H-4 FIRE STATION

a. Introduction

This design example illustrates the seismic design of a two-story fire station. The step by step procedure as shown in Tables 4-5 and 4-6 was followed almost verbatim for the design of the buildings basic structural elements. This rigid adherence to the outlined procedures was done in order to provide a clear demonstration of the use of this manual.

(1) Purpose. The objective of this problem is to demonstrate the procedure to be used for designing a building with an enhanced performance objective.

(2) Scope. The scope of this example problem includes; the design of all major structural steel members such as beams, columns, and braces, as well as the design of several example structural steel connections. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as concrete floor slabs were not considered part of the scope of this problem and are therefore not included. Additionally, this problem considers only seismic and gravity loads.

b. Building Description

(1) Function. This building functions as a fire station, and provides living quarters to station personnel as well as garage space for equipment such as fire engines.

(2) Seismic Use Group. As a fire station, this building will be required for post-earthquake recovery and as such performs a mission essential function. Therefore, this building is categorized with a seismic use group of IIIE, Essential Facilities. With the Seismic Use Group known, the structural system performance objectives are obtained from Table 4-4. Structures in seismic use group IIIE are to be designed for performance level 3; Immediate Occupancy. Ground motion B (3/4 MCE) is to be used for performance objective 3B. The minimum analysis procedure to be used is the liner elastic with R factors and linear elastic with m factors. The structure is designed first for performance objective 1A following the steps laid out in Table 4-5. After completion of the preliminary design, the enhanced performance objectives outlined in Table 4-6 for performance objective 3B are checked and the building design is updated accordingly to meet those objectives.

(3) Configuration. As shown in Figure 1, the building is rectangular in plan measuring 70 feet (21.35m) by 30 feet (9.15m). It contains a one-story low roof garage area that is connected to an adjacent two-story high roof office area and dormitory. Story height of the low roof area is 15 feet (4.58m), and of the high roof area is 11 feet (3.36m).

(4) Structural Systems. The building consists primarily of steel frame construction composed of wide flange shapes, hollow structural sections, and metal decking. However, the second floor incorporates a reinforced concrete slab. Structural systems are shown in Figure 1.

The gravity load resisting system consists of untopped metal decking that spans to open web steel joists, which span to wide flange steel beams and columns. The second floor consists of a reinforced concrete slab that spans between wide flange beams, which are supported by the same columns that support the roof decking.

The lateral load resisting system consists of both flexible and rigid diaphragms that span between steel moment frames in the transverse direction, and steel braced frames in the longitudinal direction. Roof diaphragms consisting of flexible untopped metal decking that place tributary load on the lateral load resisting system. The second floor diaphragm, however, consists of a rigid concrete slab for which torsion must be considered.

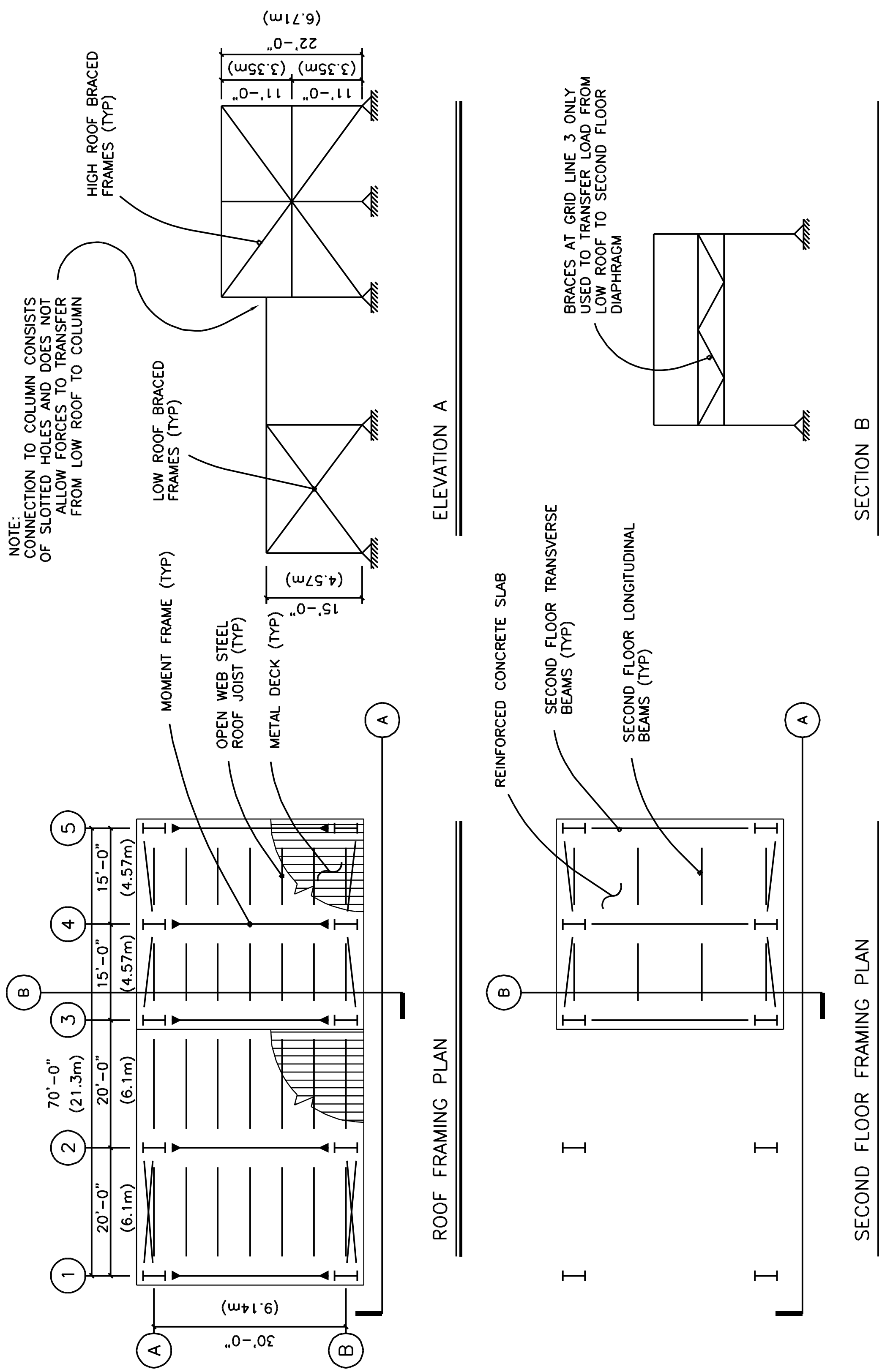


Figure. 1 Building Plan Layout

The high roof and low roof structures share a common transverse moment frame at their interface. However, in the longitudinal direction, the low roof structure is essentially isolated from the high roof structure at this location. This is accomplished by the use of a simple gravity connection consisting of elongated slotted holes that attach the low roof support beam to the moment frame column.

The building is considered regular both in plan and vertically.

(5) Choice of materials. Columns shall be designed using ASTM A572 Grade 50 and braces using ASTM A24 Grade 46. All other steel components will use ASTM A36 Grade 36. All walls not shown on the floor framing plans are intended to be nonstructural and shall be constructed so as to not impair the response of the steel moment frames and steel braced frames.

c. Preliminary building design (following steps in Table 4-5 for Life Safety). The preliminary design of the building follows the steps outlined in Table 4-5 for Performance Objective 1A. The design is then updated to meet the enhanced performance objectives laid out in Table 4-6 for Performance Objective 2A.

A-1 *Determine appropriate Seismic Use Group.* Per Table 4-3 of the manual, the building must be safe to occupy immediately after an earthquake and is required for post earthquake recovery. Therefore, it is an essential structure with a Seismic Use Group of IIIE.

A-2 *Determine Site Seismicity.* The site seismicity for this example from the MCE maps is:
$S_S = 0.80g$, and $S_1 = 0.40g$.

A-3 *Determine Site Characteristics.* For the purpose of this problem, a very dense soil and soft rock condition was assumed corresponding with a site classification of 'Class C' per Table 3-1 of the TI manual.

A-4 *Determine Site Coefficients, F_a and F_v.* From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics, the site response coefficients were interpolated as follows:

$F_a = 1.08$ (Table 3-2a)
$F_v = 1.40$ (Table 3-2b)

A-5 *Determine adjusted MCE Spectral Response Accelerations:*

$S_{MS} = F_a(S_S) = 1.08(0.80) = 0.86$ (EQ. 3-1)
$S_{M1} = F_v(S_1) = 1.40(0.40) = 0.56$ (EQ. 3-2)

A-6 *Determine Design Spectral Response Accelerations:*

$S_{DS} = (2/3)S_{MS} = (2/3)0.86 = 0.57$ (EQ. 3-3)
$S_{D1} = (2/3)S_{M1} = (2/3)0.56 = 0.37$ (EQ. 3-4)

The approximate period of the structure (based on T = 0.1N, where N = number of stories) is:

$T_{approx} = 0.1N = 0.1(2) = .2 < .5$ (EQ. 5.3.3.1-2 FEMA 302)

Since $T_{approx} = .2 < .5$, and because the structure is less then 5 stories is height; equations 3-5 and 3-6 of the TI 809-04 manual must be checked in the short period range:

$S_{DS} \leq 1.5F_a$ (EQ. 3-5)
$S_{D1} \leq 0.6F_v$ (EQ. 3-6)

Therefore, $S_{DS} = 0.57 < 1.5(1.08) = 1.62 = 1.5F_a$
$S_{D1} = 0.37 < 0.6(1.40) = 0.84 = 0.6F_v$

Use $S_{DS} = 0.57$, & $S_{D1} = 0.37$

A-7 *Determine Seismic Design Category*. With S_{DS} = 0.57, S_{D1} = 0.37, and a Seismic Use Category of IIIE, enter tables 4-2a and 4-2b to obtain a Seismic Design Category of 'D'.

A-8 *Select Structural System.*

Gravity: Steel frame with metal deck roof and reinforced concrete second floor slab. Roof deck spans between interior open web steel joists and perimeter edge beams. Open web steel joists span to transverse beams. Perimeter edge beams and interior transverse beams span between columns. Second floor slab spans between interior beams and perimeter edge beams. Second floor beams span to girders that span between columns.

