H-2 Bachelor Enlisted Quarters

a. Introduction

This design example illustrates the seismic design of a three-story Navy bachelor enlisted quarters (BEQ) building. The layout of the building is based on the Navy 1+1 module which allocates approximately 462 sq. ft. ($42.9m^2$) to a two-person living suite as indicated in Figure 1.

(1) Purpose. The purpose of this example is to illustrate the design of a representative military building in an area of high seismicity, using the provisions of FEMA 302 as modified by this document.

(2) Scope. The scope of this example problem includes; the design of all major structural members such as beams, columns, and shear walls. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as reinforced concrete slabs on grade 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. The function of a BEQ is to provide living quarters for enlisted personnel. The Department of the Navy has various standard modules for living areas. The modules can be arranged together with designated administrative and communal spaces to form the BEQ. In this example, a two-person living suite was chosen and the designated administrative and communal areas were provided on the 1st Floor. The building as indicated in Figure 1 would house 70 enlisted personnel.

(2) Seismic Use Group. Since the building is not described by any of the occupancies in Table 4-1 for special, hazardous, or essential facilities, it will be designated as a standard occupancy structure within Seismic Use Group I.

(3) Configuration. The standard Navy modules may be arranged in any desired configuration. The selected module, as shown in Figure 1, was designated for access from an interior hallway. This typical "motel" type configuration using a double-loaded interior corridor was selected as being the most efficient and economical configuration. A small reception and lounge area by the main stairway was provided at the main entrance on the 1st Floor and an additional stairway was provided at each end of the building.

(4) Structural Systems. The continuous vertical alignment of the transverse walls between the suites makes these walls ideal candidates for bearing and shear walls. Similarly, the need for fenestration at the exterior walls makes the use of longitudinal moment frames a logical choice. Precast cored concrete slabs were chosen for the framed floor system. These commercially available units are capable of spanning between the separation walls without intermediate supports. The soffit of the precast slabs forms the exposed ceiling in the service and sleeping areas. Furred ceilings in the corridors, bath, and storage areas can accommodate the heating and ventilation ducts for each of the modules. Lateral loads are transferred by the reinforced topping through dowels to the shear walls or frames and, in turn, to the foundations.

(5) Choice of materials. The bearing/shear walls could be designed as reinforced masonry rather than cast-in-place concrete. The masonry would be equally functional and in some areas of the U.S. may be more economical. Many alternatives, using cast-in-place or precast configurations, are available for the floor framing. A desirable prerequisite is that the floor be relatively stiff and have low acoustic transmission. The precast slabs with reinforced topping were chosen for the reasons discussed in subparagraph (4) above. 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 concrete frames or shear walls.

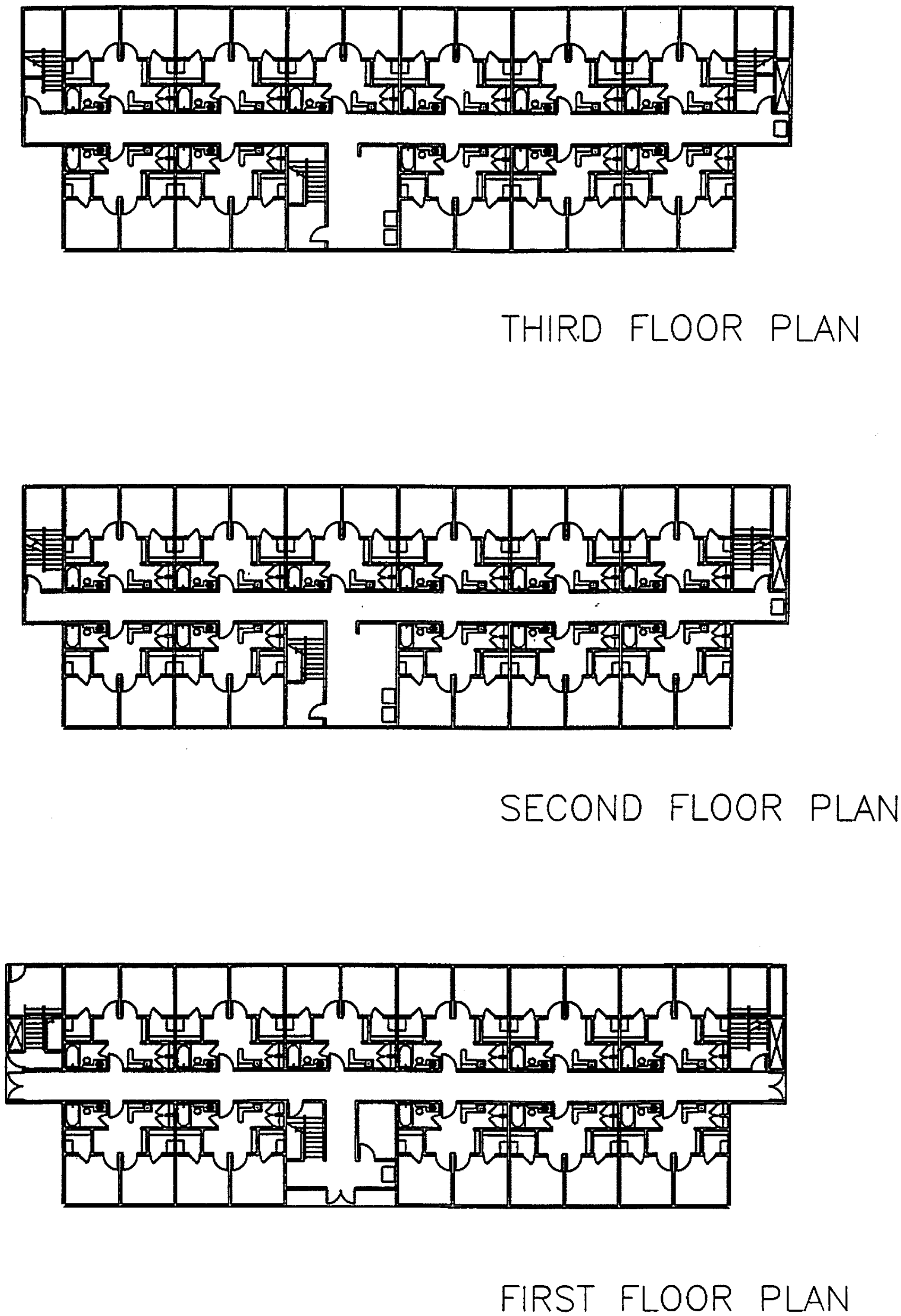
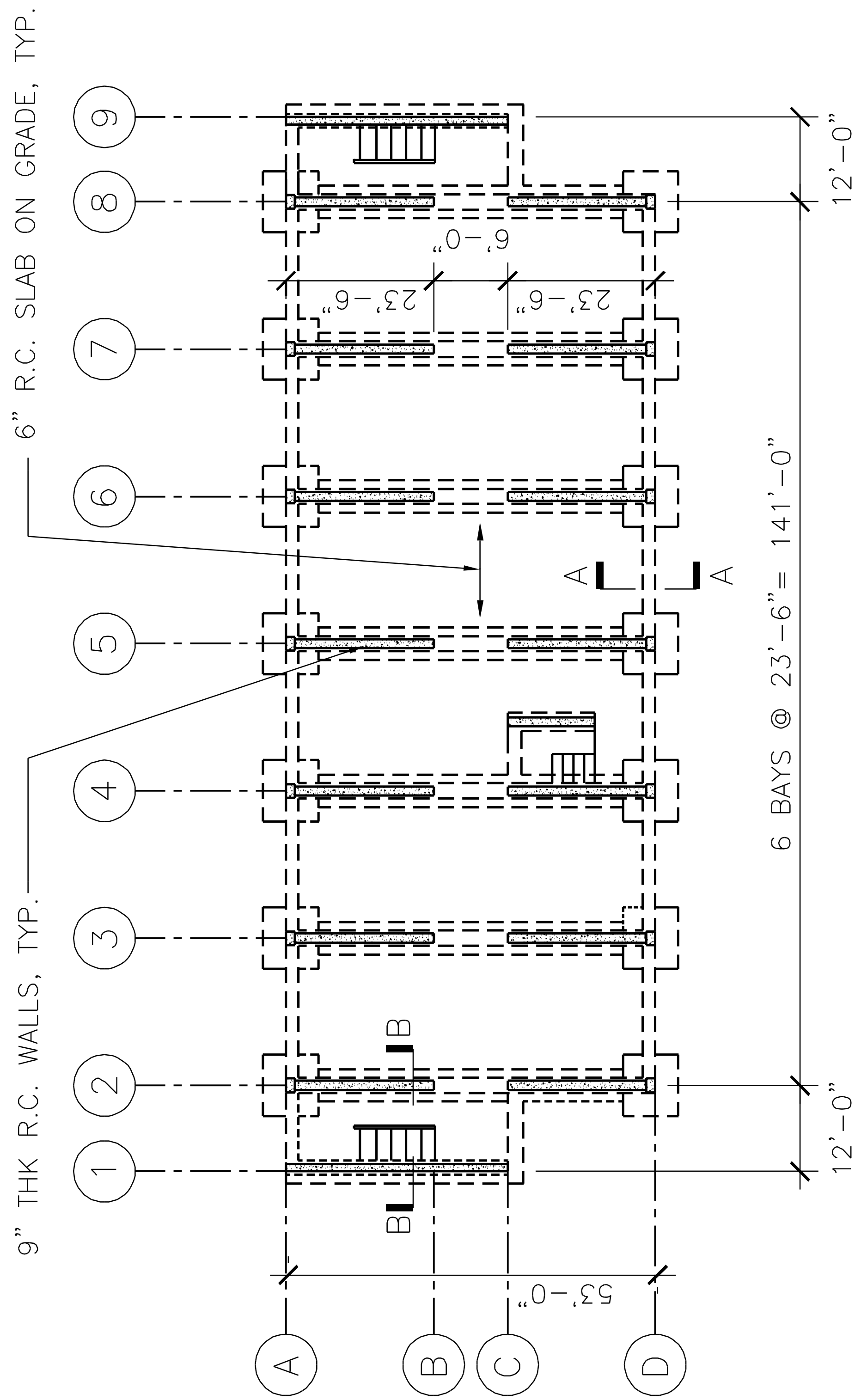
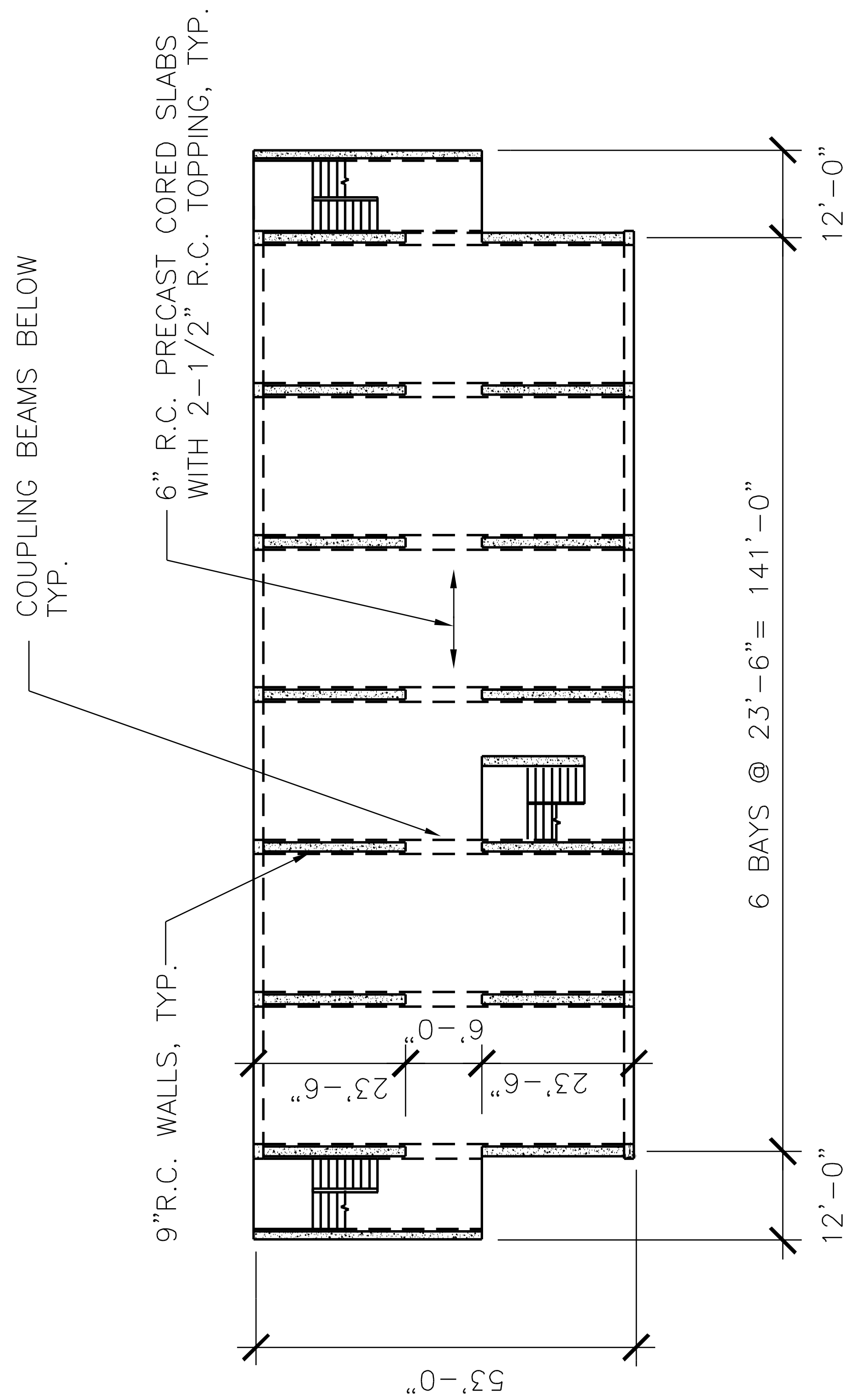


Figure 1. Architectural floor plan



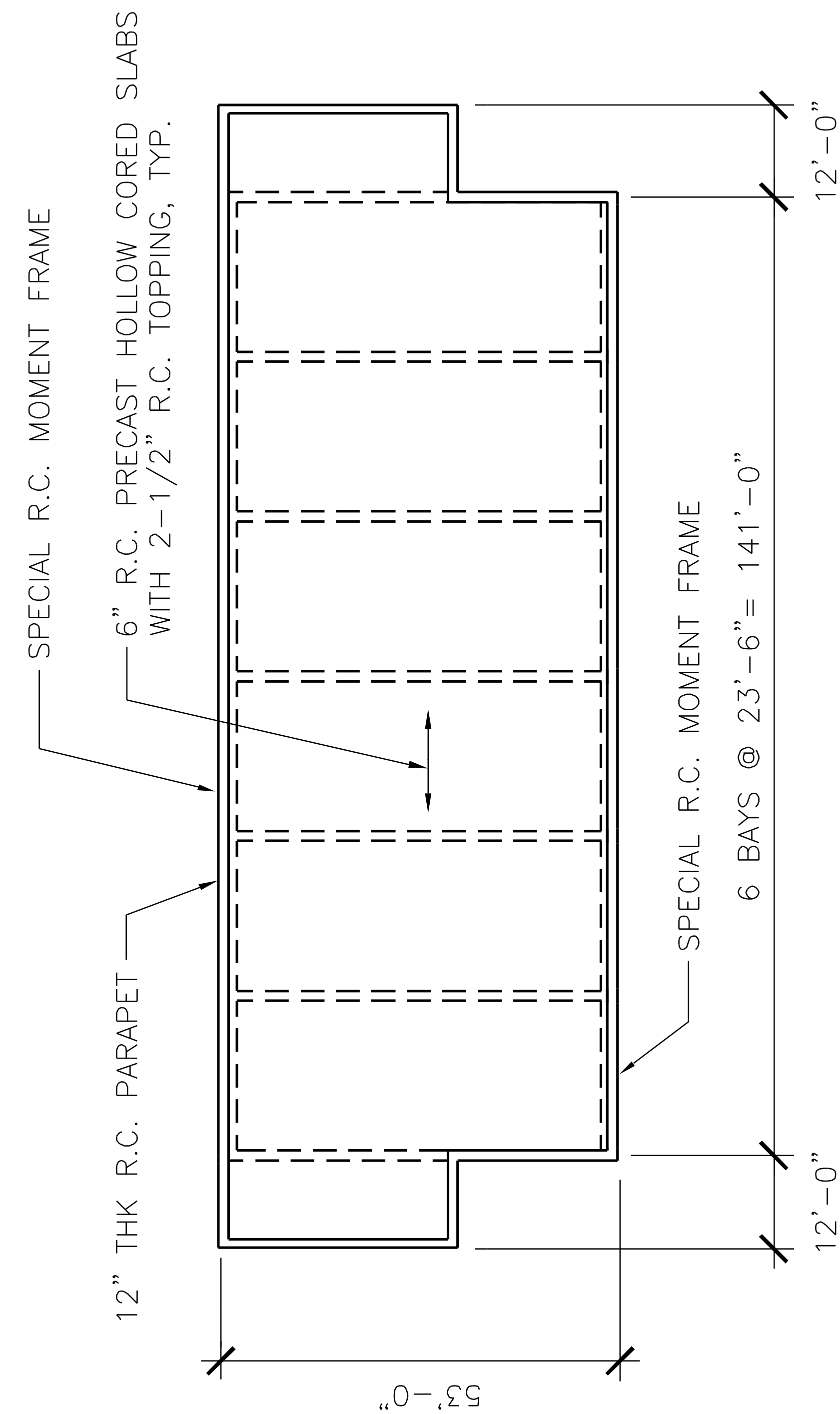
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 2. Foundation and first floor plan



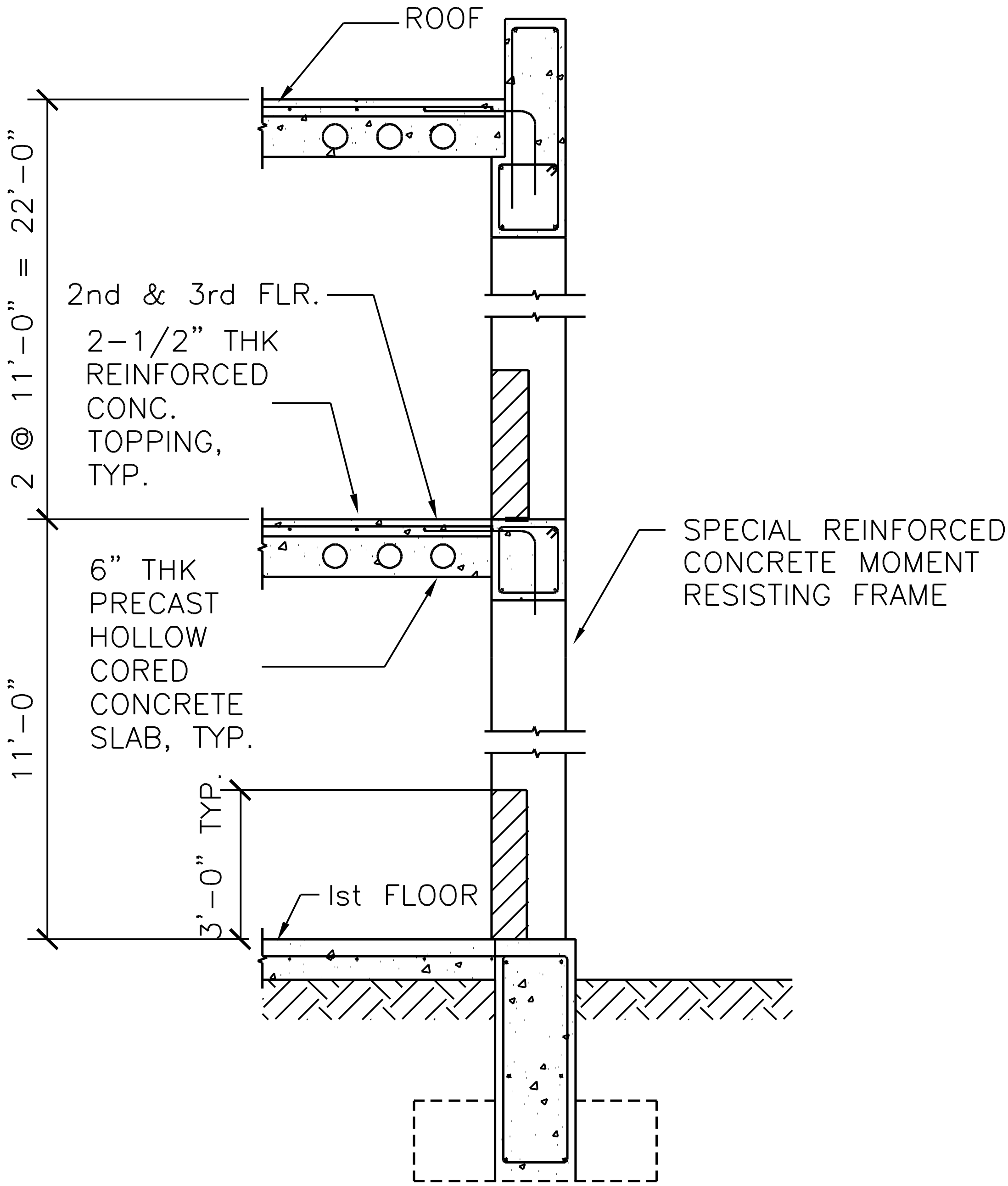
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 3. Typical floor framing plan



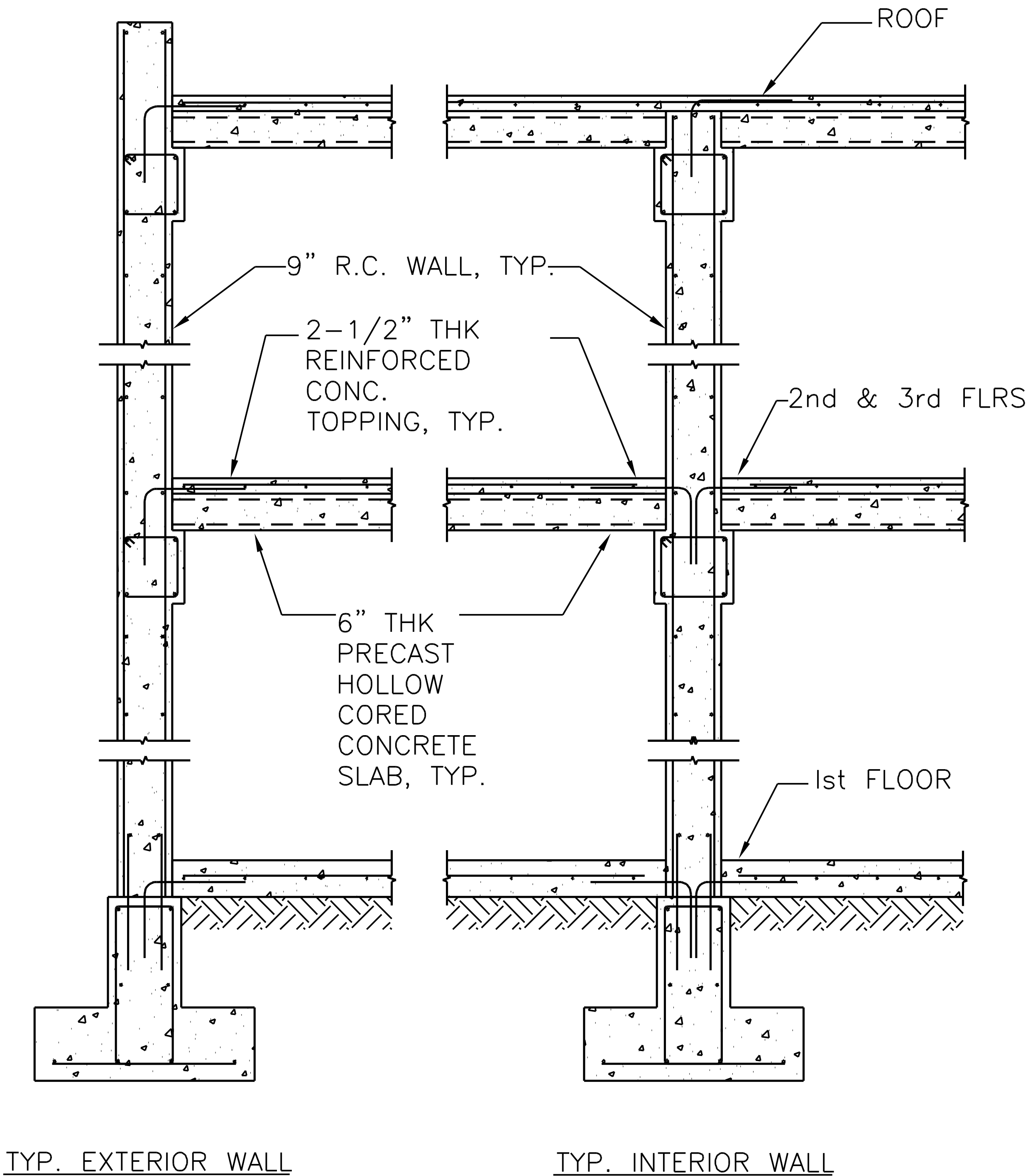
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 4. Roof framing plan



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 5. Section A-A



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 6. Section B-B

c. *Building design (following steps in Table 4-5 for Life Safety).* The design of the building follows steps outlined in Table 4-5 for Performance Objective 1A.

A-1 *Determine appropriate Seismic Use Group.* Per Table 4-3 of the manual, the building is allowed to incur moderate structural damage. Therefore, it is a standard occupancy structure with a Seismic Use Group of I.

A-2 *Determine Site Seismicity*. The site seismicity for this example from the MCE maps is assumed as:
$S_s = 1.40g$, and $S_1 = 0.50g$.

