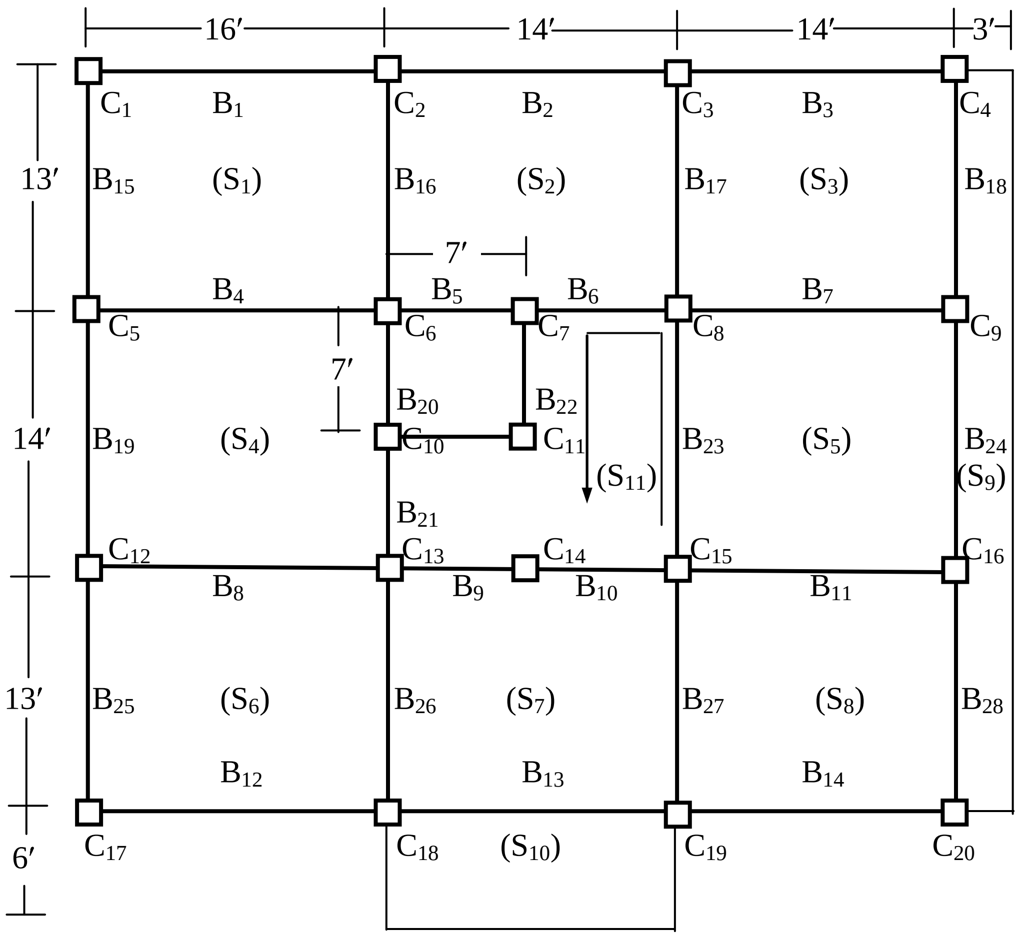
Design of a Multi-Storied RC Building



Building Plan

Building Height = 4@10′ = 40′

Loads: LL = 40 psf, FF = 20 psf, RW = 20 psf

Seismic Coefficients: Z = 0.15, I = 1.0, S = 1.0, R = 5.0

Material Properties: f'_c = 3 ksi, f_s = 20 ksi, Allowable Bearing Capacity of soil = 2 ksf

1. Design of Slabs

Largest Slab is S_4, with clear area (13′×15′).

∴ Assumed slab thickness, t = (13′ +15′)×2/180 =3.73″; i.e., 4″⇒ d = 3″ (or 2.5″ for M_{min})

∴ Self Wt.= 50 psf ⇒ DL = 50+20+20 = 90 psf = 0.09 ksf

LL = 40 psf = 0.04 ksf ⇒ Total Wt./slab area = 0.09 + 0.04 = 0.13 ksf

For design, n = 9, k = 9/(9+20/1.35) = 0.378, j = 1– k/3 = 0.874

R = ½ ×1.35×0.378×0.874 = 0.223 ksi

$A_s = M/f_s jd$ = M×12/(20×0.874×3) = M/4.37 (or M/3.64 for M_{min})

$A_{s(Temp)}$ = 0.0025 bt = 0.0025×12×4 = 0.12 in^2/′

Slab (S_1):

Slab size (12′×15′), m =12/15 = 0.80, Support condition Case 4.

M_A^+ = (0.039×0.09+0.048×0.04)×$(12)^2$ = 0.782 k′/′ ⇒ $A_{s(A)}^+$ = 0.782/4.37 = 0.18 in^2/′

M_B^+ = (0.016×0.09+0.020×0.04)×$(15)^2$ = 0.504 k′/′ ⇒ $A_{s(B)}^+$ = 0.504/3.64 = 0.14 in^2/′

M_A^- = (0.071×0.13)×$(12)^2$ = 1.329 k′/′ ⇒ $A_{s(A)}^-$ = 1.329/4.37 = 0.30 in^2/′

M_B^- = (0.029×0.13)×$(15)^2$ = 0.848 k′/′⇒ $A_{s(B)}^-$ = 0.848/4.37 = 0.19 in^2/′

Also, d_{req} = √(M_{max}/R) = √(1.329/0.223) = 2.44″, which is < 3″, OK.

Slab (S_2):

Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 3.

M_A^+ = (0.023×0.09+0.033×0.04)×$(12)^2$ = 0.488 k′/′ ⇒ $A_{s(A)}^+$ = 0.488/3.64 = 0.13 in^2/′

M_B^+ = (0.025×0.09+0.028×0.04)×$(13)^2$ = 0.570 k′/′ ⇒ $A_{s(B)}^+$ = 0.570/4.37 = 0.13 in^2/′

M_A^- = 0 ⇒ $A_{s(A)}^-$ = 0

M_B^- = (0.071×0.13)×$(13)^2$ = 1.560 k′/′⇒ $A_{s(B)}^-$ = 1.560/4.37 = 0.36 in^2/′

Also, d_{req} = √(M_{max}/R) = √(1.560/0.223) = 2.65″, which is < 3″, OK.

Slab (S_3):

Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 4.

M_A^+ = (0.032×0.09+0.037×0.04)×$(12)^2$ = 0.628 k′/′ ⇒ $A_{s(A)}^+$ = 0.628/4.37 = 0.14 in^2/′

M_B^+ = (0.023×0.09+0.028×0.04)×$(13)^2$ = 0.539 k′/′ ⇒ $A_{s(B)}^+$ = 0.539/3.64 = 0.15 in^2/′

M_A^- = (0.058×0.13)×$(12)^2$ = 1.086 k′/′ ⇒ $A_{s(A)}^-$ = 1.086/4.37 = 0.25 in^2/′

M_B^- = (0.042×0.13)×$(13)^2$ = 0.923 k′/′⇒ $A_{s(B)}^-$ = 0.923/4.37 = 0.21 in^2/′

Also, d_{req} = √(M_{max}/R) = √(1.086/0.223) = 2.21″, which is < 3″, OK.

Slab (S_4):

Slab size (13′×15′), m =13/15 = 0.87, Support condition between Case 5 and Case 9.

$M_A^+ = (0.029\times0.09+0.038\times0.04)\times(13)^2 = 0.698$ k′/′ $\Rightarrow A_{s(A)}^+ = 0.698/4.37 = 0.16$ in^2/′

$M_B^+ = (0.013\times0.09+0.020\times0.04)\times(15)^2 = 0.443$ k′/′ $\Rightarrow A_{s(B)}^+ = 0.443/3.64 = 0.12$ in^2/′

$M_A^- = (0.075\times0.13)\times(13)^2 = 1.648$ k′/′ $\Rightarrow A_{s(A)}^- = 1.648/4.37 = 0.38$ in^2/′

$M_B^- = (0.011\times0.13)\times(15)^2 = 0.322$ k′/′$\Rightarrow A_{s(B)}^- = 0.322/4.37 = 0.07$ in^2/′

Also, $d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.648/0.223)} = 2.72''$, which is < 3″, OK.

Slab (S_5):

Slab size (13′×13′), m =13/13 = 1.00, Support condition between Case 5 and Case 9.

$M_A^+ = (0.025\times0.09+0.031\times0.04)\times(13)^2 = 0.590$ k′/′ $\Rightarrow A_{s(A)}^+ = 0.590/4.37 = 0.13$ in^2/′

$M_B^+ = (0.019\times0.09+0.028\times0.04)\times(13)^2 = 0.478$ k′/′ $\Rightarrow A_{s(B)}^+ = 0.478/3.64 = 0.13$ in^2/′

$M_A^- = (0.068\times0.13)\times(13)^2 = 1.494$ k′/′ $\Rightarrow A_{s(A)}^- = 1.494/4.37 = 0.34$ in^2/′

$M_B^- = (0.016\times0.13)\times(13)^2 = 0.352$ k′/′ $\Rightarrow A_{s(B)}^- = 0.352/4.37 = 0.08$ in^2/′

Also, $d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.494/0.223)} = 2.59''$, which is < 3″, OK.

Slab (S_6):

Slab size (12′×15′), m =12/15 = 0.80, Support condition Case 4. ∴ Same design as S_1.

Slab (S_7):

Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 8.

$M_A^+ = (0.024\times0.09+0.033\times0.04)\times(12)^2 = 0.501$ k′/′ $\Rightarrow A_{s(A)}^+ = 0.501/4.37 = 0.11$ in^2/′

$M_B^+ = (0.020\times0.09+0.025\times0.04)\times(13)^2 = 0.473$ k′/′ $\Rightarrow A_{s(B)}^+ = 0.473/3.64 = 0.13$ in^2/′

$M_A^- = (0.041\times0.13)\times(12)^2 = 0.767$ k′/′ $\Rightarrow A_{s(A)}^- = 0.767/4.37 = 0.18$ in^2/′

$M_B^- = (0.054\times0.13)\times(13)^2 = 1.186$ k′/′$\Rightarrow A_{s(B)}^- = 1.186/4.37 = 0.27$ in^2/′

Also, $d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.186/0.223)} = 2.31''$, which is < 3″, OK.

Slab (S_8):

Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 4. ∴ Same design as S_3.

Slab (S_9):

One-way cantilever slab with clear span = 2.5′

∴ Required thickness, $t = (L/10)\times(0.4+f_y/100) = (2.5\times12/10)\times(0.4+40/100) = 2.4'' < 4''$, OK

∴ $w = w_{DL} + w_{FF} + w_{LL} = 50 + 20 + 40 = 110.00$ psf = 0.110 ksf

∴ $M^- = 0.11\times(2.5)^2/2 = 0.344$ k′/′ $\Rightarrow A_s^- = 0.344/4.37 = 0.08$ in^2/′

Slab (S_{10}):

One-way cantilever slab with clear span = 5.5′

$\therefore$ Required thickness, t = (5.5×12/10)×(0.4+40/100) = 5.28″⇒ 5.5″

$\therefore$ $w = w_{DL} + w_{FF} + w_{LL}$ = 68.75 + 20 + 20 = 108.75 psf = 0.109 ksf

$\therefore$ M^{-} = 0.109×(5.5)^2/2 = 1.644 k′/′ ⇒ A_s^{-} = 1.644×12/(20×0.874×(5.5–1)) = 0.25 in^2/′

Slab (S_{11}):

One-way simply supported slab with c/c span = 14′ [two 3′ landings and one 8′ flight]

Assumed LL on stairs = 100 psf

$\therefore$ Required thickness, t = (14×12/20)×(0.4+40/100) = 6.72″⇒7″; Self weight = 87.5 psf.

$\therefore$ Weight on landing, $w_1 = w_{DL} + w_{FF} + w_{LL}$ = 87.5 + 20 + 100 = 207.5 psf = 0.208 ksf

Additional weight on flights due to 6″ high stairs = ½ × (6/12)×150 psf = 0.037 ksf

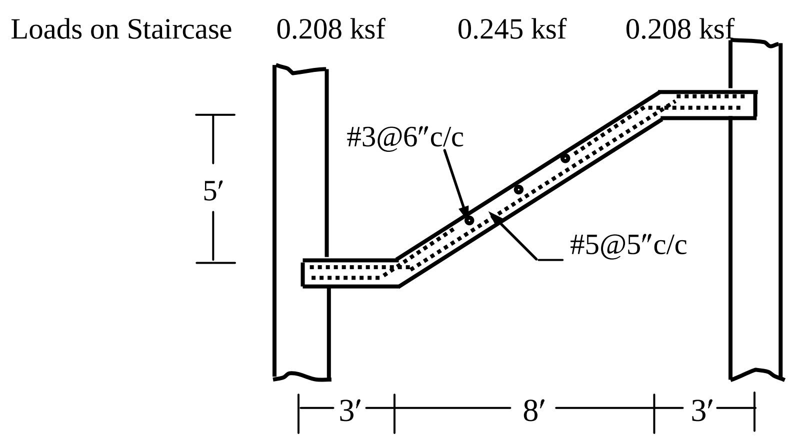
$\therefore$ Weight on flight, w_2 = 0.208 + 0.037 = 0.245 ksf

$\therefore$ $M_{max} \cong 0.245\times(14)^2/8$ = 6.003 k′/′

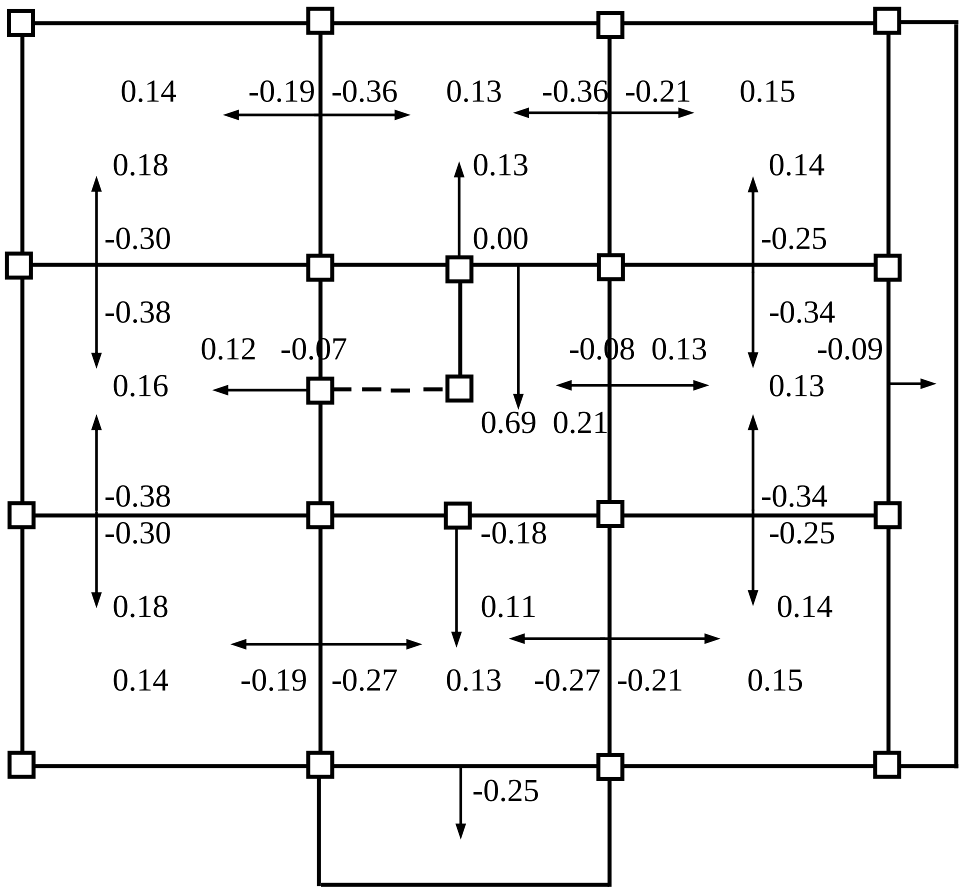
$\therefore$ $d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(6.003/0.223)}$ = 5.19″, which is < (7–1) = 6″, OK.