Lateral: As permitted by Table 7-1:

Transverse direction: Special steel concentrically braced frames.
Longitudinal direction: Special steel moment frames.

A-9 *Select R, Ω_0, & C_d factors.*

From Table 7-1:

Transverse direction - Special Steel Moment Frames	**R = 8, Ω_0 = 3, C_d = 5.5**
Longitudinal direction – Special Steel Concentrically Braced Frames	**R = 6, Ω_0 = 2, C_d = 5.0**

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

(1) Roof Decking.

Live Load (per Table 4-1 of ANSI/ASCE 7-95):		20	psf
Dead Load (estimated):	Roofing;	5	psf
	Rigid Insulation;	3	psf
	Decking;	2	psf
	Misc. (Mechanical & Electrical);	3	psf
	Total =	13	psf (0.62KN/m^2)

Total Load: Dead + Live = 20 + 13 = 33 psf (1.58KN/m^2)

Per steel deck manufacturers catalog with 3 or more 5-ft (1.53m) spans;

Choose 1-1/2" (38.1mm)-22 gage HSB36

(2) Roof Joists.

Loading: 33 psf (1.58KN/m^2) + self weight
Assume self weight $\approx$ 5 plf (0.073KN/m), and a spacing of 4-ft (1.22m) on center
Loading = 33psf(4')+5plf
= 137plf (2.00KN/m)

Per steel joist manufacturers catalog with span = 20' (6.10m), a total load = 137plf (2.00KN/m), and a live load = 80plf (1.17KN/m);

Choose 10K1 Joists @ 4-ft (1.22m) o.c.

(3) Longitudinal Roof Edge Beams. Use F_y = 36ksi. One design will be produced for the worst case and used throughout. The beam must have a minimum depth of 12" (304.8mm) to ensure installation of 3 bolts per connection in anticipation of chord/collector forces. Worst cases situation occurs at the low roof that has the longest span and largest tributary area. Also, all beams are simply supported.

Determine Design Loads;
Tributary area "A_T" = 2.5'(20') = 50-ft^2 (4.65m^2)
Dead Load = 13psf(2.5')+(estimated self wt. of 25plf)
= 57.5 plf (0.84KN/m)
Note: Due to the small tributary area, live load reduction (as per ANSI/ASCE 7-95) is not permitted.
Live Load = 2.5'(20psf)
= 50plf (0.73KN/m)
Strength requirements;
Note: Beam has continuous lateral support from attached roof deck
W_u = 1.2(57.5plf)+1.6(50plf) = 149plf (2.17KN/m)
$M_u = w_u L^2/8$ = 149plf(20')^2/8 = 7.45$^{\text{ft-kips}}$ (10.10KN-m)

Deflection requirements;

$$\Delta_{allow} = \frac{L}{240} = \frac{20'(12''/1')}{240} = 1.0'' \text{ (25.4mm)}$$ (Per ANSI/ASCE 7-95 section B.1.1)

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} = \frac{5(0.05\text{klf})(20')^4(12''/1')^3}{384(29{,}000\text{ksi})1.0''} = 6.2-\text{in}^4 \text{ (2.5X10}^6\text{ mm}^4)$$

By inspection, a W12X14 (W304.8mm X 0.20KN/m) will work due to the light loading condition

Choose W12X14 (W304.8mm X 0.20KN/m)

(4) Transverse Beams. Transverse beams are part of the moment frames. As a first approximation, it will conservatively be assumed that the beams are simply supported. Over designing for gravity loads is not expected to produce overly conservative results considering that drift due to seismic loads will probably govern the final design. One design will be produced for the worst case gravity load and used throughout. The worst case situation occurs at the low roof interior beam, which has the largest tributary area.

Determine Design Loads;
Tributary area "A_T" = 20'(30') = 600-ft^2 (55.7m^2)
Dead Load = 20psf (0.96KN/m^2) (assumed)
Live Load Reduction per ANSI/ASCE 7-95;
A_T = 600-ft^2 $\Rightarrow R_1 = 0.6$
Roof Slope < 4:12 $\Rightarrow R_2 = 1.0$
$\therefore$ Reduced Live Load = 20psf(0.6)(1.0) = 12psf (0.57KN/m^2)
$w_U = 1.2w_D+1.6w_L$ = [1.2(20psf)+1.6(12psf)]20' = 864plf (12.6KN/m)

Strength requirements; use F_y = 36ksi (248.2MPa)

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} \quad \text{with} \quad M_u = \frac{w_u L^2}{8}$$

$$\geq \frac{864\text{plf}(30')^2(12''/1')}{8(0.9)36{,}000\text{psi}} = 36-\text{in}^3 \text{ (589.9X10}^3\text{ mm}^3)$$

$$f_v V_n \geq \frac{w_u L}{2} = \frac{864\text{plf}(30')}{2} = 13^k \text{ (57.8KN)}$$

Deflection requirements;

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} \quad \text{with} \quad \Delta_{allow} = \frac{L}{240} = \frac{30'(12''/1')}{240} = 1.5'' \text{ (38.1mm)}$$

$$\geq \frac{5(0.24\text{klf})(30')^4(12''/1')^3}{384(29{,}000\text{ksi})(1.5'')} = 101-\text{in}^4 \text{ (42.0X10}^6\text{ mm}^4)$$

Compact section criteria (per AISC seismic provisions);
Try W14X26 (W355.6mm X 0.38KN/m);

$$\frac{b_f}{2t_f} = 6.0 < 8.7 = \frac{52}{\sqrt{36\text{ksi}}} = \frac{52}{\sqrt{F_y}}$$ **O.K.**

Choose W14X26 (W355.6mm X 0.38KN/m), Z = 40.2-in^3 (658.8X10^3 mm^3), f_vV_n = 69-kip (306.9KN), I = 245-in^4 (102X10^6 mm^4)

(5) Columns. Columns are sized based on the strong column /weak beam requirements of the AISC Seismic Provisions for Structural Steel Buildings (dated April 15, 1997) herein referred to as the AISC seismic provisions.

Using ASTM A572 Grade 50;

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

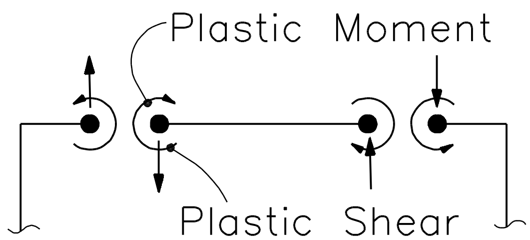
$$\sum M^*_{pb} = \sum (1.1R_yM_p + M_v)$$

where; R_y = 1.5

M_p = Plastic Moment = ZF_y = 40.2-in^3(36ksi)

$= 1{,}447^{\text{in-kips}}$ (163.5KN-m)

M_v = Additional moment due to offset of the plastic hinge from the column centerline (this is equal to the plastic shear 'V_p' times the offset distance). M_p is determined using the following free body diagram;



Note; The offset from the column centerline is determined by choosing the column depth approximately equal to the beam depth, the haunch length to be 3/4 of the beam depth, and as recommended in FEMA 267, by noting that the plastic hinge will occur another 1/3 of the beam depth beyond the toe of the haunch.

Therefore; Offset distance 'x' $= \frac{d_c}{2} + \frac{3d_b}{4} + \frac{d_b}{3} \approx 1.6d_b$

= 1.6(13.91")

= 22.25" or 1.85' (0.56m)

$$V_p = \frac{2M_p}{(L-2x)} = \frac{2(1{,}447^{\text{in-kips}})}{(30'-2(1.85'))(12''/1')} = 9.17^k \text{ (40.8KN)}$$

$$\therefore M_v = V_p(1.85') = 9.17^k(22.25'') = 204^{\text{in-kips}} \text{ (23.1KN-m)}$$

Therefore; $$\sum M^*_{pb} = 1.1(1.5)1{,}447^{\text{in-kips}} + 204^{\text{in-kips}} = 2{,}591^{\text{in-kips}} \text{ (292.8KN-m)}$$

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

where; $P_{uc} \approx$ 1.2(20psf)20'(15')(1^k/1000^{lb}) = 7.2^k (32.0KN)

$A_g \approx$ 10-in^2 (6.45X10^3 mm^2) (assumed)

$\therefore$ $P_{uc}/A_g \approx$ 0.7ksi (4.83MPa)

Therefore; $$\sum M^*_{pc} = Z_{req'd}(50\text{ksi} - 0.7\text{ksi}) = 49.3Z_{req'd}$$

and; $$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \Rightarrow Z_{req'd} \geq \frac{2{,}591^{"k}}{49.3\text{ksi}} = 53-\text{in}^3 \ \ (868.5\text{X}10^3\ \text{mm}^3)$$

Note: By inspection, shear does not govern.

Choose; W14X34 (W355.6 X 0.50KN/m), Z=54.6-in^3 ((894.7X10^3 mm^3), I = 340-in^4 (141.5X10^6 mm^4)

(6) Second Floor Slab. Per Table 9.5(a) of ACI 318-95 the minimum thickness of a one way slab when deflections are not computed is;

l/24	(one end continuous)	(governs)
l/28	(both ends continuous)	

Therefore, for a 10-ft (3.05m) span; 10'(12"/1')/24 = 5" (127.0mm)

Choose a 5" (127.0mm) thick reinforced concrete slab

(7) Second Floor Longitudinal Beams. Use F_y = 36ksi (248.2MPa)

Strength requirements;

Note: Beam has continuous lateral support from attached roof deck.