A-3 *Determine Site Characteristics.* For the purpose of this problem, a stiff soil condition was assumed corresponding to a site classification of 'Class D' per Table 3-1 of the 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:

$F_a = 1.00$ (Table 3-2a)
$F_v = 1.50$ (Table 3-2b)

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

$S_{MS} = F_a(S_S) = 1.00(1.40) = 1.40$ (EQ. 3-1)
$S_{M1} = F_v(S_1) = 1.50(0.50) = 0.75$ (EQ. 3-2)

A-6 *Determine Design Spectral Response Accelerations:*

$S_{DS} = (2/3)S_{MS} = (2/3)1.40 = 0.93$ (EQ. 3-3)
$S_{D1} = (2/3)S_{M1} = (2/3)0.75 = 0.50$ (EQ. 3-4)

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

$T_{approx} = 0.1N = 0.1(3) = 0.3 < 0.5$ (EQ. 5.3.3.1-2 FEMA 302)

Since $T_{approx} < 0.5$, and because the structure is less than 5 stories in height, Equations 3-5 and 3-6 must be checked in the short period range:

S_{MS} [$1.5F_a$ (EQ. 3-5)
S_{M1} [$0.6F_v$ (EQ. 3-6)

Therefore, $S_{MS} = 1.4 < 1.5(1.00) = 1.5$
$S_{M1} = 0.75 < 0.6(1.50) = 0.9$

Use S_{DS} = 0.93, & S_{D1} = 0.50

A-7 *Determine Seismic Design Category*. With $S_{DS} = 0.93$, $S_{D1} = 0.50$, and a Seismic Use Category of I, enter Tables 4-2a and 4-2b to obtain a Seismic Design Category of 'D'.

A-8 *Select Structural System.*

Gravity: Reinforced concrete hollow core slabs to span between transverse bearing walls.

Lateral: As permitted by table 7-1 of the manual;
Transverse direction: Special reinforced concrete shear walls.
Longitudinal direction: Special reinforced concrete moment frames.

A-9 *Select R, V_o, & C_d factors.*

From Table 7-1 of the manual:

Transverse direction:	Special reinforced concrete shear walls	**$R = 5.5$, $\Omega_o = 2.5$, $C_d = 5.0$**
Longitudinal direction:	Special reinforced concrete moment frames	**$R = 8$, $\Omega_o = 3.0$, $C_d = 5.5$**

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

(1) Roof and Floor framing

Roof and floor framing to consist of hollow core concrete slabs. Slabs span in the longitudinal direction between bearing walls.

Roof:

Determine Loading:

Live load; w_L = 20 psf (0.96KN/m^2) (per ANSI/ASCE 7-95 Table 4-1)

Dead load; w_D;

Roofing;	5 psf
Insulation;	3 psf
Suspended ceiling;	1 psf
Mechanical & Electrical;	<u>3 psf</u>
Total = w_D =	12 psf (0.57KN/m2)

Therefore, the total superimposed service load = $w_D + w_L$ = 12 psf + 20 psf = 32 psf (1.53KN/m^2). Per the hollow-core concrete slab manufacturers catalog, using a span of 23'-6" (7.17m) and the superimposed service load of 32 psf, choose a 6" (152.4mm) thick x 4'-0" (1.22m) wide hollow-core slab of normal weight concrete (wt. of slab with a 2-1/2" (63.5mm) topping = 80.3 psf or 3.84KN/m^2).

Floor:

Determine design loads:

Live load; w_L = 40 psf (1.92KN/m^2) (per ANSI/ASCE 7-95 Table 4-1)

Dead load; w_D;

Floor finish;	1 psf
Partitions;	10 psf
Suspended ceiling;	1 psf
Mechanical & Electrical;	<u>3 psf</u>
Total = w_D =	15 psf (0.72KN/m^2)

Therefore, the total superimposed service load = $w_D + w_L$ = 15 psf + 40 psf = 55 psf (2.63KN/m^2). Per the hollow-core concrete slab manufacturers catalog using a span of 23'-6", and the superimposed service load of 55 psf, choose a 6" thick x 4'-0" wide hollow-core slab of normal weight concrete (wt. of slab with a 2-1/2" topping = 80.3 psf or 3.84KN/m^2).

(2) Transverse bearing wall design

The transverse bearing walls run almost continuously through the building being interrupted only by the 6-ft. wide interior corridor. The thickness of the walls is determined based on their dual function as both shear and bearing walls; the building is located in a high seismic zone and two curtains of reinforcement are anticipated. Therefore, a wall thickness of 9-in (228.6mm). is chosen in order to accommodate the placement of two curtains of reinforcement. For gravity loads, the empirical design method of ACI 318-95 Section 14.5 will be used to check this 9-in. thickness for bearing. The empirical design method is determined to be applicable because the resultant of all factored loads is located within the middle third of the wall. Additionally, it is anticipated that reinforcement requirements will be governed by seismic considerations, and therefore, the design of the wall reinforcement will be addressed during the lateral load design.

Determine design loads;

Note: Loading will be determined per unit of wall length using a tributary width per floor or roof of 23'-6" (7.17m).

Live load;

Floor live load reduction per ANSI/ASCE 7-95;

A_T = 23.5' x 53' = 1,246-ft^2 > 400-ft^2 (115.8-m^2 >37.2m^2)

$$L = L_o\left(0.25+\frac{15}{\sqrt{A_T}}\right) > 0.4L_o \qquad \text{(EQ. 4-1 ANSI/ASCE 7-95)}$$

$$L = 40\text{psf}\left(0.25+\frac{15}{\sqrt{1{,}246-\text{ft}^2}}\right) = 27\text{psf} > 16\text{psf} = 0.4(40\text{psf}) \;\; (1.29\text{KN/m}^2 > 0.77\text{KN/m}^2)$$

Therefore, w_{FL} = 27psf (1.29KN/m^2), w_{RL} = 20psf (0.96KN/m^2)

The total roof live load per foot of wall length is;

P_{RL} = 20psf(23.5') = 470plf (6.85KN/m)

The total floor live load per foot of wall length is;

P_{FL} = 2-floors x 27psf(23.5') = 1,269plf (18.51KN/m)

Dead load;

From the design of the roof and floor framing, the roof and floor dead loads are;

w_{FD} = 80.3psf + 15psf = 95.3psf (4.56KN/m^2), w_{RD} = 80.3psf + 12psf = 92.3psf (4.42KN/m^2)

Dead load due to the wall self weight at the first floor is;

w_{WD} = 9"(1'/12")150pcf x 33'(bldg. Height) = 3,713plf (54.15KN/m)

The total dead load per foot of wall length (at the first story) is;

P_{DL} = 95.3psf(2)23.5'+92.3psf(1)23.5'+3,713plf = 10,361plf (157.10KN/m)

Factored load combinations (per ANSI/ASCE 7-95);

Load case 1: $U = 1.4D$

P_u = 1.4(10,361plf) = 14,505plf (211.53KN/m)

Load case 2: $U = 1.2D+1.6L+0.5L_r$

P_u = 1.2(10,361plf)+1.6(1,269plf)+0.5(470plf) = 14,700plf (governs) (214.38KN/m)

Load case 3: $U = 1.2D+0.5L+1.6L_r$

Pu = 1.2(10,361plf)+0.5(1,269plf)+1.6(470plf) = 13,820plf (201.55KN/m)

Check minimum wall thickness requirements per ACI 318-95 Section 14.5.3;

h = 9" > 4" **O.K.**

$$\frac{l_c}{25} = \frac{11'(12''/1')}{25} = 5.28'' < 9''(134.1\text{mm} < 228.6\text{mm}) = h$$ **O.K.**

Check strength per ACI 318-95 Section 14.5.2;

$$\phi P_{nw} = 0.55\phi f_c' A_g\left[1-\left(\frac{kl_c}{32h}\right)^2\right] \qquad \text{(EQ. 14-1 ACI 318-95)}$$

where; $\phi = 0.7$

k = 1.0 For ends are unrestrained against rotation at each floor level, but are braced against lateral translation by the orthogonal moment frames.

f_c' = 4,000psi (27.58MPa)

A_g = 9"(12"/1') = 108-in^2/ft (228.4X10^3 mm^2/m)

$$\phi P_{nw} = 0.55(0.70)4{,}000\text{psi}(108-\text{in}^2/\text{ft})\left[1-\left(\frac{1.0(11'(12''/1'))}{32(9'')}\right)^2\right] = 131.4\text{klf} \;\; (1.92\text{MN/m})$$

$P_u = 14.7\text{klf} < 131.4\text{klf} = \phi P_{nw}$ (0.21MN/m < 1.92MN/m) **O.K.**

Choose a wall thickness h = 9" (228.6mm)

(3) Corbel design

The precast hollow-core concrete slabs span between transverse walls and are supported at these walls by rectangular corbels. Corbels are 4-in (101.6mm) and 12-in. (304.8mm) deep. One design for the worst case loading will be used throughout the building.

Determine if provisions of ACI 318-95 Section 11.9.1 are satisfied;

Shear span is chosen as a = 2-in. Therefore; $\frac{a}{d} = \frac{2}{12} = 0.17 < 1.0$

Also, by inspection, $N_{uc} < V_u$ **O.K.**

Determine if provisions of ACI 318-95 Section 11.9.2 are satisfied;

Depth outside edge of bearing area = d > 0.5d **O.K.**

Determine design loads;

Note: Worst case occurs at an interior span at either the second or third floors.

Live load; $w_L = 40\text{psf}$ (1.92KN/m^2)

Note: Corbels are conservatively designed without a live load reduction.

Dead load; $w_D = 95.3\text{psf}$ (4.56KN/m^2) (calculated previously for the transverse bearing wall design)

Factored load combinations (per ANSI/ASCE 7-95);

By inspection, load case 2 governs;

$$V_u = \frac{1}{2}w_u L = \frac{1}{2}(1.2(95.3\text{psf}) + 1.6(40\text{psf}))23.5'\left(\frac{\text{kip}}{1000-\text{lb}}\right) = 2.1\text{klf} \quad (30.63\text{KN/m})$$

By inspection, the tensile loads on the support are negligible. Therefore, the requirements of ACI 318-95 Section 11.9.3.4 apply;

$N_{uc} = 0.2V_u = 0.2(2.1\text{klf}) = 0.42\text{klf}$ (6.13KN/m)

The moment at the face of support is calculated from the requirements of ACI 318-95 Section 11.9.3;

$$M_u = [V_u a + N_{uc}(h-d)]$$

where; $d = 12" - 0.75" - \frac{0.50"}{2} = 11"$ (279.4mm) (assuming #4 (10M) bars for reinforcement)

a = half of the bearing width = 2" (50.8mm)

$$\therefore M_u = 2.1\text{klf}(2")(1'/12") + 0.42\text{klf}(12"-11")(1'/12") = 0.39\frac{\text{ft}-\text{kips}}{\text{ft}} \quad (1.73\text{KN-m/m})$$

Design for shear;

$$\phi V_n \geq V_u \qquad \text{(EQ. 11-1 ACI 318-95)}$$

where; $\phi = 0.85$ per ACI 318-95 Section 11.9.3.1

$$V_n = A_{vf} f_y \mu \qquad \text{(EQ. 11-25 ACI 318-95)}$$

where; $\mu = 1.4(\lambda) = 1.4(1.0) = 1.4$

$$\therefore A_{vf} = \frac{V_u}{\phi f_y \mu} = \frac{2.1\text{klf}}{0.85(60\text{ksi})1.4} = 0.029\frac{\text{in}^2}{\text{ft}} \quad (61.3\text{mm}^2/\text{m})$$

Try 3 #3 (~10M) crossties spaced at 3-in. (76.2mm) o.c. ($A_{vf} = 0.33\text{-in}^2$ or $0.21\text{X}10^3\ \text{mm}^2$);

$\phi V_n = 0.85(27.7\text{klf}) = 23.6\text{klf} > 2.1\text{klf}(344.17\text{KN/m} > 30.63\text{KN/m}) = V_u$ **O.K.**

Check that $V_n < 0.2f'_c b_w d$ or $800 b_w d$ in accordance with ACI 318-95 Section 11.9.3.2.1;

$0.2f'_c b_w d = 0.2(4\text{ksi})(12"/\text{ft})11" = 106\text{klf} > 27.7\text{klf}(1.55\text{MN/m} > 0.40\text{MN/m}) = V_n$ **O.K.**

$800 b_w d = 800(12"/\text{ft})11" = 106\text{klf} > 27.7\text{klf} = V_n$ **O.K.**

Therefore use 3 #3 (~10M) bar crossties at 3-in. (76.2mm) o.c. vertical and spaced at 12-in. (0.30mm) o.c. horizontal

Determine reinforcement requirements for flexure 'A_f';

Per ACI 318-95 Section 11.9.3.3;

$$M_u \le \phi M_n = \phi A_f f_y\left(d - \frac{a}{2}\right) \qquad \text{where;}\quad a = \frac{A_f f_y}{0.85 f_c' b}$$

Solve for A_f per foot of wall length;

$$\therefore M_u \le \phi A_f f_y\left(d - \frac{1}{2}\left(\frac{A_f f_y}{0.85 f_c' b}\right)\right)$$

$$\Rightarrow \left[\frac{\phi(0.59)(f_y)^2}{f_c' b}\right]A_f^2 - \left[\phi f_y d\right]A_f + M_u = 0$$

The terms of the quadratic equation are evaluated as follows;

$$\left[\frac{\phi(0.59)(f_y)^2}{f_c' b}\right] = \frac{0.85(0.59)(60\text{ksi})^2}{4\text{ksi}(12")} = 37.6 - \text{kips}/\text{in}^2 \quad (0.26\text{KN/mm}^2)$$

$$\left[\phi f_y d\right] = 0.85(60\text{ksi})11" = 561 - \text{kips}/\text{in} \quad (98.2\text{KN/mm})$$

$$M_u = 0.39^k(12"/1') = 4.68^{\text{in-kips}} \quad (0.53\text{KN-m})$$

The roots of the quadratic equation are;

$$A_f \ge \frac{561 - \text{kips}/\text{in} \pm \sqrt{(561 - \text{kips}/\text{in})^2 - 4(37.6 - \text{kips}/\text{in}^2)4.68^{\text{in-kips}}}}{2(37.6 - \text{kips}/\text{in}^2)} = 0.0083 - \text{in}^2 \quad (5.35\text{mm}^2)$$

Note: The larger root is extraneous and therefore rejected

Therefore, $A_f \geqq 0.0083\text{-in}^2/\text{ft}$ ($17.5\text{mm}^2/\text{m}$)

Determine reinforcement requirements for tension A_n;
Per ACI 318-95 section 11.9.3.4;

$$N_{uc} \le \phi A_n f_y$$

$$A_n \ge \frac{N_{uc}}{\phi f_y} = \frac{0.42\text{klf}}{0.85(60\text{ksi})} = 0.0082 - \text{in}^2 \quad (5.29\text{mm}^2)$$

Therefore, $A_n \geqq 0.0082\text{-in}^2/\text{ft}$ ($17.3\text{mm}^2/\text{m}$)

Determine area of primary tension reinforcement A_s;
Per ACI 318-95 Section 11.9.3.5;

$$A_f + A_n = 0.0083 - \text{in}^2/\text{ft} + 0.0082 - \text{in}^2/\text{ft} = 0.017 - \text{in}^2/\text{ft} \quad (35.9\text{mm}^2/\text{m})$$

$$\frac{2}{3}A_{vf} + A_n = \frac{2}{3}(3(0.11 - \text{in}^2/\text{ft})) + 0.0082 - \text{in}^2/\text{ft} = 0.228 - \text{in}^2/\text{ft} \quad (482.2\text{mm}^2/\text{m}) \text{ (governs)}$$

Try 1 #4 (~10M) bar at 6" o.c.;

$$A_s = 2\left(\frac{0.20 - \text{in}^2}{\text{ft}}\right) = 0.40 - \text{in}^2/\text{ft} > 0.228 - \text{in}^2/\text{ft} \quad (845.9\text{mm}^2/\text{m} > 482.2\text{mm}^2/2) \qquad \textbf{O.K.}$$

Check minimum reinforcement requirements per ACI 318-95 Section 11.9.5;

$$\rho = \frac{A_s}{bd} \ge 0.04\left(\frac{f_c'}{f_y}\right)$$

Per foot of wall length;

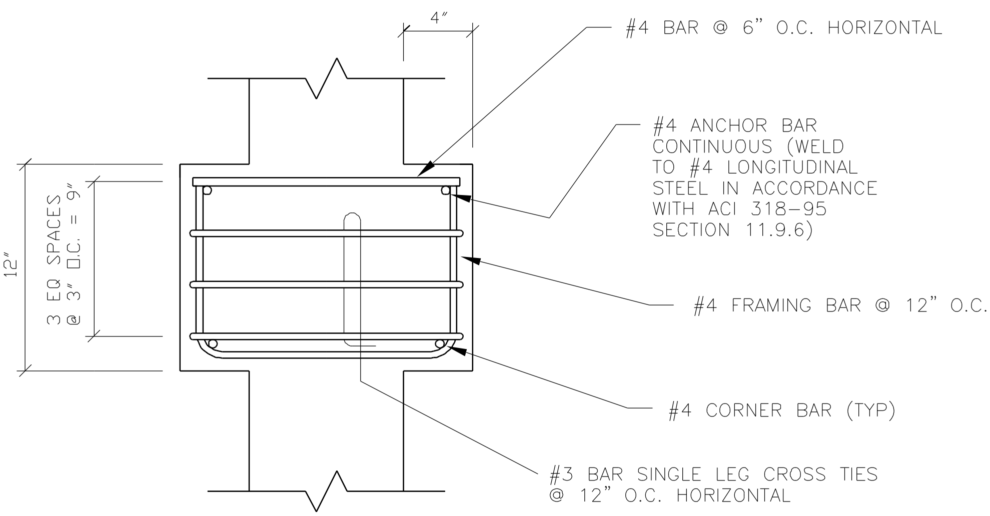
$$\rho = \frac{0.40 - \text{in}^2}{12"(11")} = 0.003 > 0.0027 = 0.04\left(\frac{4\text{ksi}}{60\text{ksi}}\right) = 0.04\left(\frac{f_c'}{f_y}\right) \qquad \textbf{O.K.}$$

Determine if additional ties are required per ACI 318-95 Section 11.9.4;

A_n provided within $(2/3)d = 2(0.11\text{-in}^2) = 0.22\text{-in}^2$ (141.9mm^2)
$(A_n)_{req'd} \geqq 0.5(A_s - A_n) = 0.5(0.40\text{-in}^2 - 0.0082\text{-in}^2) = 0.196\text{-in}^2$ (126.4mm^2)
$(A_n)_{provided} = 0.22\text{-in}^2 > 0.196\text{-in}^2 = (A_n)_{req'd}$ ($141.9\text{mm}^2 > 126.4\text{mm}^2$) **O.K.**

Therefore, use 1 #4 (~10M) bar at 6-in. (0.15m) on center for primary reinforcement

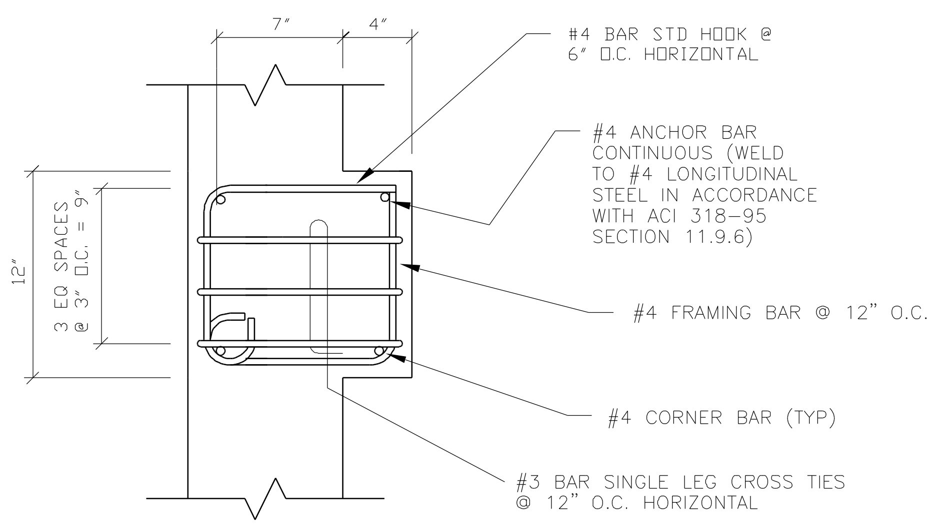
Therefore, the corbel design is as follows;



1-in = 25.4mm
#3 bar ~ 10M
#4 bar ~ 10M

At the end walls A_s must be anchored within the wall. A hoop arrangement as shown below will be used for this purpose.

The corbel design at the end walls is as follows;



See above for metric equivalents

(4) Transverse beam design

Transverse beams span over the interior 6-ft. (1.83m) wide corridor and support the floor and roof framing at these locations. The transverse beams are formed as a continuation of the corbel between the shear walls. One design for the worst case loading will be used throughout the building.

Determine design loads;
Note: Loading will be determined per unit length of the beam. Also, the worst case occurs at an interior span at either the second or third floor levels.

Tributary width per floor = 23'-6" (7.17m) (worst case conditon)

Live load;

Note: A_T = 23.5' x 6' =141-ft^2 < 400-ft^2 (13.1m^2 < 37.16m^2). Therefore, no live load reduction is allowed.

$\therefore w_{FL} = 40\text{psf} \quad (1.92\text{KN/m}^2)$

Dead load;

$w_{FD} = 95.3\text{psf} \quad (4.56\text{KPa})$ (as calculated previously)

By inspection, load case 2 (U=1.2D+1.6L+0.5L$_r$) governs;

$$w_u = [1.2(95.3\text{psf}) + 1.6(40\text{psf})]23.5' = 4.2\text{klf} \quad (61.25\text{KN/m})$$

Note: Beam has a fixed-fixed condition with the maximum positive moment occuring at mid span and the maximum negative moment occuring at its ends. The maximum shear also occurs at the member ends.

Design for flexure;

Positive moment at mid span;

$$(M_u^+)_{@\text{mid-span}} = \frac{w_u L^2}{24} = \frac{4.2\text{klf}(6')^2}{24}(12"/1') = 75.6^{\text{in-k}} \quad (8.54\text{KN-m})$$

Assume j = 0.9, and d = 12"-0.75"-0.375"-0.625"/2 = 10.6" (269.2mm) (assuming #5 (15M) bars for longitudinal steel, and #3 (~10M) bars for ties)

$$(A_s)_{\text{trial}} = \frac{M_u}{\phi f_y jd} = \frac{75.6^{\text{in-k}}}{0.9(60\text{ksi})0.9(10.6")} = 0.15-\text{in}^2 \quad (96.8\text{mm}^2)$$

Try 2-#5 bars, A_s = 2(0.31-in^2) = 0.62-in^2 (399.9mm^2)

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$$A_{s.min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200 b_w d}{f_y} \qquad \text{(EQ. 10-3 ACI 318-95)}$$

$$\therefore \rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000\text{psi}}}{60{,}000\text{psi}} = 0.00316$$

$$\text{or } \rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{200}{f_y} = \frac{200}{60{,}000\text{psi}} = 0.0033 \qquad \text{(governs)}$$

$$\rho = \frac{0.62-\text{in}^2}{17"(10.6")} = 0.0034 > \rho_{min} = 0.0033 \qquad \textbf{O.K.}$$

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \leq 175-\text{kips/in} \quad \text{(for interior exposure)} \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; f_s = 0.60(60ksi) = 36ksi (248.2MPa)

d_c = 12"-10.6" = 1.4" (35.6mm)

A = 17"(2)(1.4")/2 = 23.8-in^2 (15.35X10^3 mm^2)

$$\therefore z = 36\text{ksi}\sqrt[3]{1.4"(23.8-\text{in}^2)} = 116-\text{kips/in} < 175-\text{kips/in} \quad (30.65\text{KN/mm}) \qquad \textbf{O.K.}$$

Note: There is no need to check the upper limit on the amount of reinforcement (i.e., 0.75ρ_b) because the design is governed by the minimum amount of reinforcement.

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \quad \text{where; } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.62-\text{in}^2(60\text{ksi})}{0.85(4\text{ksi})17"} = 0.64" \quad (16.3\text{mm})$$

$$\phi M_n = 0.9(0.62-\text{in}^2)60\text{ksi}\left(10.6" - \frac{0.64"}{2}\right) = 344^{\text{in-kip}} \quad (38.87\text{KN-m})$$

$$\phi M_n = 344^{\text{in-kips}} > 75.6^{\text{in-kips}}(38.8\text{KN-m} > 8.54\text{KN-m}) = M_u^+ \qquad \textbf{O.K.}$$

Therefore, use 2 #5 (15M) bottom bars

Negative moment at member ends;

$$(M_u^-)_{@ends} = \frac{w_u L^2}{12} = 144^{in-kips} \quad (16.27KN\text{-}m)$$

By inspection, 2 #5 (15M) bars are required;

Use 2 #5 (15M) top bars

Design for shear;

Determine the strength reduction factor 'ϕ' in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength = $\phi V_n = V_u$

$$V_u = \frac{w_u L}{2} = \frac{4.2klf(6')}{2} = 12.6^k \quad (56.0KN)$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; L = 6' (1.83m) (clear span between shear walls)

$M_{pr1} = M_{pr2}$, and is calculated using $f_s = 1.25f_y$, and $\phi = 1.0$ as follows;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right) \quad \text{where;} \quad a = \frac{1.25A_s f_y}{0.85f_c' b}$$

$$a = \frac{1.25(0.62 - in^2)60ksi}{0.85(4ksi)17"} = 0.80" \quad (20.3mm)$$

$$\therefore M_{pr} = 1.25(0.62 - in^2)60ksi\left(10.6" - \frac{0.80"}{2}\right) = 474^{in-kips} \quad (53.56KN\text{-}m)$$

W = factored gravity load along span = w_u = 4.2klf (61.3KN/m)

$\therefore$ W = Lw_u = 6'(4.2klf) = 25.2^k (112.1KN)

$$\therefore V_e = \frac{2(474^{in-kips})(1'/12")}{6'} + \frac{25.2^k}{2} = 25.8^k \quad (114.8KN)$$

$$V_u = 12^k < 25.8^k (53.4KN < 114.8KN) = V_e$$

Therefore, ϕ = 0.6

Note: This design considers gravity loads only and will be reevaluated in the lateral load design (step B-11 of Table 4-5).