⇒ A_s^{+} = 6.003×12/(20×0.874×(7–1)) = 0.69 in^2/′; i.e., #5@5″c/c

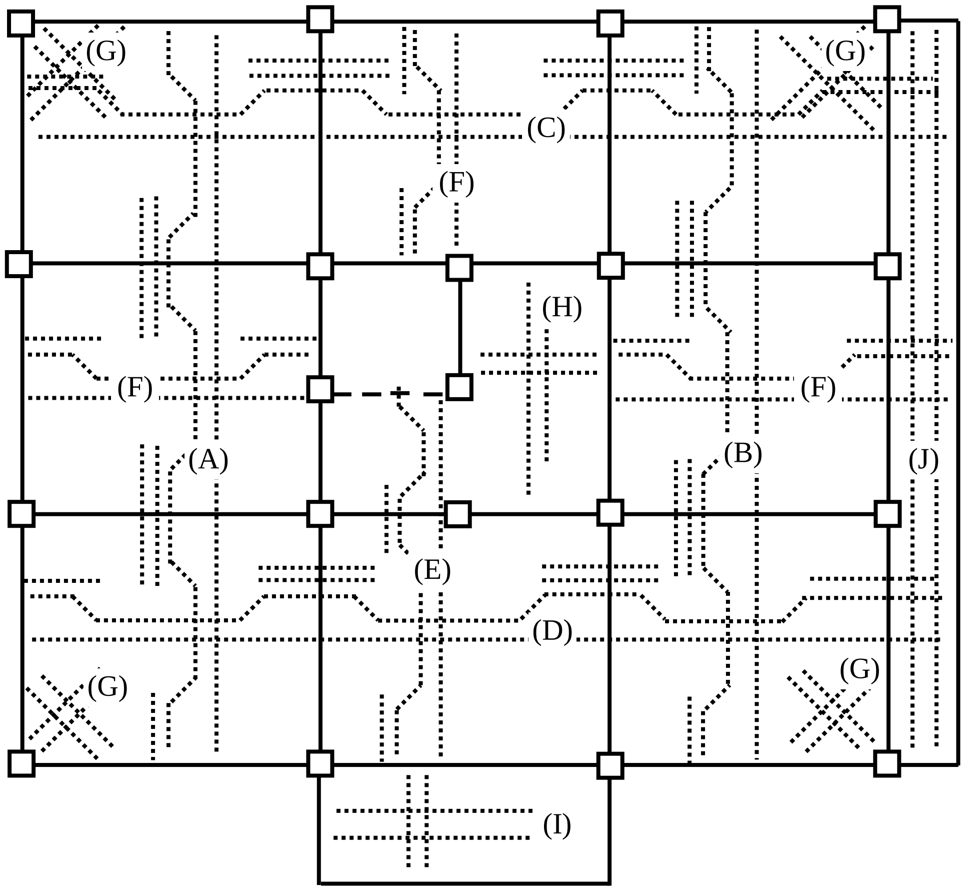
$A_{s(Temp)}$ = 0.0025 bt = 0.0025×12×7 = 0.21 in^2/′; i.e., #3@6″c/c



Staircase Reinforcements



Required Slab Reinforcement ($in^2/'$) from Flexural Design
[Note: $A_{s(Temp)}$ = 0.12 $in^2/'$ and S_{max} = 2t, must be considered in all cases]



Slab Reinforcements

(A): #3@7″c/c, alt. ckd. with (1#3, 1#4) extra top

(B): #3@8″c/c, alt. ckd. with (2#4) extra top

(C): #3@8″c/c, alt. ckd. with (2#4) extra top

(D): #3@8″c/c, alt. ckd. with (1#3, 1#4) extra top

(E): #3@8″c/c, alt. ckd. with (1#4) extra top

(F): #3@8″c/c, alt. ckd. with (1#3) extra top

(G): Corner Reinforcement – #3@7″c/c at top and bottom, parallel to Slab diagonals

(H): Staircase Reinforcement – #5@5″c/c Main (bottom), #3@6″c/c Temperature Rod

(I): S_{10} Reinforcement – #3@5″c/c at Main (top), #3@10″c/c Temperature Rod

(J): S_9 Reinforcement – Temperature Rod #3@8″c/c both ways

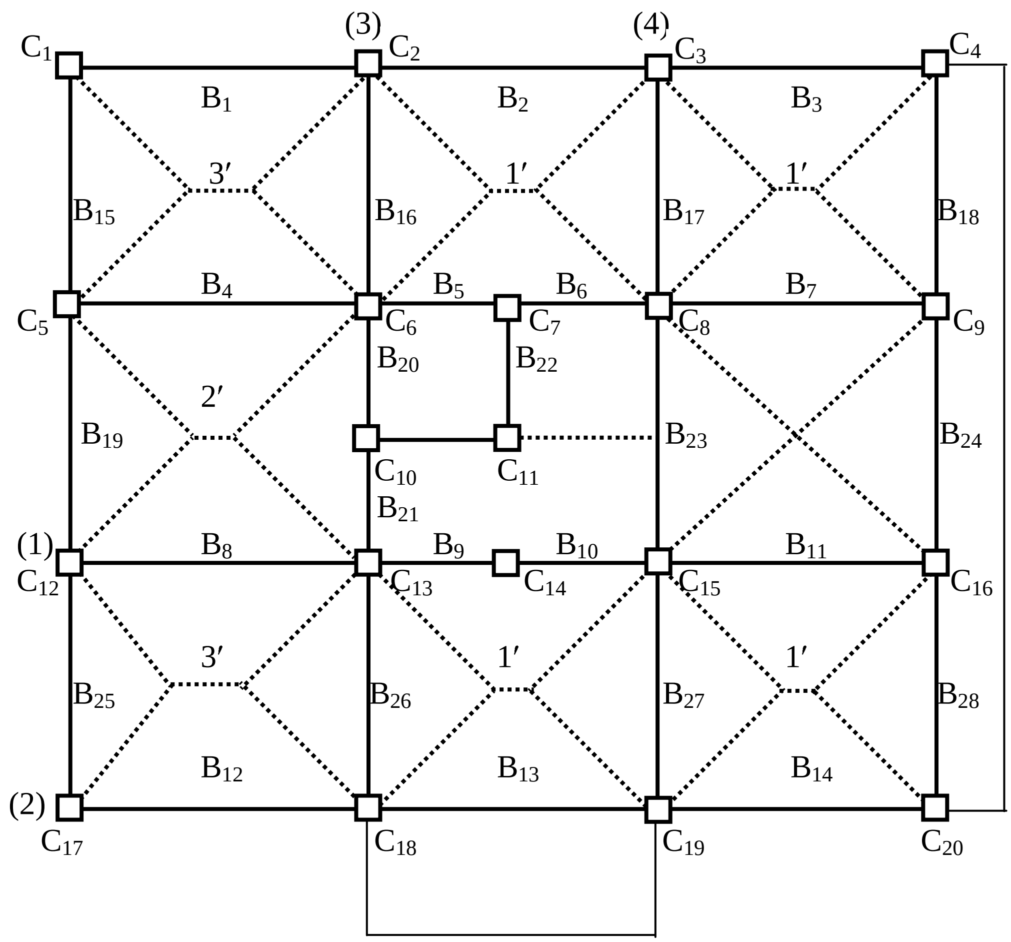
2. Vertical Load Analysis of Beams and Columns

Beams are assumed to be 12″×12″ below the slab.

∴ Self-weight of Beams = (12″×12″)×150/144 = 150 lb/′ = 0.15 k/′

Weight of 5″ Partition Walls (PW) = (5″/12)×9′×120 = 450 lb/′ = 0.45 k/′

∴ Weight of 10″ Exterior Walls (EW) = 0.90 k/′



Load Distribution from Slab to Beam

Frame (1) [$B_{8-9-10-11}$]:

Slab-load on B_8 = [13/2×(16+3)/2+14/2×(16+2)/2]× 0.13 = 16.22 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 16.22/16 + 0.15 + 0.45 = 1.61 k/′

Slab-load on B_{9-10} ≅ 13/2×(14+1)/2×0.13 +(14×14–7×7)/2×(0.208+0.245)/2

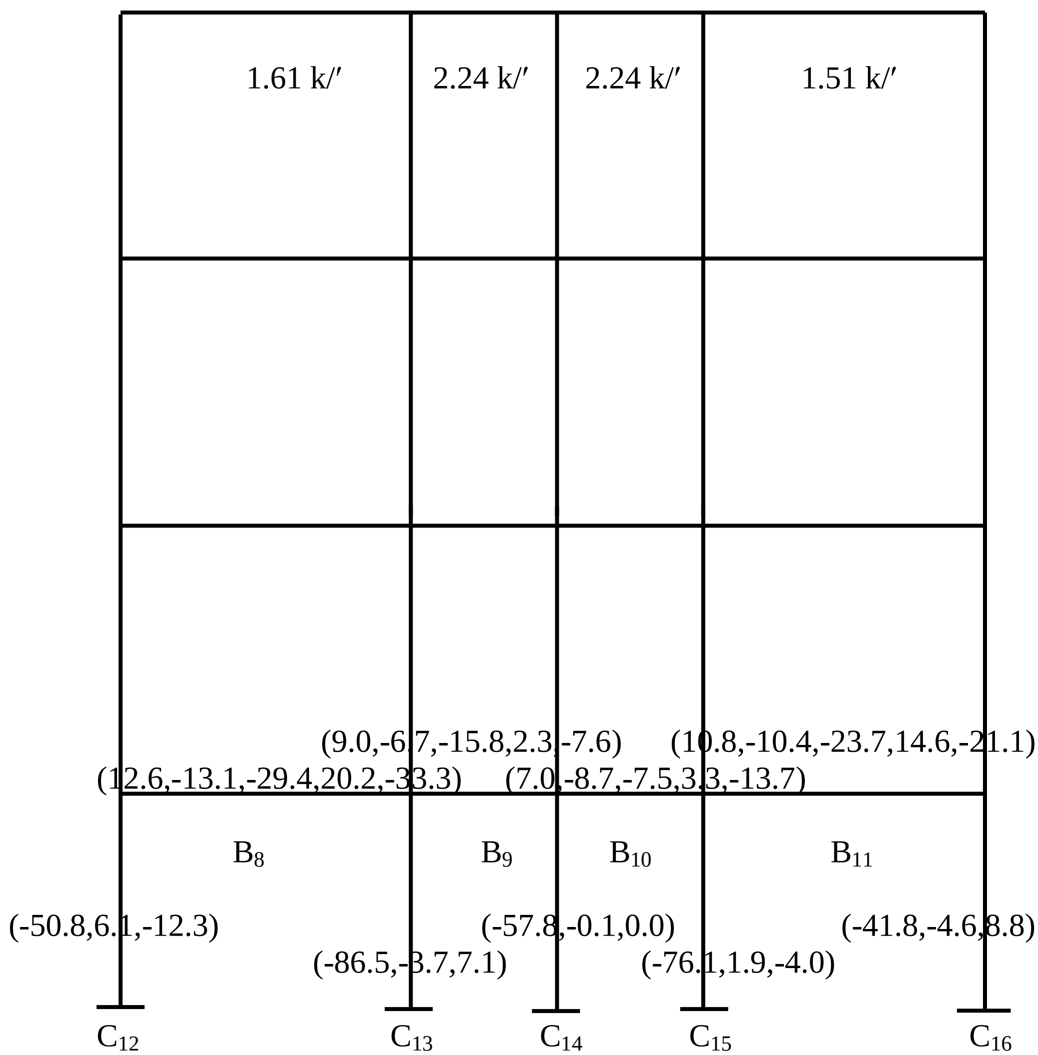
= 6.34 + 16.65 = 22.99 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 22.99/14 + 0.15 + 0.45 = 2.24 k/′

Load from Slabs to B_{11} = [13/2×(14+1)/2+14/2×(14)/2]× 0.13 = 12.71 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 12.71/14 + 0.15 + 0.45 = 1.51 k/′

[One cannot use the ACI Coefficients here due to large differences in adjacent Spans]



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (1) from Vertical Load Analysis

Frame (2) [$B_{12\text{-}13\text{-}14}$]:

Slab-load on B_{12} = [13/2×(16+3)/2]× 0.13 = 8.03 ^{k}

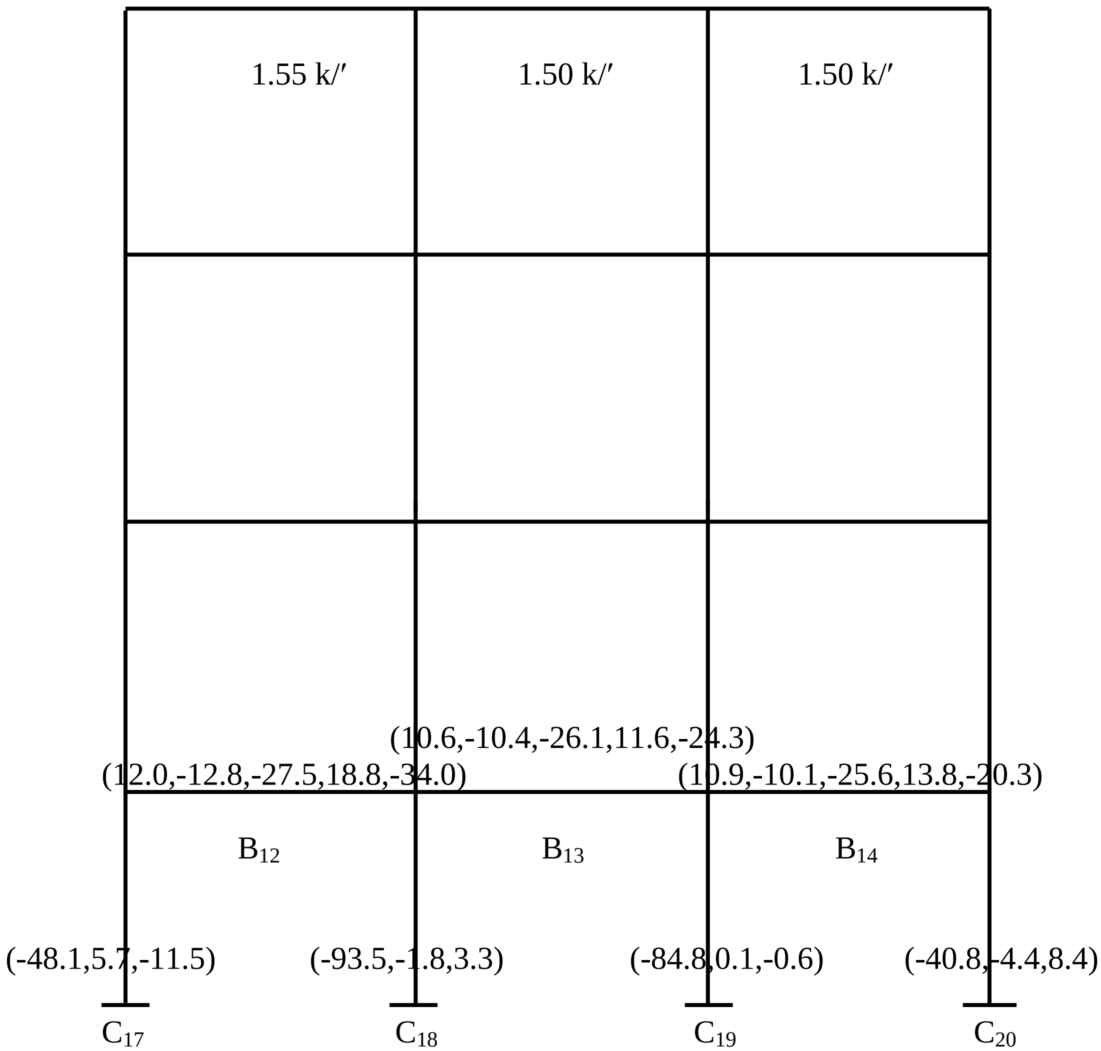
∴ Equivalent UDL (+ Self Wt. and EW) ≅ 8.03/16 + 0.15 + 0.90 = 1.55 k/′