$w_u = 1.2w_{DL}+1.6w_{LL}$

where; w_{DL} = (5/12)'(150pcf)10'+10psf(10')+(1psf+3psf+1psf)10'

Partition wt.
Beam Spacing
Floor finish wt.
Suspended ceiling wt.
Miscellaneous wt.

w_{DL} = 775plf (11.30KN/m)

w_{LL} = 40psf(10') = 400plf (5.83KN/m) (Per ANSI/ASCE 7-95 Table 4-1 Residential)

w_u = 1.2(0.775klf)+1.6(0.400klf) = 1.57klf (22.90KN/m)

$$M_u = \frac{1.57\text{klf}(15')^2}{8} = 44.2^{\text{ft-kips}} = 530^{\text{in-kips}} \ \ (59.0\text{KN/m})$$

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} = \frac{530^{\text{in-kips}}(1000^{\text{lb/k}})}{0.9(36{,}000\text{psi})} = 16.4-\text{in}^3 \ \ (268.7\text{X}10^3\ \text{mm}^3)$$

$$f_v V_n \geq \frac{w_u L}{2} = \frac{1{,}570\text{plf}(15')}{2} = 12^k \ \ (53.4\text{KN})$$

Deflection requirements;

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} = \frac{5(0.40\text{klf})(15')^4(12"/1')^3}{384(29{,}000\text{ksi})0.75"} = 20.95-\text{in}^4 \ \ (8.72\text{X}10^6\ \text{mm}^4)$$

Choose W10X17 (W254mm X 0.25KN/m), Z = 18.7-in^3 (306.4X10^3 mm^3), I = 81.9-in^4 (34.1X10^6 mm^4), F $_v$V$_n$ =47.2-kip (209.9KN)

(8) Transverse Second Floor Beams. Use F_y = 36ksi (248.2MPa). Produce one design for the worst case and use throughout. Worst case situation occurs at the interior bay.

Strength requirements;

Note: Beam has continuous lateral support from attached slab.

$w_u = 1.2w_{DL}+1.6w_{LL}$

where; w_{DL} = [(5/12)'(150pcf)+17plf(1/10')+12psf]15'+30plf

Self wt.
Tributary width
Concrete floor slab wt.
Partitions & Misc. wts.
Long beam wts.

w_{DL} = 1,173plf (17.11KN/m)
w_{LL} = 40psf(15') = 600plf (8.75KN/m)

Note: Floor live load reduction, per ANSI/ASCE 7-95, is not permitted because this is a one way slab and the floor live load = 40psf < 100psf (1.92KN/m^2 < 4.79KN/m^2).

w_u = 1.2(1.218klf)+1.6(0.60klf) = 2.42klf (35.29KN/m)

$$M_u = \frac{2.42\text{klf}(30')^2}{8} = 272^{\text{ft-kips}} = 3264^{\text{in-kips}} \ \ (368.8\text{KN-m})$$

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} = \frac{3264^{\text{in-kips}}(1000^{\text{lb/k}})}{0.9(36{,}000\text{psi})} = 101-\text{in}^3 \ \ (1.66\text{X}10^6\ \text{mm}^3)$$

$$f_v V_n \geq \frac{w_U L}{2} = \frac{2{,}420\text{plf}(30')}{2} = 36.3^k \ \ (161.5\text{KN})$$

Deflection requirements;

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} = \frac{5(0.60\text{klf})(30')^4(12''/1')^3}{384(29{,}000\text{ksi})1.5''} = 251.4-\text{in}^4 \ \ (104.6\text{X}10^6\ \text{mm}^4)$$

Choose W21X50 (W533.4mm X 0.73KN/m), Z = 110-in^3 (1.80X10^6 mm^3), I = 984-in^4 (410X10^6 mm^4), $f_v V_n$ = 154-kip

d. Equivalent Lateral Force Procedure

B-1 *Calculate Fundamental Period, T:*

$$T_a = C_t h_n^{3/4} \qquad \text{(EQ. 5.3.3.1-1 FEMA 302)}$$

Transverse direction;	C_t = 0.035	Moment frame resisting 100% of the seismic force.
Longitudinal direction;	C_t = 0.020	Braced frame system.
	h_n = 22-ft (6.71m)	Height to highest level.

Therefore;

$T_a = 0.035(22')^{3/4} = 0.36\,\text{sec}$ transverse

$T_a = 0.020(22')^{3/4} = 0.20\,\text{sec}$ longitudinal

B-2 *Determine Dead Load,'W':*

Building weights were calculated on spread sheet and are shown in Figures 2 and 3. Total seismic weight is;

W = 129-kips (574KN)

B-3 *Calculate Base Shear, V:*

$$V = C_s W \qquad \text{(EQ. 5.3.2 FEMA 302)}$$

where; $$C_s = \frac{S_{DS}}{R} \qquad \text{(EQ. 3-7)}$$

$$C_s < \frac{S_{D1}}{TR} \qquad \text{(EQ. 3-8)}$$

$$C_s > 0.044 S_{DS} \qquad \text{(EQ. 3-9)}$$

Longitudinal direction;

R = 6 (Table 7-1)

C_s = 0.57/6 = 0.095 < 0.37/{0.2(0.6)} = 0.31
> 0.044(0.57) = 0.025

Transverse direction;

R = 8 (Table 7-1)

$C_s = 0.57/8 = 0.071$ $\quad < 0.37/\{0.36(8)\} = 0.13$
$\quad > 0.044(0.57) = 0.025$

Therefore; $\mathbf{C_{s,long} W = 0.095(129^k) = 12.2^k}$ **(54.3KN),** $\mathbf{C_{s,trans} W = 0.071(129^k) = 9.1^k}$ **(40.5KN)**

B-4 *Calculate Vertical Distribution of Forces.*

Note: The building will behave as two separate structures. In the transverse direction, the single story structure will distribute some tributary loads to the common lateral load resisting moment frame at the interface with the two-story structure, but will otherwise behave independently in this direction. In the longitudinal direction, the single story and two story structures are completely independent due to the elongated slotted holes in their adjoining connection. Therefore the two structures will be analyzed independently of each other.

Divide the base shear between the single story and the two story structures

The base shear will be divided between the structures based on the ratio of their masses;

Single story weight = 26.1^k (116.1KN)
Two story weight = $19.8^k + 82.2^k = 102^k$ (453.7KN)

Therefore;
Base shear for the single story building;

$$V_{\text{single story, trans}} = \frac{26.1^k}{128^k}(9.1^k) = 1.9^k \text{ (8.45KN)}$$

$$V_{\text{single story, long}} = \frac{26.1^k}{128^k}(12.2^k) = 2.5^k \text{ (11.12KN)}$$

Base shear for the two-story building;

$$V_{\text{two story,trans}} = \frac{102^k}{128^k}(9.1^k) = 7.2^k \text{ (32.03KN)}$$

$$V_{\text{two story, long}} = \frac{102^k}{128^k}(12.2^k) = 9.7^k \text{ (43.15KN)}$$

Calculate vertical distribution on two story structure;

$$F_x = C_{vx} V \qquad \text{(EQ. 5.3.4-1 FEMA 302)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(EQ. 5.3.4-2 FEMA 302)}$$

where; k = 1 in both directions for the building period is less than 0.5 sec.

The calculations are tabularized below*;

Story	w_i (kips)	h_i (ft)	$w_i x h_i$ (ft-kips)	C_{vx}	$C_{vx} x V_t$ (kips)	$C_{vx} x V_L$ (kips)
Roof	19.8	22	435.6	0.33	2.34	3.15
2^{nd}	82.2	11	904.2	0.67	4.86	6.55
SUM =	102		1339.8	1.00	7.20	9.70

Note: For metric equivalent; 1-kip = 4.448KN, 1-ft = 0.30m, 1-ft-kip = 1.36KN-m

Therefore;

Transverse direction;
$\mathbf{F_{roof} = 2.34^k}$ **(10.41KN)**
$\mathbf{F_{2nd} = 4.86^k}$ **(21.62KN)**

Longitudinal direction;
$\mathbf{F_{roof} = 3.15^k}$ **(14.01KN)**
$\mathbf{F_{2nd} = 6.55^k}$ **(29.13KN)**

ASSEMBLY WEIGHTS (PSF):

DIAPHRAGMS:

Level - High Roof

Built-Up roofing;		5.00		
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00		
1-1/2"-22 GA Decking (galvanized);		1.90		
10K1 Open Web Steel Joists @ 4' o.c.; 5plf(1/4') =		1.25		
W12x14 Edge Beam @ equivalent 15' o.c.; 14plf(1/15') =		0.93		
W14X26 Beam @ avg spacing of 10' o.c.; 26plf(1/10') =		2.60		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	18.7	psf	(0.9KN/m^2)

Level - Low Roof

Built-Up roofing;		5.00		
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00		
1-1/2"-22 GA Decking (galvanized);		1.90		
10K1 Open Web Steel Joists @ 4' o.c.; 5plf(1/4') =		1.30		
W12x14 Edge Beam @ equivalent 15' o.c.; 14plf(1/15') =		0.93		
W14X26 Beam @ avg spacing of 13' o.c.; 26plf(1/13') =		1.00		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	17.1	psf	(0.82KN/m^2)

Level - Second Floor

Floor Finish;		1.00		
5" Thick Concrete Floor Slab; (5/12)'(150pcf) =		62.50		
W10x17 Beam @ avg spacing of 7.5' o.c.; 17plf(1/7.5') =		2.27		
W21x50 Girders @ avg spacing of 10' o.c.; 50plf(1/10') =		5.00		
Partitions (10 psf per FEMA 310 section 3.5.2.1);		10.00		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	84.8	psf	(4.06KN/m^2)

METAL SIDE WALLS:

Metal Siding;		1.00		
Girts;		1.00		
2" insulation @ 1.0psf/in., 1.0x2 =		2.00		
W14X34 Col @ avg spacing of 17.5'; 34plf(1/17.5') =		1.94		
	Total =	5.9	psf	(0.3KN/m^2)

Figure 2. Calculation of component weights.