Determine if only minimum shear reinforcement is required per ACI 318-95 Section 11.5.6.1;

$$V_c = 2\sqrt{f_c'}b_w d = 2\sqrt{4000psi}\,17"(11")(1 - k/1000 - lb) = 23.7^k \quad (105.4KN)(\text{EQ. 11-1 ACI 318-95})$$

$$V_u = 12.6^k < 14.2^k (53.4KN < 63.2KN) = 0.6(23.7^k) = \phi V_c$$

Therefore, provide only minimum shear reinforcement per ACI 318-95 Section 11.5.5.3;

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

Determine spacing for #3 (~10M) hoops;

$$s = \frac{A_v f_y}{50b_w} = \frac{2(0.11 - in^2)60{,}000psi}{50(17")} = 15.5" \quad (393.7mm)$$

or $s = \frac{d}{2} = \frac{11"}{2} = 5.5"$ (139.7mm) (governs) (Per ACI 318-95 Section 11.5.4.1)

or $s = 24"$ (609.7mm)

Therefore, use #3 (~10M) hoops at 5" (127.0mm) o.c. over full length of beam

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;	$C_t = 0.020$	Shear wall system
Longitudinal direction;	$C_t = 0.030$	Reinforced concrete moment frame Resisting 100% of seismic forces
	h_n = 33-ft (10.06m)	Height to highest level

Therefore;

$$T_a = 0.020(33')^{3/4} = 0.28\,\text{sec} \qquad \text{transverse}$$

$$T_a = 0.030(33')^{3/4} = 0.41\,\text{sec} \qquad \text{longitudinal}$$

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

Note: For determining the base shear, the proportions of the moment frame elements are initially guessed. This is judged to provide adequate results because most of the weight of the building is due to the shear walls and the pre-cast concrete floor and roof framing thereby making the moment frames a relatively small percentage of the total building weight. The reproportioning of the moment frame elements is judged to have negligible effect on the base shear. The beams are proportioned first using a rule of thumb of one inch (25.4mm) of depth for every foot (0.305m) of span with the width being conservatively taken as 3/4 of the depth. The columns are then proportioned to have the same dimensions as the beams. The columns are proportioned as such because, by inspection, they support very little axial load and function primarily in flexure with a loading similar to that of the beams (pre-cast concrete planks spanning between bearing walls provide the primary gravity support, and the concrete frames are left to support only their own self weight).

Try a 24" (609.6mm) deep x 18" (457.2mm) wide floor beam and column, and 16" (406.4mm) deep x 18" (457.2mm) wide roof beam;

Check proportion requirements of ACI 318-95 Section 21.3, and 21.4;

Floor & Roof Beams;

Clear span = 21.5'>4(24")(1'/12") = 8' (2.44m)floor or >4(16")(1'/12") = 5.33' (1.63m)roof **O.K.**

$\frac{b}{h} = \frac{18"}{24"} = 0.75 > 0.3$ (floors) $\frac{b}{h} = \frac{18"}{16"} = 1.13 > 0.3$ (roof) **O.K.**

$b = 18" > 10"$ (floors) $b = 18" > 10"$ (roof) **O.K.**

(457.2mm > 254.0mm) (457.2mm > 254.0mm)

$b = 18" \leq W + \frac{3}{2}h = 54"$ (floors) $b = 18" \leq W + \frac{3}{2}h = 54"$ (roof) **O.K.**

(1371.6mm) (1371.6mm)

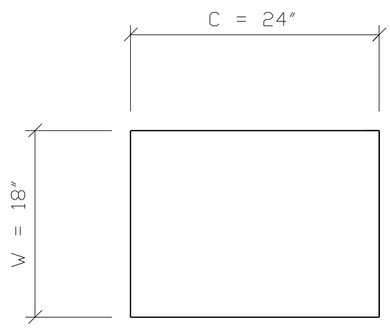
Column;

$\frac{W}{C} = \frac{18"}{24"} = 0.75 > 0.4$ **O.K.**

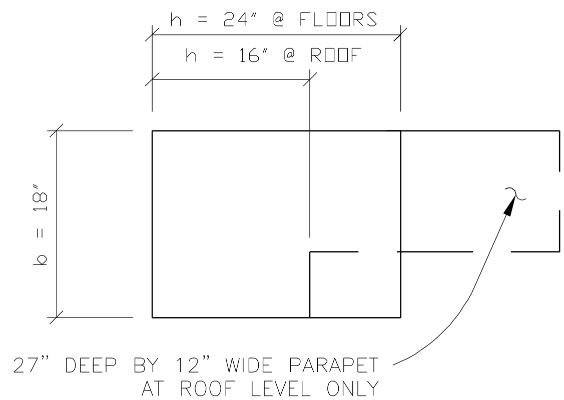
$W = 18" > 12"$ (457.2mm > 304.8mm) **O.K.**

Therefore, the moment frame cross sections are as follows;

Columns:



Beams:



1-in = 25.4mm

Building weights are calculated on spread sheet and are shown in figures 3, and 4.
Total seismic weight is;

$$W = 4{,}428^{k} \quad (19.70\text{MN})$$

B-3 *Calculate Base Shear, V:*

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

where;

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

and,

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

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

Transverse direction;

$$C_{s,trans} = \frac{S_{DS}}{R} = \frac{0.93}{5.5} = 0.17$$

$$C_{s,trans} = 0.17 < 0.33 = \frac{0.50}{0.28(5.5)} = \frac{S_{D1}}{TR}$$

$$C_{s,trans} = 0.17 > 0.041 = 0.044(0.93) = 0.044 S_{DS}$$

Therefore, $V_{trans} = C_s W = 0.17(4{,}428^{k}) = 753^{k}$ (3.35MN)

Longitudinal direction;

$$C_{s,long} = \frac{S_{DS}}{R} = \frac{0.93}{8} = 0.12$$

$$C_{s,long} = 0.12 < 0.15 = \frac{0.50}{0.41(8)} = \frac{S_{D1}}{TR}$$

$$C_{s,long} = 0.12 > 0.041 = 0.044(0.93) = 0.044 S_{DS}$$

Therefore, $V_{long} = C_s W = 0.12(4{,}428^{k}) = 531^{k}$ (2.36MN)

$V_{trans} = 753^{k}$ (3.35MN), $V_{long} = 531^{k}$ (2.36MN)

ASSEMBLY WEIGHTS (PSF):

DIAPHRAGMS:

Level - Roof				
Built-Up roofing;		5.00		
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00		
6" Thick hollow core concrete slab;		49.00		
2-1/2" Concrete topping; 2.5"(1'/12")150pcf =		31.30		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	92.3	psf	(4.42KN/m^2)
Level - Floors				
Floor finish;		1.00		
Partitions (10psf per FEMA 310 section 3.5.2.1);		10.00		
6" Thick hollow core concrete slab;		49.00		
2-1/2" Concrete topping; 2.5"(1'/12")150pcf =		31.30		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	95.3	psf	(4.56KN/m^2)

TRANSVERSE WALLS:

9" Thick reinforced concrete shear walls; 9"(1'/12")150pcf =	112.50	psf	(5.39KN/m^2)

LONGITUDINAL WALLS:

Infill;				
6" Thick medium weight concrete block wall grouted at 48" o.c.;		40.00	psf	
Windows;				
Glass, frame and sash;		8.00	psf	
Columns;				
18" wide by 24" deep columns (use 23.5' spacing); 18"(24")(1-ft^2/144-in^2)(150pcf)(1/23.5') =		19.15	psf	
Spandral Beams at floors;				
24" deep by 18" wide spandrel beams (use 11' tributary height); 24"(18")(1-ft^2/144-in^2)(150pcf)(1/11') =		40.91	psf	
Spandral Beams and parapet at roof;				
16" deep by 18" wide spandrel beams (use 5.5' tributary height); 16"(18")(1-ft^2/144-in^2)(150pcf)(1/5.5') =		54.55		
27" deep by 12" wide parapet; 27"(12")(1-ft^2/144-in^2)(150pcf)(1/5.5') =		61.36		
	Total =	115.9	psf	(5.55KN/m^2)

Figure 7. Calculation of component weights

BUILDING WEIGHTS (KIPS):

Item Desc.	Grid Line	Width or Trib. ht. (ft)	Length (ft)	Number	Trib Area $(ft)^2$	Unit Wt. (psf)	Weight Trans. (kips)	Weight Long. (kips)	Weight Total (kips)
Level - Roof									
Diaph.	A1-C2	29.5	12	1	354	92.3	32.7	32.7	32.7
Diaph.	A2-D8	53	141	1	7473	92.3	689.8	689.8	689.8
Diaph.	A8-D9	29.5	12	1	354	92.3	32.7	32.7	32.7
Longitudinal Walls									
Beam	A, C, D	5.5	165	2	1815	115.9	210.4		210.4
Glass[1]	A, C, D	3.46	165	2	1142	8.0	9.1		9.1
Columns[1]	A, C, D	3.46	165	2	1142	19.2	21.9		21.9
Transverse Walls									
Wall	2 to 8	5.5	53	7	2041	112.5		229.6	229.6
Wall	1 & 9	5.5	29.5	2	325	112.5		36.5	36.5
Parapet	2 & 8	1.71	23.5	2	80	112.5		9.0	9.0
Parapet	1 & 9	1.71	29.5	2	101	112.5		11.4	11.4
						Total Roof Tributary Weight =	996	1042	1283
Level - Floor									
Diaph.	A1-C2	29.5	12	1	354	95.3	33.7	33.7	33.7
Diaph.	A2-D8	53	141	1	7473	95.3	712.2	712.2	712.2
Diaph.	A8-D9	29.5	12	1	354	95.3	33.7	33.7	33.7
Longitudinal Walls									
Beam	A, C, D	11	165	2	3630	40.9	148.5		148.5
Infill	A, C, D	3	165	2	990	40.0	39.6		39.6
Glass[2]	A, C, D	6	165	2	1980	8.0	15.8		15.8
Columns[3]	A, C, D	9	165	2	2970	19.2	56.9		56.9
Transverse Walls									
Wall	2 to 8	11	53	7	4081	112.5		459.1	459.1
Wall	1 & 9	11	29.5	2	649	112.5		73.0	73.0
						Typical Floor Tributary Weight =	1040	1312	1573
								Total Building Weight =	4428

[1] Trib height at roof = 1/2(story height) - depth to bottom of beam = 5.5' - 16.0" - 8.5" = 3.46'

[2] Trib height at floor = story height - beam depth -infill depth = 11' - 2' - 3' = 6'

[3] Trib height at floor = story height - beam depth = 11' - 2' = 9'

1-in = 25.4mm
1-ft = 0.30m
1-kip = 4.448KN

Figure 8. Calculation of building weights

B-4 *Calculate Vertical Distribution of Forces.*

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

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

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

The calculations are tabularized below*;

Story Level	w_i (kips)	h_i (ft)	$w_i x h_i$ (ft-kips)	C_{vx}	$C_{vx} x V_{trans}$ $=F_{x,trans}$ (kips)	$C_{vx} x V_{long}$ $=F_{x,long}$ (kips)
Roof	1283	33	42337	0.45	338	239
3^{rd}	1573	22	34597	0.37	276	195
2^{nd}	1573	11	17299	0.18	138	98
SUM =	4428		94232	1.00	753	531

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

Therefore;

Transverse direction;
$F_{roof} = 338^k$
$F_{3rd} = 276^k$
$F_{2nd} = 138^k$

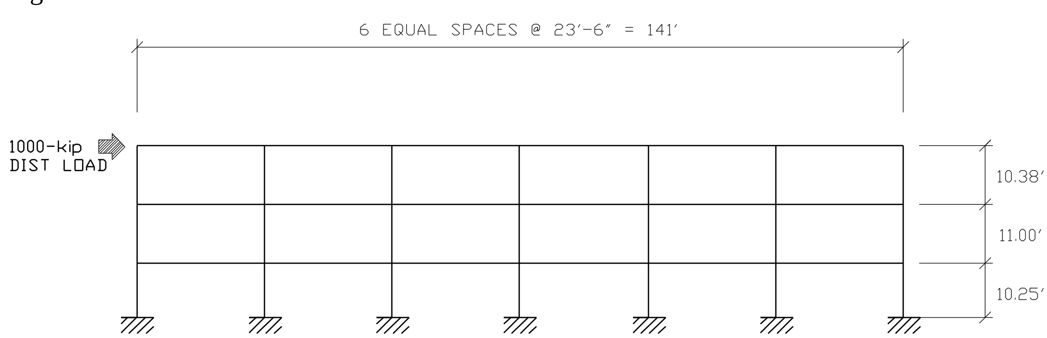
Longitudinal direction;
$F_{roof} = 239^k$
$F_{3rd} = 195^k$
$F_{2nd} = 98^k$

B-5 *Perform Static Analysis.*

General;

Because the diaphragms are rigid, relative rigidities of the lateral load resisting elements must be determined in order to establish the distribution of seismic loads. In the transverse direction, the shear walls are analyzed based on their flexural and shear deformations of a cantilever wall using closed form equations. In the longitudinal direction, moment frames are analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0) to determine their rigidity. Increased flexibility due to cracking for both the shear walls and the moment frames was accounted for by using cracked section properties in accordance with Section 10.11.1 of ACI 318-95.

The following diagram shows the computer model geometry used to model the longitudinal moment frames. Relative rigidities are determined for each floor level. For example, the stiffness at the roof is determined by applying a 1000^k load at the roof level (distributed uniformly along the length of the frame at that level) with no other loads acting on the model. The deflection of the frame is then taken as the average of all nodes at that level.



1-ft = 0.30m
1-kip = 4.448KN

Cracked section properties are determined as follows;

Floor Beams;

$$I_{cracked} = 0.35I_g = 0.35\left[\frac{1}{12}(18")(24")^3\right] = 7{,}258 - in^4 \ (3.02X10^9 \ mm^4),$$

Area = 432-in^2 (278.6X10^3 mm^2)

Roof Beams;

$$I_{cracked} = 0.35I_g = 0.35\left[\frac{1}{12}(18")(16")^3\right] = 2{,}150 - in^4 \ (0.89X10^9 \ mm^4),$$

Area = 288-in^2 (185.8X10^3 mm^2)

Note: It is conservative not to include the parapet in the roof beam cross section

Columns;

$$I_{cracked} = 0.70I_g = 0.70\left[\frac{1}{12}(18")(24")^3\right] = 14{,}515 - in^4 \ (6.04X10^9 \ mm^4),$$

Area = 432-in^2 (278.6X10^3 mm^2)

Therefore, the relative rigidities of the longitudinal moment frames are as follows;

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{5.26"} = 190^{kips/in} = 2{,}280^{kips/ft} \ (33.25MN/m)$$

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{2.35"} = 426^{kips/in} = 5{,}106^{kips/ft} \ (74.46MN/m)$$

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{0.61"} = 1{,}639^{kips/in} = 19{,}672^{kips/ft} \ (286.89MN/m)$$

Shear walls are analyzed using the following equation;

$$\Delta = \frac{Vh^3}{3E_cI_{cr}} + \frac{1.2Vh}{A_{cr}G}$$

(formula from the "Reinforced Masonry Handbook" by J. Amrhein, 5th ed. Section 4-3)

$$R = \frac{1{,}000^k}{\Delta}$$

where; V = lateral force on pier in (kips)
h = story to story height (in)
E_c = Modulus of elasticity of cracked concrete = 3,605ksi (for f_c' = 4,000psi) (24.86MPa for f_c' = 27.58MPa)
$I_{cr} = 0.70I_g$ = Moment of inertia of cracked wall
$A_{cr} = A_g$ = Gross wall area; thickness x length
G = Shear modulus = $E_c/(2(1+v))$, with v = 0.18.
G = 1,528ksi (10.53MPa)

Note: The shear walls consist of two walls in series. These walls are connected by a flexible floor beam, but are not a coupled wall system. Calculations for the stiffness of the walls at each floor level are tabularized below;

Note: Calculations use V = 1000^k (4.448MN). Also, I_{cr} uses a value of 0.7 > 0.35 as specified in the code because it is anticipated that the walls are relatively understressed and that a value of 0.35 would be overly conservative.

Floor Level	Grid Lines	h (in)	$I_{cr} = 0.7(tL^3/12)$ t (in)	L (in)	I_{cr} ($x10^3$-in^4)	A_{cr} (in^2)	Δ (in)	$\Sigma\Delta_{below}$ (in)	K = $1^k/\Sigma\Delta_{below}$ (kips/ft)
Second	2 to 8	132	9	282	11,774	2,538	5.89E-05	5.89E-05	1,415
Third	2 to 8	132	9	282	11,774	2,538	5.89E-05	1.18E-04	707
Roof	2 to 8	132	9	282	11,774	2,538	5.89E-05	1.77E-04	472
Second	1 & 9	132	9	354	23,290	3,186	4.17E-05	4.17E-05	2,000
Third	1 & 9	132	9	354	23,290	3,186	4.17E-05	8.33E-05	1,000
Roof	1 & 9	132	9	354	23,290	3,186	4.17E-05	1.25E-04	667

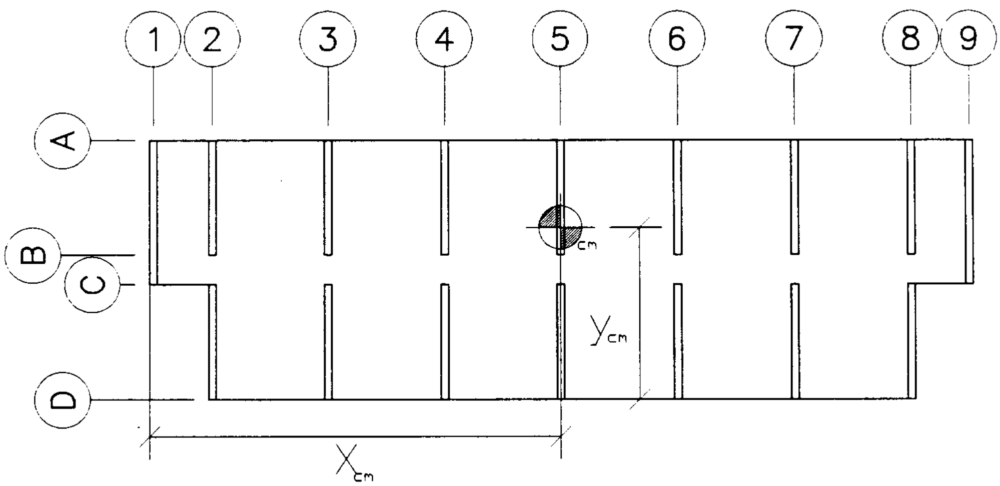
1-in = 25.4m
$1\text{-in}^2 = 0.645X10^3 \text{ mm}^2$
$1\text{-in}^4 = 0.416X10^6 \text{ mm}^4$

B-6 *Determine cr and cm.*

Center of mass, cm;

By inspection, the center of mass in the longitudinal direction lies at the plan centroid. Therefore, it is only necessary to calculate the center of mass in the transverse direction.

The center of mass is defined as; $y_{cm} = \dfrac{\sum w_i y_i}{\sum w_i}$



For the calculations which follow, it is convenient to use an equivalent weight for exterior walls in pounds per square foot (N/m^2). Unit weights and tributary areas were previously calculated in the seismic weights section of this problem.

Roof;
Walls on grid lines A, C, and D;

$$\text{Equivalent wall wt.} = 115.9\text{psf} + 19.15\text{psf}\left(\frac{3.46'}{5.5'}\right) + 8\text{psf}\left(\frac{3.46'}{5.5'}\right) = 133\text{psf} \quad (6.37\text{KN/m}^2)$$

Walls on grid lines 1 through 9;
Equivalent wall wt. = Actual wall wt. = 112.5psf (5.39KN/m^2)

Floors;
Walls on grid lines A, C, and D;

$$\text{Equivalent wall wt.} = 40.91\text{psf} + 19.15\text{psf}\left(\frac{9'}{11'}\right) + 8\text{psf}\left(\frac{6'}{11'}\right) + 40\text{psf}\left(\frac{3'}{11'}\right) = 71.9\text{psf} \quad (3.44\text{KN/m}^2)$$

Walls on grid lines 1 through 9;
Equivalent wall wt. = Actual wall wt. = 112.5psf (5.39KN/m^2)

Calculations for the location of the mass centroid are tabularized below;

LOCATION OF MASS CENTROID OF ROOF IN THE TRANSVERSE DIRECTION

Element	Grid Line	Width or Height (ft)	Length (ft)	y_i (ft)	Area (ft^2)	Unit wt. (psf)	Number	w_i (kips)	w_iy_i (ft-kips)
Diaph	A1-C2	29.5	12	38.25	354	92.3	1	33	1,250
	A2-D8	53	141	26.5	7473	92.3	1	690	18,279
	A8-C9	29.5	12	38.25	354	92.3	1	33	1,250
Trans Walls	A1-C1	7.21	29.5	38.25	213	112.5	1	24	915
	A9-C9	7.21	29.5	38.25	213	112.5	1	24	915
	A2-D2 to A8-D8	5.5	53	26.5	291.5	112.5	7	230	6,083
Trans Parapet	C2-D2 & C8-D8	2	24	12	40	113	2	9	106
Long Walls	A1-A9	5.5	165	53	907.5	133	1	121	6,397
	C1-C2	5.5	12	23.5	66	133	1	9	206
	C8-C9	5.5	12	23.5	66	133	1	9	206
	D2-D8	5.5	141	0	775.5	133	1	103	0
Therefore, for the roof level;							SUM =	1,283	35,608

y_{cm} = 27.75 ft (8.46m) x_{cm} = 82.50 ft (25.2m)

LOCATION OF MASS CENTROID OF FLOOR IN THE TRANSVERSE DIRECTION

Element	Grid Line	Width or Height (ft)	Length (ft)	y_i (ft)	Area (ft^2)	Unit wt. (psf)	Number	w_i (kips)	w_iy_i (ft-kips)
Diaph	A1-C2	29.5	12	38.25	354	95.3	1	34	1,290
	A2-D8	53	141	26.5	7473	95.3	1	712	18,873
	A8-C9	29.5	12	38.25	354	95.3	1	34	1,290
Trans Walls	A1-C1	11	29.5	38.25	324.5	112.5	1	37	1,396
	A9-C9	11	29.5	38.25	324.5	112.5	1	37	1,396
	A2-D2 to A8-D8	11	53	26.5	583	112.5	7	459	12,166
Long Walls	A1-A9	11	165	53	1815	71.9	1	130	6,916
	C1-C2	11	12	23.5	132	71.9	1	9	223
	C8-C9	11	12	23.5	132	71.9	1	9	223
	D2-D8	11	141	0	1551	71.9	1	112	0
Therefore, at the floor level;							SUM =	1,573	43,775

y_{cm} = 27.83 ft (8.48m) x_{cm} = 82.50 ft (25.2m)

1-in = 25.4mm 1psf = 47.9KN/m^2
1-ft = 3.0mm 1-kip = 4.448KN
1-ft^2 = 0.093m^2 1-ft-kip=1.36KN-m

Center of rigidity, cr;

Due to the symmetrical layout of the lateral load resisting elements, the center of rigidity can be located by inspection.

For both the floors and the roof; **y_{cr} = 26.5 ft (8.08m)** **x_{cr} = 82.5 ft (25.2m)**

B-7 *Perform torsional analysis.*

The load distributed to each element of the lateral load resisting system in a particular direction can be determined by its stiffness relative to all the other elements in that direction.

The direct shear component is F_v;

$$F_v = \frac{R_i}{\sum R_i} V$$

Additionally, there are two torsional components; F_{tors}, and F_{acc};

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

where; e = eccentricity due to the displacement of the center of mass to the center of rigidity

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

where; e_{acc} = eccentricity to account for possible errors in calculating the cm or cr.