Slab-load on B_{13} = [13/2×(14+1)/2]×0.13 = 6.34^k

∴ Equivalent UDL (+ Self Wt. and EW) ≅ 6.34/14 + 0.15 + 0.90 = 1.50 k/′

Load from Slabs to B_{14} = [13/2×(14+1)/2]× 0.13 = 6.34 ^{k}

∴ Equivalent UDL (+ Self Wt. and EW) ≅ 6.34/14 + 0.15 + 0.90 = 1.50 k/′



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (2) from Vertical Load Analysis

Frame (3) [$B_{16-20-21-26}$]:

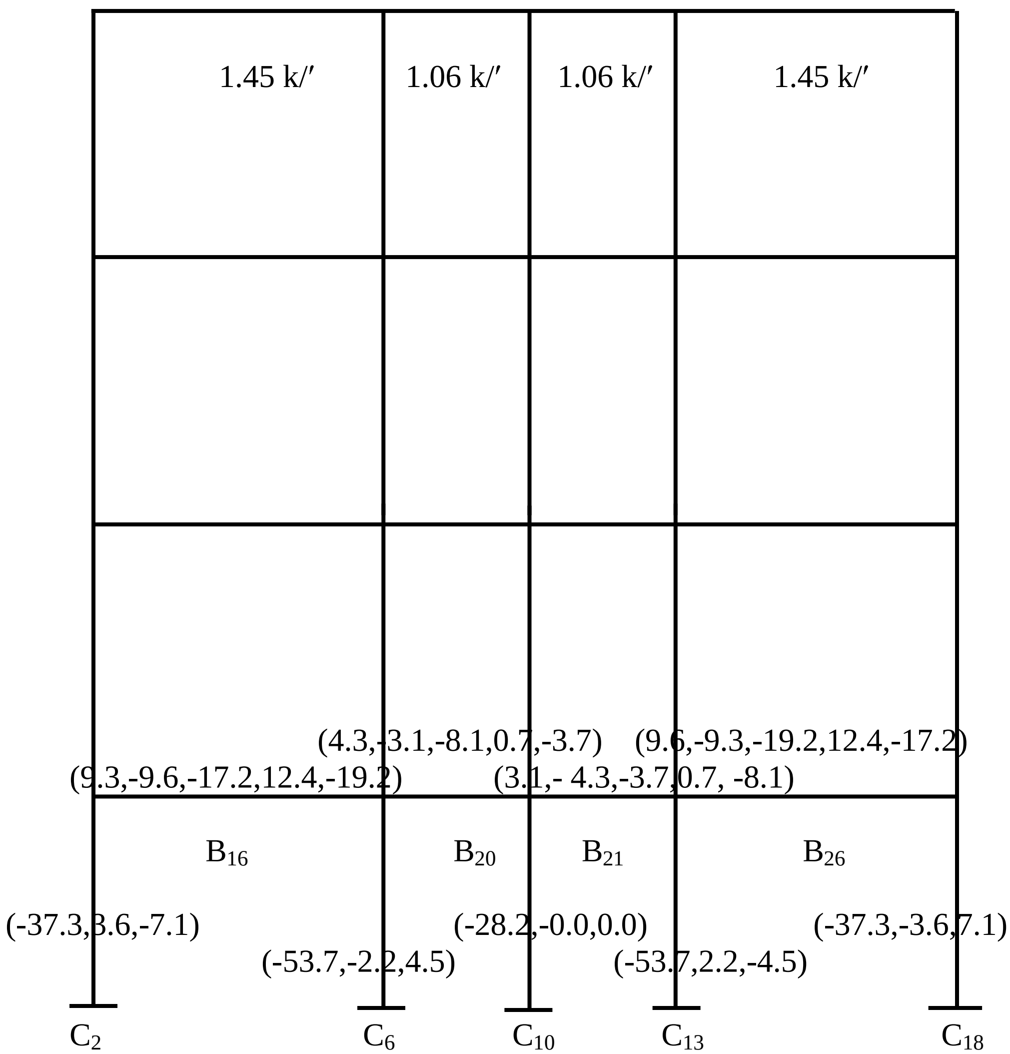
Slab-load on B_{16} and B_{26} = [13/2×(13)/2+13/2×(13)/2]× 0.13 = 10.99 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 10.99/13 + 0.15 + 0.45 = 1.45 k/′

Slab-load on B_{20-21} ≅ [14/2×(14)/2]× 0.13 = 6.37 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 6.37/14 + 0.15 + 0.45 = 1.06 k/′

[One cannot use ACI Coefficients here due to large differences in adjacent Spans]



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (3) from Vertical Load Analysis

Frame (4) [$B_{17-23-27}$]:

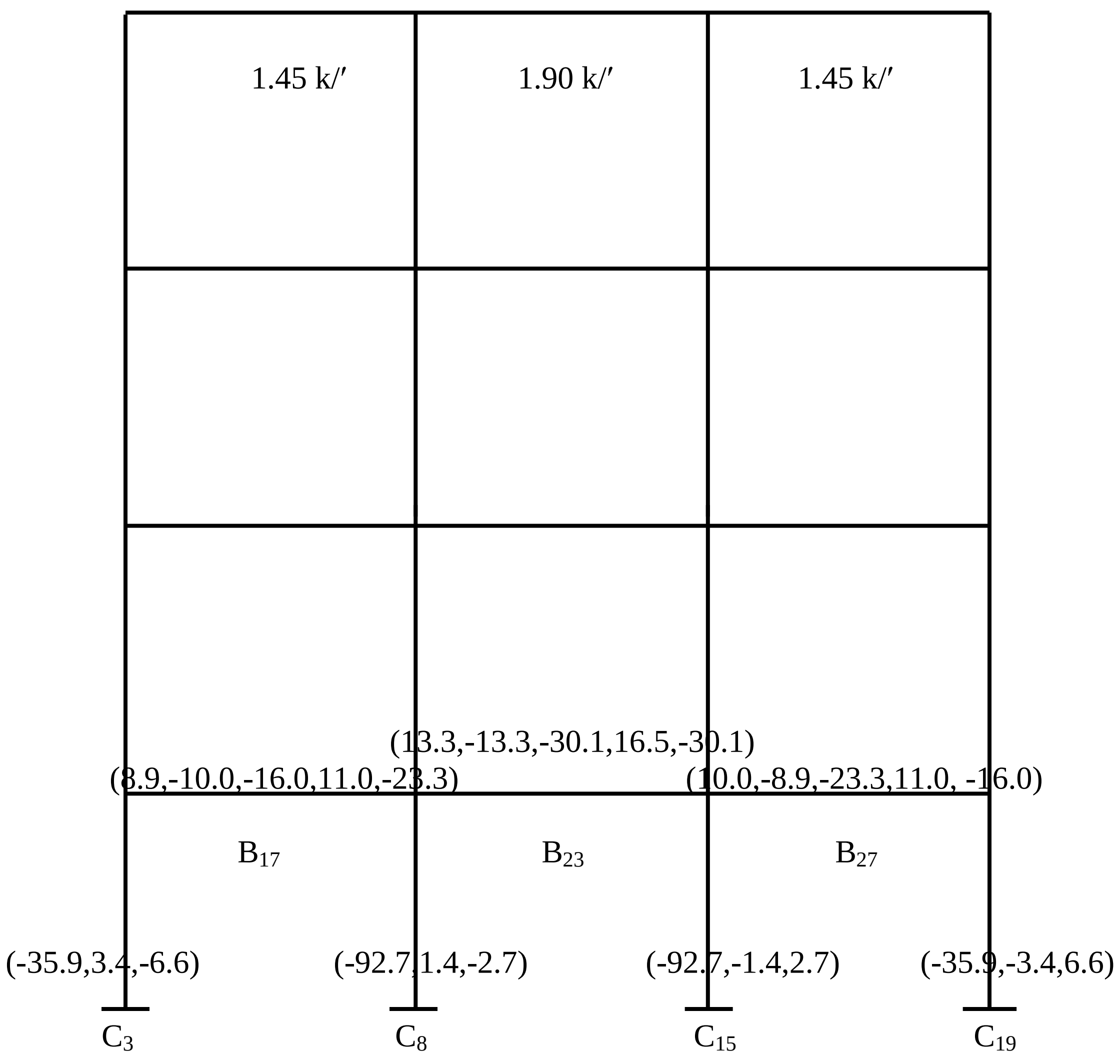
Slab-load on B_{17} and B_{27} = [13/2×(13)/2+13/2×(13)/2]× 0.13 = 10.99 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 10.99/13 + 0.15 + 0.45 = 1.45 k/′

Slab-load on B_{23} = [14/2×(14)/2]× 0.13 = 6.37^k

∴ Equivalent UDL (+ Self Wt., EW, S_9) ≅ 6.37/14 +0.15 + 0.90 + 3×0.13 = 1.90 k/′

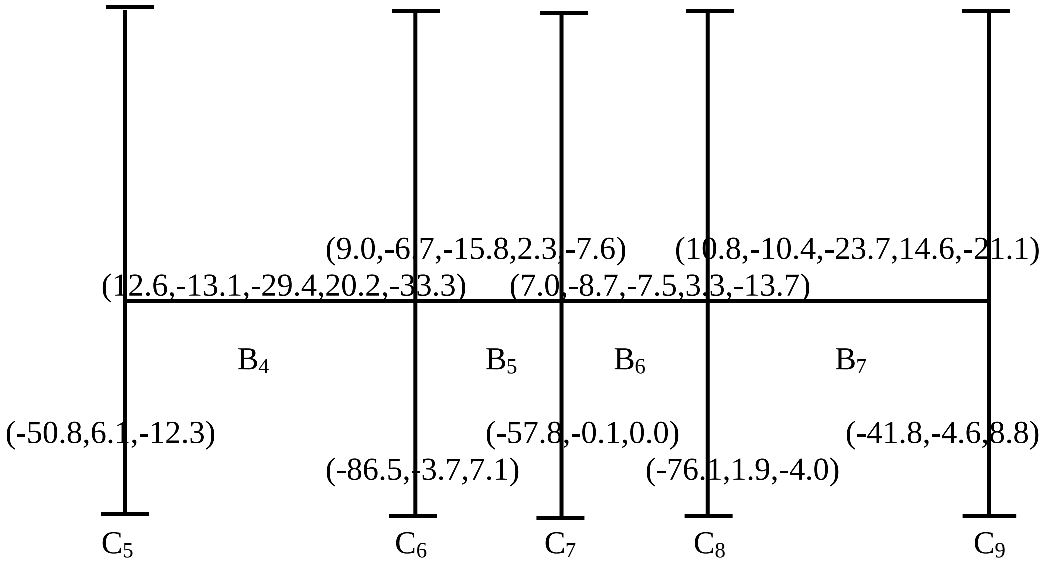
[Here, the EW is considered because the exterior beam B_{24} is more critical. It has the same slab load as B_{23} in addition to self-weight and EW]



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (4) from Vertical Load Analysis

Frame [$B_{4-5-6-7}$]:

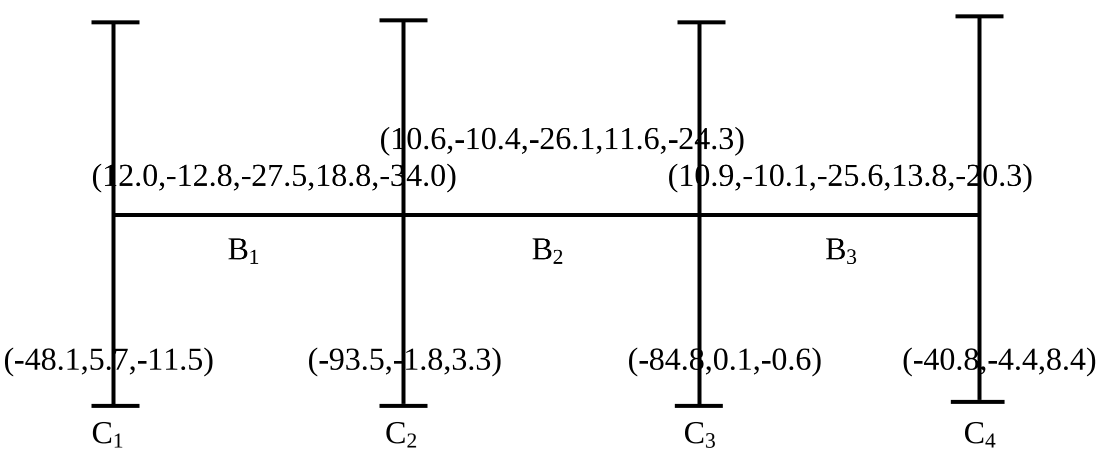
Similar to Frame (1) [$B_{8-9-10-11}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{4-5-6-7}$] from Vertical Load Analysis

Frame [B_{1-2-3}]:

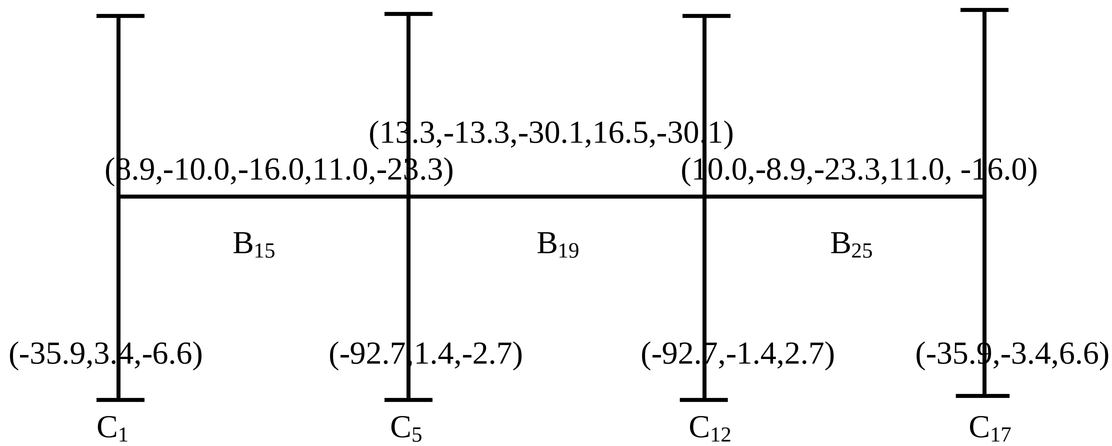
Similar to Frame (2) [$B_{12-13-14}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [B_{1-2-3}] from Vertical Load Analysis