BUILDING WEIGHTS (KIPS):

Item Desc.	Width or Height (ft)	Length (ft)	Number	Trib Area $(ft)^2$	Unit Wt. (psf)	Weight Trans. (kips)	Weight Long. (kips)	Weight Total (kips)
Level - High Roof								
Diaph.	30	30	1	900	18.7	16.8	16.8	16.8
Longitudinal Walls (above and below the diaphragm)								
Below	5.5	30	2	330	5.9	2.0		2.0
Transverse Walls (above and below the diaphragm)								
Below	5.5	30	1	165	5.9		1.0	1.0
Below	3.5	30	1	105	5.9		0.6	0.6
Total High Roof Tributary Weight =						18.8	18.4	20.4
Level - Low Roof								
Diaph.	30	40	1	1200	17.1	20.6	20.6	20.6
Longitudinal Walls (above and below the diaphragm)								
Below	7.5	40.0	2	600	5.9	3.6		3.6
Transverse Walls (above and below the diaphragm)								
Above	3.5	30.0	1	105	5.9		0.6	0.6
Below	7.5	30.0	1	225	5.9		1.3	1.3
Total Low Roof Tribrtary Weight =						24.1	22.5	26.1
Level - Second Floor								
Diaph.	30	30	1	900	84.8	76.3	76.3	76.3
Longitudinal Walls (above and below the diaphragm)								
Above	5.5	30.0	2	330	5.9	2.0		2.0
Below	5.5	30.0	2	330	5.9	2.0		2.0
Transverse Walls (above and below the diaphragm)								
Above	5.5	30.0	1	165	5.9		1.0	1.0
Below	5.5	30.0	1	165	5.9		1.0	1.0
Total Second Floor Tributary Weight =						80.2	78.3	82.2
Total Building Weight =								129

Note: For metric conversions; 1-ft = 0.30m, 1-ft^2 = 0.093m^2, 1psf = 47.88N/m^2, 1-kip = 4.448KN

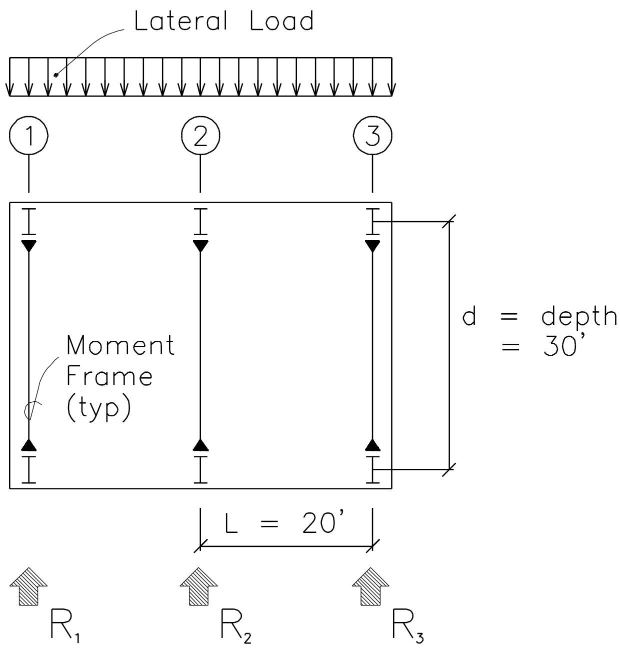
Figure 3. Calculation of building weights.

B-5 *Perform Static Analysis.*

General;

The roof diaphragms are composed of metal decking without fill and are therefore flexible diaphragms. Therefore, loads at the roof will be distributed to the lateral load resisting elements by the tributary area method. The second floor diaphragm is composed of a reinforced concrete slab and is considered a rigid diaphragm. Loads at the second floor will be distributed to the lateral load resisting elements in accordance with their rigidities.

Perform static analysis on single story structure;
Transverse direction:



1-ft = 0.30m

Lateral load: $w_u = F_{trans}/2L = 1.9^k/40' = 47.5\text{plf}$ (0.69KN/m)

Diaphragm moment: $M_u = \frac{w_u L^2}{8} = \frac{47.5\text{plf}(20')^2}{8} = 2{,}375^{ft-lbs}$ (3.22KN-m)

Chord forces: $T = C = \frac{M_u}{d} = \frac{2{,}375^{ft-lbs}}{30'} = 79.2 - \text{lb}$ (352.3N)

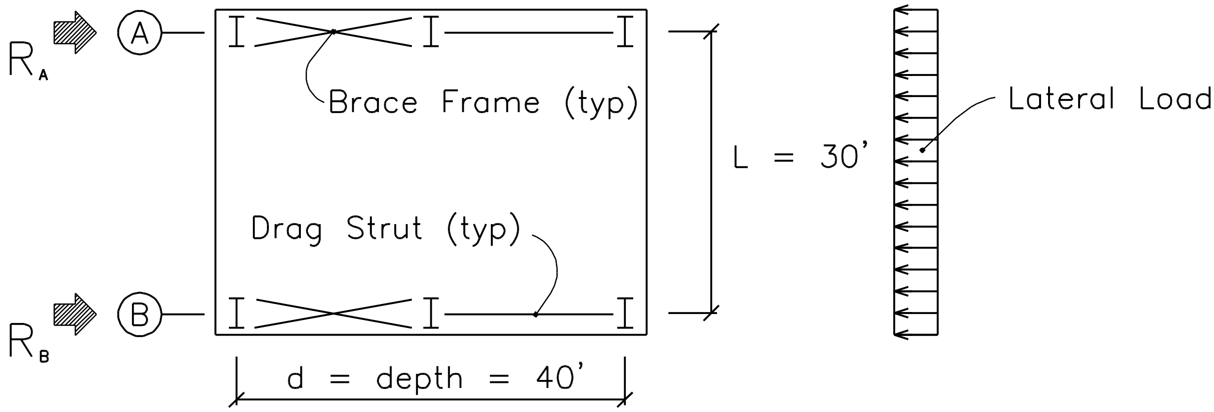
Reactions on moment frames:

End frames; $R_1 = R_3 = w_u$x(tributary length) = 47.5plf(10') = 475-lb (2.11KN)

Interior frame; R_2 = 47.5plf(20') = 950-lb (4.23KN)

Unit shear: v = R/d = 475-lb/30' = 16plf (0.23KN/m)

Longitudinal direction:



1-foot = 0.30m

Lateral load: $w_u = F_{trans}/L = 2.5^k/30' = 83.3plf$ (1.22KN/m)

Diaphragm moment: $M_u = \frac{w_u L^2}{8} = \frac{83.3plf(30')^2}{8} = 9{,}371^{ft-lbs}$ (12.71KN-m)

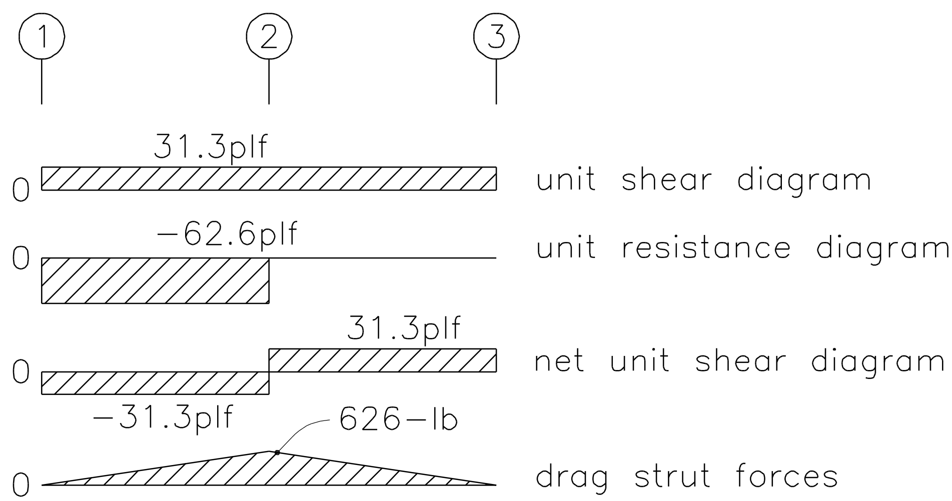
Chord forces: $T = C = \frac{M_u}{d} = \frac{9{,}371^{ft-lbs}}{40'} = 234 - lb$ (1.04KN)

Reactions on braced frames:

Both side frames; $R_A = R_B = w_u x(\text{tributary length}) = 83.3plf(15') = 1{,}250\text{-lb}$ (5.56KN)

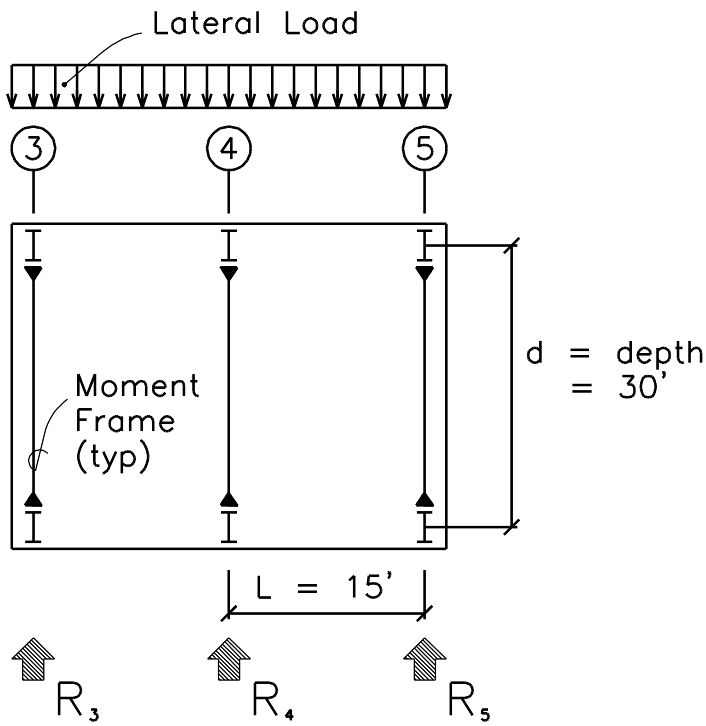
Unit shear: $v = R/d = 1{,}250\text{-lb}/40' = 31.3plf$ (0.46KN/m)

Drag strut forces:



1-lb = 4.448N
1psf = 14.58KN/m

Perform static analysis on two-story structure;
Roof Level
Transverse direction:



1-ft = 0.30m

Lateral load: $w_u = F_{trans}/2L = 2.34^k/30' = 78$ plf (1.14KN/m)

Diaphragm moment: $M_u = \dfrac{w_u L^2}{8} = \dfrac{78\text{plf}(15')^2}{8} = 2{,}194^{\text{ft}-\text{lbs}}$ (2.98KN-m)