The shear component and the two torsional components add together to determine the total force transferred to an element of the lateral load resisting system. Calculations for the distribution of forces to the lateral load resisting elements at each floor level are tabularized below;

Roof;

V_{trans} =	338	kips	e_{trans} =	0	ft	$e_{trans,acc}$ =	8.25	ft
V_{long} =	239	kips	e_{long} =	1.25	ft	$e_{long,acc}$ =	2.65	ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	d_{xi} (1) (ft)	d_{yi} (1) (ft)	$R_x d_y$ or $R_y d_x$ (k-ft/ft)	$R_x d_y^2$ or $R_y d_x^2$ (k-ft/ft)	F_v (kips)	F_t (2) (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	667	-	82.5	-	55027.5	4539769	28.40	0.00	5.71	34.1
	A2-B2 & C2-D2 (3)	944	-	70.5	-	66552.0	4691916	40.20	0.00	6.91	47.1
	A3-B3 & C3-D3 (3)	944	-	47	-	44368.0	2085296	40.20	0.00	4.61	44.8
	A4-B4 & C4-D4 (3)	944	-	23.5	-	22184.0	521324	40.20	0.00	2.30	42.5
	A5-B5 & C5-D5 (3)	944	-	0	-	0.0	0	40.20	0.00	0.00	40.2
	A6-B6 & C6-D6 (3)	944	-	23.5	-	22184.0	521324	40.20	0.00	2.30	42.5
	A7-B7 & C7-D7 (3)	944	-	47	-	44368.0	2085296	40.20	0.00	4.61	44.8
	A8-B8 & C8-D8 (3)	944	-	70.5	-	66552.0	4691916	40.20	0.00	6.91	47.1
	A9-C9	667	-	82.5	-	55027.5	4539769	28.40	0.00	5.71	34.1
Long	A2-A8	-	2280	-	26.5	60420	1601130	119.37	0.67	1.42	121.5
	D2-D8	-	2280	-	26.5	60420	1601130	119.37	-0.67	1.42	120.8
	ΣR_i =	7942	4560			J_p =	26878870				

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

[2] Note: only positive components are added

[3] Note: there are two walls in parallel with a total rigidity of; 2 x 472-kips/ft = 944-kips/ft

Note: For metric equivalents; 1-kip/ft = 14.58KN/m, 1-ft = 0.30m, 1-kip = 4.48KN

Second Floor;

V_{trans} = 138 kips e_{trans} = 0 ft $e_{trans,acc}$ = 8.25 ft

V_{long} = 98 kips e_{long} = 1.33 ft $e_{long,acc}$ = 2.65 ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	d_{xi}[1] (ft)	d_{yi}[1] (ft)	$R_x d_y$ or $R_y d_x$ (k-ft/ft)	$R_x d_y^2$ or $R_y d_x^2$ (k-ft/ft)	F_v (kips)	F_t[2] (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	2,000	-	82.5	-	165000.0	13612500	11.61	0.00	1.91	13.5
	A2-B2 & C2-D2[3]	2,830	-	70.5	-	199515.0	14065808	16.42	0.00	2.31	18.7
	A3-B3 & C3-D3[3]	2,830	-	47	-	133010.0	6251470	16.42	0.00	1.54	18.0
	A4-B4 & C4-D4[3]	2,830	-	23.5	-	66505.0	1562868	16.42	0.00	0.77	17.2
	A5-B5 & C5-D5[3]	2,830	-	0	-	0.0	0	16.42	0.00	0.00	16.4
	A6-B6 & C6-D6[3]	2,830	-	23.5	-	66505.0	1562868	16.42	0.00	0.77	17.2
	A7-B7 & C7-D7[3]	2,830	-	47	-	133010.0	6251470	16.42	0.00	1.54	18.0
	A8-B8 & C8-D8[3]	2,830	-	70.5	-	199515.0	14065808	16.42	0.00	2.31	18.7
	A9-C9	2,000	-	82.5	-	165000.0	13612500	11.61	0.00	1.91	13.5
Long	A2-A8	-	19672	-	26.5	521308	13814662	48.77	0.69	1.37	50.8
	D2-D8	-	19672	-	26.5	521308	13814662	48.77	-0.69	1.37	50.1
	ΣR_i =	23810	39344			J_p =	98614614				

Third Floor;

V_{trans} = 276 kips e_{trans} = 0 ft $e_{trans,acc}$ = 8.25 ft

V_{long} = 195 kips e_{long} = 1.33 ft $e_{long,acc}$ = 2.65 ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	d_{xi}[1] (ft)	d_{yi}[1] (ft)	$R_x d_y$ or $R_y d_x$ (k-ft/ft)	$R_x d_y^2$ or $R_y d_x^2$ (k-ft/ft)	F_v (kips)	F_t[2] (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	1000	-	82.5	-	82500.0	6806250	23.23	0.00	4.41	27.6
	A2-B2 & C2-D2[3]	1414	-	70.5	-	99687.0	7027934	32.85	0.00	5.33	38.2
	A3-B3 & C3-D3[3]	1414	-	47	-	66458.0	3123526	32.85	0.00	3.55	36.4
	A4-B4 & C4-D4[3]	1414	-	23.5	-	33229.0	780882	32.85	0.00	1.78	34.6
	A5-B5 & C5-D5[3]	1414	-	0	-	0.0	0	32.85	0.00	0.00	32.8
	A6-B6 & C6-D6[3]	1414	-	23.5	-	33229.0	780882	32.85	0.00	1.78	34.6
	A7-B7 & C7-D7[3]	1414	-	47	-	66458.0	3123526	32.85	0.00	3.55	36.4
	A8-B8 & C8-D8[3]	1414	-	70.5	-	99687.0	7027934	32.85	0.00	5.33	38.2
	A9-C9	1000	-	82.5	-	82500.0	6806250	23.23	0.00	4.41	27.6
Long	A2-A8	-	5106	-	26.5	135309	3585689	97.55	0.82	1.64	100.0
	D2-D8	-	5106	-	26.5	135309	3585689	97.55	-0.82	1.64	99.2
	ΣR_i =	11898	10212			J_p =	42648559				

See page H2-24 for metric equivalents

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

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

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

Evaluation of r_{max};

For the longitudinal moment frames, r_{max} is taken as the maximum of the sum of the shears in any two adjacent columns in the plane of the frame divided by the story shear. The portal method is used as an approximation for the distribution of shear in the columns.

Therefore, since each story has 7 columns per frame, 2 frames per story, and the frames are each equally loaded;

$$r_{max} = \frac{2}{12} = 0.17 \text{ at each floor level}$$

For the transverse shear walls, r_{max} is taken as the shear in the most heavily loaded wall divided by $10/l_w$. Where l_w is the wall length in feet divided by the story shear. At every story level, the most heavily loaded wall occurs at either grid lines 1 or 9.

Therefore, at each story level;

$$\frac{10}{l_w} = \frac{10}{29.5'} = 0.339$$

At the third story level;

$$r_{max} = \frac{34.1^k}{338^k}(0.339) = 0.034$$

At the second story level;

$$r_{max} = \frac{27.6^k}{276^k}(0.339) = 0.034$$

At the first story level;

$$r_{max} = \frac{13.7^k}{138^k}(0.339) = 0.034$$

Calculations to determine the redundancy factor are tabularized below;

Story Level	Earthquake Direction	r_{max}	A_x (ft^2)	r_x
Third	Transverse	0.034	8,181	-4.5 < 1
	Longitudinal	0.170	8,181	0.70 < 1.0
Second	Transverse	0.034	8,181	-4.50 < 1.0
	Longitudinal	0.170	8,181	0.70 < 1.0
First	Transverse	0.034	8,181	-4.50 < 1.0
	Longitudinal	0.170	8,181	0.70 < 1.0

Therefore, for all story levels; $r_{x,trans}$ = 1.0, $r_{x,long}$ = 1.0

B-9 *Determine need for overstrength factor* Ω_o.

Per Paragraph 5.2.6.3.2 of FEMA 302, collector elements, splices, and their connections to resisting elements shall be designed for the load combinations of Section 5.2.7.1 of FEMA 302. Therefore, for these force controlled elements the following seismic load will be used;

For both the transverse or the longitudinal direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \qquad \text{(EQ. 4-6)}$$
$$E = 3Q_E \pm 0.2(0.93)D$$
$$E = 3Q_E \pm 0.186D$$

B-10 *Calculate combined load effects.*

Load combinations per ANSI/ASCE 7-95 are as follows;

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

However, per Paragraph 4-6;

$E = \rho Q_E$ 6 $0.2S_{DS}D$
$= 1.0Q_E$ 6 $0.2(0.93)D$
$= Q_E$ 6 $0.186D$

Therefore, Equations 4 and 5 become;

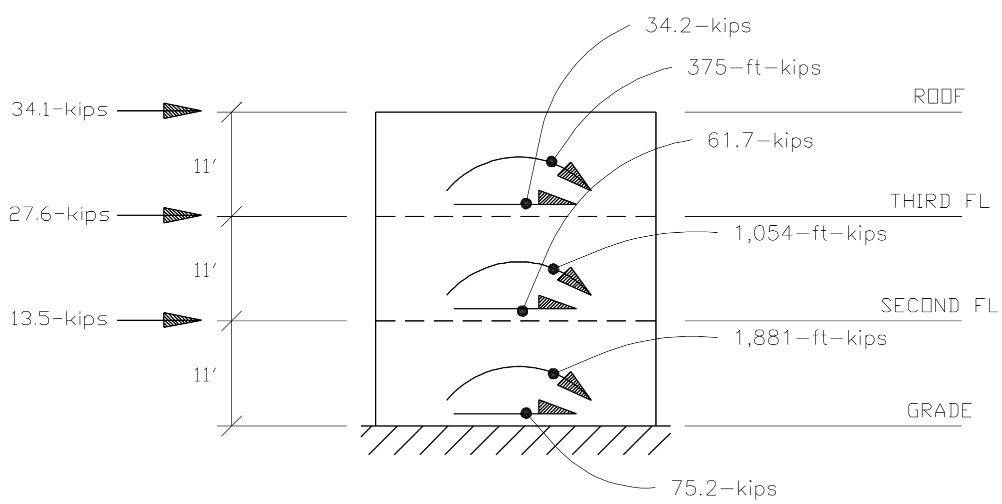
(4a) $1.386D+Q_E+0.5L$
(5a) $0.714D+Q_E$

B-11 *Determine structural member sizes.*

(1) Transverse shear wall design; use f_c' = 4,000psi (27.58MPa), and f_y = 60ksi (413.7MPa)

Per Paragraph 7-2.f. (1), shear walls will be designed in accordance with ACI 318-95 as modified by the provisions given in Chapter 9 of FEMA 303.

Walls on grid lines 1 and 9;
Determine design loads;
The following diagram shows the maximum shear and overturning moment at each level;



1-ft = 0.30m, 1-kip = 4.448KN

Therefore, at the base of the wall V_{max} = 75.2-kips (0.33MN), and M_{max} = 1,881$^{ft\text{-}kips}$ (2.55MN-m).

Design for shear;
Determine if reinforcement is required to resist shear;

$$V_u \le A_{cv}\sqrt{f_c'} \qquad \text{(per ACI 318-95 Section 21.6.2.1)}$$

$$V_u = 75.2^k < 202^k (334.5KN < 898.5KN) = (29.5')9"\sqrt{4{,}000psi}(12"/1') = A_{cv}\sqrt{f_c'}$$

Therefore, only minimum reinforcement is required.

By inspection, only one curtain of reinforcement is required. However, because a 9-in. thick wall is being used, it is decided to use two curtains in order to distribute the reinforcement more evenly throughout the section. Also, per ACI 318-95 Section 21.6.2.1, the minimum reinforcement required need only comply with Section 14.3 of ACI 318-95. Therefore, the minimum ratio of vertical reinforcement area to gross concrete area is 0.0012 (for bars smaller than number 5), and the minimum ratio of horizontal reinforcement area to gross concrete area is 0.0020 (also for bars smaller than number 5).

Try 2 curtains of #4 (~10M) bar at 12-in. (308.8mm) on center each way;
Check minimum reinforcement per ACI 318-95 Section 14.3;

$$\rho_h = \frac{2A_{sh}}{bt} = \frac{2(0.20-in^2)}{12"(9")} = 0.0037 > 0.0020 = \rho_{req'd} \qquad \textbf{O.K.}$$

$$\rho_v = \frac{2A_{sv}}{bt} = \frac{2(0.20-in^2)}{12"(9")} = 0.0037 > 0.0012 = \rho_{req'd} \qquad \textbf{O.K.}$$

Check spacing per ACI 318-95 Section 14.3.5;

18" < 3 x (wall thickness) = 3(9") = 27" Therefore, s_{max} = 18" (457.2mm)

$s_{vert} = s_{horiz}$ = 12" < s_{max} = 18" (308.8mm < 457.2mm) **O.K.**

Check capacity;

$$V_u \le \phi V_n \qquad \text{with } \phi = 0.6 \qquad \text{(EQ. 11-1 ACI 318-95)}$$

Since h_w/l_w = 33'/29.5' = 1.12 < 2.0 in accordance with Section 21.6.5.3 of ACI 318-95;

$$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y) \qquad \text{(EQ. 21-7 ACI 318-95)}$$

where; A_{cv} = 29.5'(9")(12"/1') = 3,186-in^2 (2.05X10^6 mm^2)
f_c' = 4,000psi (46.90MPa)
α_c = 3.0 for h_w/l_w = 1.4 < 1.5
ρ_n = 0.0037
f_y = 60ksi (413.7MPa)

Note: The more conservative Equation 21-6 could have been used due to the light loading.

$$\therefore \phi V_n = \frac{0.6(3{,}186-in^2)}{1000-kips/lb}\left(3\sqrt{4{,}000psi} + 0.0037(60{,}000psi)\right) = 787^k \ (3.50MN)$$

$$\phi V_n = 787^k > 75.2^k = V_u \quad (3.50MN > 0.33MN) \qquad \textbf{O.K.}$$

Design for flexural and axial loads;
Determine axial loads acting at each story;
Note: The tributary width of diaphragm is 6-ft., and the story to story height is 11-ft (3.36m).
Dead loads;

At the roof; P_{RD} = 92.3psf(6') + 112.5psf(5.5') = 1,173plf (17.11KN/m)
At the floors; P_{FD} = 95.3psf(6') + 112.5psf(11') = 1,810plf (26.40KN/m)

Live loads; (Live loads are unreducable due to the small tributary area)

At the roof; P_{RL} = 20psf(6') = 120plf (1.75KN/m)
At the floors; P_{FL} = 40psf(6') = 240plf (3.50KN/m)

Confirm that walls are structural;
Per FEMA 302 Section 9.1.1.13, walls with P_u > 0.35P_o shall not be considered to contribute to the calculated strength of the structure for resisting earthquake induced forces. By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = [1.386(1.17+2(1.81))klf + 0.5(2(0.24))klf]29.5' = 203^k (902.9KN)

The nominal strength of the wall is given by;

$$P_\circ = 0.85f_c^{'}(A_g - A_{st}) + f_y A_{st} \qquad \text{(EQ. 10-2 ACI 318-95)}$$

where; A_g = 29.5'(12"/1')9" = 3,186-in^2 (2.05X10^6 mm^2)
$A_{st} = l_w t \rho_v$ = 29.5'(12"/1')9"(0.0037) = 11.8-in^2 (7.61X10^3 mm^2)
f_c' = 4,000psi (46.90MPa)

$$P_\circ = 0.85(4{,}000\text{psi})(3{,}186 - \text{in}^2 - 11.8 - \text{in}^2) + 60{,}000\text{psi}(11.8 - \text{in}^2) = 11{,}500^k \text{ (51.15MN)}$$

$$\therefore P_u = 203^k << 4{,}025^k = 0.35P_\circ \text{ (902.9KN<<18.70MN)} \qquad \text{Therefore, walls are structural}$$

Determine if boundary elements are required;
This requirement will be checked at the first story level only for this is the worst case condition, and if boundary elements are not required at that level they will not be required at the levels above.
Per FEMA 302 Section 9.1.1.13, boundary elements are not required if;

(1) $P_u \le 0.10 A_g f_c^{'}$

and either

(2) $\dfrac{M_u}{V_u l_w} \le 1.0$

or (3) $V_u \le 3A_{cv}\sqrt{f_c^{'}}$, and $\dfrac{M_u}{V_u l_w} \le 3.0$

Check condition (1);
By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;
P_u = 203^k << 1,273^k = 0.1(29.5')(12"/1')9"(4,000psi)(1^k/1000-lb) = 0.1$A_g f_c$' **O.K.**
(902.9KN<<5.66MN)

Check condition (2);
By inspection, M_u and V_u are the same for all load cases. Therefore;

$$\frac{M_u}{V_u l_w} = \frac{1{,}881^{ft-kips}}{75.2^k(29.5')} = 0.85 < 1.0 \qquad \textbf{O.K.}$$

Therefore, no boundary elements are required

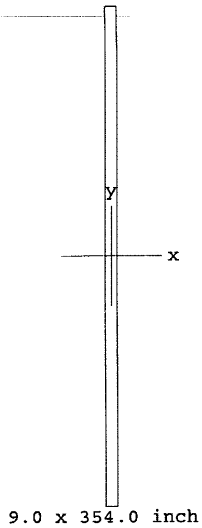
Determine if wall has adequate capacity for flexural and axial loads combined;
Per FEMA 302 Section 9.1.1.13, walls subject to combined flexural and axial loads shall be designed in accordance with Sections 10.2 and 10.3 of ACI 318-95 except that Section 10.3.6 of ACI 318-95 and the nonlinear strain requirements of 10.2.2 do not apply. To satisfy these requirements, an analysis program entitled 'PCACOL' produced by the Portland Cement Association was utilized. This program produces an interaction diagram for the wall cross section and plots the loads acting on the section.

Note: At the time of writing of this problem, the current version of PCACOL program was developed for ACI 318-89. However, there are no changes between the 95 code and the 89 code which will affect the results of this shear wall analysis.

Determine design loads;
Load case 4a; P_u = [1.386(1.17+2(1.81))klf + 0.5(2(0.24))klf]29.5' = 203^k (902.9KN)
M_u = 1.0(1,881-ft-kips) = 1,881-ft-kips (2.55MN-m)
Load case 5a; P_u = [0.714(1.79+2(1.81))klf]29.5' = 114^k (507.1KN)
M_u = 1.0(1,881-ft-kips) = 1,881-ft-kips (2.55MN-m)

Figure 9 shows the design interaction diagram (obtained from PCACOL) for the shear wall section. The section has a 9-in. (220.6mm) thick web reinforced with two curtains of reinforcement each having #4 (~10M) vertical and horizontal bars spaced at 12-in. (308.8mm) on center with f_c' = 4,000psi (46.90MPa) and f_y = 60,000psi (413.7MPa). Two design load combinations are listed. The point marked "1" represents the P_u-M_u combination corresponding to load case 4a, and the point marked "2" represents the P_u-M_u combination corresponding to load case 5a. This figure shows that the walls have sufficient capacity for axial and overturning forces.

Therefore, use 2 curtains of #4 (~10M) bars at 12" (308.8mm) o.c. each way



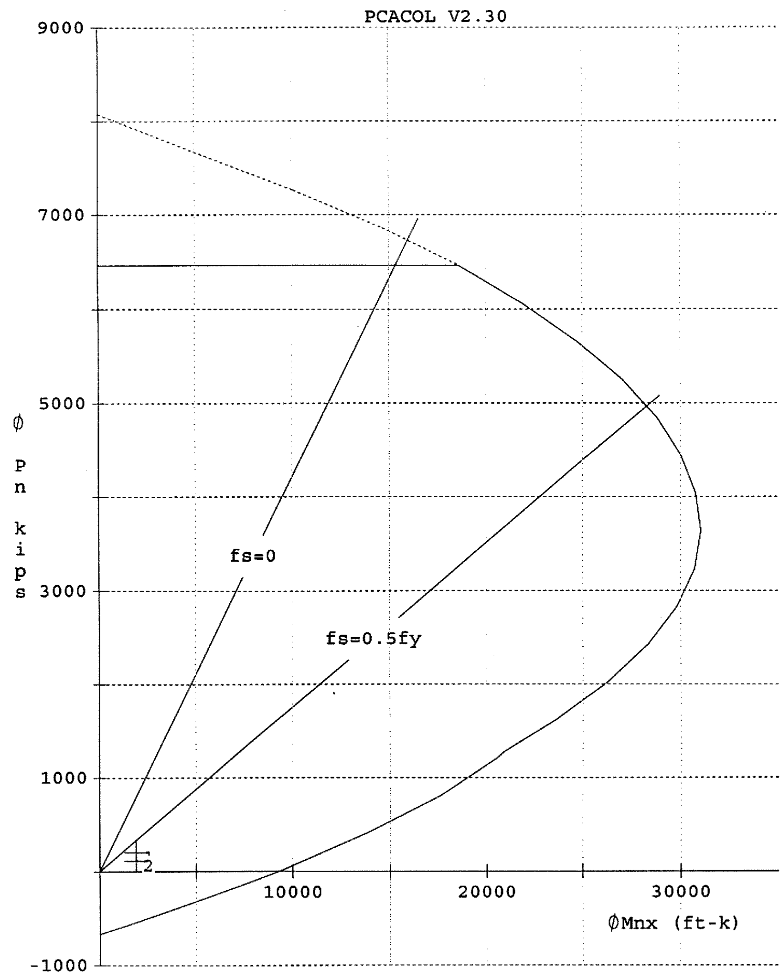
f'c = 4.0 ksi

fy = 60.0 ksi

Confinement: Tied
clr cover = 0.68 in
spacing = 6.62 in
62-#4 at 0.39%
As = 12 in^2

Ix = 33271398 in^4
Iy = 21506 in^4

Xo = 0.00 in
Yo = 0.00 in



File name: H:\USERS\SBURNS\H2\WALL1-9.COL

Project: TI 809-04	Material Properties:
Column Id:	Ec = 3834 ksi eu = 0.003 in/in
Engineer:	fc = 3.40 ksi Es = 29000 ksi
Date: Time:	Beta1 = 0.85
Code: ACI 318-89	Stress Profile: Block
Units: in-lb	phi(c) = 0.70, phi(b) = 0.90

X-axis slenderness is not considered.

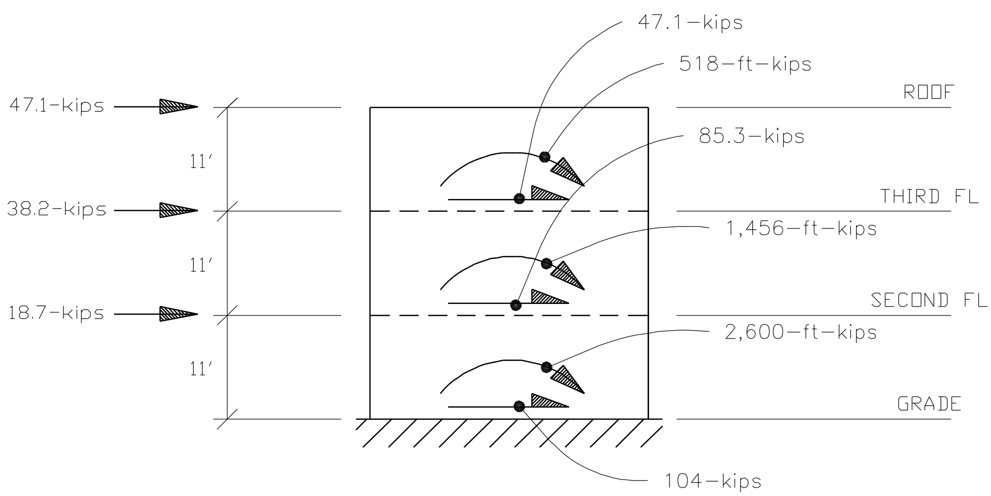
Note: For metric equivalents; 1-in = 25.4mm, 1-ft-kip = 1.356KN-m, 1-ksi = 6.895MPa

Figure 9. Design strength interaction diagram for shear wall section on grid lines 1 and 9

Walls on grid lines 2 through 8;

One design will be produced for the worst case and used for all walls on grid lines 2 through 8. By inspection, the worst case situation occurs on either grid line 2 or 8 between grid lines C and D because these walls carry the largest shear, largest overturning moment, and smallest axial load (the walls moment capacity increases with an increase in axial load).

Determine design loads;
The following diagram shows the maximum shear and overturning moment at each level;



1-ft = 0.30m, 1-kip = 4.448KN

Therefore, at the base of the wall V_{max} = 104-kips (462.6KN), and M_{max} = 2,600-ft-kips (3.53MN-m).

Design for shear;
By inspection, only minimum reinforcement will be required.

Try 2 curtains of #4 (~10M) bar at 12-in. (308.8mm) on center each way as used for walls on grid lines 1 and 9;

Check capacity;

$V_u \leq \phi V_n$ with $\phi = 0.6$ (EQ. 11-1 ACI 318-95)

Since h_w/l_w = 33'/26.5' = 1.25 < 2.0 in accordance with Section 21.6.5.3 of ACI 318-95;

$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y)$ (EQ. 21-7 ACI 318-95)

where; A_{cv} = 23.5'(9")(12"/1') = 2,538-in^2 (1.64X10^6 mm^2)

f_c' = 4,000psi (46.90MPa)

α_c = 3.0 for h_w/l_w = 1.3 < 1.5

ρ_n = 0.0037

f_y = 60ksi (413.7MPa)

Note: The more conservative Equation 21-6 could have been used due to the light loading.

$$\therefore \phi V_n = \frac{0.6(2{,}538-\text{in}^2)}{1000-\text{kips/lb}}\left[3\sqrt{4{,}000\text{psi}} + 0.0037(60{,}000\text{psi})\right] = 627^k \quad (2.79\text{MN})$$

$$\phi V_n = 627^k > 104^k = V_u \quad (2.79\text{MN} > 462.6\text{KN})$$ **O.K.**

Design for flexural and axial loads;
Determine axial loads acting at each story;
Note: The tributary width of diaphragm is 23.5-ft(1/2) = 11.8-ft. (3.60m), and the story to story height is 11-ft (3.36m).

