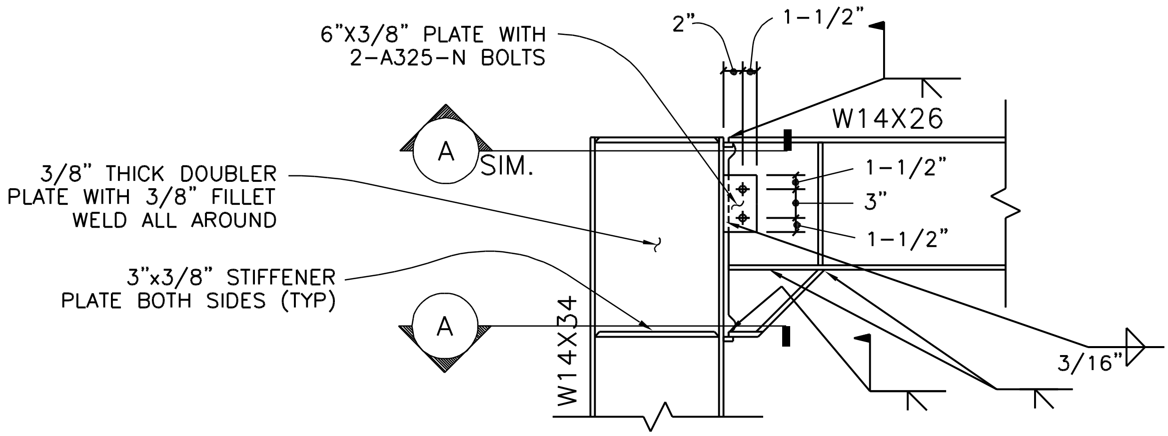
The required weld size 's' is;

$$s = \frac{F}{(0.707)L(\phi F_W)} = \frac{11.5^k}{0.707(12")31.5\text{ksi}} = 0.043" < \frac{3"}{16} \quad (1.09\text{mm} < 4.76\text{mm})$$

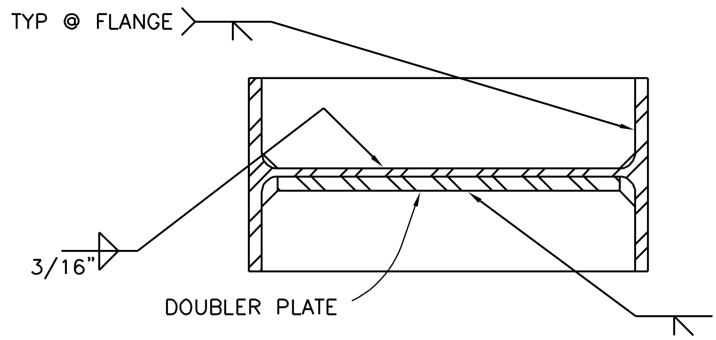
Therefore, the minimum weld size governs.

**Choose 6"x3/8" (152.4mmx9.53mm) plate with two 3/16" (4.76mm) welds to the column flange and two 3/4" (19.05mm) A325-N bolts at the beam web**

The final design of the low roof moment connection is as follows;




MOMENT FRAME DETAIL FOR LOW ROOF




A CONTINUITY PLATE DETAIL

1-in = 25.4mm
1plf = 14.58n/m

Longitudinal direction;

Braced frames;

A typical brace connection located at the foot of the column will be designed;

Determine demand;

It is decided to design the brace connection using the maximum load that can be delivered to it by the brace;

Tension; $R_u = R_yF_yA_g = 1.5(46\text{ksi})4.0\ 9\text{-in}^2 = 282^k$ (1.25MN)

Compression; $R_u = F_{cr}A_g = \frac{\phi_c P_n}{\phi_c} = \frac{29^k}{0.85} = 34.1^k$ (151.7MN)

Try a 3/8" (9.53mm) thick gusset plate;

Design welds to gusset plate; Use E70XX electrodes and ASTM A36 for the gusset plate

Minimum weld size = 3/16" (4.76mm) (per AISC LRFD 2nd ed. table J2.4, based on gusset plate thickness)

Maximum weld size = 3/16" (4.76mm) (per AISC LRFD 2nd ed. J2.2b.(b), based on brace thickness)

Required length of four 3/16" (4.76mm) welds;

$$l_w = \frac{R_u}{4(0.707)s(\phi F_w)} = \frac{282^k}{4(0.707)0.188"(31.5ksi)} = 16.8-\text{in} \quad \text{Say 17-in (431.8mm)}$$

Check base metal;

$$\phi R_n = \phi(0.6F_y)A_g = 0.9(0.6(46ksi)4(17")0.25" = 373^k > 282^k \qquad \textbf{O.K.}$$

(1.66MN > 1.25MN)