Chord forces: $T = C = \dfrac{M_u}{d} = \dfrac{2{,}194^{\text{ft}-\text{lbs}}}{30'} = 73-\text{lb}$ (0.32KN)

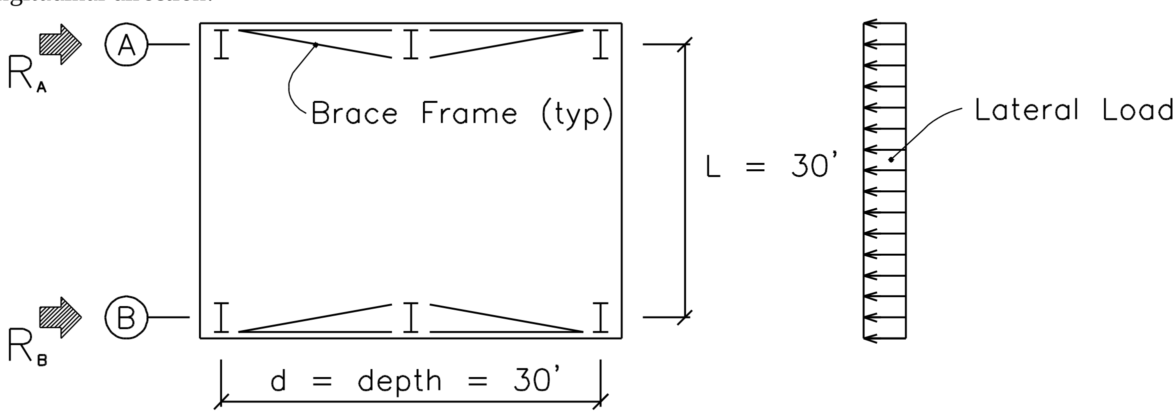
Reactions on moment frames:

End frames; $R_3 = R_5 = w_u$x(tributary length) = 78plf(7.5') = 585-lb (2.60KN)

Interior frame; R_4 = 78plf(15') = 1,170-lb (5.20KN)

Unit shear: v = R/d = 585-lb/30' = 19.5 plf (0.28KN/m)

Longitudinal direction:



1-ft = 0.30m

Lateral load: $w_u = F_{long}/L = 3.15^k/30' = 105plf$ (1.53KN/m)

Diaphragm moment: $M_u = \frac{w_u L^2}{8} = \frac{105plf(30')^2}{8} = 11{,}813^{ft-lbs}$ (16.02KN-m)

Chord forces: $T = C = \frac{M_u}{d} = \frac{11{,}813^{ft-lbs}}{30'} = 394 - lb$ (1.75KN)

Reactions on braced frames:

Both side frames; $R_A = R_B = w_u x$(tributary length) = 105plf(15') = 1,575-lb (7.01KN)

Unit shear: v = R/d = 1,575-lb/30' = 52.5plf (0.67KN/m)

Drag strut forces: $T_{drag} = C_{drag}$ = R/2 = 1,575-lb/2 = 788-lb (3.51KN)

Second Floor Level

Note: The relative rigidities of the vertical lateral load resisting elements must be determined in order to establish the distribution of seismic loads. The transverse moment frames were analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0) to determine their stiffness. The size of the braces, in the moment frame at the interface between the high and low roof structures, was assumed, as they have not yet been designed. The following diagram shows the computer model input and results. Haunch properties are calculated as shown below. The deflection 'Δ' was taken at the point of applied loading.

Frame with truss;

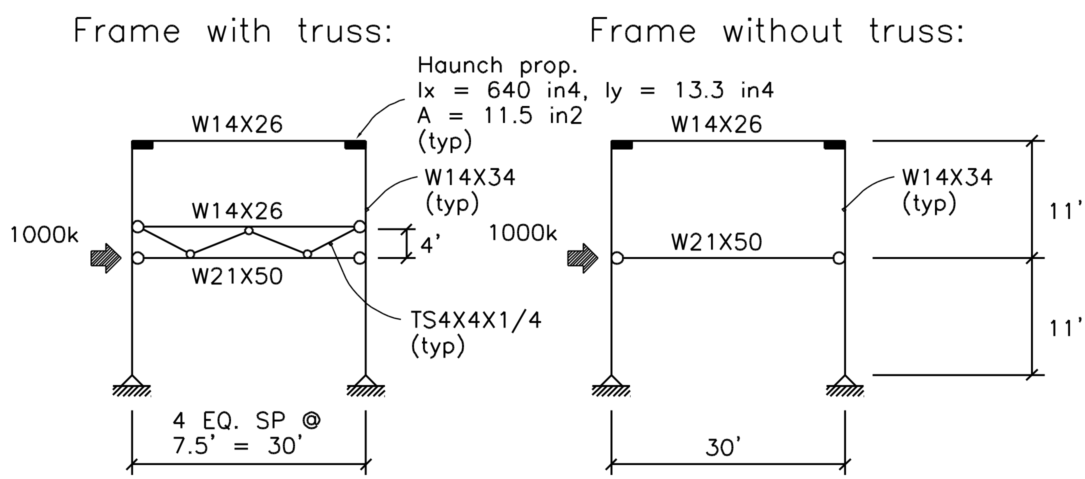
$\Delta_3 = 63.9"$ ($1.62X10^3$ mm)

$K_3 = \frac{1000^k}{63.9"} = 15.7^{k/in}$ ($2.75^{KN/mm}$)

Frame without truss;

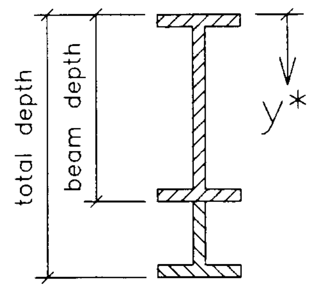
$\Delta_{4,5} = 216"$ ($5.49X10^3$ mm)

$K_{4,5} = \frac{1000^k}{216"} = 4.63^{k/in}$ ($0.81^{KN/mm}$)



1-in = 25.4mm
1-ft = 0.30m
1-in^2 = 645.2 mm^2
1-in^4 = $416.2X10^3$ mm^4
1-kip = 4.448KN

Haunch properties are calculated on spreadsheet as follows;



Beam =	W14X26
Beam I (in) =	245.00
Beam A (in^2) =	7.69
Beam depth (in) =	13.91
Flange Thickness (in) =	0.420
Web Thickness (in) =	0.255
Flange Width (in) =	5.025
Total Depth (in) =	20.91
Haunch Depth (in) =	7.00

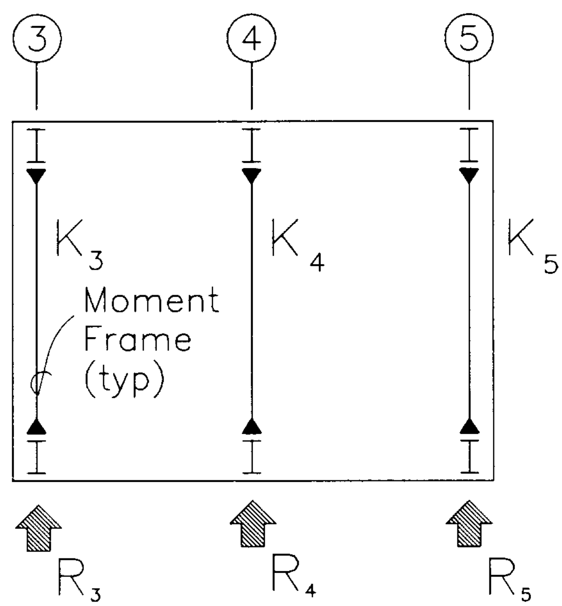
y* (in) =	10.98
Haunch I_x (in^4) =	640
Total A (in^2) =	11.48
Haunch I_y (in^4) =	13.3
Haunch S_x (in^3) =	58
Haunch Z_x (in^3) =	43.24

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

Therefore, in the transverse direction:

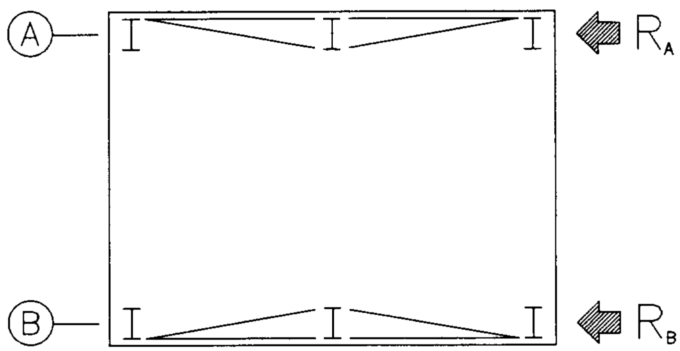
$$R_{trans} = F_{2^{nd}}\left(\frac{K_i}{\sum K_i}\right) \qquad \text{where; } \sum K_i = 15.7^{k/in} + 2(4.63^{k/in}) = 25^{k/in} \ \text{(4.38kn/MM)}$$

$$\text{Therefore; } R_3 = 4.86^k\left(\frac{15.7^{k/in}}{25^{k/in}}\right) = 3.05^k \ \text{(13.6KN)} \quad R_4 = R_5 = 4.86^k\left(\frac{4.63^{k/in}}{25^{k/in}}\right) = 0.90^k \ \text{(4.45KN)}$$



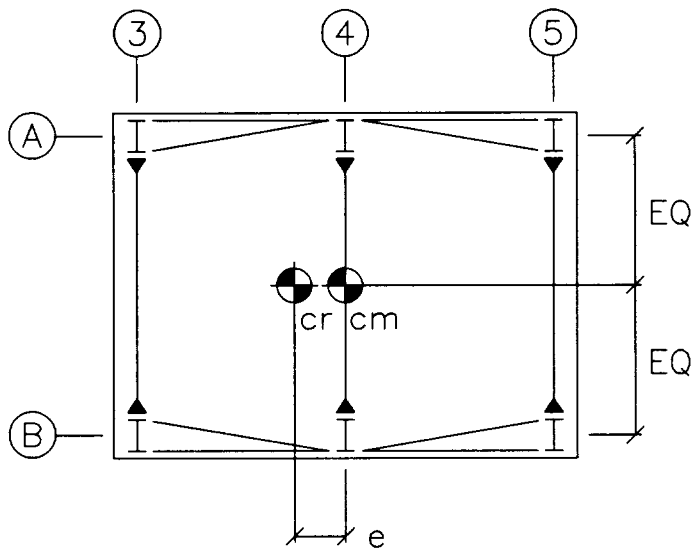
In the longitudinal direction:

$$R_{long} = F_{2^{nd}}\left(\frac{K_i}{\sum K_i}\right) = F_{2^{nd}}\left(\frac{1}{2}\right) \qquad \text{Therefore;} \qquad R_A = R_B = 6.55^k\left(\frac{1}{2}\right) = 3.28^k \ (13.13\text{KN})$$



B-6 *Determine cr and cm.*

Note: Because the roof diaphragms are composed of flexible elements, torsion is only an issue for the second floor diaphragm. Also, by inspection, the cm (center of mass) is located at the geometric plan center of the second floor diaphragm. Additionally, the center of rigidity in the longitudinal direction is also located at the geometric plan center. However, the center of rigidity in the transverse direction must be calculated.