Dead loads;

At the roof; P_{RD} = 92.3psf(11.8') + 112.5psf(5.5') = 1,708plf (24.91KN/m)

At the floors; P_{FD} = 95.3psf(11.8') + 112.5psf(11') = 2,362plf (34.45KN/m)

Live loads; (Live loads are unreducable due to the small tributary area)

At the roof; P_{RL} = 20psf(11.8') = 236plf (3.44KN/m)

At the floors; P_{FL} = 40psf(11.8') = 472plf (6.88KN/m)

Confirm that walls are structural;

Per FEMA 302 Section 9.1.1.13, walls with P_u > $0.35P_o$ shall not be considered to contribute to the calculated strength of the structure for resisting earthquake induced forces. By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = [1.386(1.71+2(2.36))klf + 0.5(0.236+2(0.472))klf]29.5' = 280^k (1.25MN)

The nominal strength of the wall is given by;

$$P_\circ = 0.85f_c^{'}(A_g - A_{st}) + f_y A_{st} \qquad \text{(EQ. 10-2 ACI 318-95)}$$

where; A_g = 23.5'(12"/1')9" = 2,538-in^2 (1.64X10^6 mm^2)

A_{st} = $l_w t\rho_v$ = 23.5'(12"/1')9"(0.0037) = 9.39-in^2 (6.06X10^3 mm^2)

f_c' = 4,000psi (46.9MPa)

$$P_\circ = 0.85(4{,}000\text{psi})(2{,}538 - \text{in}^2 - 9.39 - \text{in}^2) + 60{,}000\text{psi}(9.39 - \text{in}^2) = 9{,}161^k \text{ (40.75MN)}$$

$$\therefore P_u = 280^k << 3{,}206^k = 0.35P_\circ \text{ (1.25MN << 14.26MN)} \qquad \text{Therefore, walls are structural}$$

Determine if boundary elements are required;

This requirement will be checked at the first story level only for this is the worst case condition, and if boundary elements are not required at that level they will not be required at the levels above.

Per FEMA 302 Section 9.1.1.13, boundary elements are not required if;

(1) $P_u \le 0.10A_g f_c^{'}$

and either

(2) $\dfrac{M_u}{V_u l_w} \le 1.0$

or (3) $V_u \le 3A_{cv}\sqrt{f_c^{'}}$, and $\dfrac{M_u}{V_u l_w} \le 3.0$

Check condition (1);

By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = 280^k < $1{,}015^k$ = 0.1(23.5')(12"/1')9"(4,000psi)(1^k/1000-lb) = $0.1A_g f_c$' (1.25MN < 4.51MN)

O.K.

Check condition (2);

By inspection, M_u and V_u are the same for all load cases. Therefore;

$$\frac{M_u}{V_u l_w} = \frac{2{,}600^{ft-kips}}{104^k(23.5')} = 1.06 > 1.0 \qquad \textbf{N.G.}$$

Therefore, check condition 3;

$$V_u = 104^k < 3(9")282"\sqrt{4{,}000\text{psi}} = 482^k \text{ (0.46MN < 2.14MN)} \qquad \textbf{O.K.}$$

$$\frac{M_u}{V_u l_w} = 1.06 < 3.0 \qquad \textbf{O.K.}$$

Therefore, no boundary elements are required

Determine if wall has adequate capacity for flexural and axial loads combined;

Determine design loads;

Load case 4a; P_u = [1.386(1.71+2(2.36))klf + 0.5(2(0.472))klf]23.5' = 221^k (0.98MN)

M_u = 1.0(2,600-ft-kips) = 2,600-ft-kips (3.53MN-m)

Load case 5b; P_u = [0.714(1.71+2(2.36))klf]23.5' = 108^k (480.4KN-m)

M_u = 1.0(2,600-ft-kips) = 2,600-ft-kips (3.53MN-m)

Figure 10 shows the design interaction diagram (obtained from PCACOL) for the shear wall section. As in Figure 9 only two design load combinations are listed. The point marked "1" represents the P_u-M_u combination corresponding to load case 4a, and the point marked "2" represents the P_u-M_u combination corresponding to load case 5a. This figure shows that the walls have sufficient capacity for axial and overturning forces.

Therefore, use 2 curtains of #4 (~10M) bars at 12" (308.8mm) o.c. each way

(2) Longitudinal moment frame design: use f_c' = 4,000psi (46.90MPa), f_y = 60ksi (413.7MPa)

Per paragraph 7-4.f, special moment frames (SMF's) are frames conforming to the requirements of Sections 21.1 through 21.5 of ACI 318-95 in addition to the ACI 318-95 requirements for ordinary moment frames (OMF's).

Determine design loads;
The RISA-2D model, used previously to determine the relative rigidity of the frames, was reanalyzed for lateral loading. The resultant forces acting on the frame at each floor level, determined in section 3d of this problem solution, were distributed over the length of the structure at each floor level. Note that there are no live loads acting on the frame, and the dead loads are due only to the frames self weight, weight of windows, and non-structural infill walls. Cracked section properties in accordance with ACI 318-95 Section 10.11.1 were used.

Note: The RISA-2D program does not distribute loads over rigid end offsets. Therefore, distributed loads acting within the rigid end offsets were added as point loads on the accompanying node.

Dead loads;

At the roof ; w_{RD} = distributed load due to weight of beam and parapet
w_{RD} = {16"(18")+27"(12")}150pcf(1^k/1000-lb)/144-in^2/ft^2 = 0.638klf (9.30KN/m)
P_{RD} = point load due to tributary weight of column and parapet over rigid end offsets
P_{RD} = {18"(24")}150pcf(3.46')(1^k/1000-lb)/144-in^2/ft^2+0.638klf(2') =2.83^k (12.6KN)

Note: The tributary 3.46-ft (1.06m) column height was calculated in the seismic weights section of this problem.

At the floor; w_{FD} = distributed load due to weight of beam + infill + windows
w_{FD} = {18"(24")}150pcf(1^k/1000-lb)/144in^2/ft^2+…
…+{3'(40psf)+6'(8psf)}(1^k/1000-lb) = 0.618klf (9.01KN/m)

Note: The tributary 6-ft glass height was calculated in the seismic weights section of this problem.

P_{FD} = point load due to tributary weight of column and over rigid end offsets
P_{FD} = {18"(24")}150pcf(11')(1^k/1000-lb)/144in^2/ft^2 = 4.95^k (22.02KN)

The earthquake story forces were uniformly distributed over a length equal to the length of the frame minus the length of the rigid end offsets.

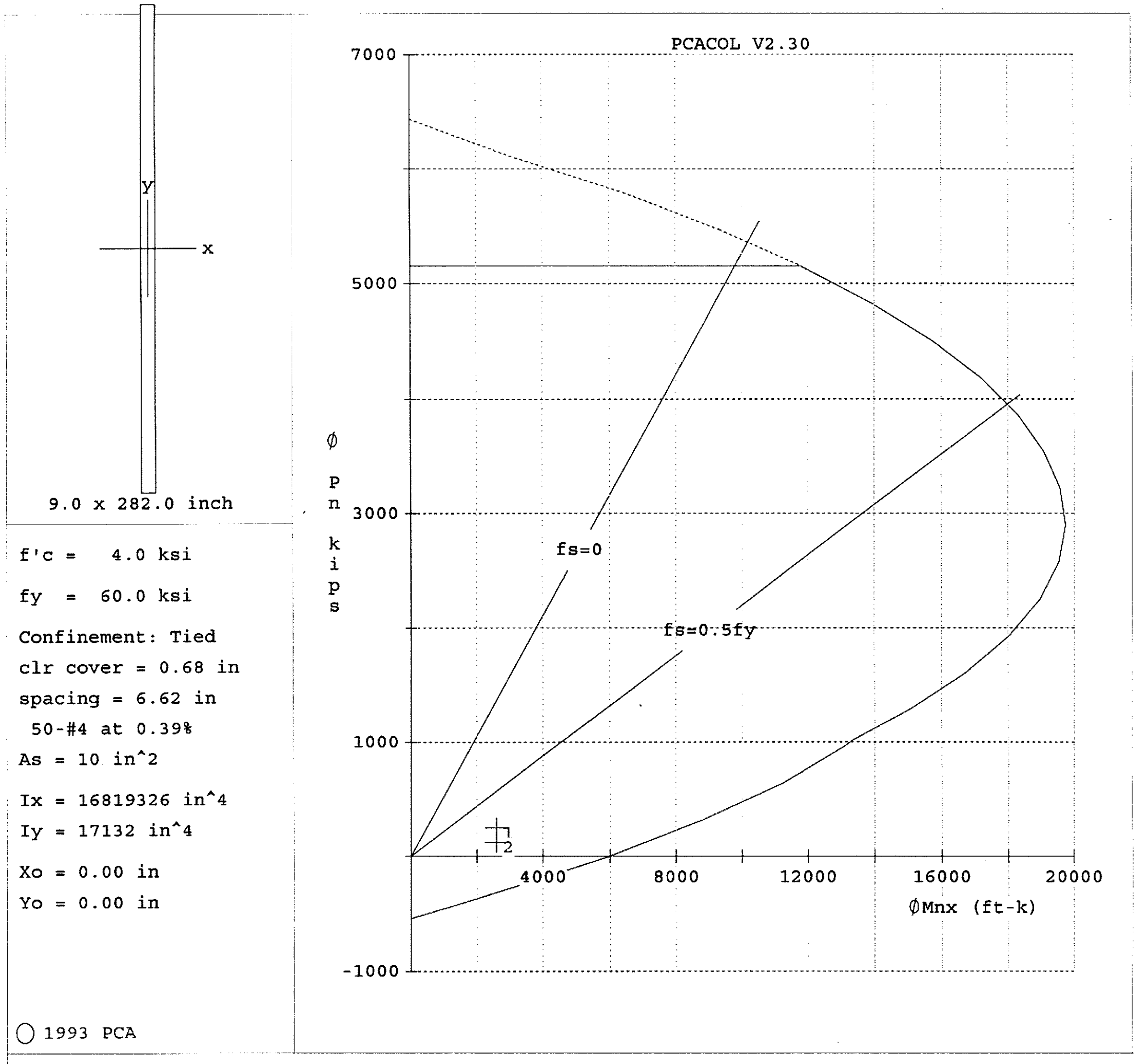
At the roof; $(Q_E)_R$ = 121.5^k/(141'-12(1')) = 0.941klf (13.72KN/m)
At the 3rd floor; $(Q_E)_3$ = 100^k/(141'-12(1')) = 0.775klf (11.30KN/m)
At the 2nd floor; $(Q_E)_2$ = 50.8^k/(141'-12(1')) = 0.394klf (5.75KN/m)

Design beams;

Check that axial loading may be ignored for beam design per Section 21.3.1.1 of ACI 318-95;

From the analysis output, the largest compressive axial load in a floor beam is 11.6^k (51.6KN) (due to load combination 5a), and the largest compressive axial load in a roof beam is 16.8^k (74.73KN) (due to load combination 4a).

At the roof; $\frac{A_g f_c'}{10} = \frac{16''(18'')4\text{ksi}}{10} = 115^k > 16.8^k$ (511.5KN > 74.73KN) **O.K.**



Licensed To: Licensee name not yet specified.

File name: H:\USERS\SBURNS\H2\WALL2-8.COL

Project: TI 809-04	Material Properties:
Column Id:	Ec = 3834 ksi eu = 0.003 in/in
Engineer:	fc = 3.40 ksi Es = 29000 ksi
Date: Time:	Beta1 = 0.85
Code: ACI 318-89	Stress Profile: Block
Units: in-lb	phi(c) = 0.70, phi(b) = 0.90

X-axis slenderness is not considered.

Note: For metric equivalents; 1-in = 25.4mm, 1-ft-kip = 1.356KN-m, 1-ksi = 6.895MPa

Figure 10. Design strength interaction diagram for shear wall section on grid lines 2 through 8

At the floors, $\frac{A_g f_c'}{10} = \frac{18"(24")4\text{ksi}}{10} = 173^k > 11.6^k$ (0.77MN > 51.6KN) **O.K.**

Therefore, axial loading may be ignored in beam design

The moment and shear diagrams resulting from the RISA-2D analysis for the beams are shown in Figures 11 and 12 for lateral loads and gravity loads respectively. Inspection of these diagrams shows that there is relatively little variation in forces amongst beams at the second and third floor levels, and relatively little variation amongst roof beams. Therefore, it is decided to produce two beam designs, one for the second and third floor levels and one for the roof beams.

Design second and third floor moment frame beams;

Design for flexure;
Negative moment at face of column;
By inspection, the governing load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$M_u^- = 1.386(-24.43^{ft-k}) + (-157.40^{ft-k}) = -191^{ft-k} \quad (259.0\text{KN-m})$$

Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{191^{ft-k}(12"/1')}{0.9(60\text{ksi})0.9(21.5")} = 2.19 - \text{in}^2 \quad (1.41X10^3\ \text{mm}^2)$$

Try 5 #6 (~20M) bars; $A_s = 5(0.44\text{-in}^2) = 2.20\text{-in}^2$ ($1.42X10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

$$d = 24" - 2" - 0.375" - (0.75"/2) = 21.25" \quad (539.8\text{mm})$$

Check spacing of bars in one layer per ACI 318-95 Section 7.6.1;

$$2(\text{clear cover} + d_{stirrup}) + \Sigma d_{bar} + (\text{No. spaces}) \times 1"$$
$$= 2(2" + 0.375") + 5(0.75") + 4(1") = 12.5" < 18" = \text{beam width} \quad (317.5\text{mm} < 457.2\text{mm})$$ **O.K.**

Check capacity;

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{2.20 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18"} = 2.16" \quad (54.9\text{mm})$$

$$\phi M_n = 0.9(2.20 - \text{in}^2)60\text{ksi}\left(21.25" - \frac{2.16"}{2}\right)(1'/12") = 200^{ft-k} \quad (271.2\text{KN-m})$$

$$\phi M_n = 200^{ft-k} > 191^{ft-k} = M_u \quad (271.2\text{KN-m} > 259.0\text{KN-m})$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s \sqrt[3]{d_c A} \le 145^{k/in} \quad (25.39\text{KN/mm})$$ (EQ. 10-5 ACI 318-95)

where; $f_s = 0.60(60\text{ksi}) = 36\text{ksi}$ (248.2MPa)
$d_c = 24" - 21.25" = 2.75"$ (69.9mm)
$A = 18"(2)(24"-21.25")/5 = 19.8\text{-in}^2$ ($12.8X10^3$ mm^2)

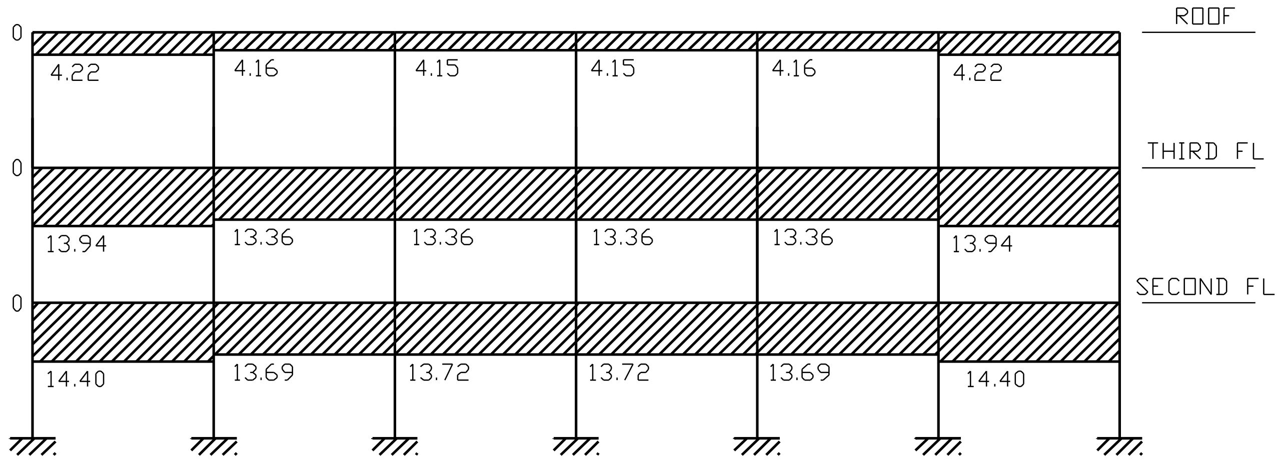
$$\therefore z = 36\text{ksi}\sqrt[3]{2.75"(19.8 - \text{in}^2)} = 136^{k/in} < 145^{k/in} \quad (23.82\text{KN/mm} < 25.39\text{KN/mm})$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

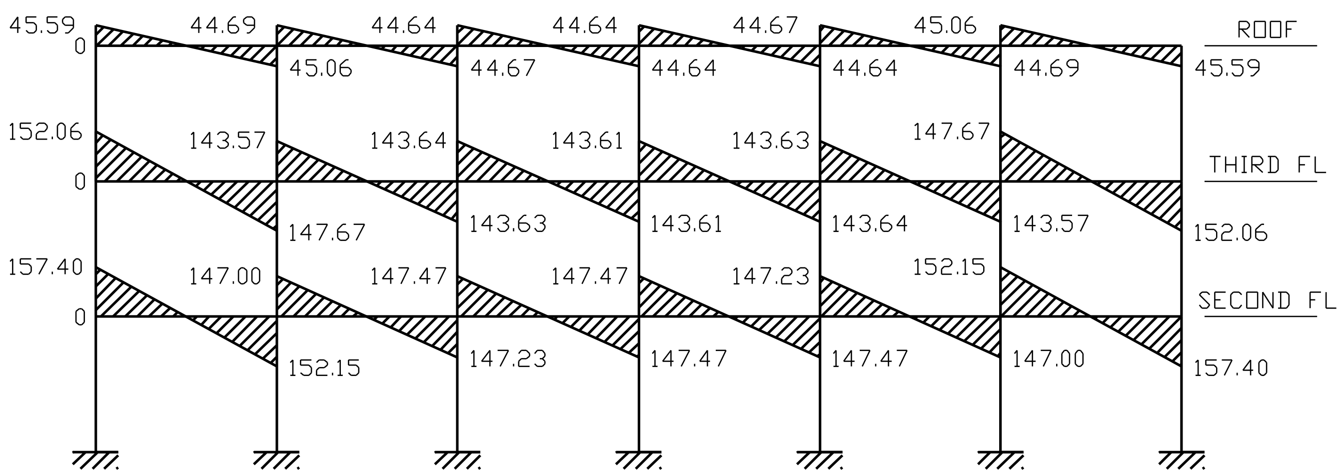
$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \ge \frac{200 b_w d}{f_y}$$ (EQ. 10-3 ACI 318-95)

$$\text{or } \rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}000\text{psi}} = 0.00333 \quad \text{(governs)}$$

SHEAR DIAGRAM (KIPS):

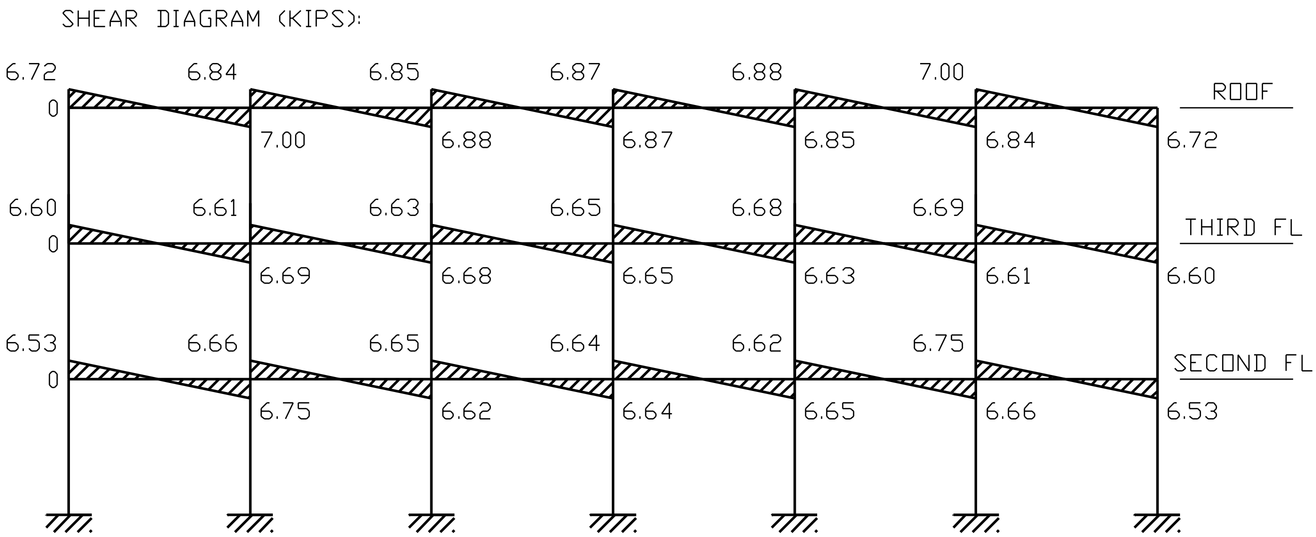


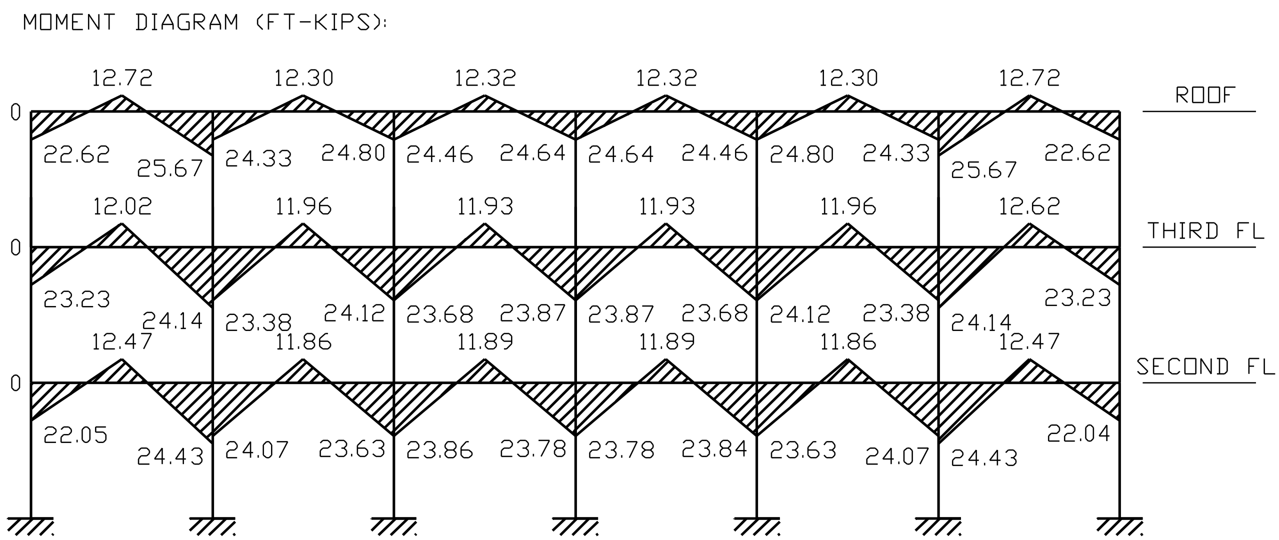
MOMENT DIAGRAM (FT-KIPS):



Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 11. Shear and moment diagrams for frame beams due to lateral loading





Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 12. Shear and moment diagrams for frame beams due to gravity loads

$$\rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000psi}}{60{,}000psi} = 0.00316$$

$$\rho = \frac{2.20 - in^2}{18"(21.25")} = 0.00575 > 0.00316 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10f_c'A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

$$0.75\rho_b = 0.75\left[\frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right)\right] = 0.75\left[\frac{0.85(0.85)4ksi}{60ksi}\left(\frac{87{,}000}{87{,}000 + 60{,}000psi}\right)\right] = 0.0214$$

and $0.75\rho_b = 0.0214 < 0.025$

$\therefore \rho = 0.00575 < 0.0214 = \rho_{max}$ **O.K.**

Therefore, choose 5 #6 top bars at column face

Positive moment at face of column;
By inspection, the governing load combination is 5a ($U = 0.714D + Q_E = 0.5L$);

$$M_u^+ = 0.714(-24.43^{ft-k}) + 157.40^{ft-k} = +140^{ft-k} \quad (189.8KN\text{-}m)$$

Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{140^{ft-k}(12"/1')}{0.9(60ksi)0.9(21.5")} = 1.61 - in^2 \quad (1.04x10^3\ mm^2)$$

Try 5 #5 (15M) bars; $A_s = 5(0.31\text{-}in^2) = 1.55\text{-}in^2$ ($1.00x10^3\ mm^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 1.5" – 0.375" – (0.625"/2) = 21.81" (554.0mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.55 - in^2(60ksi)}{0.85(4ksi)18"} = 1.52" \quad (38.6mm)$$

$$\phi M_n = 0.9(1.55 - in^2)60ksi\left(21.81" - \frac{1.52"}{2}\right)(1'/12") = 147^{ft-k} \quad (199.3KN\text{-}m)$$

$$\phi M_n = 147^{ft-k} > 140^{ft-k} = M_u \quad (199.3KN\text{-}m > 189.8KN\text{-}m)$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \le 145^{k/in} \quad (25.39KN/mm)$$ (EQ. 10-5 ACI 318-95)

where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
d_c = 24" – 21.81" = 2.19" (55.6mm)
A = 18"(2)(24"-21.81")/5 = 15.8 -in² ($10.19x10^3\ mm^2$)

$$\therefore z = 36ksi\sqrt[3]{2.19"(15.8 - in^2)} = 117^{k/in} < 145^{k/in} \quad (20.49KN/mm < 25.39KN/mm)$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \ge \frac{200 b_w d}{f_y}$$ (EQ. 10-3 ACI 318-95)

$$\text{or } \rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}000psi} = 0.00333$$ (governs)

$$\rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000psi}}{60{,}000psi} = 0.00316$$

$$\rho = \frac{1.55 - in^2}{18''(21.81'')} = 0.00395 > 0.00333 = \rho_{min}$$ **O.K.**

Check moment strength at face of joint per ACI 318-95 Section 21.3.2.2;

$$\frac{1}{2}(\phi M_n^-)_{@joint} = \frac{1}{2}(200^{ft-k}) = 100^{ft-k} < 147^{ft-k} = (\phi M_n^+) \quad (135.6KN\text{-}m < 199.3KN\text{-}m)$$ **O.K.**

Therefore, choose 5 #5 (15M) bottom bars at column face

Positive moment at midspan;
By inspection, the governing load combination is 1(U = 1.4D);

$$M_u^+ = 1.4(12.5^{ft-k}) = 17.5^{ft-k} \quad (23.7KN\text{-}m)$$

Due to the load demand at this section, steel will be governed by detailing requirements. By inspection, the minimum reinforcement requirements of ACI 318-95 Section 10.5, and 21.3.2.1 will govern. Since the steel at the column face is very close to the minimum, it is not possible to terminate any bars.