Frame [$B_{15-19-25}$]:

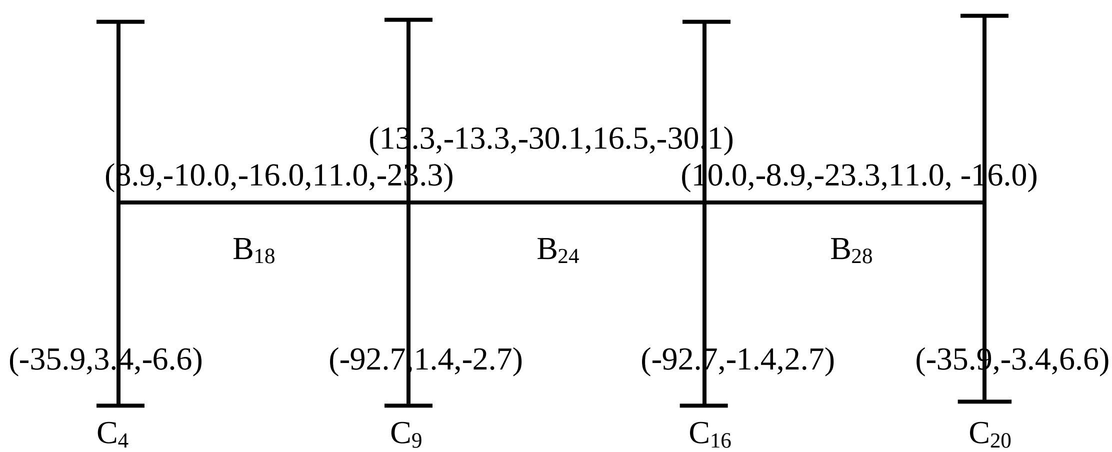
Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{15-19-25}$] from Vertical Load Analysis

Frame [$B_{18-24-28}$]:

Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{18-24-28}$] from Vertical Load Analysis

3. Lateral Load Analysis of Beams and Columns

Seismic Coefficients: $Z = 0.15$, $I = 1.0$, $S = 1.0$, $R = 5.0$

For RCC structures, $T = 0.073\times(40/3.28)^{3/4} = 0.476$ sec, which is <0.7 sec $\Rightarrow V_t = 0$.

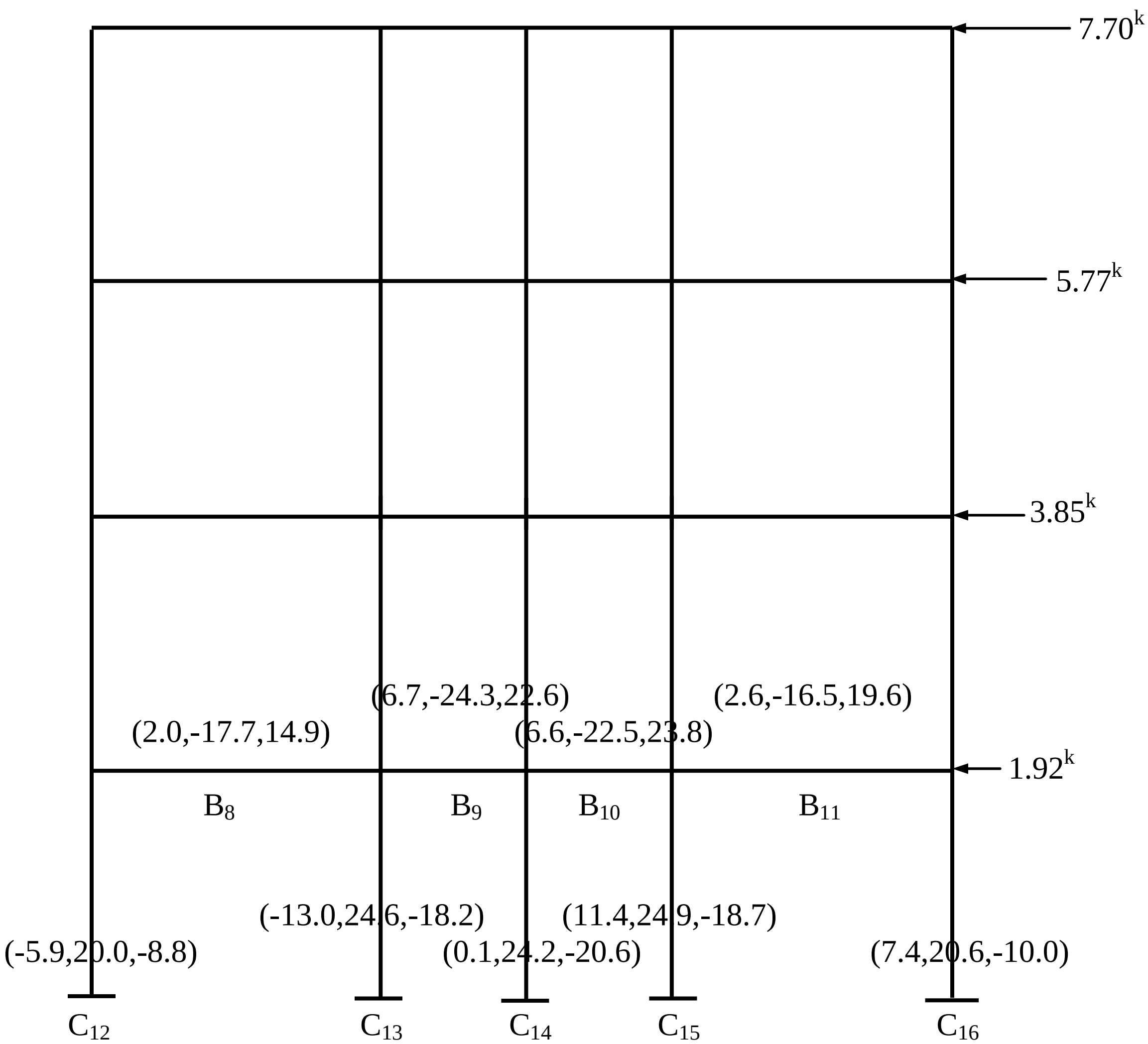
$$C = 1.25\ S/T^{2/3} = 2.05 \leq 2.75 \qquad [h = 40' = \text{Building Height in ft}]$$

$\therefore$ Base Shear, $V = (ZIC/R)\ W = 0.15\times1.0\times2.05/5.0\ W = 0.0615\ W$

$\therefore$ For equally loaded stories, $F_i = (h_i/\sum h_i)V \Rightarrow F_1 = 0.1V$, $F_2 = 0.2V$, $F_3 = 0.3V$, $F_4 = 0.4V$

Frame (1) [$B_{8\text{-}9\text{-}10\text{-}11}$]:

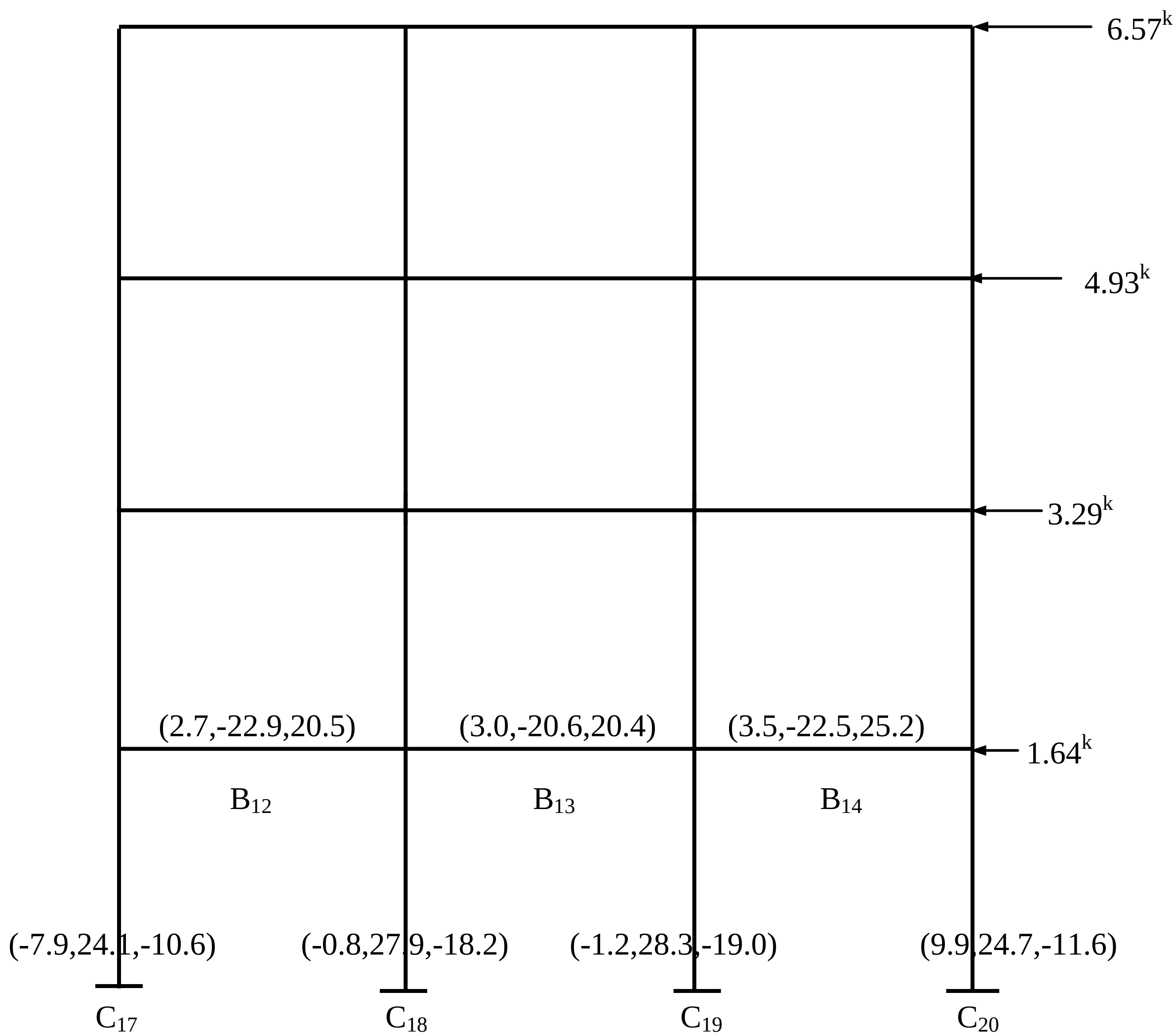
$W = 4\times(1.61\times16 + 2.24\times7 + 2.24\times7 + 1.51\times14) = 313.04^k \Rightarrow V = 0.0615W = 19.25^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (1) from Lateral Load Analysis

Frame (2) [$B_{12\text{-}13\text{-}14}$]:

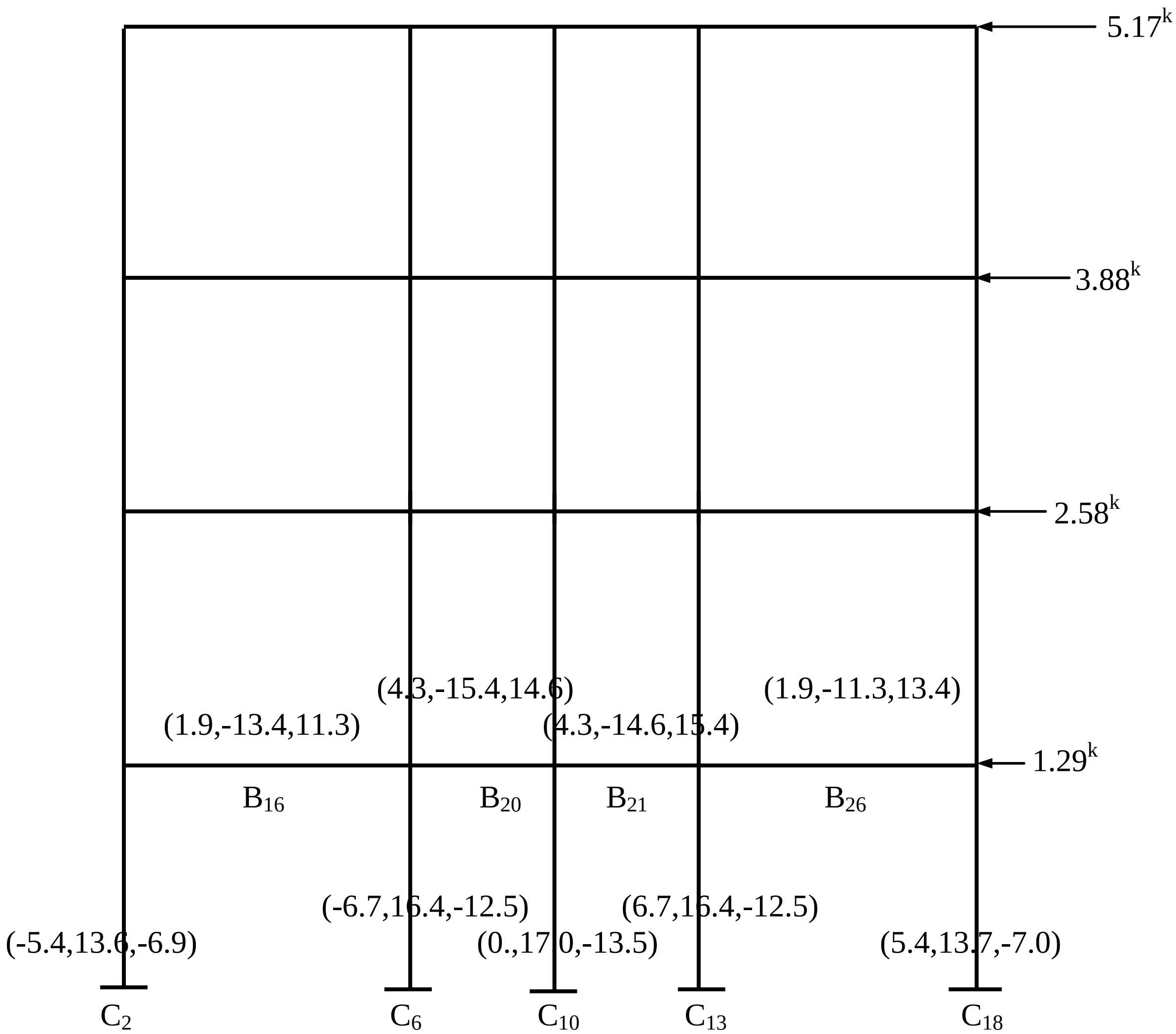
$W = 4\times(1.55\times16 + 1.50\times14 + 1.50\times14) = 267.20^k \Rightarrow V = 0.0615W = 16.43^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (2) from Lateral Load Analysis

Frame (3) [$B_{16-20-21-26}$]:

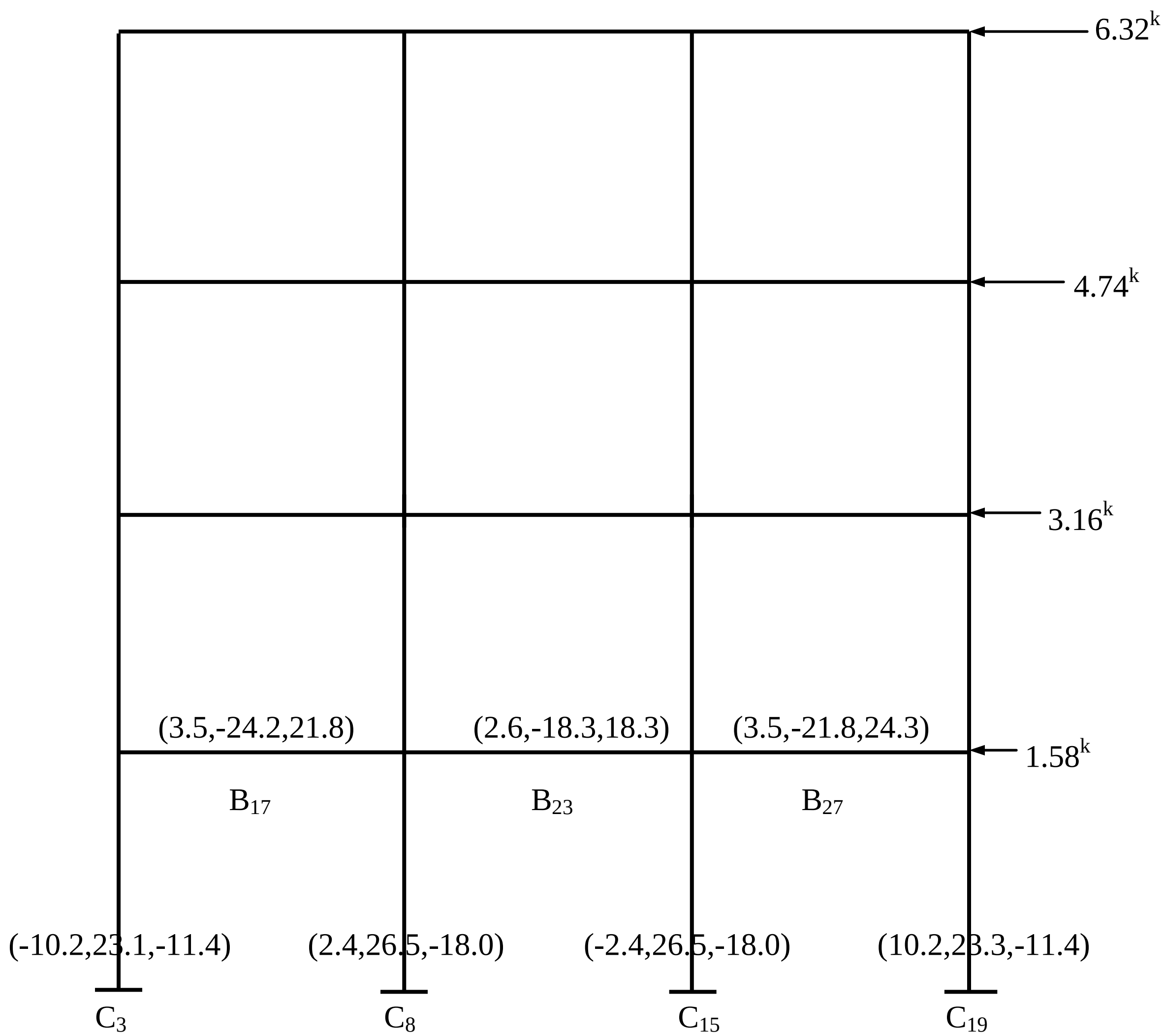
$W = 4\times(1.45\times13 + 1.06\times7 + 1.06\times7 + 1.45\times13) = 210.16^k \Rightarrow V = 0.0615W = 12.92^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (3) from Lateral Load Analysis

Frame (4) [$B_{17-23-27}$]:

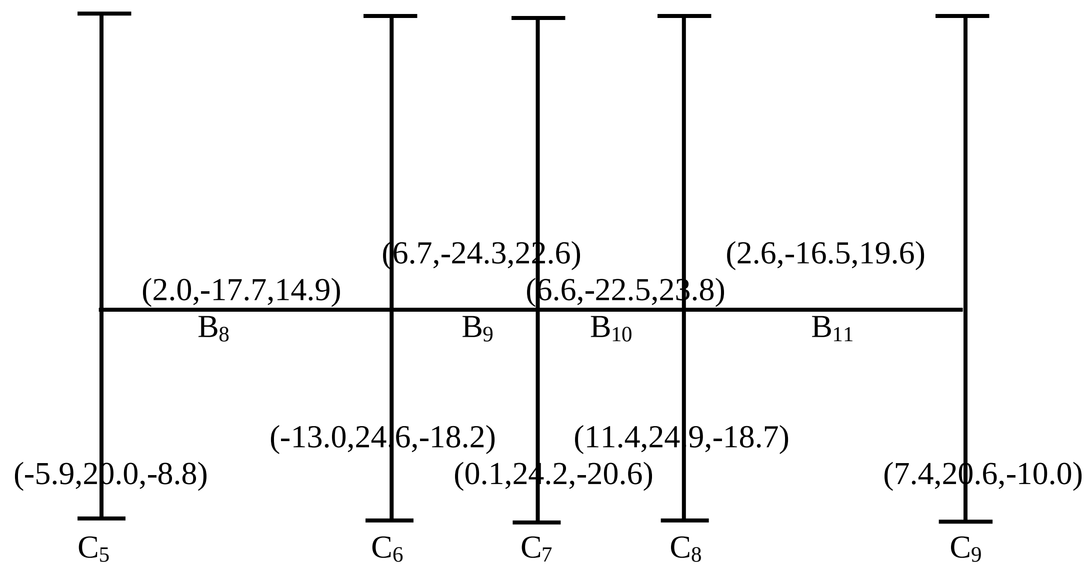
$W = 4\times(1.45\times13 + 1.90\times14 + 1.45\times13) = 257.20^k \Rightarrow V = 0.0615W = 15.81^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (4) from Lateral Load Analysis

Frame [$B_{4\text{-}5\text{-}6\text{-}7}$]:

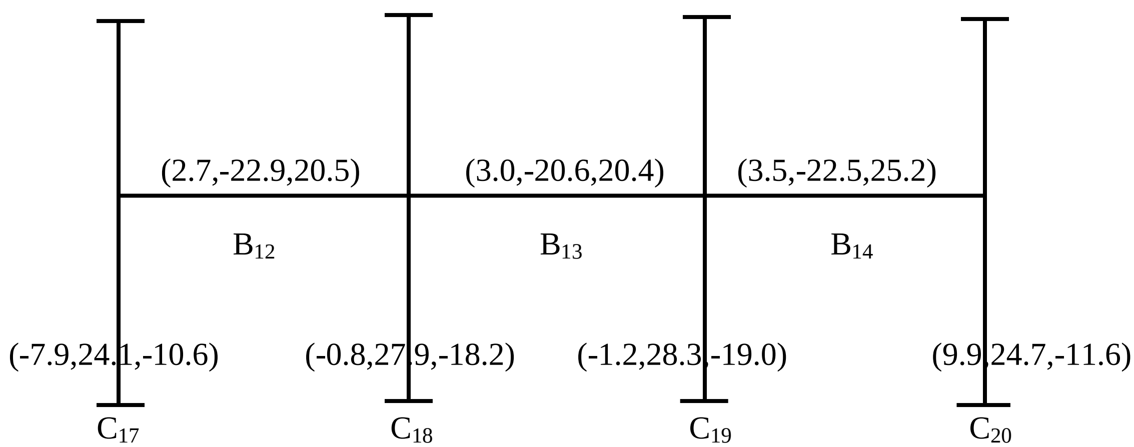
Similar to Frame (1) [$B_{8\text{-}9\text{-}10\text{-}11}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{4\text{-}5\text{-}6\text{-}7}$] from Lateral Load Analysis

Frame [$B_{1\text{-}2\text{-}3}$]:

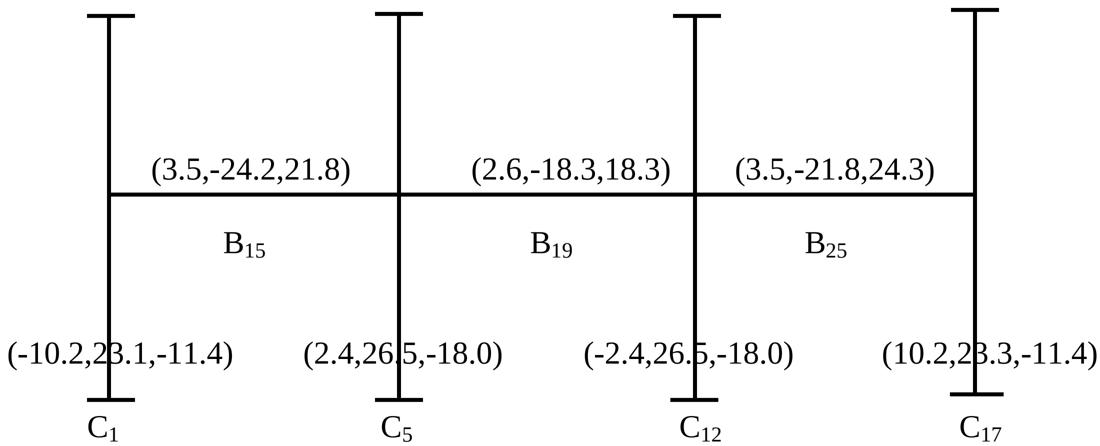
Similar to Frame (2) [$B_{12\text{-}13\text{-}14}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{1\text{-}2\text{-}3}$] from Lateral Load Analysis

Frame [$B_{15-19-25}$]:

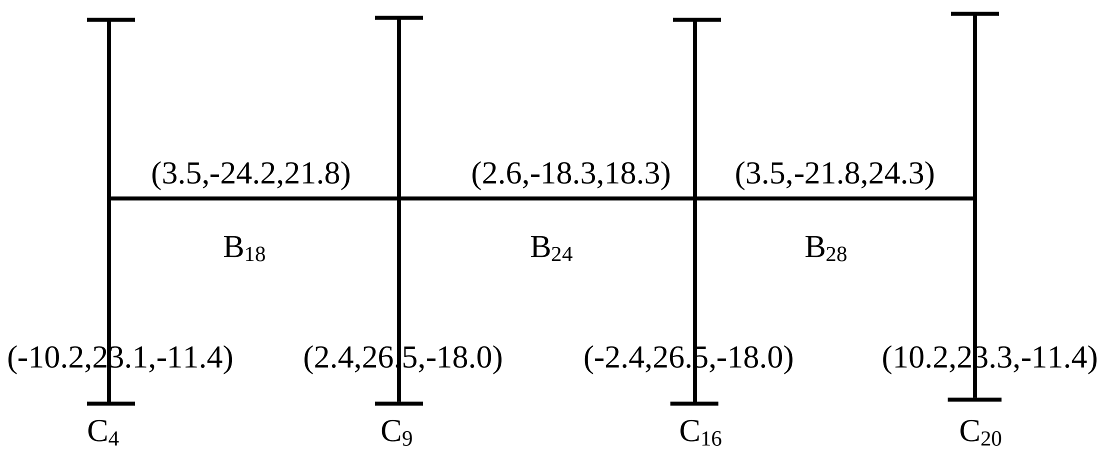
Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{15-19-25}$] from Lateral Load Analysis

Frame [$B_{18-24-28}$]:

Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{18-24-28}$] from Lateral Load Analysis

4. Combination of Vertical and Lateral Loads

The Design Force (i.e., AF, SF or BM) will be the maximum between the following two combinations

(i) Vertical Force = DL+LL

(ii) Combined Vertical and Lateral Force = 0.75 (DL+LL+EQ); i.e., 0.75 times the combined force from Vertical and Lateral Load Analysis.

The design Shear Forces and Bending Moments for various beams are calculated below using the two options mentioned above.

4.1 Load Combination for Beams

Frame (1) [$B_{4-5-6-7}$] and [$B_{8-9-10-11}$]:

Beams	SF_1(V)	SF_1(L)	SF_1(D)	SF_2(V)	SF_2(L)	SF_2(D)
B_4, B_8	12.6	±2.0	12.6	-13.1	±2.0	-13.1
B_5, B_9	9.0	±6.7	11.8	-6.7	±6.7	-10.1
B_6, B_{10}	7.0	±6.6	10.2	-8.7	±6.6	-11.5
B_7, B_{11}	10.8	±2.0	10.8	-10.4	±2.0	-10.4

Beams	BM_1(V)	BM_1(L)	BM_1(D)	BM_0(V=D)	BM_2(V)	BM_2(L)	BM_2(D)
B_4, B_8	-29.4	±17.7	-35.3	20.2	-33.3	±14.9	-36.2
B_5, B_9	-15.8	±24.3	6.4, -30.1	2.3	-7.6	±22.6	11.3, -22.7
B_6, B_{10}	-7.5	±22.5	11.3, -22.5	3.3	-13.7	±23.8	7.6, -28.1
B_7, B_{11}	-23.7	±16.5	-30.2	14.6	-21.1	±19.6	-30.5

Frame (2) [B_{1-2-3}] and [$B_{12-13-14}$]:

Beams	SF_1(V)	SF_1(L)	SF_1(D)	SF_2(V)	SF_2(L)	SF_2(D)
B_1, B_{12}	12.0	±2.3	12.0	-12.8	±2.3	-12.8
B_2, B_{13}	10.6	±3.0	10.6	-10.4	±3.0	-10.4
B_3, B_{14}	10.9	±3.5	10.9	-10.1	±3.5	-10.2

Beams	BM_1(V)	BM_1(L)	BM_1(D)	BM_0(V=D)	BM_2(V)	BM_2(L)	BM_2(D)
B_1, B_{12}	-27.5	±22.9	-37.8	18.8	-34.0	±20.5	-40.9
B_2, B_{13}	-26.1	±20.6	-35.0	11.6	-24.3	±20.4	-33.5
B_3, B_{14}	-25.6	±22.5	-36.1	13.8	-20.3	±25.2	-34.1

Frame (3) [$B_{16\text{-}20\text{-}21\text{-}26}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_{16}	9.3	±1.9	9.3	-9.6	±1.9	-9.6
B_{20}	4.3	±4.3	6.5	-3.1	±4.3	-5.6
B_{21}	3.1	±4.3	5.6	-4.3	±4.3	-6.5
B_{26}	9.6	±1.9	9.6	-9.3	±1.9	-9.3

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_{16}	-20.5	±13.4	-23.0	12.4	-22.9	±11.3	-22.9
B_{20}	-9.5	±15.4	5.5, -17.6	0.7	-4.2	±14.6	8.2, -13.7
B_{21}	-4.2	±14.6	8.2, -13.7	0.7	-9.5	±15.4	5.5, -17.6
B_{26}	-22.9	±11.3	-22.9	12.4	-20.5	±13.4	-23.0

Frame (4) [$B_{15\text{-}19\text{-}25}$], [$B_{17\text{-}23\text{-}27}$] and [$B_{18\text{-}24\text{-}28}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_{15}, B_{17}, B_{18}	8.9	±3.5	9.3	-10.0	±3.5	-10.1
B_{19}, B_{23}, B_{24}	13.3	±2.6	13.3	-13.3	±2.6	-13.3
B_{25}, B_{27}, B_{28}	10.0	±3.5	10.1	-8.9	±3.5	-9.3

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_{15}, B_{17}, B_{18}	-16.0	±24.2	6.2, -30.2	11.0	-23.3	±21.8	-33.8
B_{19}, B_{23}, B_{24}	-30.1	±18.3	-36.3	16.5	-30.1	±18.3	-36.3
B_{25}, B_{27}, B_{28}	-23.3	±21.8	-33.8	11.0	-16.0	±24.2	6.2, -30.2

Other Beams:

1. Beam B_{22} -

Approximately designed as a simply supported beam under similar load as B_{20}.