Locate cr_{trans};

$$cr_{trans} = \frac{\sum K_i x_i}{\sum K_i} = \frac{15.7^{k/in}(0) + 4.63^{k/in}(15'+30')}{15.7^{k/in} + 2(4.63^{k/in})} = 8.35' \ (2.56\text{m})$$

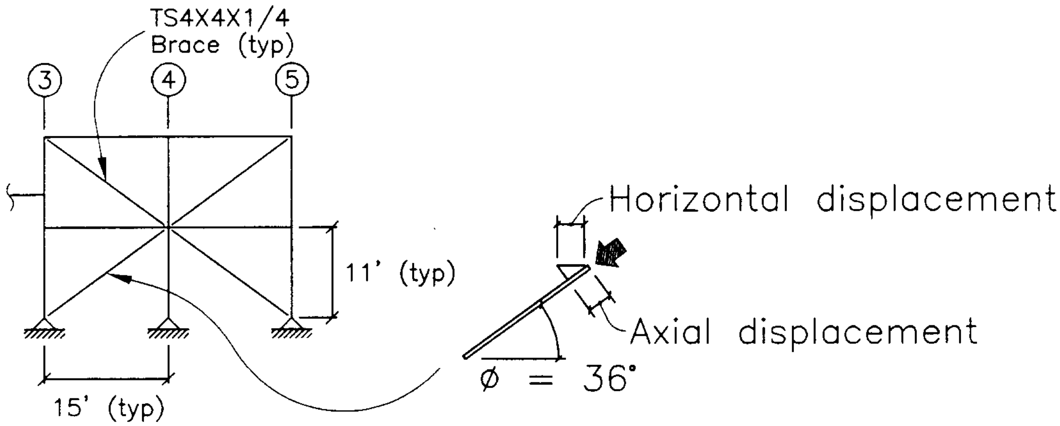
Therefore the eccentricity is;

$$e_{trans} = 15' - 8.35' = 6.65' \ (2.03\text{m})$$

B-7 *Perform torsional analysis.*

Note: In order to determine the polar torsional rigidity 'J_p', the longitudinal brace elements had to be assumed as they have not yet been designed. The analysis could have proceeded using a polar torsional rigidity based on the transverse moment frames alone, but the results would have been overly conservative for the moment frames (this is because the relatively stiffer braced frames and their effect to increase J_p would have been ignored). Therefore, the braced frame elements were chosen as hollow structural sections consisting of TS4X4X1/4.

Calculate Stiffness of braced frames;



1-in = 25.4mm
1-ft = 0.30m
TS4X4X1/4 = TS 101.6mmX101.6mmX6.35mm

Consider a typical brace;

$P_{horiz} = P_{axial} \cos\phi$

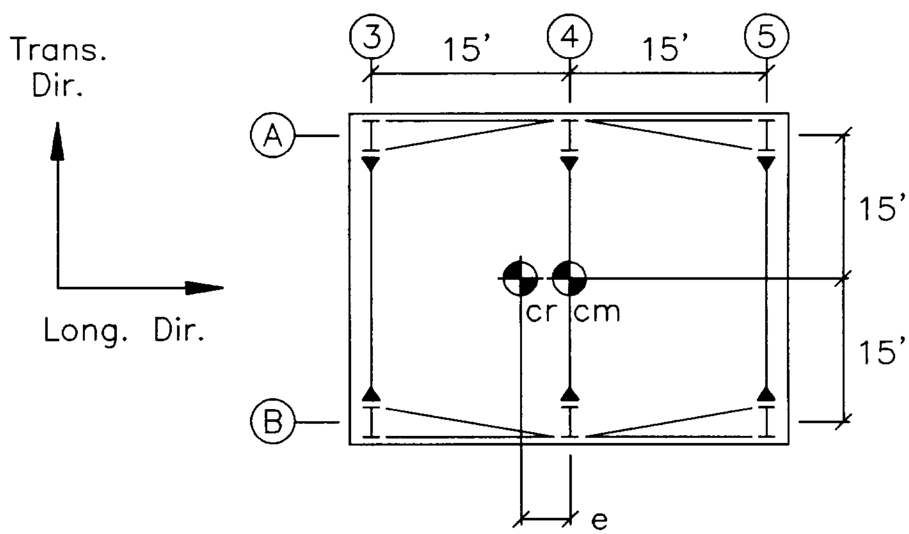
using small angle approximation;

$\Delta_{horiz} = \dfrac{\Delta_{axial}}{\cos\phi}$ Therefore; $K_{horiz} = \dfrac{P_{horiz}}{\Delta_{horiz}} = \dfrac{AE}{L}\cos^2\phi$

For two braces;

$$K_{horiz} = \frac{2AE}{L}\cos^2\phi = \frac{2(3.59\text{in}^2)29{,}000\text{ksi}}{18.6\text{ft}(12''/1')}\left(\frac{15}{18.6}\right)^2 = 607^{k/in}\ (106.3\text{KN/mm})$$

Torsional loads are calculated as follows;



1-ft = 0.30m

$$F_{tors} = \frac{Rd_i}{\sum Rd_i^2} Ve$$

$$F_{acc} = \frac{Rd_i}{\sum Rd_i^2} Ve_{acc}$$

where;

e_{acc} = 0.05% of the building dimension perpendicular to the direction considered.

$e_{acc.\,trans}$ = 0.05(30') = 1.5' (0.46m)

$e_{acc.\,long}$ = 0.05(30') = 1.5' (0.46m)

The calculations were done on spreadsheet and are tabularized below;

V_{trans} =	4.86	kips	e_{trans} =	6.65	ft	$e_{trans.acc}$ =	1.5 ft
V_{long} =	6.55	kips	e_{long} =	0	ft	$e_{long,acc}$ =	1.5 ft

EQ Direction	Element (grid line)	R_{yi} (k/in)	R_{xi} (k/in)	d_{xi}[(1)] (ft)	d_{yi}[(1)] (ft)	R_xd_y or R_yd_x (k-ft/in)	$R_xd_y^2$ or $R_yd_x^2$ (k-ft/in)	F_t[(2)] (kips)	F_{acc} (kips)	$F_{total} = F_t + F_{acc}$ (kips)
Trans	3	15.7	-	8.35	-	131.1	1095	-0.0153	0.0035	0.003
	4	4.63	-	6.65	-	30.8	205	0.0036	0.0008	0.004
	5	4.63	-	21.65	-	100.2	2170	0.0117	0.0026	0.014
Long	A	-	607	-	15	9105	136575	0.0000	0.3234	0.323
	B	-	607	-	15	9105	136575	0.0000	0.3234	0.323

[1]Note: distance 'd' is measured from the cr J_p = 276620

[2] Note: only positive components are added

1-ft = 0.30m
1-kip = 4.448KN
1klf = 0.175KN/mm
1kip-ft/in = 0.053KN-m/mm

B-8 *Determine need for redundancy factor,* ρ.

The building has a seismic design category of D, and therefore, per paragraph 4-4, the redundancy factor is calculated as follows;

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_x}} \qquad \text{(EQ. 4-1)}$$

Evaluation of r_{max};

For braced frames (except at the first story of the high roof structure);

$$r_{max} = \left(\frac{1}{4}\right) \approx 0.25 \qquad \text{(four braces having equal loads at an angle } \theta \approx 36^\circ\text{)}$$

For braced frames at the first story of the high roof structure;

$$r_{max} = \frac{(3.15^k + 6.55^k + 0.323^k)\left(\frac{1}{4}\right)}{3.15^k + 6.55^k} = 0.26 \qquad \text{(includes torsion effects)}$$

For moment frames (except at the first story of the high roof structure);

$$r_{max} = 1/2 \qquad \text{(two columns per frame with equal shears)}$$

For moment frames at the first story of the high roof structure;

$$r_{max} = \frac{3.05^k + 0.003^k + 0.585^k}{2.34^k + 4.86^k} = 0.51 \qquad \text{(includes torsion effects)}$$

B-9 *Determine need for overstrength factor,* $\Omega_{\circ}$.

Per paragraph 8.6.2 of FEMA 302, connections for diagonal bracing members and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or $\Omega_{\circ}$ times the design seismic force.