Check capacity per ACI 318-95 Section 21.3.2.2;

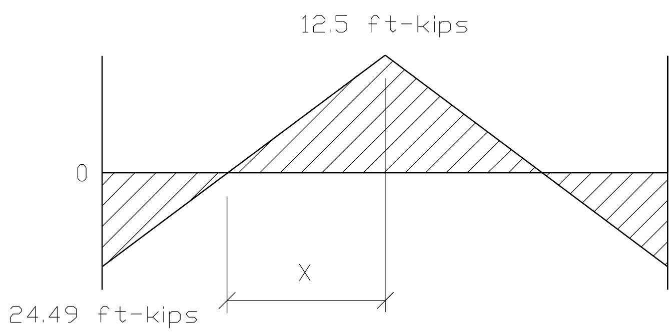
$$\frac{1}{4}(\phi M_n^-)_{@joint} = \frac{1}{4}(200^{ft-k}) = 50^{ft-k} < 147^{ft-k} = (\phi M_n^+) \quad (67.8KN\text{-}m < 199.3KN\text{-}m)$$ **O.K.**

Check development of positive moment reinforcement at inflection points per ACI 318-95 Section 12.11;

$$l_d \le \frac{M_n}{V_u} + l_a$$ (EQ. 12-2 ACI 318-95)

where; $M_n = M_u = 147^{ft\text{-}k}/0.9 = 163^{ft\text{-}k}$ (221.0KN-m)
V_u; V_u is evaluated at the inflection point as follows;

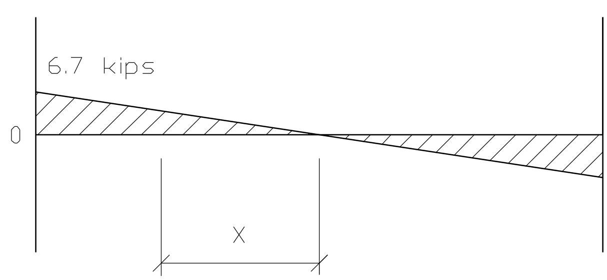
Inflection point is located from the beam centerline using similar triangles;



1-ft-kip = 1.356KN-m

$$x = \frac{21.5'/2}{(24.5^{ft-k} + 12.5^{ft-k})}(12.5^{ft-k}) = 3.63' \quad (1.11m)$$

Similarly, V_u is determined at the inflection point;



1-kip = 4.448KN

$$V_u = \frac{6.7^k}{(21.5'/2)}(3.63') = 2.26^k \quad (10.05KN)$$

Also,

l_a = minimum (d, $12d_b$)
d = 21.31" (541.3mm)
$12d_b$ = 12(0.625") = 7.5" (190.5mm) (governs)

$$\therefore l_d \le \frac{163^{ft-k}}{2.26^k}(12"/1') + 7.5" = 873" \quad (22.17X10^3 \text{ mm})$$

$$l_d = \frac{f_y \alpha \beta \gamma}{25\sqrt{f_c'}} d_b$$ where; α = 1.3 (> than 12" (304.8mm) conc. per ACI 318-95 Section 12.2.2)

β = 1.0 (uncoated reinforcement)
γ = 0.8 (#5 (15M) bars)

$$\therefore l_d = \frac{60{,}000psi(1.3)1.0(0.8)}{25\sqrt{4{,}000psi}}(0.625") = 24.7" \quad (637.4mm)$$

$$l_d = 24.7" < 873" = \frac{M_u}{V_u} + l_a$$ **O.K.**

Negative moment at mid span;
There is no negative moment occuring at mid span. Therefore, sreel will be governed by detailing requirements. Per ACI 318-95 Section 21.3.2.1, at least two bars are required.

Try 3 #6 (~20M) bars (assuming two of the top steel bars at the column face are to be cut-off); A_s = 3(0.44-in^2) = 1.32-in^2 (0.85X10^3 mm^2), and d = 21.25" (539.8mm)

Check capacity;

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.32 - in^2(60ksi)}{0.85(4ksi)18"} = 1.29" \quad (32.8mm)$$

$$\phi M_n = 0.9(1.32 - in^2)60ksi\left(21.25" - \frac{1.29"}{2}\right)(1'/12") = 122^{ft-k} \quad (165.4KN\text{-}m)$$

$$\frac{1}{4}(\phi M_n^-)_{@joint} = \frac{1}{4}200^{ft-k} = 100^{ft-k} < 122^{ft-k} = \phi M_n \quad (135.6KN\text{-}m < 165.4KN\text{-}m)$$ **O.K.**

Therefore, choose 3 #6 (~20M) bars top steel at mid span

Determine cut-off point for two #6 top steel bars in accordance with ACI 318-95 section 12.10;

Determine the critical section;
By inspection, the governing load combination is 4a (U = 1.386D + Q_E + 0.5L). This combination governs because it requires the longest span from column face for the steel. At column face the moment capacity of the beam was calculated as ϕM_n = 191$^{ft\text{-}kips}$ (259.0KN-m), and at mid span the required strength is M_u = 1.386(12.5$^{ft\text{-}kips}$) = 17.3$^{ft\text{-}kips}$ (23.5KN-m).

The slope of the moment diagram is;

$$\frac{dM_u}{dx} = \frac{-191^{ft-k} - (1.386(12.5^{ft-kips}))}{21.5'/2} = -19.4^k \quad (86.29KN)$$

The capacity of the 2 remaining #6 bars was previously calculated at ϕM_n = 122$^{ft\text{-}kips}$ (165.4KN-m). Therefore, the distance from the face of column to where $M_u = \phi M_n$ = 122$^{ft\text{-}kips}$ is;

$$x = \frac{122^{ft-k} - 191^{ft-k}}{-19.4^{ft-k}} = 3.6', \text{ Say } 4' \quad (1.22m)$$

Determine the extension past the critical section;

$$\frac{l_d}{d_b} = \frac{f_y \alpha\beta\gamma}{25\sqrt{f_c^{'}}}$$ where; $\alpha = 1.3$ (> than 12" (304.8mm) conc. per ACI 318-95 Section 12.2.2)

$\beta = 1.0$ (uncoated reinforcement)

$\gamma = 1.0$ (#6 (~20M) bars)

$$\therefore l_d = \frac{60{,}000\text{psi}(1.3)1.0(0.8)}{25\sqrt{4{,}000\text{psi}}}(0.75") = 29.6" = 2.47' < 4' \quad (0.75\text{m} < 1.22\text{m})$$

48" + d = 48" + 21.25" = 69.3", Say 5'-9" (1.75m) (governs)

48" + $12d_b$ = 48" + 12(0.75") = 57" (1.45m)

Therefore, cut off 2 #6 (~20M) top steel bars at 5'-9" (1.75m) from face of column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \leq V_u$

By inspection, load combination 4a ($U = 1.386D + Q_E + 0.5L$) provides the largest factored shear force;

$$(V_u)_{max} = 1.386(6.75^k) + 14.4^k = 23.8^k \quad (105.9\text{KN})$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; L =21.5' (6.56m) (beam clear span)

M_{pr1} and M_{pr2} are calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f_c^{'}b}$$

W/2 = gravity load reaction = 21.5'(w_{FD}/2)

At the beam left side using top bars;

$$a = \frac{1.25(2.20 - \text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18"} = 2.70" \quad (68.6\text{mm})$$

$$\therefore M_{pr1} = 1.25(2.20 - \text{in}^2)60\text{ksi}\left(21.25" - \frac{2.70"}{2}\right)(1'/12") = 274^{\text{ft-k}} \quad (371.5\text{KN-m})$$

At the beam right side using bottom bars;

$$a = \frac{1.25(1.55 - \text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18"} = 1.90" \quad (48.3\text{mm})$$

$$\therefore M_{pr2} = 1.25(1.55 - \text{in}^2)60\text{ksi}\left(21.81" - \frac{1.90"}{2}\right)(1'/12") = 202^{\text{ft-k}} \quad (273.9\text{KN-m})$$

$$\text{Also, } \frac{W}{2} = \frac{L(1.386w_{FD})}{2} = \frac{21.5'(1.386(0.618\text{klf}))}{2} = 9.20^k \quad (40.9\text{KN})$$

$$(V_e)_{max} = \frac{274^{\text{ft-k}} + 202^{\text{ft-k}}}{21.5'} + \frac{9.20^k}{2} = 26.7^k \quad (118.8\text{KN})$$

$$(V_u)_{max} = 23.8^k < 26.7^k = (V_e)_{max} \quad (105.9\text{KN} < 118.8\text{KN})$$ **Therefore, ϕ = 0.6**

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 26.7^k > 11.9^k = \frac{1}{2}(V_u)_{max}$ (118.8KN > 52.9KN)

Condition (2); From the computer RISA-2D analysis, the largest axial load in a floor beam is 11.45^k (50.9KN).

$$\frac{A_g f_c^{'}}{20} = \frac{24"(18")4\text{ksi}}{20} = 86.4^k > 11.45^k \quad (384.3\text{KN} > 50.9\text{KN})$$ **Therefore, $V_c = 0$**

Design stirrups;

Note: There are 5 top and bottom longitudinal bars, and per ACI 318-95 Section 7.10.5.3, three stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an extra interior cross tie will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of column face; 2d = 2(21.81") = 43.6" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(3(0.11-in^2))60ksi(21.81")}{26.7^k} = 9.70" \quad (246.4mm)$$

Spacing based on detail requirements;

s = d/4 = 21.25"/4 = 5.31" (134.9mm)
s = 8(0.625") = 5" (127.0mm) (governs)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

Therefore, within 4-ft. (1.22m) of column face, provide stirrups consisting of 3 legs of #3 (~10M) bars at 5-in. (127.0mm) o. c.

For the remainder of the beam;

Spacing based on strength requirements;

At 4-ft (1.22m) from column face;

$$(V_e)_{@2h} = (V_e)_{max} - \frac{(V_e)_{max} - (V_e)_{min}}{L} 2h$$

where; $(V_e)_{max} = 26.7^k$ (calculated previously)

$$(V_e)_{min} = \frac{274^{ft-k} + 197^{ft-k}}{21.5'} - \frac{9.20^k}{2} = 17.3^k \quad (77.0KN)$$

$$(V_e)_{@2h} = 26.7^k - \left(\frac{26.7^k - 17.3^k}{21.5'}\right)4' = 25^k \quad (111.2KN)$$

$$\therefore s = \frac{0.6(3(0.11-in^2))60ksi(21.25")}{25^k} = 10.1" \quad (256.5mm) \qquad \text{(governs)}$$

Spacing based on detail requirements;

s = d/2 = 21.25"/2 = 10.6" (269.2mm)

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11-in^2)60{,}000psi}{50(18")} = 22" \quad (558.8mm)$$

For the remainder of the beam, provide stirrups consisting of 3 legs of #3 (~10M) bars at 10-in. (254.0mm) o.c. max

Check transverse reinforcement requirements at longitudinal bar cut-offs;

Determine shear at the cut-off point;

$(V_u)_{max}$ at column face = 23.8^k (105.9KN) (previously calculated)
$(V_u)_{max}$ at beam mid span = 14.4^k (64.1KN) ($1.0Q_E = 1.0(14.4^k)$)

Therefore, at 5'-9" from column face;

$$(V_u)_{@5'-9"} = 23.8^k - \left(\frac{23.8^k - 14.4^k}{21.5'}\right)5.75' = 21.3^k \quad (94.7KN)$$

Note: The cut-off point occurs outside of the length identified in ACI 318-95 Section 21.3.3.1 (i.e., 2h from column face). Therefore, ϕV_c can be included in the section capacity ϕV_n.

$$\phi V_s = \frac{\phi A_v f_y d}{s} = \frac{0.6(3(0.11-in^2))60ksi(21.25")}{10"} = 25.2^k \quad (112.1KN)$$

$$V_c = 2\sqrt{f'_c}\, b_w d \qquad \text{(EQ. 11-3 ACI 318-95)}$$

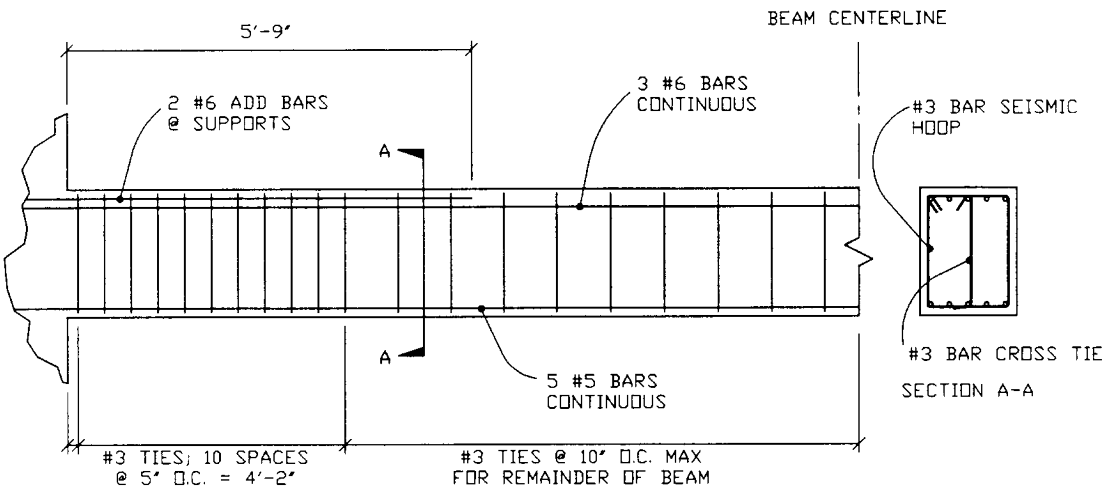
$$\phi V_c = 0.6\left(2\sqrt{4{,}000\text{psi}}\right)18''(21.25'')(1^k/1{,}000^{lb}) = 29^k \quad (129.0\text{KN})$$

$$\phi V_n = \phi(V_c + V_s) = 0.6\left(29^k + 25.2^k\right) = 32.5^k \quad (144.6\text{KN})$$

$$V_u = 18.8^k < 21.7^k = \frac{2}{3}(32.5^k) = \frac{2}{3}\phi V_n \qquad \textbf{O.K.}$$

Therefore, no additional transverse reinforcement is required at bar cut-offs

Therefore, the second and third floor beam design is as follows;



#3 bar ~ 10M bar
#5 bar = 15M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design roof level moment frame beams;

Design for flexure;
Negative moment at face of column;
By inspection, the governing load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$M_u^- = 1.386(-25.67^{\text{ft-kips}}) + (-45.59^{\text{ft-kips}}) = -81.2^{\text{ft-kips}} \quad (110.1\text{KN-m})$$

Assume j = 0.9, and d = h – 2.5" = 16" – 2.5" = 13.5" (342.9mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{81.2^{\text{ft-kips}}(12''/1')}{0.9(60\text{ksi})0.9(13.5'')} = 1.49 - \text{in}^2 \quad (0.96\text{X}10^3\ \text{mm}^2)$$

Try 5 #5 (15M) bars; $A_s = 5(0.31\text{-in}^2) = 1.55\text{-in}^2 > 1.49\text{-in}^2$ ($1.00\text{X}10^3\ \text{mm}^2 > 0.96\text{X}10^3\ \text{mm}^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 16" – 1.5" – 0.375" – (0.625"/2) = 13.81" (350.8mm)

Check capacity;

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = \frac{1.55 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18''} = 1.52'' \quad (38.6\text{mm})$$

$$\phi M_n = 0.9(1.55 - \text{in}^2)60\text{ksi}\left(13.81'' - \frac{1.52''}{2}\right)(1'/12'') = 91^{\text{ft-kips}} \quad (123.4\text{KN-m})$$

$$\phi M_n = 91^{\text{ft-kips}} > 72.9^{\text{ft-kips}} = M_u \quad (12.34\text{KN-m} > 98.8\text{KN-m}) \qquad \textbf{O.K.}$$

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$z = f_s\sqrt[3]{d_cA} \le 145^{k/in}$ (25.4KN/m) (EQ. 10-5 ACI 318-95)

where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
d_c = 16" – 13.81" = 2.19" (55.6mm)
A = 18"(2)(16"-13.81")/5 = 15.8-in^2 (10.2X10^3 mm^2)

$\therefore z = 36ksi\sqrt[3]{2.19"(15.8 - in^2)} = 117^{k/in} < 145^{k/in}$ (20.49KN/mm < 25.4KN/mm) **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (previously calculated)

$$\rho = \frac{1.55 - in^2}{16"(13.81")} = 0.00702 > 0.00333 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10f_c'A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

and $0.75\rho_b = 0.0214 < 0.025$ (previously calculated)

$$\therefore \rho = \frac{1.55 - in^2}{16"(13.81")} = 0.00701 < 0.0214 = \rho_{max}$$ **O.K.**

Therefore, choose 5 #5 (15M) top bars at column face

Positive moment at face of column;
By inspection, the governing load combination is 5a ($U = 0.714D + Q_E$);

$M_u^+ = 0.714(22.62^{ft-kips}) + 45.59^{ft-kips} = +61.7^{ft-kips}$ (83.7KN-m)

Assume j = 0.9, and d = h – 2.5" = 16" – 2.5" = 13.5" (342.9mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{61.7^{ft-kips}(12"/1')}{0.9(60ksi)0.9(13.5")} = 1.13 - in^2$$ (0.73X10^3 mm^2)

Try 5 #5 (15M) bars; A_s = 5(0.31-in^2) = 1.55-in^2 (1.00X10^3 mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 16" – 1.5" – 0.375" – (0.625"/2) = 13.8" (350.5mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85f_c'b}$$

$$a = \frac{1.55 - in^2(60ksi)}{0.85(4ksi)18"} = 1.52" \text{ (38.6mm)}$$

$$\phi M_n = 0.9(1.55 - in^2)60ksi\left(13.8" - \frac{1.52"}{2}\right)(1'/12") = 91^{ft-kips} \text{ (123.4KN-m)}$$

$\phi M_n = 91^{ft-kips} > 61.7^{ft-kips} = M_u$ (123.4KN-m > 83.7KN-m) **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$z = f_s\sqrt[3]{d_cA} \le 145^{k/in}$ (25.4KN/mm) (EQ. 10-5 ACI 318-95)

where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
d_c = 16" – 13.8" = 2.20" (55.9mm)
A = 18"(2)(16"-13.8")/5 = 15.8 –in^2 (10.19X10^3 mm^2)

$\therefore z = 36ksi\sqrt[3]{2.20"(15.8 - in^2)} = 118^{k/in} < 145^{k/in}$ (20.7KN/mm < 25.4KN/mm) **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (previously calculated)

$$\rho = \frac{1.55 - in^2}{16"(13.8")} = 0.0070 > 0.00333 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10f_c'A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

$0.75\rho_b = 0.0214$ (previously calculated) and $0.75\rho_b = 0.0214 < 0.025$

$$\therefore \rho = \frac{1.55 - in^2}{16''(13.8'')} = 0.0070 < 0.0214 = \rho_{max}$$ **O.K.**

Check moment strength at face of joint per ACI 318-95 Section 21.3.2.2;

$$\frac{1}{2}(\phi M_n^-)_{@joint} = \frac{1}{2}(91^{ft-kips}) = 45.5^{ft-kips} < 91^{ft-kips} = (\phi M_n^+) \quad (61.7KN\text{-}m < 123.4KN\text{-}m)$$

O.K.

Therefore, choose 5 #5 (15M) bottom bars at column face

Positive moment at mid span;
Because the longitudinal reinforcement is close to the minimum requirement, it is not possible to terminate any bars along the beams length. By inspection, all other detailing and strength requirements are satisfied.

Therefore, choose 5 #5 (15M) bottom bars at mid span

Negative moment at mid span;
As was demonstrated for the second and third floor beams, it is possible to cut off two of the 5 #5 (15M) top bars. However, in order to reduce the number of repetitive calculations it is decided to continue the 5 #5 (15M) bars along the member length. By inspection, all detailing and strength requirements are satisfied.

Therefore, choose 5 #5 (15M) top bars at mid span

Design for shear;
Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u$

By inspection, load combination 4a ($U = 1.386D + Q_E + 0.5L$) provides the largest factored shear force;

$$(V_u)_{max} = 1.386(7.0^k) + 4.22^k = 13.9^k \quad (61.8KN)$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; L =21.5' (beam clear span)

M_{pr1} and M_{pr2} are calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_sf_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_sf_y}{0.85f_c'b}$$

W/2 = gravity load reaction = 21.5'(w_{RD}/2) (6.56m)

Since $M_{pr1} = M_{pr2}$;

$$a = \frac{1.25(1.55 - in^2)60ksi}{0.85(4ksi)18''} = 1.90'' \quad (48.3mm)$$

$$\therefore M_{pr1} = M_{pr2} = 1.25(1.55 - in^2)60ksi\left(13.8'' - \frac{1.90''}{2}\right)(1'/12'') = 125^{ft-kips} \quad (169.5KN\text{-}m)$$

$$\text{Also, } \frac{W}{2} = \frac{L(1.386w_{RD})}{2} = \frac{21.5'(1.386(0.638klf))}{2} = 9.51^k \quad (42.3KN)$$

$$(V_e)_{max} = \frac{125^{ft-kips} + 125^{ft-kips}}{21.5'} + \frac{9.51^k}{2} = 16.4^k \quad (92.9KN)$$

$$(V_u)_{max} = 13.9^k < 16.4^k = (V_e)_{max} \quad (61.8KN < 72.9KN)$$ **Therefore, $\phi = 0.6$**

Determine if $V_c = 0$ per ACI 318-95 section 21.3.4.2;

Condition (1); $(V_e)_{max} = 16.4^k > 7.0^k = \frac{1}{2}(V_u)_{max}$ (72.9KN > 31.1KN)

Condition (2); From the computer RISA-2D analysis, the largest axial load in a roof beam is 16.8^k (74.7KN).

$$\frac{A_g f_c'}{20} = \frac{18"(16")4\text{ksi}}{20} = 57.6^k > 16.8^k \quad (256.2\text{KN} > 74.7\text{KN})$$

Therefore, $V_c = 0$

Design stirrups;

Note: As in the case of the floor beams, there are 5 top and bottom longitudinal bars and per ACI 318-95 Section 7.10.5.3, three stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an extra interior cross tie will be used.
Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of column face; 2d = 2(13.88") = 27.8" Say 2.5' (0.76m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(3(0.11-\text{in}^2))60\text{ksi}(13.8")}{16.4^k} = 10.0" \quad (3.05\text{m})$$

Spacing based on detail requirements;
s = d/4 = 13.8"/4 = 3.45" (87.6mm) (governs)
s = 8(0.50") = 4" (101.6mm)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

Therefore, within 2.5-ft. (0.76m) of column face, provide stirrups consisting of 3 legs of #3 (~10M) bars at 3.5-in. (88.9mm) o. c.

For the remainder of the beam;
Spacing based on strength requirements;
At 2.5-ft from column face;

$$(V_e)_{@2h} = (V_e)_{max} - \frac{(V_e)_{max} - (V_e)_{min}}{L} 2h$$

where; $(V_e)_{max} = 16.4^k$ (72.9KN) (calculated previously)

$$(V_e)_{min} = \frac{125^{\text{ft-kips}} + 125^{\text{ft-kips}}}{21.5'} - \frac{9.51^k}{2} = 6.9^k \quad (30.7\text{KN})$$

$$(V_e)_{@2h} = 16.4^k - \left(\frac{16.4^k - 6.9^k}{21.5'}\right) 2.5' = 15.3^k \quad (68.1\text{KN})$$

$$\therefore s = \frac{0.6(3(0.11-\text{in}^2))60\text{ksi}(13.8")}{15.3^k} = 10.7" \quad (271.8\text{mm})$$

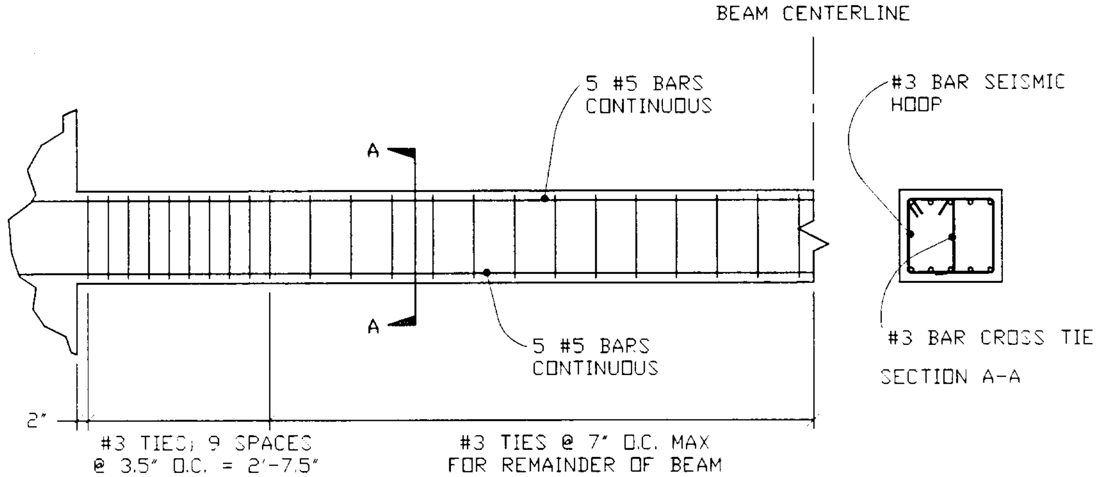
Spacing based on detail requirements;
s = d/2 = 13.8"/2 = 6.9" (175.3mm) (governs)

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11-\text{in}^2)60{,}000\text{psi}}{50(18")} = 22" \quad (558.8\text{mm})$$

For the remainder of the beam, provide stirrups consisting of 3 legs of #3 (~10M) bars at 7-in. (177.8mm) o.c. max

Therefore, the roof beam design is as follows;



#3 bar ~ 10M bar
#5 bar = 15M bar
1-ft = 0.30m
1-in = 25.4mm

Design splices for beams;

Longitudinal reinforcement for beams shall be spliced using welded splices or mechanical connectors in conformance with ACI 318-95 Section 21.2.6.1. Only alternate bars in each layer of longitudinal reinforcement shall be spliced at a section. The center to center distance between splices of adjacent bars shall be 24-in. or more.