Check shear/tension rupture;

$$\phi R_n = \phi t\left(0.6F_y\left(\frac{2l_w}{\cos 30^\circ}\right) + F_u w_1\right)$$

where; $w_1 = 4.5"+2l_w(\tan 30^\circ) = 4.5"+2(17")0.577 = 24.1"$ (612.1mm)

$$\therefore \phi R_n = 0.75(0.375")\left(0.6(36ksi)\left(\frac{2(17")}{0.866}\right) + 58ksi(24.1")\right) = 632^k < 282^k \qquad \textbf{O.K.}$$

(2.81MN < 1.25MN)

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

$$\phi R_n = 0.75(0.375")(0.6(58ksi)2(17") + 36ksi(4.5")) = 378^k > 282^k \qquad \textbf{O.K.}$$

(1.68MN > 1.25MN)

Check tensile capacity;

$$\phi R_n = \phi w_1 F_u t = 0.75(24.1")58ksi(0.375") = 348^k > 282^k \qquad \textbf{O.K.}$$

(1.55MN > 1.25MN)

Check compressive capacity;

$$\phi R_n = \phi F_y w_1 t = 0.9(36ksi)24.1"(0.375") = 293^k > 34.1^k \qquad \textbf{O.K.}$$

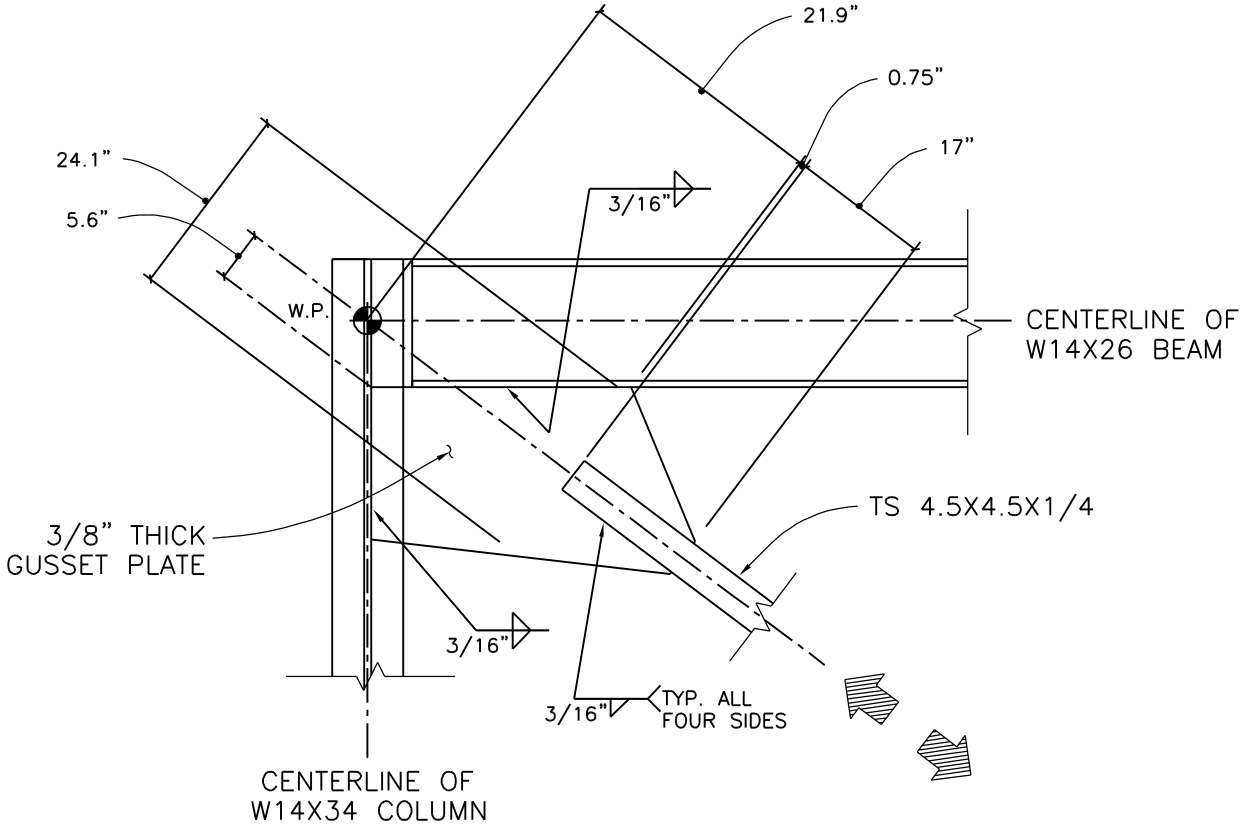
(1.30MN > 0.15MN)

Check buckling of the gusset plate;

$$\phi R_n = \phi\left(\frac{4000t^3\sqrt{F_y}}{l_1}\right) = 51.9^k > 34.1^k \qquad (230.9KN > 157.7KN) \qquad \textbf{O.K.}$$

**Choose four 3/16" x 17" (4.76mmx431.8mm) long fillet welds with a 3/8" (9.53mm) thick gusset plate**

The final design of the low roof brace frame connection at the roof is as follows;




1-in = 25.4mm
1plf = 14.58N/m