Therefore, for these *force controlled elements* the following seismic load will be used;

Longitudinal direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \qquad \text{(EQ. 4-6)}$$
$$E = 2Q_E \pm 0.2(0.57)D$$
$$E = 2Q_E \pm 0.114D$$

Transverse direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \qquad \text{(EQ. 4-7)}$$
$$E = 3Q_E \pm 0.2(0.57)D$$
$$E = 3Q_E \pm 0.114D$$

B-10 *Calculate combined load effects.*

Load combinations per ANSI/ASCE 7-95 and TI 809-04 are as follows;

(1) $1.4D$
(2) $1.2D+1.6L+0.5L_r$
(3) $1.2D+0.5L+1.6L_r$
(4) $1.2D+E+0.5L$
(5) $0.9D+E$

However, per paragraph 4-6 of TI 809-04;

$$E = rQ_E \pm 0.2 S_{DS} D$$
$$= 1.0Q_E \pm 0.2(0.57)D$$
$$= Q_E \pm 0.114D$$

Therefore, equations 4 and 5 become;

(4a) $1.314D+Q_E+0.5L$
(4b) $1.086+Q_E+0.5L$
(5a) $1.014D+Q_E$
(5b) $0.786D+Q_E$

B-11 *Determine structural member sizes.*

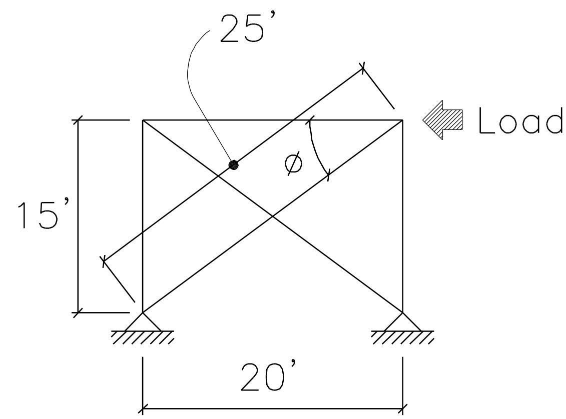
(1) Braced Frames (Low Roof).

Determine Design Loads:

Per paragraph 7-3.a (5) (a), structural steel braced frames will conform to the requirements of the AISC seismic provisions for structural steel buildings.

Axial load per braced frame;

Note: All load combinations reduce to $P_{horiz} = Q_E = F_{roof}/2$



1-ft = 0.30m

Therefore; $$P_u = P_{axial} = \frac{P_{horiz}}{2\cos f} = \frac{2.5^k / 2}{2(0.8)} = 0.78^k \ \ (3.47KN)$$

Two brace elements per brace frame

Design Members (use HSS w/ F_y = 46ksi or 317.2MPa):

Per paragraph 7-3.b (3), the effective out-of-plane unbraced length is equal to two thirds of the total length.

Therefore; KL = (2/3)25’ = 16.7’ or 200” (5.09m)

Per AISC seismic provisions; $$\frac{KL}{r} \le \frac{1000}{\sqrt{F_y}} \qquad \frac{h}{t} \le l_p = \frac{110}{\sqrt{F_y}}$$

Therefore; $$r \ge \frac{KL\sqrt{F_y}}{1000} = \frac{200''\sqrt{46^{ksi}}}{1000} = 1.36-\text{in} \ \ (34.5mm)$$

$$l_p = \frac{110}{\sqrt{46^{ksi}}} = 16.2$$

Try TS 4X4X1/4;

From AISC column load tables for an effective length KL = 17’ $f_c P_n = 42^k > 0.78^k = P_u$ **O.K.**

$f_t P_n = 0.9(46^{ksi})3.59-\text{in}^2 = 149^k > 0.86^k$ (662.8KN > 3.83KN) **O.K.**

r = 1.51” > 1.36” = $r_{req'd}$ **O.K.**

h/t = 16 < 16.2 = l_p **O.K.**

Choose TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm)

(2) Chord/Collector Elements (Low Roof).

Determine Design Loads:

Seismic;

Note: Only Chord/Collector elements in the plane of the braced frames will be designed. Chord/Collector elements perpendicular to the braced frame are a part of the transverse moment frames. From the static analysis the axial loads are determined as follows;

Maximum Chord Force = 79.2-lb (0.35KN)
Maximum Collector Force = 626-lb (2.78KN) (governs)

Gravity;

w_D = 18.7psf(2.5') = 46.8plf (0.68KN/m)
w_L:
There is no live load reduction per ANSI/ASCE 7-95 for A_T = 2.5'(20') = 50-ft^2 < 200-ft^2 (4.65m^2 < 18.58m^2).
Therefore w_L = 20psf(2.5') = 50plf (0.73KN/m)
Therefore; $w_U = 1.314w_D + 0.5w_L$ = 1.314(46.8plf)+0.5(50plf) = 86.5plf (1.26KN/m)

$$M_u = \frac{w_u L^2}{8} = \frac{86.5\text{plf}(20')^2}{8} = 4.33^{\text{ft-kips}} \ (5.87\text{KN-m})$$

$$P_u = \Omega_\circ(Q_E) = 2(626-\text{lb}) = 1.25^k \ (5.56\text{KN})$$ (Per FEMA 302 Section 8.6.2)

Design Members:
Check the W12X14 choosen previously.
Capacity;

$\phi_b M_n = 47^{\text{ft-kips}}$ Per AISC LRFD 2nd edition load factor design selection table.

$\phi_c P_n = 110^k$ Per AISC LRFD 2nd edition table 3-36 with bending about the x-axis only:

$$\frac{KL}{r} = \frac{1.0(20')(12''/1')}{4.62''} = 52 \Rightarrow \phi_c F_{cr} = 26.54\text{ksi} \ (174.0\text{MPa}),$$

and $\phi_c P_n = A_g(\phi_c F_{cr}) = 4.16-\text{in}^2(26.54\text{ksi}) = 110^k$ (489.3KN)

Check which interaction equation to use;

$$\frac{P_u}{\phi_c P_n} = \frac{1.25^k}{110^k} = 0.011 < 0.2 \Rightarrow \text{AISC LRFD EQ. H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{1.25}{2(110^k)} + \left(\frac{4.33^{\text{ft-kips}}}{47^{\text{ft-kips}}} + 0\right) = 0.097 < 1.0$$ **O.K.**

Choose W12X14 (W304.8mmX0.204KN/m)

(3) Braced Frames (High Roof).

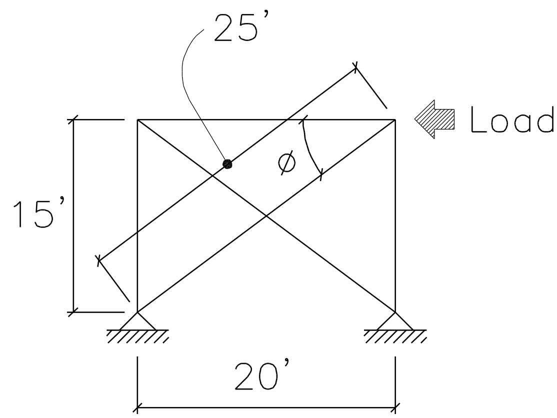
Note: Braces will be designed for the worst case loading at the first story level.

Determine Design Loads:

$P_{horiz} = Q_E = (F_{roof} + F_{2nd})/2 + F_{torsion}$
$= (3.15^k + 6.55^k)/2 + 0.323^k = 5.17^k$ (23.0KN)

Axial load per brace;

$$P_u = P_{axial} = \frac{P_{horiz}}{2\cos\phi} = \frac{5.17^k}{2(0.806)} = 3.21^k \ (14.3\text{KN})$$



1-ft = 0.30m

Design Members (use HSS w/ F_y = 46ksi, or 317.2MPa):

Per paragraph 7-3.b (3), the effective out-of-plane unbraced length is equal to two thirds of the total length.

Therefore; KL = (2/3)18.6’ = 12.4’ or 149” (3.78m)

Per AISC seismic provisions; $\frac{KL}{r} \le \frac{1000}{\sqrt{F_y}}$ $\frac{h}{t} \le l_p = \frac{110}{\sqrt{F_y}}$

Therefore; $r \ge \frac{KL\sqrt{F_y}}{1000} = \frac{149"\sqrt{46^{ksi}}}{1000} = 1.01-in\ \ (2.57mm)$

$l_p = \frac{110}{\sqrt{46^{ksi}}} = 16.2$

Try TS 4X4X1/4;

From AISC column load tables for an effective length KL = 13’

$f_c P_n = 68^k > 3.21^k (302.5KN > 14.3KN) = P_u$ **O.K.**

$f_t P_n = 0.9(46^{ksi})3.59 - in^2 = 149^k > 3.21^k (662.8KN > 14.3KN)$ **O.K.**

r = 1.51” > 1.01” = $r_{req'd}$ **O.K.**

h/t = 16 < 16.2 = l_p **O.K.**

Choose TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm)

(4) Chord/Collector Elements (High Roof).

Determine Design Loads:

Seismic;

Note: Only Chord/Collector elements in the plane of the braced frames will be designed for Chord/Collector elements perpendicular to the braced frame are a part of the transverse moment frames. From the static analysis, the axial loads were determined as follows;

Maximum Chord Force = 73-lb (0.32KN)
Maximum Collector Force = 788-lb (3.51KN) (governs)

By inspection with the design for the low roof, a W12X14 is adequate.

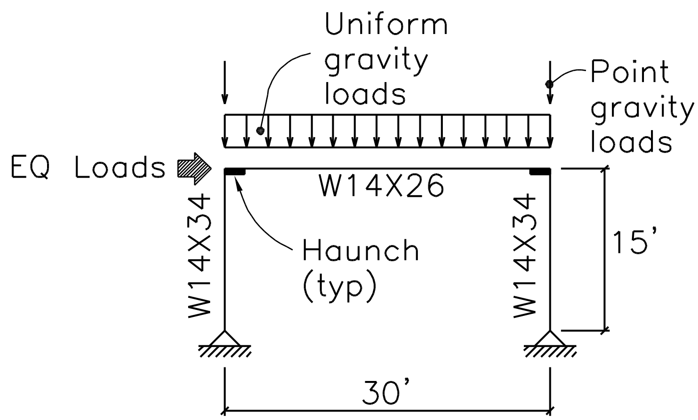
Choose W12X14 (W304.8mmX0.204KN/m)

(5) Moment Frames (Low Roof).