Design columns;

General;
The columns support only their own self weight, the weight of windows, and some minor infill. Therefore, the columns will first be checked to see if they are true columns or instead may be proportioned only for flexure.

Determine if ACI 318-95 Section 21.4 applies;
This section applies only if the column is proportioned to resist an axial compressive force exceeding $A_g f_c'/10$. All columns have the same cross sectional area therefore it is only necessary to check the most heavily loaded column. From the RISA-2D analysis the most heavily loaded column is and end column located at the first story ($E = 32.55^k$, and $D = 32.58^k$). By inspection, the controling load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$P_u = 1.386(32.58^k) + 32.55^k = 77.7^k \quad (345.6KN)$$

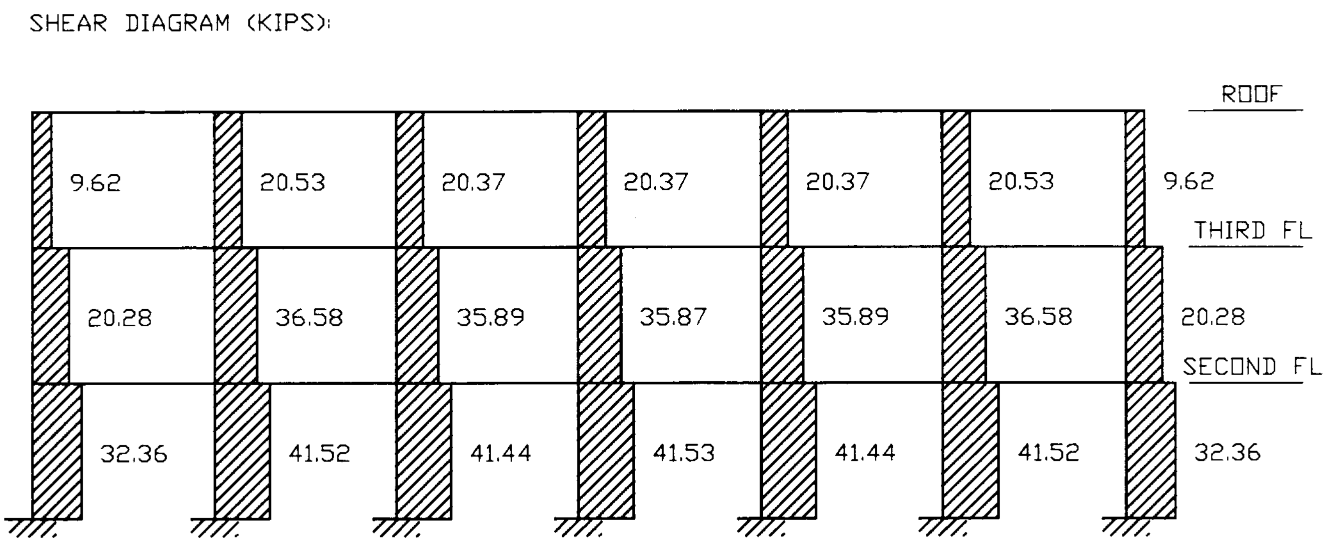
$$\frac{A_g f_c'}{10} = \frac{18"(24")4ksi}{10} = 173^k \quad (769.5KN)$$

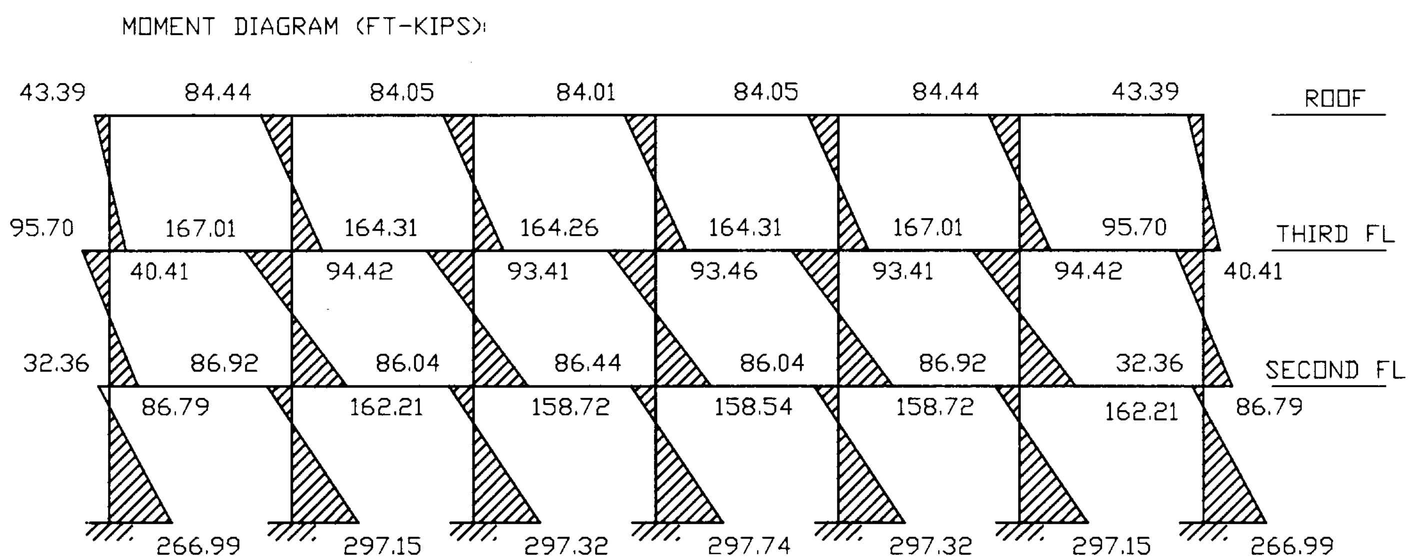
$$P_u = 77.7^k < 173^k = \frac{A_g f_c'}{10} \quad (345.6KN < 769.5KN)$$

Therefore, columns shall be proportioned primarily for flexure

Note: It is good practice to apply the strong column/weak beam criteria of ACI 318-95 Section 21.4.2.2 even though it is not required. Therefore, in this solution, ACI 318-95 Section 21.4.2.2 shall be condsidered.

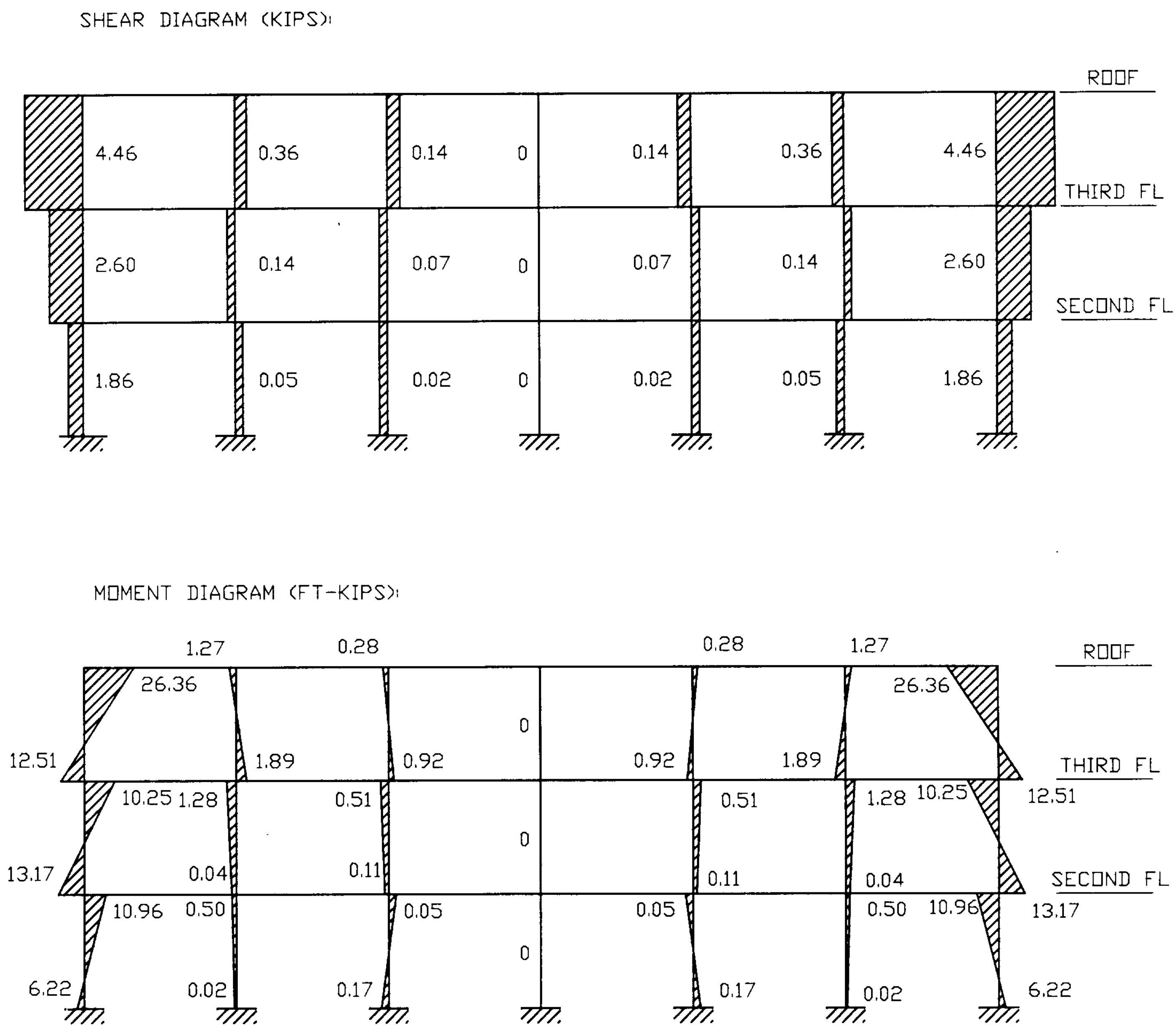
The moment and shear diagrams for the columns are shown in Figures 13 and 14 for lateral loads and gravity loads respectively. Inspection of these diagrams shows that there is relatively little variation in forces amongst columns in a story level with the exception of the end columns at the second and third story





Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 13. Shear and moment diagrams for frame columns due to lateral loading



Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 14. Shear and moment diagrams for frame columns due to gravity loading

level. Therefore, one design will be produced for the first story columns, one for the interior second story columns, one for the interior third story columns, one design for the second and third story end columns.

Design columns for first story level;
Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$);

$$M_u = 1.386(6.22^{ft-kips}) + 298^{ft-kips} = 307^{ft-kips} \quad (416.3KN\text{-}m)$$

$$V_u = 1.386(1.86^k) + 41.53^k = 44.1^k \quad (196.2KN)$$

Determine if ACI 318-95 section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For first story columns at the second floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{ft-kips} + 200^{ft-kips}) = 416.4^{ft-kips} \quad (564.6KN\text{-}m)$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{ft-kips} \quad (282.3KN\text{-}m)$$

However, $M_u = 307^{ft-kips} > 208^{ft-kips}$ (416.3KN-m > 282.3KN-m). Therefore, strong column/weak beam criteria does not govern design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{307^{ft-kips}(12''/1')}{0.9(60ksi)0.9(21.5'')} = 3.53 - in^2 \quad (2.28X10^3\ mm^2)$$

Try 6 #7 (20M) bars (on both faces of the column); $A_s = 6(0.60\text{-}n^2) = 3.60\text{-}in^2$ ($2.32X10^3\ mm^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 2" – 0.375" – (0.875"/2) = 21.19" (538.2mm)

By inspection, the 6 #7 (~20M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{3.60 - in^2(60ksi)}{0.85(4ksi)18''} = 3.53'' \quad (89.7mm)$$

$$\phi M_n = 0.9(3.60 - in^2)60ksi\left(21.19'' - \frac{3.53''}{2}\right)(1'/12'') = 315^{ft-k} \quad (427.1KN\text{-}m)$$

$$\phi M_n = 315^{ft-kips} > 307^{ft-kips} = M_u \quad (427.1KN\text{-}m > 416.3KN\text{-}m) \qquad \textbf{O.K.}$$

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (calculated previously in the beam design)

$$\rho = \frac{3.60 - in^2}{18''(21.19'')} = 0.00944 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00944 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \leq 145^{k/in} \quad (25.4KN/m) \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; fs = 0.6(60ksi) = 36ksi (248.2MPa)
dc = 24" – 21.19" = 2.81" (71.4mm)
A = 18"(2)(24" – 21.19")/6 = 16.86-in² ($10.87X10^3\ mm^2$)

$$\therefore z = 36ksi\sqrt[3]{2.81''(16.86 - in^2)} = 130^{k/in} < 145^{k/in} \quad (22.8KN/mm < 25.4KN/mm) \qquad \textbf{O.K.}$$

Therefore, provide 6 #7 (~20M) longitudinal bars on opposite faces of the column

Design for shear;
Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u = 44.1^k$ (196.2KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L}$$

where; L =9.25' (2.82m) (column clear span)
$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right),\ \text{where } a = \frac{1.25A_s f_y}{0.85f'_c b}$$

Therefore;

$$a = \frac{1.25(3.60 - in^2)60ksi}{0.85(4ksi)18"} = 4.41"\quad (112.0mm)$$

$$\therefore M_{pr1} = 1.25(3.60 - in^2)60ksi\left(21.19" - \frac{4.41"}{2}\right)(1'/12") = 427^{ft\text{-}k}\quad (579.0KN\text{-}m)$$

$$\therefore (V_e)_{max} = \frac{2(427^{ft\text{-}k})}{9.25'} = 92.3^k\quad (410.6KN)$$

$$(V_u)_{max} = 44.1^k < 92.3^k = (V_e)_{max}\quad (196.2KN < 410.6KN)$$

Therefore, ϕ = 0.6

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 92.3^k > 22.05^k = \frac{1}{2}(V_u)_{max}$ (410.6KN > 98.1KN)

Condition (2); As previously mentioned, the largest axial load in a first story column is 77.7^k.

$$P_u = 1.386(32.58^k) + 32.55^k = 77.7^k\quad (345.6KN)$$

$$\frac{A_g f'_c}{20} = \frac{24"(18")4ksi}{20} = 86.4^k > 77.7^k\quad (384.3KN > 345.6KN$$

Therefore, V_c = 0

Design stirrups;
Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with two extra interior cross ties will be used.
Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s}\qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(21.19") = 42.4" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - in^2))60ksi(21.19")}{92.3^k} = 3.64"\ \text{Say } 3.5"\ (88.9mm)\qquad \text{(governs)}$$

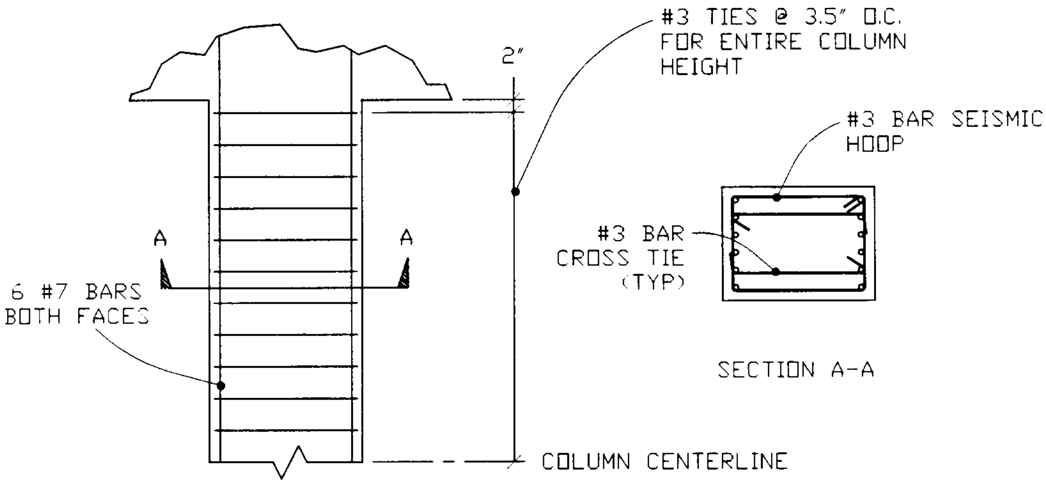
Spacing based on detail requirements;

s = d/4 = 21.19"/4 = 5.30" (134.6mm)
s = 8(0.875") = 7.0" (177.8mm)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

For the remainder of the column;
Since Ve is constant over the length of the column s = 3.5-in. (88.9mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 3.5-in. (88.9mm) o.c. over the entire column length

Therefore, the first floor column design is as follows;
Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#7 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design interior columns for the second story level;

Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$);

$$M_u = 1.386(1.28^{\text{ft-kips}}) + 167.01^{\text{ft-kips}} = 169^{\text{ft-kips}} \quad (229.2\text{KN-m})$$

$$V_u = 1.386(0.14^k) + 36.58^k = 36.8^k \quad (163.7\text{KN})$$

Determine if ACI 318-95 Section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{\text{ft-k}} + 200^{\text{ft-k}}) = 416.4^{\text{ft-k}} \quad (564.6\text{KN-m})$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{\text{ft-k}} \quad (282.3\text{KN-m})$$

Since $M_u = 169^{\text{ft-kips}} < 208^{\text{ft-kips}}$ (229.2KN-m < 282.3KN-m) strong column/weak beam criteria governs design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{208^{\text{ft-kips}}(12''/1')}{0.9(60\text{ksi})0.9(21.5'')} = 2.39 - \text{in}^2 \quad (1.54\text{X}10^3\ \text{mm}^2)$$

Try 6 #6 (~20M) bars (on both faces of the column); $A_s = 6(0.44\text{-n}^2) = 2.64\text{-in}^2$ ($1.70\text{X}10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);
d = 24" – 2" – 0.375" – (0.750"/2) = 21.25" (539.8mm)
By inspection, the 6 #6 (~20M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{2.64 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18''} = 2.59'' \quad (65.8\text{mm})$$

$$\phi M_n = 0.9(2.64-in^2)60ksi\left(21.25"-\frac{2.59"}{2}\right)(1'/12") = 237^{ft-kips} \quad (321.4KN\text{-}m)$$

$$\phi M_n = 237^{ft-kips} > 208^{ft-kips} = M_u \quad (321.4KN\text{-}m > 282.3KN\text{-}m)$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (calculated previously in the beam design)

$$\rho = \frac{2.64-in^2}{18"(21.25")} = 0.00690 > 0.00333 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00690 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \le 145^{k/in} \quad (25.4KN/mm)$$ (EQ. 10-5 ACI 318-95)

where; $fs = 0.6(60ksi) = 36ksi$ (248.2MPa)

$dc = 24" - 21.25" = 2.75"$ (69.9mm)

$A = 18"(2)(24" - 21.25")/6 = 16.50\text{-}in^2$ ($10.64X10^3$ mm^2)

$$\therefore z = 36ksi\sqrt[3]{2.75"(16.50-in^2)} = 128^{k/in} < 145^{k/in} \quad (22.4KN/mm < 25.4KN/mm)$$ **O.K.**

Therefore, provide 6 #6 (~20M) longitudinal bars on opposite faces of the column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u = 36.8^k$ (163.7KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1}+M_{pr2}}{L}$$

where; L =9.00' (2.75m) (column clear span)

$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d-\frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f_c'b}$$

Therefore;

$$a = \frac{1.25(2.64-in^2)60ksi}{0.85(4ksi)18"} = 3.24" \quad (82.3mm)$$

$$\therefore M_{pr1} = 1.25(2.64-in^2)60ksi\left(21.25"-\frac{3.24"}{2}\right)(1'/12") = 324^{ft-kips} \quad (439.3KN\text{-}m)$$

$$\therefore (V_e)_{max} = \frac{2(324^{ft-kips})}{9.0'} = 72^k \quad (320.3KN)$$

$$(V_u)_{max} = 36.8^k < 72^k = (V_e)_{max} \quad (163.7 < 320.3KN)$$ **Therefore, $\phi = 0.6$**

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 72^k > 18.4^k = \frac{1}{2}(V_u)_{max}$ (320.3KN > 81.8KN)

Condition (2); From the RISA-2D analysis, the largest axial load in an interior second story column is;

$$P_u = 1.386(31.91^k) + 0.64^k = 49^k \quad (218.0KN)$$

$$\frac{A_g f_c'}{20} = \frac{24"(18")4ksi}{20} = 86.4^k > 49^k \quad (384.3KN > 218.0KN)$$ **Therefore, $V_c = 0$**

Design stirrups;

Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an two extra interior cross ties will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(21.25") = 42.5" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - in^2))60ksi(21.25")}{72^k} = 4.68" \text{ Say } 4.5" \text{ (114.3mm)} \qquad \text{(governs)}$$

Spacing based on detail requirements;

s = d/4 = 21.25"/4 = 5.31" (138.9mm)
s = 8(0.750") = 6.0" (152.4mm)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

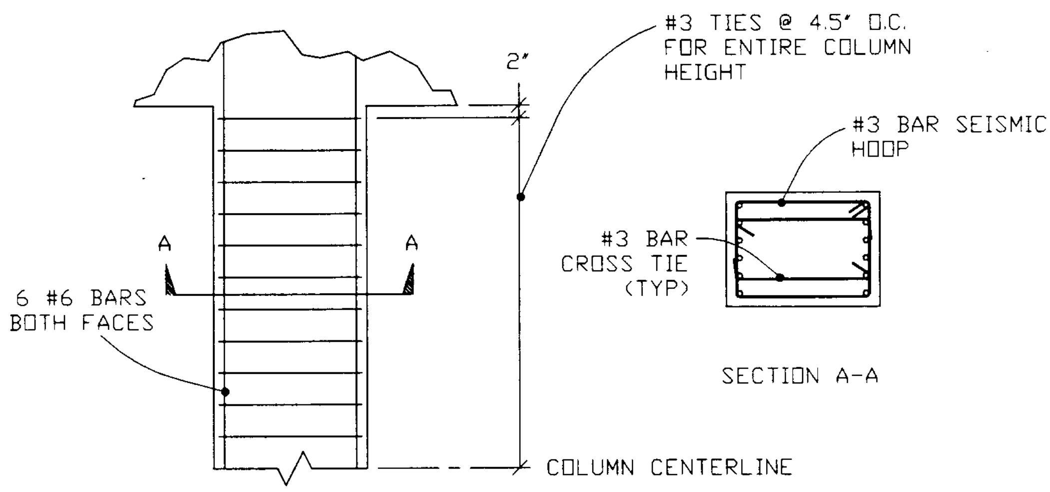
For the remainder of the column;

Since Ve is constant over the length of the column s = 4.5-in. (114.3mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 4.5-in. o.c. over the entire column length

Therefore, the second story interior column design is as follows;

Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design interior columns for the third story level;

Determine design loads;

By inspection, the governing load combination is 4a for all loads (U = 1.386D + Q_E + 0.5L);

$$M_u = 1.386(1.89^{ft-kips}) + 94.42^{ft-kips} = 97^{ft-kips} \text{ (131.5KN-m)}$$

$$V_u = 1.386(0.36^k) + 20.53^k = 21^k \text{ (93.4KN)}$$

Determine if ACI 318-95 Section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{ft-kips} + 200^{ft-kips}) = 416.4^{ft-kips} \quad (564.6KN\text{-}m)$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{ft-k} \quad (282.3KN\text{-}m)$$

Since M_u = 97 $^{ft-kips}$ < $208^{ft-kips}$ (131.5KN-m < 282.3KN-m) strong column/weak beam criteria governs design. Also, because the design moment is the same as used at the second story level, the same design will result.