$\therefore$ Maximum SF $\cong 1.06\times7/2 = 3.71$ k

and Maximum positive BM $\cong 1.06\times7^2/8 = 6.49$ k′

2. Edge Beam for S_{10} -

Uniformly distributed load on S_{10} = 0.109 ksf

Uniformly distributed load on Edge Beam = $0.109\times5' = 0.55$ k/′

$\therefore$ Clear Span = 13′ $\Rightarrow V_{max} \cong 0.55\times(13)/2 = 3.6$ k; $M^{\pm} \cong 0.55\times(13)^2/10 = 9.3$ k′

4.2 Load Combination for Columns

The column forces are shown below as [AF (k), BM_{1y}, BM_{1x} (k′)]

Columns	Frame	(V)	(L_x)	0.75(V+L_x)	0.75(V-L_x)	(L_y)	0.75(V+L_y)	0.75(V-L_y)
C_1, C_{17}	2, 4	-84.0, 5.7, 3.4	-7.9, 24.1, 0	-68.9, 22.4, 2.6	-57.1, -13.8, 2.6	-10.2, 0, 23.1	-70.7, 4.3, 19.9	-55.4, 4.3, -14.8
C_2, C_{18}	2, 3	-130.8, -1.8, 3.6	-0.8, 27.9, 0	-98.7, 19.6, 2.7	-97.5, -22.3, 2.7	-5.4, 0, 13.6	-102.2, -1.4, 12.9	-94.1, -1.4, -7.5
C_3, C_{19}	2, 4	-120.7, 0.1, 3.4	-1.2, 28.3, 0	-91.4, 21.6, 2.6	-89.6, -21.2, 2.6	-10.2, 0, 23.1	-98.2, 0.1, 19.9	-82.9, 0.1, -14.8
C_4, C_{20}	2, 4	-76.7, -4.4, 3.4	9.9, 24.7, 0	-50.1, 15.2, 2.6	-65.0, -21.8, 2.6	-10.2, 0, 23.1	-65.2, -3.3, 19.9	-49.9, -3.3, -14.8
C_5, C_{12}	1, 4	-143.5, 6.1, 1.4	-5.9, 20.0, 0	-112.1, 19.6, 1.1	-103.2, -10.4, 1.1	2.4, 0, 26.5	-105.8, 4.6, 20.9	-109.4, 4.6, -18.8
C_6, C_{13}	1, 3	-140.2, -3.7, -2.2	-13.0, 24.6, 0	-114.9, 15.7, -1.7	-95.4, -21.2, -1.7	-6.7, 0, 16.4	-110.2, -2.8, 10.7	-100.1, -2.8, -14.0
C_7, C_{14}	1	-57.8, -0.1, 0	0.1, 24.2, 0	-43.3, 18.1, 0	-43.4, -18.1, 0	0., 0, 0.	-43.4, -0.1, 0	-43.4, -0.1, 0
C_8, C_{15}	1, 4	-168.8, 1.9, -2.7	11.4, 24.9, 0	-118.1, 20.1, 1.1	-135.2, -17.3, 1.1	2.4, 0, 26.5	-124.8, 1.4, 20.9	-128.4, 1.4, -18.8
C_9, C_{16}	1, 4	-134.5, -4.6, -2.7	7.4, 20.6, 0	-95.3, 12.0, -2.0	-106.4, -18.9, -2.0	1.7, 0, 18.9	-99.6, -3.5, 12.2	-102.2, -3.5, -16.2
C_{10}	3	-28.2, 0, 0.0	0, 0, 0	-21.2, 0, 0.0	-21.2, 0, 0.0	0., 0, 17.0	-21.2, 0.0, 12.8	-21.2, 0.0, -12.8

Besides, the design force on C_{11} is assumed to be 3.71^k; i.e., the SF at support of B_{22}.

In this work, only one size and reinforcements will be chosen for all the columns. For this purpose, the columns (C_8, C_{15}) are chosen as the model because they provide the most critical design conditions.

The designed column should therefore satisfy the following design conditions,

(1) Compressive Force = 168.8^k, Bending Moments BM_{1x} = 2.7 k′, BM_{1y} = 1.9 k′.

(2) Compressive Force = 118.1^k, Bending Moments BM_{1x} = 1.1 k′, BM_{1y} = 20.1 k′.

(3) Compressive Force = 135.2^k, Bending Moments BM_{1x} = 1.1 k′, BM_{1y} = 17.3 k′.

(4) Compressive Force = 124.8^k, Bending Moments BM_{1x} = 20.9 k′, BM_{1y} = 1.4 k′.

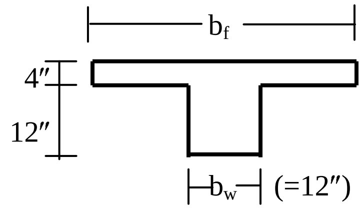
(5) Compressive Force = 128.4^k, Bending Moments BM_{1x} = 18.8 k′, BM_{1y} = 1.4 k′.

5. Design of Beams

Flexural Design

As shown for Slab Design, $n = 9$, $k = 0.378$, $j = 0.874$ and $R = 0.223$ ksi

The section shown is chosen for all the beams.



$\therefore$ For 1 layer of rods, $d = 16 - 2.5 = 13.5''$, $d' = 2.5''$

$M_c = Rbd^2 = 0.223 \times 12 \times (13.5)^2/12 = 40.6$ k′

The M_{max} is 40.9 k′ (in B_1 and B_{12})

$\Rightarrow$ Almost all beams are Singly Reinforced.

In the only doubly reinforced beams, the extra moment (40.9 – 40.6 = 0.3 k′) is negligible and expected to be absorbed within the necessary reinforcements on the other side.

$\therefore A_s = M/f_s jd = M \times 12/(20 \times 0.874 \times 13.5) = M/19.67$

For T-beams (possible for positive moments),

$A_s = M/f_s(d - t/2) = M \times 12/(20 \times [13.5 - 2]) = M/19.17$

Shear Design

For Shear, $V_c = 1.1\sqrt{(f_c')}b_w d = 1.1\sqrt{(3000)} \times 12 \times 13.5/1000 = 9.8^k$

$V_{c1} = 3\sqrt{(f_c')}b_w d = 26.6^k$, $V_{c2} = 5\sqrt{(f_c')}\, b_w d = 44.4^k$

The Maximum Design Shear Force here [for B_{19}, B_{23}, B_{27} in Frame (4)] is

$= 13.3 - 1.90 \times (12/2 + 13.5)/12 = 10.2^k$, which is $> V_c$, but $< V_{c1}$ and V_{c2}.

$\therefore S_{max} = d/2 = 6.75''$, $12''$ or $A_v/(0.0015 b_w) = 0.22/(0.0015 \times 12) = 12.2'' \Rightarrow S_{max} = 6.75''$

Spacing of #3 Stirrups, $S = A_v f_v d/(V - V_c) = 0.22 \times 20 \times 13.5/(V - 9.8) = 59.4/(V - 9.8)$

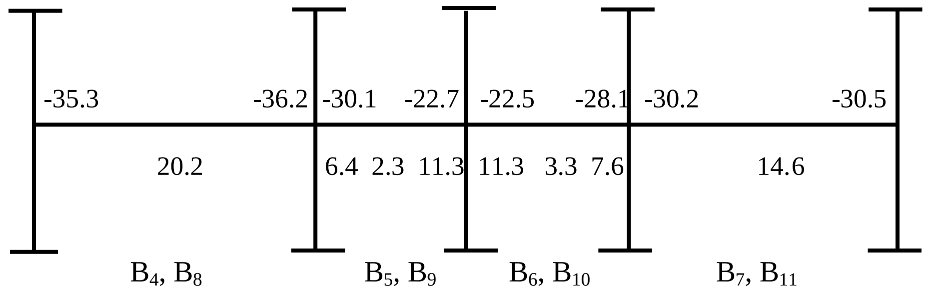
$= 143.5''$, when $V = 10.2^k$

$\therefore$ The design is governed by $S_{max} = 6.75''$

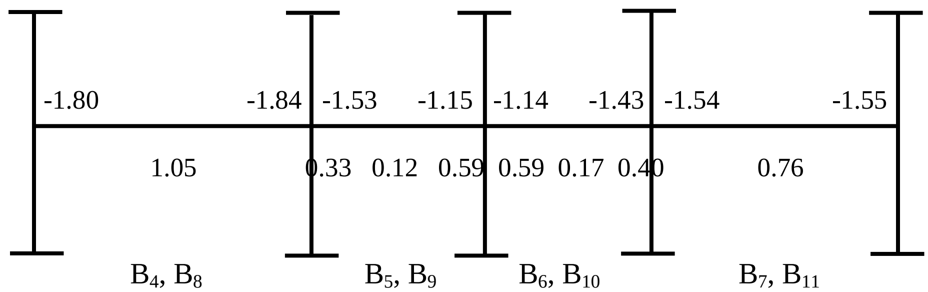
The rest of the design concentrates mainly on flexural reinforcements.

Frame (1) [$B_{4\text{-}5\text{-}6\text{-}7}$] and [$B_{8\text{-}9\text{-}10\text{-}11}$]:

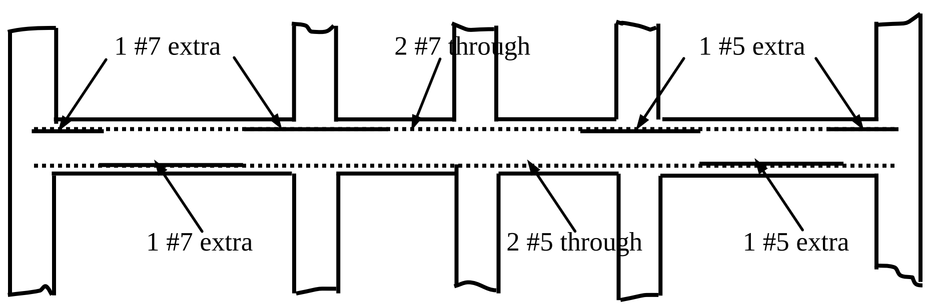
The design moments (k′) are



The flexural reinforcements (in^2) are

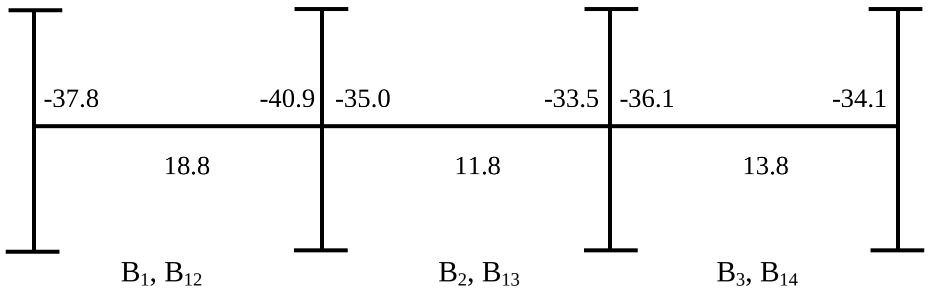


The reinforcements are arranged as follows

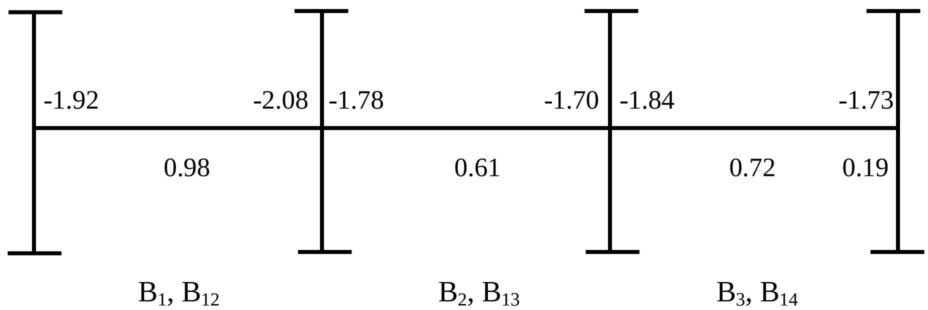


Frame (2) [$B_{1\text{-}2\text{-}3}$] and [$B_{12\text{-}13\text{-}14}$]:

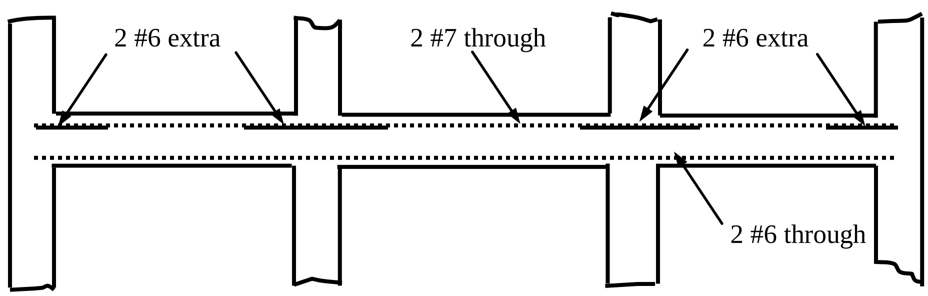
The design moments (k′) are



The flexural reinforcements (in^2) are

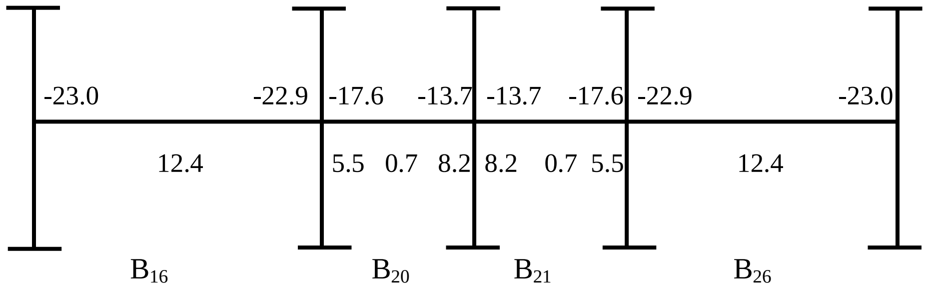


The reinforcements are arranged as follows

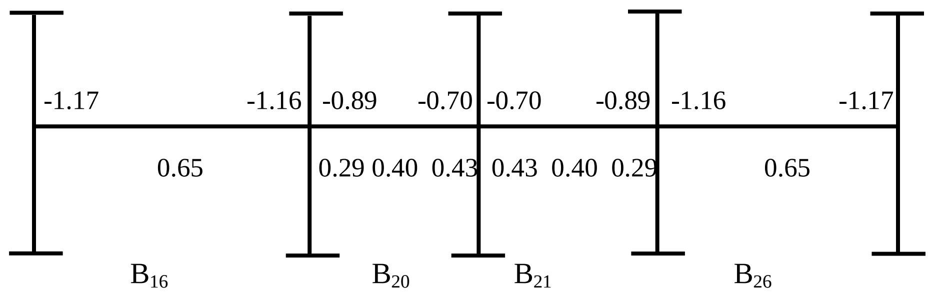


Frame (3) [$B_{16-20-21-26}$]:

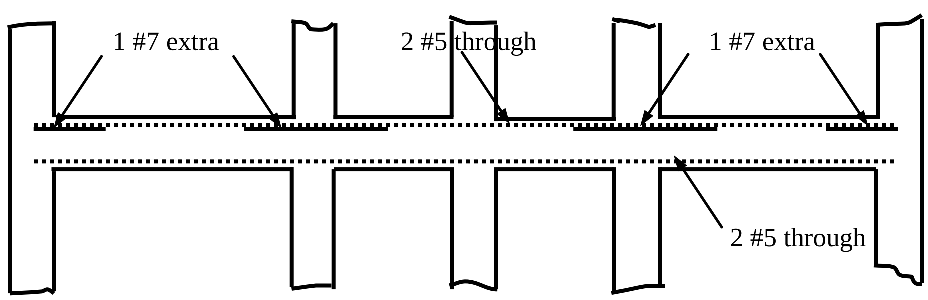
The design moments (k′) are



The flexural reinforcements (in^2) are

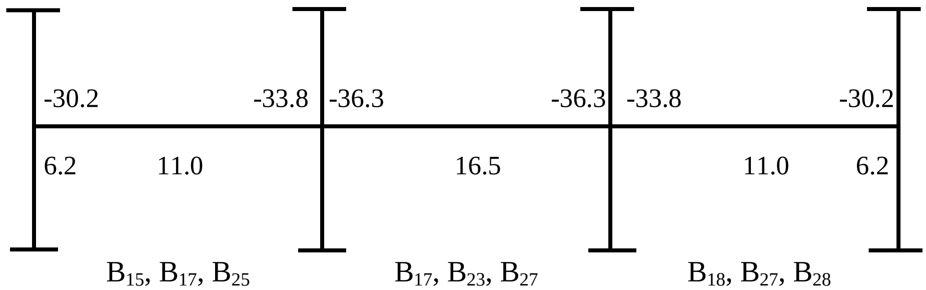


The reinforcements are arranged as follows

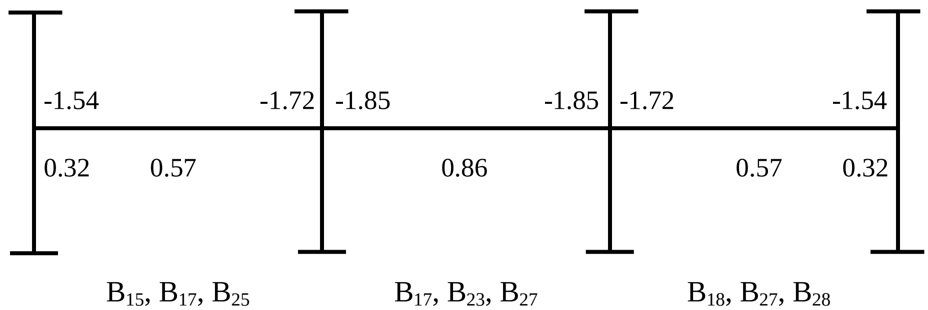


Frame (4) [$B_{15-17-18}$], [$B_{17-23-27}$] and [$B_{25-27-28}$]:

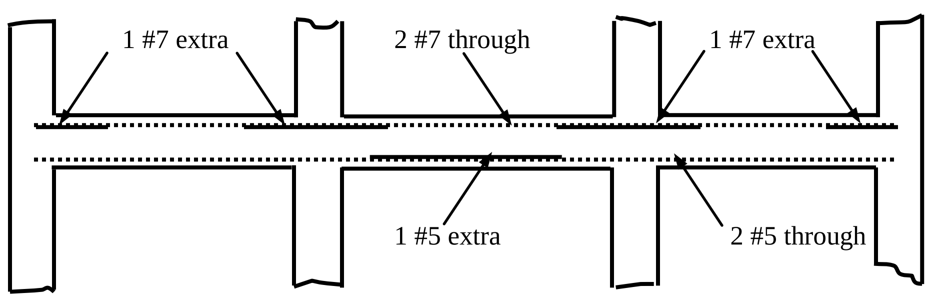
The design moments (k′) are



The flexural reinforcements (in^2) are



The reinforcements are arranged as follows



6. Design of Columns

The designed column should satisfy the five design conditions mentioned before (in the load combination for columns).

(1) Compressive Force = 168.8^k, Bending Moments BM_{1x} = 2.7 k′, BM_{1y} = 1.9 k′

(2) Compressive Force = 118.1^k, Bending Moments BM_{1x} = 1.1 k′, BM_{1y} = 20.1 k′

(3) Compressive Force = 135.2^k, Bending Moments BM_{1x} = 1.1 k′, BM_{1y} = 17.3 k′

(4) Compressive Force = 124.8^k, Bending Moments BM_{1x} = 20.9 k′, BM_{1y} = 1.4 k′

(5) Compressive Force = 128.4^k, Bending Moments BM_{1x} = 18.8 k′, BM_{1y} = 1.4 k′

To choose an assumed section, it will be designed only for an axial force slightly greater than the first of those conditions [since condition (1) has additional moments also]; and the design will be checked against the other conditions.

Assume the design axial load = 175^k

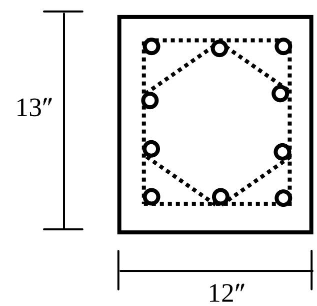
The following formula is valid for tied columns

$175 = 0.85\,(0.25f_c'A_g + A_sf_s) = 0.85\,A_g(0.25\,f_c' + p_gf_s)$

$\therefore p_g = 0.03 \Rightarrow 175 = 0.85\,A_g\,(0.25\times3 + 0.03\times20)$

$\Rightarrow A_g = 152.21\ in^2$

$\therefore$ Choose (12″×13″) section with 10 #6 bars and #3 ties @12″ c/c.



$\therefore p_g = 4.4/(12\times13) = 0.028$, $m = f_y/0.85f_c' = 40/2.55 = 15.69$

$\therefore$ The column has two axes with different dimensions and steel arrangements

For the strong axis, load eccentricity for balanced condition,

$e_{bx} = (0.67\,p_g\,m + 0.17)\,d = (0.67\times0.028\times15.69 + 0.17)\times(13\text{-}2.5) = 4.90'' = 0.41'$

For the weak axis, load eccentricity for balanced condition,

$e_{by} = (0.67\,p_g\,m + 0.17)\,d = (0.67\times0.028\times15.69 + 0.17)\times(12\text{-}2.5) = 4.43'' = 0.37'$

All the eccentricities involved here are much less than e_{bx} and e_{by}, so compression governs in each case.

$S_x = (1/c)\,[bh^3/12 + (2n-1)\sum A_{si}\,(h_i - h/2)^2]$

$\quad = (1/6.5)\,[12\times13^3/12 + (2\times9-1)\times2\times\{1.32\times(4.0)^2 + 0.88\times(1.0)^2\}] = 453.1\ in^3$

$S_y = (1/6.0)\,[13\times12^3/12 + (2\times9-1)\times2\times\{1.76\times(3.5)^2 + 0.44\times(0)^2\}] = 434.2\ in^3$

For condition (1)

$f_a = P/A_g = 168.8/156 = 1.08$ ksi, $F_a = 0.34(1+p_g m)f_c' = 0.34(1+0.028\times15.69)\times3 = 1.47$ ksi,

$f_{bx} = M_x/S_x$, $F_{bx} = 0.45f_c' = 1.35$ ksi, $f_{by} = M_y/S_y$, $F_{by} = 0.45f_c' = 1.35$ ksi

$\therefore f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} = 1.08/1.47 + (2.7\times12/453.1)/1.35 + (1.9\times12/434.2)/1.35$

$= 0.74 + 0.05 + 0.04 = 0.83 < 1$ (OK)

$\therefore$ Condition (1) is satisfied.

For condition (2)

$(118.1/156)/1.47 + (1.1\times12/453.1)/1.35 + (20.1\times12/434.2)/1.35$

$= 0.51 + 0.02 + 0.41 = 0.95 < 1$ (OK)

$\therefore$ Condition (2) is satisfied.

For condition (3)

$(135.2/156)/1.47 + (1.1\times12/453.1)/1.35 + (17.3\times12/434.2)/1.35$

$= 0.59 + 0.02 + 0.35 = 0.96 < 1$ (OK)

$\therefore$ Condition (3) is satisfied.

For condition (4)

$(124.8/156)/1.47 + (20.9\times12/453.1)/1.35 + (1.4\times12/434.2)/1.35$

$= 0.54 + 0.41 + 0.03 = 0.98 < 1$ (OK)

$\therefore$ Condition (4) is satisfied.

For condition (5)

$(128.4/156)/1.47 + (18.8\times12/453.1)/1.35 + (1.4\times12/434.2)/1.35$

$= 0.56 + 0.37 + 0.03 = 0.96 < 1$ (OK)

$\therefore$ Condition (5) is satisfied.

$\therefore$ The assumed section is chosen for all the columns.

7. Design of Footings

The following sample footings will be designed.

1. Individual Footing: A footing under C_5 will be designed for column load 143.5^k (plus footing weight).
2. Combined Footing: A footing under C_6, C_7 and C_8 will be designed for column loads 140.2^k, 57.8^k and 168.8^k (plus footing weights) combined.

The allowable bearing capacity of the soil is 2.0 ksf

[This can be determined from field tests like SPT, CPT or from lab test for unconfined compression strength. The formula for bearing capacity has factors for soil cohesion (N_c), foundation width (N_γ) and depth (N_q). These are functions of the angle of friction ϕ. With a factor of safety 3.0, N_c approximately equals to 6.0 and neglecting N_γ and N_q, the unconfined compression strength equals to the allowable bearing capacity].

<u>Individual Footing under C_5:</u>

Column load = $143.5^k \Rightarrow$ Footing load $\cong 143.5^k \times 1.1 \cong 157.9^k$

$\therefore$ Footing area $= 157.9\ ^k/2.0$ ksf $= 78.9$ ft$^2 \cong 9.00' \times 9.00'$

$\therefore$ Effective bearing pressure $= 143.5/(9.00)^2 = 1.77$ ksf

Column size $= 12''\times13''$, Effective depth of footing = d

$\therefore$ Punching Shear area $A_p = 2 \times (12 + d + 13 + d)\, d = 4 \times (12.5 + d)\, d$

Punching Shear strength $= 2\sqrt{(f_c')} = 2\sqrt{(3000)} = 110$ psi $= 0.110$ ksi

$\therefore 0.110\ \{4 \times (12.5 + d)\, d\} = 143.5 - 1.77 \times (12+d) \times (13+d)/(12)^2$

$\Rightarrow d^2 + 12.5\, d = 327.5 - (12+d) \times (13+d)/35.62 \Rightarrow d = 12.09''$

$\therefore$ Take footing thickness, $t = 16.5'' \Rightarrow d = 12.5''$

Flexural Shear strength $= 1.1\sqrt{(f_c')} = 1.1\sqrt{(3000)} = 60.2$ psi $= 0.0602$ ksi $= 8.68$ ksf

Maximum flexural shear force $= 1.77$ ksf $\times\{(9.00-12/12)/2-d\}' = 1.77\times(4.0-d)$ k/$'$

$\therefore 8.68\times1\times d = 1.77\times(4.0-d) \Rightarrow d = 1.77 \times 4.0/(8.68 + 1.77) = 0.68' = 8.14'' < 12.5''$, OK.

Total Maximum bending moment, $M = 1.77\times\{(9.00\text{-}12/12)/2\}^2/2 \times 9.00 = 127.56$ k$'$

$\therefore$ Depth required by M is $= \sqrt{(M/Rb)} = \sqrt{\{127.56/(0.223\times9.00)\}} = 7.97'' < 12.5''$, OK.

$\therefore A_s = M/f_s jd = 127.56\times12/(20\times0.874\times12.5) = 7.01$ in^2

Minimum reinforcement $= (0.2/f_y)bd = (0.2/40)\times9.00\times12\times12.5 = 6.75$ in$^2 < A_s$

$\therefore$ Provide 12 #7 bars in each direction.

<u>Combined Footing under C_6-C_7-C_8:</u>

Column loads are 140.2^k, 57.8^k and 168.8^k

$\therefore$ Resultant = 140.2 + 57.8 +168.8 = 366.8^k

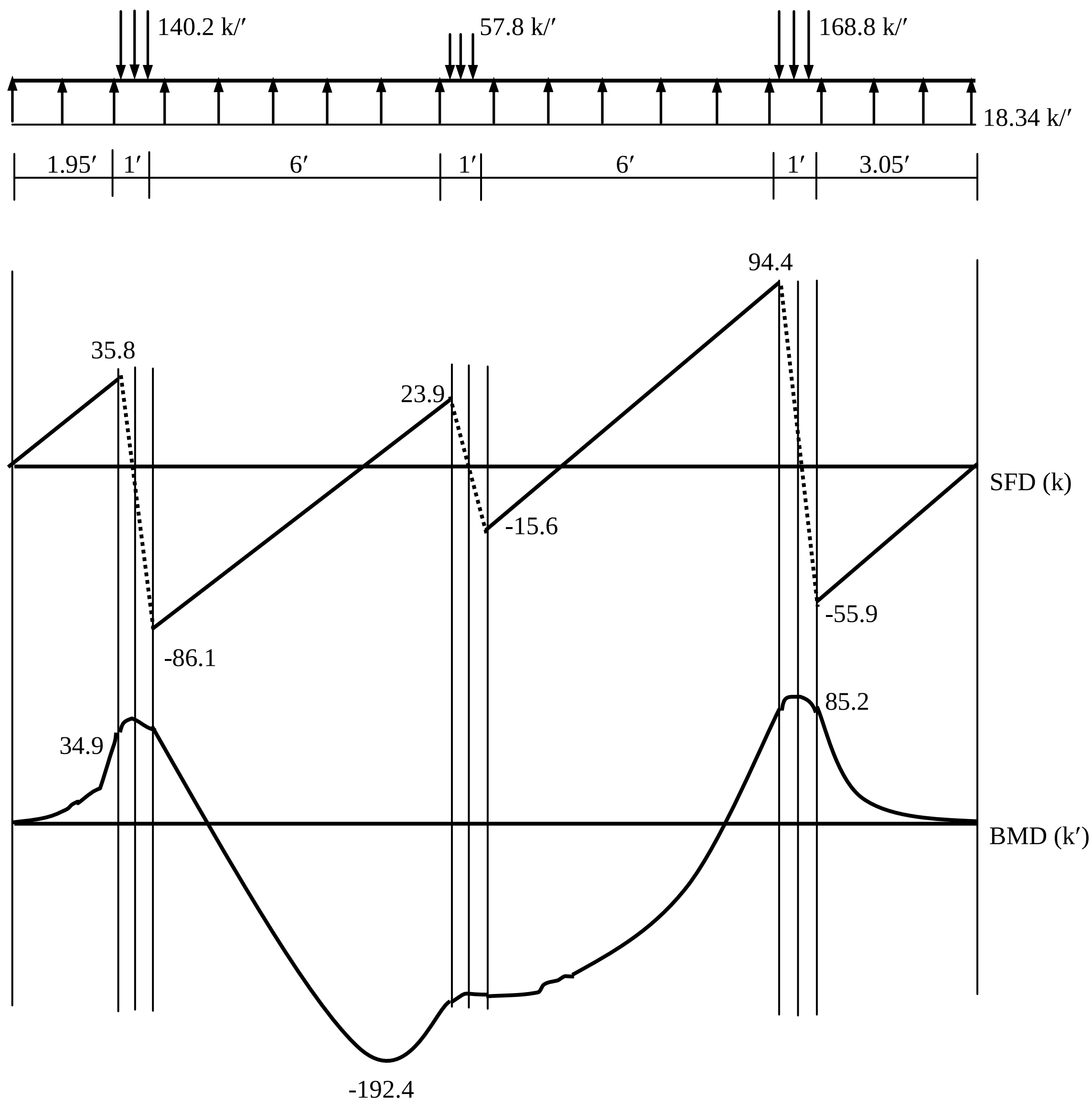
$\therefore$ Footing area = $366.8^k \times 1.1/2.0$ ksf = 201.74 ft$^2 \cong 20' \times 10'$

Distance of resultant from the first column

= (140.2 $\times$0 + 57.8 $\times$7 +168.8 $\times$14)/366.8 = 7.55′

$\therefore$ Effective bearing pressure = 366.8/(20$\times$10) = 1.834 ksf; i.e., 18.34 k/′ along the length.