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

Determine Design Loads:

w_L:
Live load reduction per ANSI/ASCE 7-95;
$A_T = 20'(30') = 600\text{-ft}^2\ (55.7\text{m}^2) \Rightarrow R_1 = 0.6$
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore w_L = 20'(20\text{psf})0.6 = 240\text{plf}\ \ (3.50\text{KN/m})$
$w_D = 20'(17.1\text{psf})$
$= 342\text{plf}\ \ (4.99\text{KN/m})$
$P_D = 5.9\text{psf}(7.5')(20') = 885\text{-lb}\ (3.94\text{KN})$ (point load due to wt. of side walls)
$E = 1.0Q_E = 0.95^k\ (4.23\text{KN})$ (applied as a uniform load of $0.95^k/30' = 31.7\text{plf}\ (0.46\text{KN/m})$ along the beam length)



1-in = 25.4mm
1-ft = 0.30m
1-lb = 4.448N

Design Members:

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

General;

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 equation 4a; $1.314D+Q_E$. After comparing the frame deflection to the allowable story drift, an investigation was undertaken to ensure that plastic hinges would form in their predetermined locations (within the beam at the toe of the haunch as opposed to the face of the column). Last, a check on the strength requirements of the frame was completed.

Drift requirements;

Calculated drift; $d_{calc} = 0.2''\ \ (5.1\text{mm})$

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

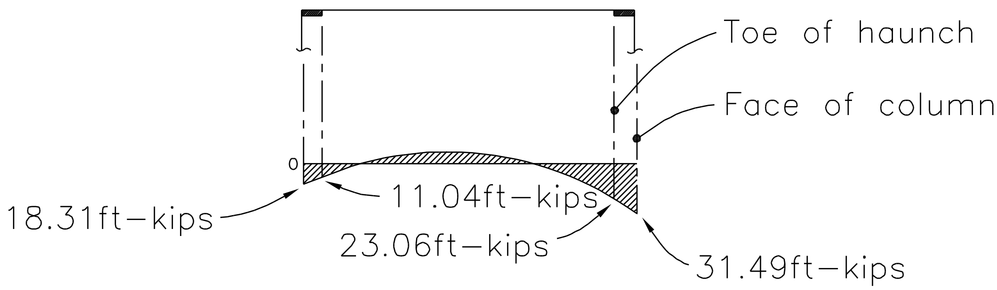
$\Delta_{allow} = 0.025(15'(12''/1')) = 4.5''\ (114.3\text{mm})$

Therefore; $C_d \text{ x } d_{calc} = 5.5(0.2'') = 1.10''(27.9\text{mm}) < \Delta_{allow} = 4.5''\ \ (114.3\text{mm})$ **O.K.**

Check plastic hinge location;

Note: At the time of writing of this problem, it is industry practice to ensure formation of a plastic hinge at the toe of the haunch (rather then at the face of the column) by keeping the ratio of stresses at the toe of the haunch relative to the face of the column greater then 1.2. Also, after the formation of a hinge on one side of the beam, the other side must be checked to ensure that the hinge does not form some where else along the length of the beam. The method is demonstrated as follows;

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



1-ft-kip = 1.36KN-m

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

The stress ratio on the right side is;

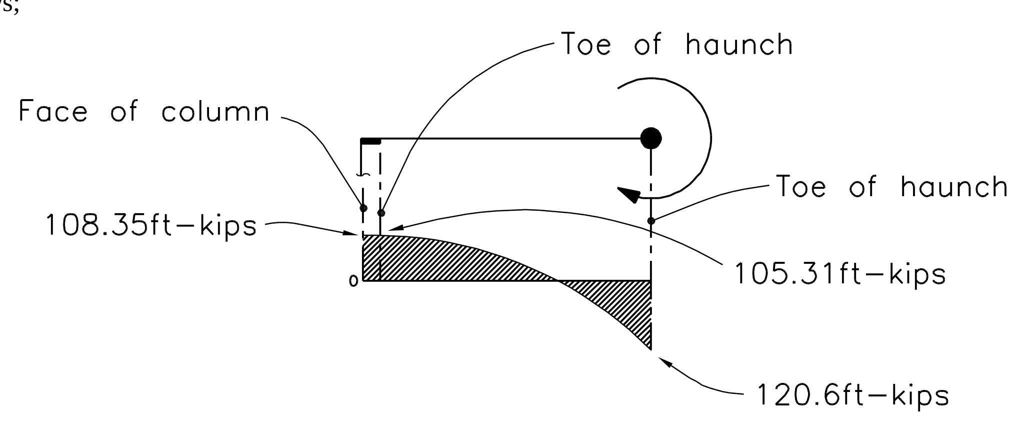
$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{\text{toe}} / S_{x,\text{toe}}}{M_{\text{face}} / S_{x,\text{haunch}}} = \frac{23.06^{\text{ft-kips}}(12''/1')/35.3-\text{in}^3}{31.49^{\text{ft-kips}}(12''/1')/58-\text{in}^3} = 1.2 \qquad \textbf{O.K.}$$

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.

$$M_p = Z_x F_y = 40.2-\text{in}^3(36\text{ksi}) = 120.6^{\text{ft-kips}} \ (164\text{KN-m})$$

$$M_y = S_x F_y = 35.3-\text{in}^3(36\text{ksi}) = 106^{\text{ft-kips}} \ (144\text{KN-m})$$

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



1-ft-kip = 1.36KN-m

The resulting stress ratio on the left side is;

$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{toe} / S_{x,toe}}{M_{face} / S_{x,haunch}} = \frac{105.31^{ft-kips}(12''/1') / 35.3-in^3}{108.35^{ft-kips}(12''/1') / 58-in^3} = 1.6 > 1.2 \qquad \textbf{O.K.}$$

Strength requirements;

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

$M_{u,beam} = 23.06^{ft-kips}(31.3KN - m), V_{u,beam} = 6.37^{k}(28.33KN)$

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

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

$\therefore M_{u,beam} = 23.06^{ft-kips} < 109^{ft-kips} = f_b M_n$ (31.3KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 6.37^{k} < 69^{k} = f_v V_n$ (28.33KN < 306.9KN) **O.K.**

Check unbraced length of beam flanges (per AISC seismic provisions);

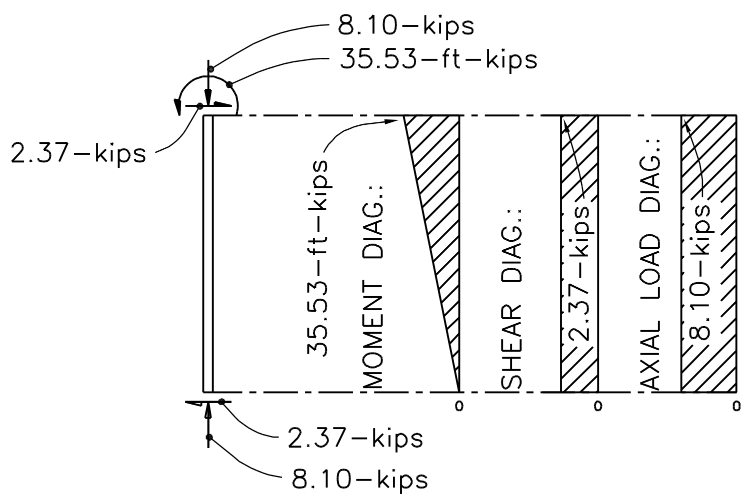
Try 5' on center (same spacing as the perpendicular floor joists);

$$L_b = 5' < 6.25' = \frac{2500(1.08'')(1'/12'')}{36ksi} = \frac{2500r_y}{F_y} \quad (1.53m < 1.91m) \qquad \textbf{O.K.}$$

Therefore provide lateral support to beam flanges at 5-ft (1.53m) o.c.

Column:
Columns are loaded both in flexure and axially and are therefore beam columns. Additionally, the frames the columns are in are not braced against sidesway and therefore magnified moments due to sidesway must be determined.

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 = 4.448KN
1-ft-kip = 1.36KN

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. Therefore, the total axial load is as follows;

$$P_{u.total} = 8.10^k + 0.3(0.78^k \sin\phi) = 8.10^k + 0.3\left(0.78^k\left(\frac{15'}{25'}\right)\right) = 8.24^k \text{ (36.7KN)}$$

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

M_{nt} was determined by placing a restraint against sidesway at the top of the column (a translation fixed restraint) and re-running the RISA-2D analysis with the governing load combination.

$M_{nt} = +28.74^{ft-kips}$ (38.97KN-m) (calculated reaction at the restraint = 0.91^k) (4.05KN)

M_{lt} was determined by removing all loads from the RISA-2D analysis, including the restraint placed at the top of the column in determining M_{nt}, and then placing only the opposite of the reaction determined previously (-0.91^k) at the top of the column and re-running the RISA-2D analysis.

$M_{lt} = +6.83^{ft-kips}$ (9.26KN-m)

B_1 is determined as follows;

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

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

$P_u = 8.24^k$ (36.7KN) (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 ($200.0\text{X}10^3$ MPa)

$K_x = 1.0$ (sidesway inhibited)

$$\frac{K_x L}{r_x} = \frac{2.3(15')(12''/1')}{5.65''} = 73.4$$

$$P_{e1} = \frac{\pi^2(29{,}000\text{ksi})10.0 - \text{in}^2}{(73.4)^2} = 531^k \text{ (2.36MN)}$$

$$B_1 = \frac{0.85}{\left(1 - \frac{8.24^k}{531^k}\right)} = 0.86 < 1.0$$ 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; $\Delta_{oh} = 0.2''$ (from RISA-2D analysis)

$\sum H = 0.95^k$ (4.23KN) (lateral loading, "Q_E")

L = 15'(12"/1') = 180" (4.58m) (column height)

$\sum P_u = 8.10^k + 7.18^k + 2(0.14^k) = 15.6^k$ (69.4KN)

orthogonal effects; $0.3(0.78^k(15'/25')) = 0.14^k$ (0.62KN)

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

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

Therefore; $\quad M_u = B_1M_{nt} + B_2M_{lt} = 1.0(28.74^{ft-kips}) + 1.02(6.83^{ft-kips}) = 35.7^{ft-kips}$ (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;

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

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

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

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 71.5 = \lambda_p$$ **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 $\frac{KL}{r}$, $\quad K_y = 1.0$ due to the braced frames

K_x is determined using AISC alignment charts as follows;

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

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

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

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

Therefore, from the charts; $K_x = 2.3$

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

$$\frac{K_yL_y}{r_y} = \frac{1.0(15'(12"/1'))}{1.53"} = 117.6 \qquad \text{(governs)}$$

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

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

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

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

$\phi_bM_{nx} = 112^{ft-kips}$ 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.**