Therefore, use the same design for the third story interior columns as used at the second story interior columns

Design end columns for the second and third story levels;
Determine design loads;
By inspection, the governing load combination is 4a for all loads (U = 1.386D + Q_E + 0.5L), and the worst case column occurs at the second floor level;

$$M_u = 1.386(10.25^{ft-kips}) + 95.7^{ft-kips} = 110^{ft-kips} \quad (149.2KN\text{-}m)$$

$$V_u = 1.386(2.60^k) + 20.28^k = 24^k \quad (106.8KN)$$

Determine if ACI 318-95 section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(200^{ft-kips}) = 240^{ft-kips} \quad (325.4KN\text{-}m)$$

$$\Rightarrow (\phi M_n)_{col} \geq 120^{ft-kips} \quad (162.7KN\text{-}m)$$

Since M_u = $110^{ft-kips}$ < $120^{ft-kips}$ strong column/weak beam criteria governs design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{120^{ft-kips}(12"/1')}{0.9(60ksi)0.9(21.5")} = 1.38 - in^2 \quad (0.89X10^3\ mm^2)$$

Try 6 #5 (15M) bars (on both faces of the column); A_s = 6(0.31-n^2) = 1.86-in^2 (1.20X10^3 mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 1.5" – 0.375" – (0.625"/2) = 21.81" (554.0mm)

By inspection, the 6 #5 (15M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

ρ_{min} = 0.00333 (calculated previously in the beam design)

$$\rho = \frac{1.86 - in^2}{18"(21.81")} = 0.00473 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.86 - in^2(60ksi)}{0.85(4ksi)18"} = 1.82" \quad (46.2mm)$$

$$\phi M_n = 0.9(1.86 - in^2)60ksi\left(21.81" - \frac{1.82"}{2}\right)(1'/12") = 175^{ft-k} \quad (237.3KN\text{-}m)$$

$$\phi M_n = 175^{ft-k} > 120^{ft-k} = M_u \quad (237.3KN\text{-}m > 162.7KN\text{-}m) \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00473 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$z = f_s\sqrt[3]{d_c A} \leq 145^{k/in}$ (25.4KN/mm) (EQ. 10-5 ACI 318-95)

where; $fs = 0.6(60ksi) = 36ksi$ (288.2MPa)

$dc = 24" - 21.81" = 2.19"$ (55.6mm)

$A = 18"(2)(24" - 21.81")/6 = 13.14\text{-in}^2$ ($8.48X10^3$ mm^2)

$\therefore z = 36ksi\sqrt[3]{2.19"(13.14 - in^2)} = 110^{k/in} < 145^{k/in}$ (19.26KN/mm < 25.4KN/mm) **O.K.**

Therefore, provide 6 #5 (15M) longitudinal bars on opposite faces of the column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \leq V_u = 24^k$ (106.8KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L}$$

where; L =8.71' (2.66m) (column clear span)

$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f_c' b}$$

Therefore;

$$a = \frac{1.25(1.86 - in^2)60ksi}{0.85(4ksi)18"} = 2.28" \quad (57.9mm)$$

$$\therefore M_{pr1} = 1.25(1.86 - in^2)60ksi\left(21.81" - \frac{2.28"}{2}\right)(1'/12") = 240^{ft-k} \quad (325.4KN\text{-}m)$$

$$\therefore (V_e)_{max} = \frac{2(240^{ft-k})}{8.71'} = 55^k \quad (244.6KN)$$

$(V_u)_{max} = 24^k < 55^k = (V_e)_{max}$ (106.8KN < 244.6KN) **Therefore, ϕ = 0.6**

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 55^k > 12^k = \frac{1}{2}(V_u)_{max}$ (244.6KN > 53.4KN)

Condition (2); From the RISA-2D analysis, the largest axial load in an interior second floor column is;

$P_u = 1.386(19.25^k) + 18.09^k = 44.8^k$ (199.3KN)

$\frac{A_g f_c'}{20} = \frac{24"(18")4ksi}{20} = 86.4^k > 44.8^k$ (384.3KN > 199.3KN) **Therefore, V_c = 0**

Design stirrups;

Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an two extra interior cross ties will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \quad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; $2d = 2(21.81") = 43.62"$ Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - in^2))60ksi(21.81")}{55^k} = 6.28" \text{ Say } 6.0" \quad (152.4mm)$$

Spacing based on detail requirements;

$s = d/4 = 21.81"/4 = 5.5"$ (139.9mm)

s = 8(0.625") = 5.0" (127.0mm) (governs)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

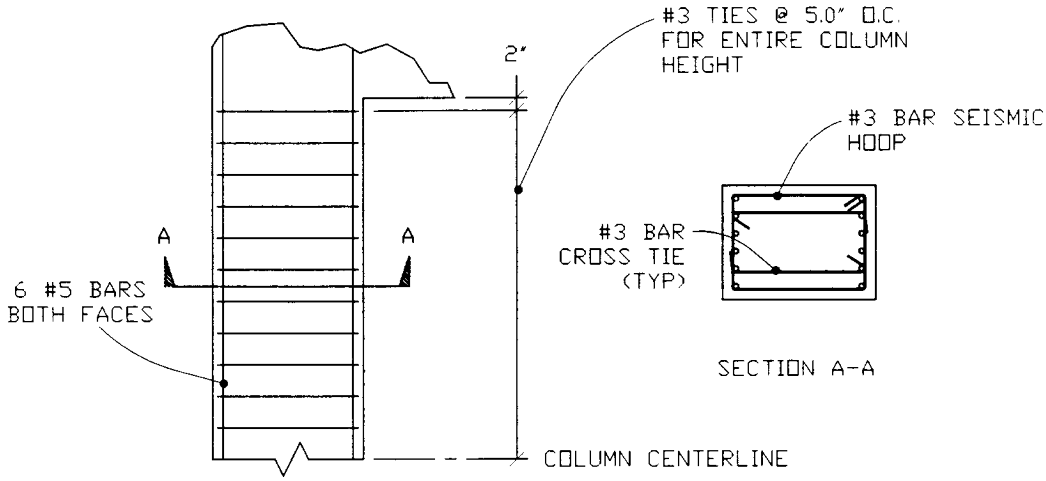
For the remainder of the column;

Since V_e is constant over the length of the column s = 5.0 -in. (127.0mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 5.0-in. (127.0mm) o.c. over the entire column length

Therefore, the second and third story end column design is as follows;

Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design splices for columns;

Longitudinal reinforcement for columns shall be spliced using welded splices or mechanical connectors in conformance with ACI 318-95 Section 21.4.3.2. Splices are allowed only within the center half of the column, and only alternate bars will be spliced at a section. The center to center distance between splices of adjacent bars shall be 24-in. (0.61m) or more.

Design joints;

Note: All joints have the same cross-sectional dimensions. Therefore, the following design for transverse reinforcement applies to all joints at each floor level in the frame. Also, the shear strength check will be performed only once for the worst case joint in the frame.

Design transverse reinforcement for confinement;

Per ACI 318-95 Section 21.5.2.1, the total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations;

$$A_{sh} = 0.3\left(\frac{sh_c f_c'}{f_{yh}}\right)\left[\left(\frac{A_g}{A_{ch}}\right)-1\right] \qquad \text{(EQ. 21-3 ACI 318-95)}$$

$$A_{sh} = 0.09\frac{sh_c f_c'}{f_{yh}} \qquad \text{(EQ. 21-4 ACI 318-95)}$$

where; h_c = 24" - 2(2") – 0.375" = 19.63" (498.6mm)
f_{yh} = 60ksi (413.7MPa)

$f_c' = 4ksi$ (27.6MPa)
$A_g = 18"(24") = 432\text{-}in^2$ ($278.6X10^3\ mm^2$)
$A_{ch} = (18" - 2(2"))(24"-2(2")) = 280\text{-}in^2$ ($180.6X10^6\ mm^2$)
$A_{sh} = 4(0.11\text{-}in^2) = 0.44\text{-}in^2$ ($0.28X10^3\ mm^2$)(using #3 (~10M) seismic hoops w/ 4 legs)

$$\therefore s = \frac{A_{sh}}{\left\{0.3\left(\frac{h_c f_c'}{f_{yh}}\right)\left[\left(\frac{A_g}{A_{ch}}\right)-1\right]\right\}}\frac{0.44-in^2}{\left\{0.3\left(\frac{19.63"(4ksi)}{60ksi}\right)\left[\left(\frac{432-in^2}{280-in^2}\right)-1\right]\right\}} = 2.07" \text{ Say } 2" \text{ (50.8mm)}$$

$$\text{or}\quad s = \frac{A_{sh}}{\left\{0.09h_c\frac{f_c'}{f_{yh}}\right\}} = \frac{0.44-in}{\left\{0.09(19.63")\left(\frac{4ksi}{60ksi}\right)\right\}} = 3.74" \text{ (95.0mm)}$$

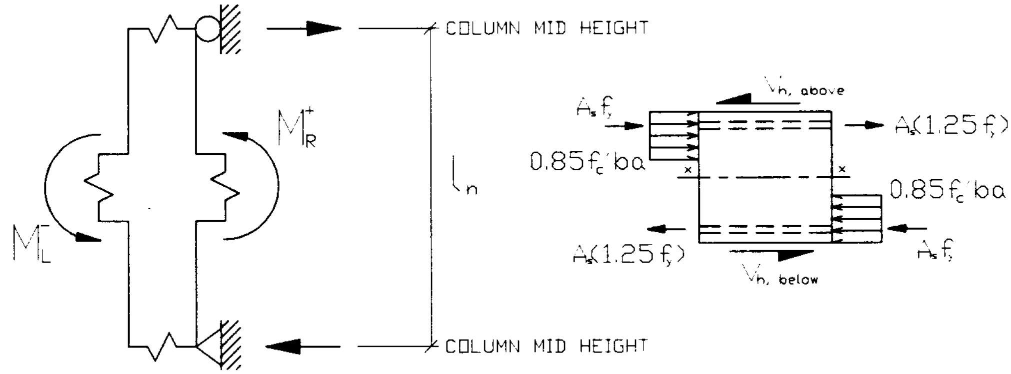
Provide stirrups consisting of 4 legs of #3 (~10M) bars at 2.0-in. (50.8mm) o.c. throughout joint

Determine (worst case) loading;

By inspection, the worst case loading occurs at an interior joint at the second floor level. It is assumed that beams in adjoining floors have formed plastic hinges at their junctions with the column, and that the end moments in the beams are resisted equally by the columns above and below the joint.

$$V_h \approx \frac{\sum(M_{pr})_{beams@joint}}{l_n}$$

where; $\Sigma(M_{pr})_{beams@joint} = 274^{ft\text{-}k} + 202^{ft\text{-}k} = 476^{ft\text{-}k}$ (645.5KN-m)
l_n = average of column clear spans between first and second floors
$l_n = (9.25' + 9')/2 = 9.13'$ (2.78m)



$$V_h = \frac{476^{ft-k}}{9.13'} = 52.1^k \text{ (231.7KN)}$$

$$T_1 = 1.25A_s^- f_y = 1.25(5(0.44 - in^2))60ksi = 165^k \text{ (733.9KN)}$$

$$C_2 = T_2 = 1.25A_s^+ f_y = 1.25(2(0.31 - in^2))60ksi = 46.5^k \text{ (206.8KN)}$$

Joint shear is evaluated at section x-x as follows;

$$\therefore V_{joint} = V_h - T_1 - C_2 = 52.1^k - 165^k - 46.5^k = -160^k \text{ (711.7KN)}$$

Determine shear capacity;
Per ACI 318-95 Section 21.5.3, the nominal shear strength of the joint shall be taken as;

$$\phi V_n = \phi V_c = \phi 12\sqrt{f_c'}A_j$$

where; $A_j = 18"(24") = 432\text{-}in^2$ ($278.6X10^3\ mm^2$)

$$\phi = 0.85$$

$$\therefore \phi V_n = 0.85(12)\sqrt{4{,}000\text{psi}}\,432 - \text{in}^2 = 279^k \;(1.24\text{MN})$$

Check capacity of joints;

$$\phi V_n = 279^k > 160^k = V_{joint} \quad (1.24\text{MN} > 711.7\text{KN})$$ **O.K.**

Determine embedment length of beam reinforcement within joint

Beam longitudinal steel will be anchored in the core of the end columns using a standard 90-deg hook. These bars consist of three sizes. The embedment length for each bar size is determined based on ACI 318-95 section 21.5.4 as follows;

$$l_{dh} = \frac{f_y d_b}{\left(65\sqrt{f_c'}\right)} \qquad \text{(EQ. 21-5 ACI 318-95)}$$

where; l_{dh} shall not be less than $8d_b$ or 6-in. (152.4mm)
f_y = 60,000psi (413.7MPa)
d_b = 0.5" (12.7mm) for #4 (~10M) bars, 0.625" (15.9mm) for #5 (15M) bars, and 0.75" (19.1mm) for #6 (~20M) bars
fc' = 4,000psi

The embedment length for each bar size is calculated in the table below;

Bar Size (#)	Bar Diameter (in)	$8d_b$ (in)	$l_{dh} = f_y d_b/(65\text{sqrt}(f_c'))$ (in)	l_{dh} (in)
4	0.500	4.00	7.3	7.5
5	0.625	5.00	9.1	9.5
6	0.750	6.00	10.9	11

#4 bar ~ 10M bar
#5 bar = 15M bar
#6 bar ~ 20M bar
1-in = 25.4mm

Design diaphragm;

By inspection, the maximum diaphragm shear occurs at the landing along grid lines 1 or 2. At this location a 15-ft. (4.58m) length of diaphragm must transmit the shear to the shear wall on grid line 1. This shear has been previously determined to be $v_u = 34.1^k$ (151.7KN) and includes torsional effects.

Determine shear capacity of diaphragm; use f_c' = 4,000psi (27.6MPa), and f_y = 60ksi (413.7MPa)

Capacity of diaphragm is determined from ACI 318-95 Section 21.6.5.2;

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_n f_y) \qquad \text{(EQ. 21-6 ACI 318-95)}$$

Try minimum reinforcement in conformance with ACI 318-95 Section 21.6.2;

$$\rho_{min} = 0.0018$$

$$\therefore \phi V_n = 0.85(2.5")(15')(12"/1')\left[2\sqrt{4{,}000\text{psi}} + 0.0018(60{,}000\text{psi})\right](1^k/1000^{lb}) = 89.7^k \quad (399.0\text{KN})$$

$$\phi V_n = 89.7^k > 34.1^k = V_u \quad (399.0\text{KN} > 151.7\text{KN})$$ **O.K.**

Therefore, design for minimum reinforcement;

Determine spacing of #4 (~10M) bars;

$$\rho_{min} = \frac{0.20 - \text{in}^2}{s(2.5")} = 0.0018 \Rightarrow s = 44.4" \;(1.13\text{m})$$

However, minimum spacing per ACI 318-95 Section 7.12.2.2 is;

s = 5 x (slab thickness) = 12.5" (317.5mm) (governs)

or s = 18" (457.2mm)

Also, per ACI 318-95 section 21.6.5.5, reinforcement must be equal in both directions.

Therefore, use #4 (~10M) bars at 12-in. (304.8mm) o.c. each way for slab reinforcement

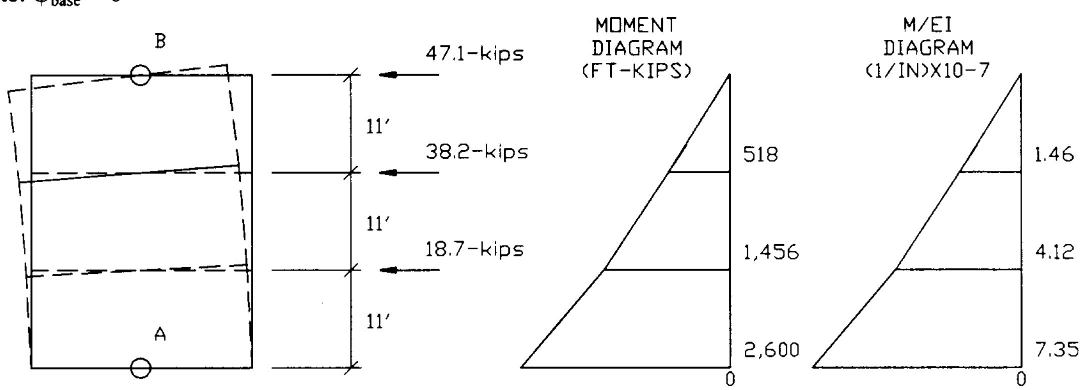
Design transverse coupling beams;

Transverse coupling beams connect transverse shear walls between grid lines B and C and were designed previously for gravity loads. These beams are considered flexible and are intended not to restrict the independent cantilever action of the walls. Instead, these beams serve only to transfer gravity loads between the walls acting in bearing. However, they must be checked to ensure that they can accommodate the in-plane rotations of the walls under lateral loads. The worst case condition occurs at the roof on grid lines 2 and 8 where the rotations are largest. The rotation of a shear wall at the roof level id determined by the moment area method.

$$\phi_{roof} - \phi_{base} = \int_A^B \frac{M_u}{E_c I_{cr}} dx$$

where; E_c = modulus of elasticity of cracked concrete = 3,605ksi ($24.9 x10^3$ MPa)
$I_{cr} = 0.7 I_g$ = moment of inertia of cracked wall = $11{,}774 x10^3$-in^4 ($4.90X10^{12}$ mm^4)

Note: $\phi_{base} = 0$



1-kip = 4.448KN 1-ft-kip = 1.356KN-m 1-in = 25.4mm

$$\phi_{roof} = \left[\frac{1}{2}(1.46) + (1.46) + \frac{1}{2}(4.12 - 1.46) + 4.12 + \frac{1}{2}(7.35 - 4.12)\right] 132\text{"} x10^{-7} = 0.000122 \text{rad} = 0.007 \deg$$

The moment acting in the beam due to this rotation will now be calculated. Due to symmetry, this moment is the same at both ends of the beam and puts the beam into reverse curnature. This moment will be added to the moment caused by gravity loads, and will then be checked against the beams flexural capacity. Additionally, to insure that a shear failure will not occur, the shear reinforcement will be revised to that required for the beams nominal flexural capacity per ACI 318-95 Section 21.3.4.1.

Using the moment area method, the relationship between the beams end rotation and the moment causing that rotation can be obtained;

$$2\phi = \frac{1}{2}\left(\frac{M}{E_c I_{cr}}\right)\frac{L}{2} \quad \text{or} \quad M = \frac{8\phi E_c I_{cr}}{L}$$

where; L = the beam length = 6' (1.83m)
E_c = 3,605ksi ($24.9 x10^3$ MPa)
$I_{cr} = 0.35 I_g = 0.35(1/12)(17")(12")^3$ = 857-in^4 ($356.7X10^6$ mm^4)

$$\therefore M = \frac{8(0.000122)\text{rad}(3{,}605\text{ksi})857 - \text{in}^4}{6'(12"/1')} = 41.8^{\text{in-k}} \quad (56.7\text{KN-m})$$

The moment due to gravity loads was previously calculated as $M_u = 144^{\text{in-k}}$ (195.3KN-m), and the capacity was previously calculated as $\phi M_n = 344^{\text{in-k}}$ (466.5KN-m).

Therefore;

$$M_u = 41.8^{in-k} + 144^{in-k} = 186^{in-k} < 344^{in-k} = \phi M_n \quad (252.2\text{KN-m} < 466.8\text{KN-m})$$ **O.K.**

Design for shear;

In the gravity load design section of this problem, V_e was calculated as 25.8^k (114.8KN), and V_u was calculated as 12^k (53.4KN). It was also established that $\phi = 0.6$.
Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $V_e = 25.8^k > 6^k = \frac{1}{2}V_u$

Condition (2); By inspection, the axial load is negligible. **Therefore, $V_c = 0$**

Design stirrups;

$$V_s = \frac{A_v f_y d}{s}$$ (EQ. 11-15 ACI 318-95)

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(10.6") = 21.2" Say 2' (0.61m)

$$s = \frac{\phi A_v f_y d}{V_e} = \frac{0.6(2(0.11 - in^2))60ksi(10.6")}{25.8^k} = 3.25" \quad (82.6mm)$$

Spacing based on detail requirements;

s = d/4 = 10.6"/4 = =2.65" Say 2.5" (63.5mm) (governs)
s = 8(0.625") = 5" (127.0mm)
s = 24(0.375") = 9" (228.8mm)
s = 12" (304.8mm)

Therefore, within 2-ft. (0.61m) of the end of the beams, provide stirrups consisting of 2 legs of #3 (~10M) bars at 2.5-in. (63.5mm) o. c.

For the remainder of the beam;
Spacing based on strength requirements;
Since Ve is constant over the member length; s = 3.25" Say 3" (76.2mm) (governs)
Spacing based on detail requirements;

s = d/2 = 13.88"/2 = 6.94" (176.3mm)

$$A_v = 50\frac{b_w s}{f_y}$$ (EQ. 11-13 ACI 318-95)

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11 - in^2)60{,}000psi}{50(18")} = 22" \quad (558.8mm)$$

For the remainder of the beam, provide stirrups consisting of 2 legs of #3 (~10M) bars at 3-in. (76.2mm) o.c. max

B-12 *Determine allowable drift and PΔ effect.*

Per TI 809-04 Table 6-1, the allowable interstory drift is $0.020h_{sx}$. The calculated story drifts are to be multiplied by the C_d factors listed in Table 7-1 before comparison to the allowable drift.

For both the transverse and the longitudinal directions, the allowable story drift is;

$$\delta_{allow} = 0.020(11')(12"/1') = 2.64" \quad (67.1mm)$$

Check drift in the transverse direction;

$C_d = 5.0$ (for specially reinforced concrete shear walls per Table 7-1)

The worst case condition occurs at walls along grid lines 2 or 8. The displacements at a level relative to ground are calculated using the moment area method as follows;

$$\Delta_{roof} = \left\{\frac{1}{2}(1.46)\frac{2}{3} + \frac{1}{2}(4.12 - 1.46)\frac{5}{3} + (1.46)\frac{3}{2} + \frac{1}{2}(7.35 - 4.12)\frac{8}{3} + (4.12)\frac{5}{2}\right\}(132)^2 x10^{-7} = 0.034 - \text{in}$$

(0.86mm)

$$\Delta_{2^{nd}\,floor} = \left\{\frac{1}{2}(4.12 - 1.46)\frac{2}{3} + (1.46)\frac{1}{2} + \frac{1}{2}(7.35 - 4.12)\frac{5}{3} + (4.12)\frac{3}{2}\right\}(132)^2 x10^{-7} = 0.018 - \text{in}$$

(0.46mm)

$$\Delta_{3^{rd}\,floor} = \left\{\frac{1}{2}(7.35 - 4.12)\frac{2}{3} + (4.12)\frac{1}{2}\right\}(132)^2 x10^{-7} = 0.006 - \text{in} \ \ (0.15\text{mm})$$

Third story drift; $\delta_3 = \Delta_{roof} - \Delta_{3^{rd}\,floor} = 0.034\text{"} - 0.018\text{"} = 0.016 - \text{in}$ (0.41mm) (governs)

Second story drift; $\delta_2 = \Delta_{3^{rd}\,floor} - \Delta_{2^{nd}\,floor} = 0.018\text{"} - 0.006\text{"} = 0.012 - \text{in}$ (0.30mm)

First story drift; $\delta_1 = \Delta_{2^{nd}\,floor} - \Delta_{1^{st}\,floor} = 0.006\text{"} - 0.0\text{"} = 0.006 - \text{in}$ (0.15mm)

$$C_d x \delta_3 = 5.0(0.016\text{"}) = 0.08\text{"} < 2.64\text{"} = \delta_{allow} \ \ (2.03\text{mm} < 67.1\text{mm})$$

Therefore, story drift is satisfied in the transverse direction

Check drift in the longitudinal direction;

$C_d = 5.5$ (for specially reinforced concrete moment frames per Table 7-1)

The displacements at a level relative to ground were calculated in the RISA-2D analysis and are as follows;

$\Delta_{roof} = 1.085 - \text{in}$ (27.6mm)
$\Delta_{2^{nd}\,floor} = 0.710 - \text{in}$ (18.0mm)
$\Delta_{3^{rd}\,floor} = 0.279 - \text{in}$ (7.1mm)

Third story drift; $\delta_3 = \Delta_{roof} - \Delta_{2^{nd}\,floor} = 1.085\text{"} - 0.710\text{"} = 0.375 - \text{in}$ (9.5mm)

Second story drift; $\delta_2 = \Delta_{2^{nd}\,floor} - \Delta_{3^{rd}\,floor} = 0.710\text{"} - 0.279\text{"} = 0.431 - \text{in}$ (10.9mm) (governs)

First story drift; $\delta_1 = \Delta_{2^{nd}\,floor} - \Delta_{1^{st}\,floor} = 0.279\text{"} - 0.0\text{"} = 0.279 - \text{in}$ (7.1mm)

$$C_d x \delta_3 = 5.5(0.431\text{"}) = 2.37\text{"} < 2.64\text{"} = \delta_{allow} \ \ (60.2\text{mm} < 67.1\text{mm})$$

Therefore, story drift is satisfied in the longitudinal direction

Per FEMA 302 P-delta effects need not be considered when the following equation is equal to or less than 0.10:

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

where; P_x = total vertical design load at and above level x without load factors
Δ = story drift at level x
V_x = seismic story shear force at level x
h_{sx} = story height below level x

Check the requirements for P-delta effects in the transverse direction;

$$\theta_{\text{roof}} = \frac{(1{,}283^{k} + 0.20\text{ksf}(8{,}181 - \text{ft}^2))0.016''}{338^{k}(10.38')(12''/1')5.0} = 0.00022 << 0.10$$

$$\theta_{3^{rd}\text{ floor}} = \frac{(1{,}283^{k} + 1{,}573^{k} + (0.20 + 0.16)\text{ksf}(8{,}181 - \text{ft}^2))0.012''}{276^{k}(11')(12''/1')5.0} = 0.00038 << 0.10$$

$$\theta_{2^{nd}\text{ floor}} = \frac{(4{,}428^{k} + (0.20 + 2(0.16))\text{ksf}(8{,}181 - \text{ft}^2))0.006''}{138^{k}(10.25')(12''/1')5.0} = 0.00061 << 0.10$$

Therefore, P-delta effects need not be considered in the longitudinal direction

Check the requirements for P-delta effects in the longitudinal direction;

$$\theta_{\text{roof}} = \frac{(1{,}283^{k} + 0.20\text{ksf}(8{,}181 - \text{ft}^2))0.375''}{239^{k}(10.38')(12''/1')5.5} = 0.007 < 0.10$$

$$\theta_{3^{rd}\text{ floor}} = \frac{(1{,}283^{k} + 1{,}573^{k} + (0.20 + 0.16)\text{ksf}(8{,}181 - \text{ft}^2))0.431''}{195^{k}(11')(12''/1')5.5} = 0.018 << 0.10$$

$$\theta_{2^{nd}\text{ floor}} = \frac{(4{,}428^{k} + (0.20 + 2(0.16))\text{ksf}(8{,}181 - \text{ft}^2))0.279''}{98^{k}(10.25')(12''/1')5.5} = 0.037 << 0.10$$

Therefore, P-delta effects need not be considered in the transverse direction