$\therefore$ Considering the column loads to be uniformly distributed over the column width = 1′, the footing loads and corresponding SFD and BMD are shown below



Column size = 12″×13″, Effective depth of footing = d

$\therefore$ Punching Shear area $A_p = 2 \times (12 + d + 13 + d)\, d = 4\times(12.5+d)\, d$

Punching Shear strength = $2\sqrt{(f_c')}$ = 110 psi = 0.110 ksi

$\therefore 0.110\{4\times(12.5 + d)\, d\} = 168.8 - 1.834\times (12 + d)\times(13 + d)/(12)^2$

$\Rightarrow d^2 + 12.5\, d = 445.5 - (12+d)\times(13+d)/34.57 \Rightarrow d = 15.55''$

$\therefore$ Take footing thickness, t = 20″ $\Rightarrow$ d = 16″

Flexural Shear strength = $1.1\sqrt{(f_c')}$ = 60.2 psi = 0.0602 ksi = 8.68 ksf

Maximum shear force (according to SFD) = 94.4^k

$\therefore$ Maximum flexural shear force = 94.4 – 18.25d [d is in ft]

$\therefore 8.68 \times 10\times d = 94.4 - 18.25d \Rightarrow d = 0.899' = 10.78'' < 16''$, OK.

Maximum bending moment (according to BMD) = 192.4 k′

$\therefore$ Depth required by M is = $\sqrt{(M/Rb)} = \sqrt{\{192.4/(0.223\times10)\}} = 9.29'' < 16''$, OK.

$\therefore A_s^{(+)} = M/f_s jd = 192.4\times12/(20\times0.874\times16) = 192.4/23.3 = 8.26\ in^2$, at top

and $A_s^{(-)} = 85.2/23.3 = 3.66\ in^2$, at bottom

Minimum reinforcement = $(0.2/f_y)bd = (0.2/40) \times10\times12\times16 = 9.60\ in^2 > A_s$

$\therefore$ Provide 16 #7 bars at top and bottom.

Width of the transverse beams under columns = 12 + 16 = 28″

Load per unit length under C_8 = 168.8/10 = 16.88 k/′

$\therefore$ Maximum bending moment = $16.88\times [(10-13/12)/2]^2/2 = 167.76$ k′

$\therefore$ Depth required by M is = $\sqrt{(M/Rb)} = \sqrt{\{167.76\times12/(0.223\times28)\}} = 17.96'' > 16''$.

$\therefore$ Provide d = 18″, increase the thickness to 22″ and put transverse rods at bottom

$\therefore A_s^{(+)} = 167.76\times12/(20\times0.874\times18) = 6.40\ in^2$, at bottom [Note: d = 18″ here]

Similarly, $A_s^{(+)}$ under $C_6 = 6.40\times140.2/168.8 = 5.31\ in^2$

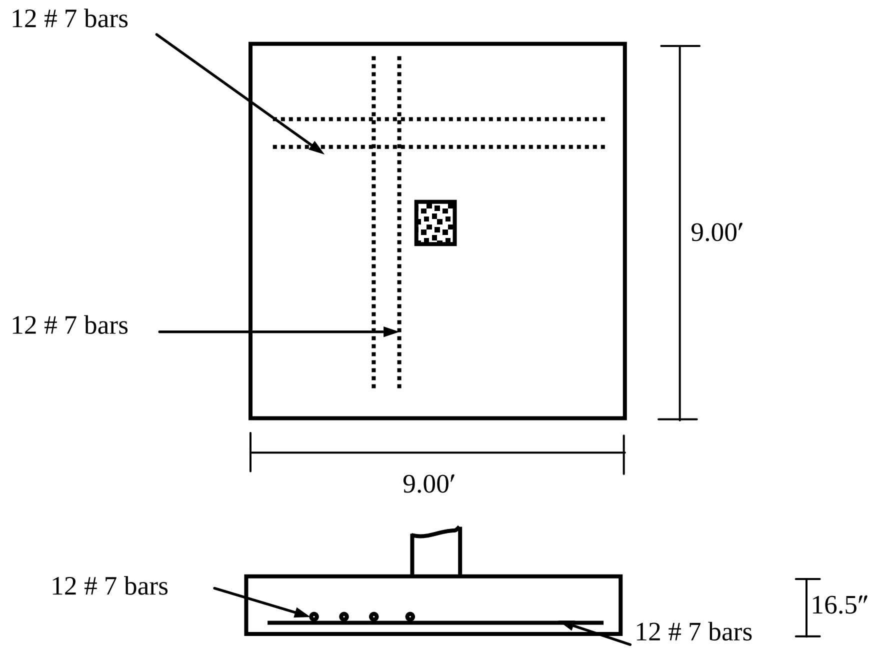
and $A_s^{(+)}$ under $C_7 = 6.40\times57.8/168.8 = 2.19\ in^2$

$\therefore A_{s(min)} = (0.2/f_y)bd = (0.2/40)\times28\times18 = 2.52\ in^2 < A_s^{(+)}$ for all columns except C_7

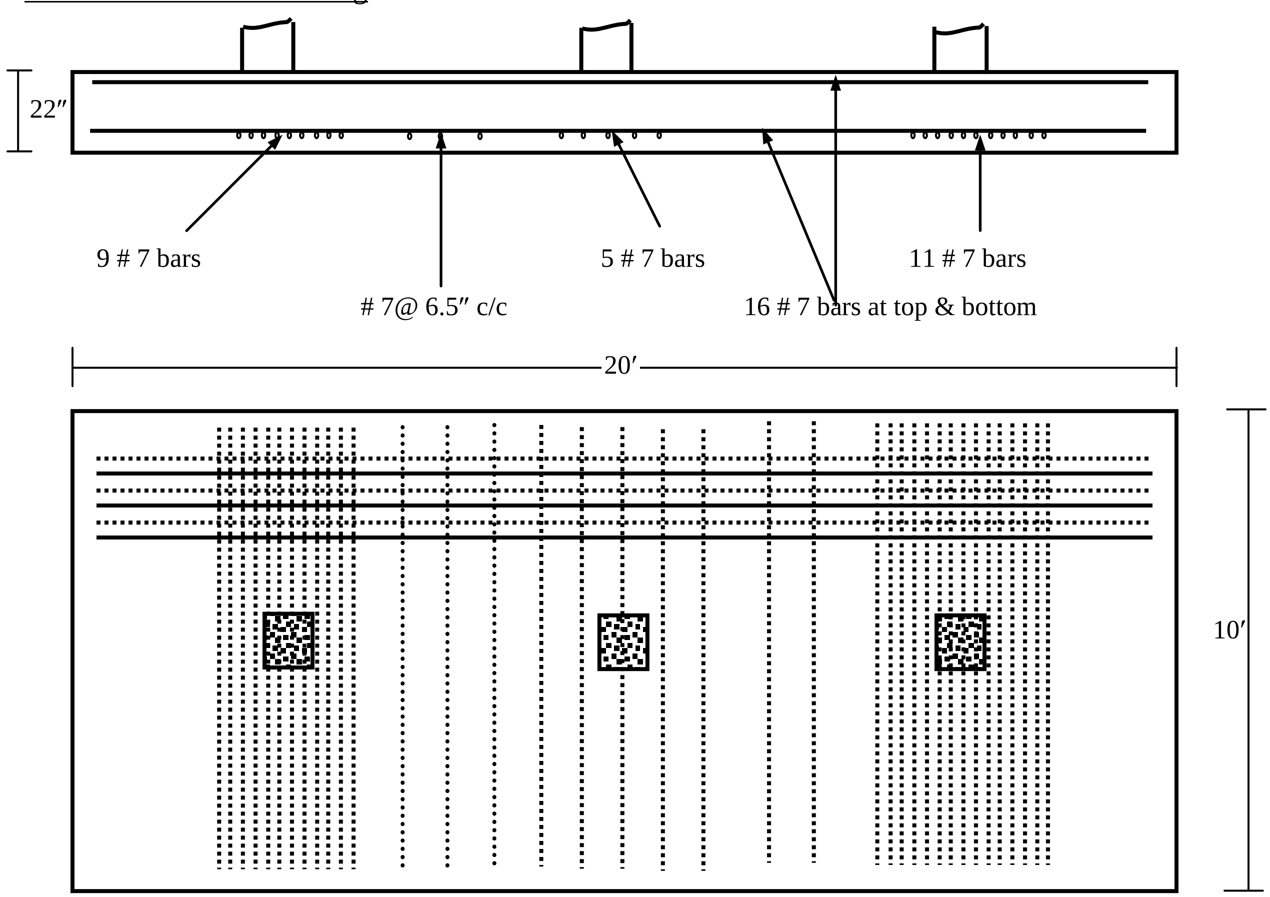
$\therefore$ Provide 9#7, 5#7 and 11#7 bars at bottom within the width of the transverse beams (28″) under each column (in placing 11 #7 bars in 28″, be careful about minimum spacing).

Elsewhere, provide $(0.2/f_y)bd = 1.08\ in^2/ft$ (i.e., #7 @ 6.5″ c/c).

Details of Individual Footing



Details of Combined Footing



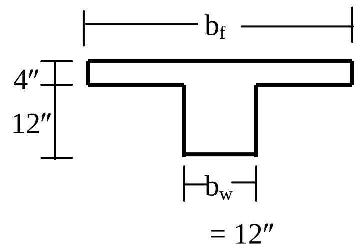
5. Design of Beams (USD)

Material Properties: $f'_c = 3$ ksi, $f_y = 40$ ksi, $\therefore f_c = 0.85f'_c = 2.55$ ksi

Flexural Design

$p_{max} = (0.75\ \alpha f'_c/f_y)\ [87/(87+f_y)] = 0.0277$, $R_u = \phi\ p_{max}\ f_y\ [1 - 0.59 p_{max}\ f_y/f'_c] = 0.781$ ksi

The section shown is chosen for all the beams.



$\therefore$ For 1 layer of rods, $d = 16 - 2.5 = 13.5''$, $d' = 2.5''$

$M_c = R_u bd^2 = 0.781 \times 12 \times (13.5)^2/12 = 142.3$ k′

The M_{max} is 61.4 k′ (in B_1 and B_{12})

$\Rightarrow$ All the beams are Singly Reinforced.

$A_s = (f_c/f_y)[1 - \sqrt{\{1 - 2M/(\phi f_c bd^2)\}}]bd$

$= (2.55/40) \times [1 - \sqrt{\{1 - 2M \times 12/(0.9 \times 2.55 \times 12 \times 13.5^2)\}}] \times 12 \times 13.5$

$= 10.33\ [1 - \sqrt{\{1 - M/209.1\}}]$

For T-beams (possible for positive moments), b_f is the minimum of

(i) $16t_f + b_w = 76''$, (ii) Simple Span/4 $\cong 0.6L \times 12/4 = 1.8L('')$, (iii) c/c

$\Rightarrow$ Since L varies between 7′~18′ and c/c between 16′~18′, it is conservative and simplified to assume $b_f = 12''$; i.e., calculations are similar to rectangular beam.

Shear Design

$V_c = 2\sqrt{(f_c')}b_w d = 2\sqrt{(3000)} \times 12 \times 13.5/1000 = 17.7^k$

$V_{c1} = 6\sqrt{(f_c')}b_w d = 53.2^k$, $V_{c2} = 10\sqrt{(f_c')}\ b_w d = 88.7^k$

The Maximum Design Shear Force here [for B_{19}, B_{23}, B_{27} in Frame (4)] is

$V_d = 20.3 - 2.85 \times (12/2+13.5)/12 = 15.3^k \Rightarrow V_n = V_d/\phi = 18.4^k$

$\therefore V_n > V_c$, but $< V_{c1}$ and V_{c2}.

$\therefore S_{max} = d/2 = 6.75''$, $24''$ or $A_v f_y/50b_w = 0.22 \times 40000/(50 \times 12) = 14.7'' \Rightarrow S_{max} = 6.75''$

Spacing of #3 Stirrups, $S = A_v f_y d/(V_n - V_c) = 0.22 \times 40 \times 13.5/(V_n - 17.7) = 118.8/(V_n - 17.7)$

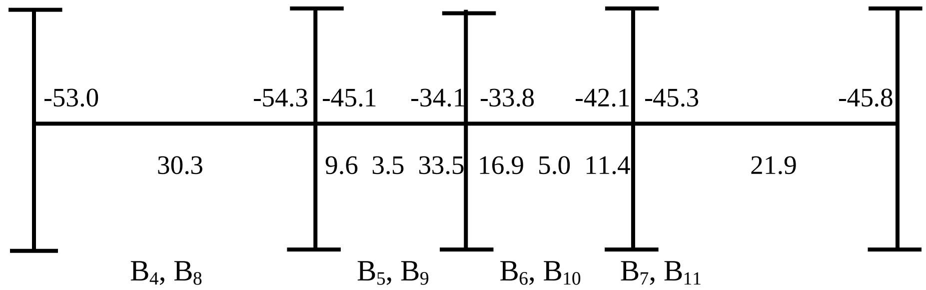
$= 169.7''$, when $V = 18.4^k$

$\therefore$ The design is governed by $S_{max} = 6.75''$

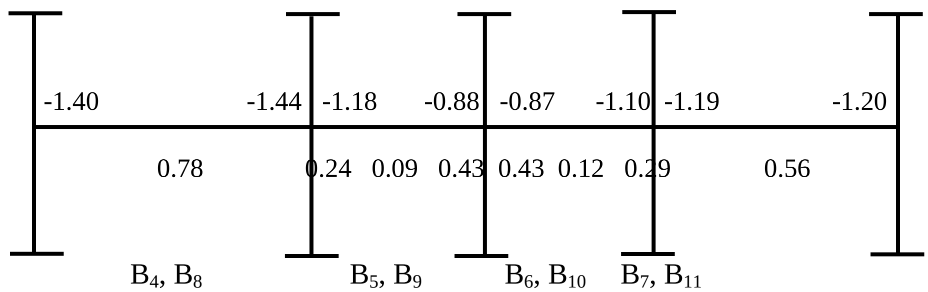
The rest of the design concentrates mainly on flexural reinforcements.

Frame (1) [$B_{4-5-6-7}$] and [$B_{8-9-10-11}$]:

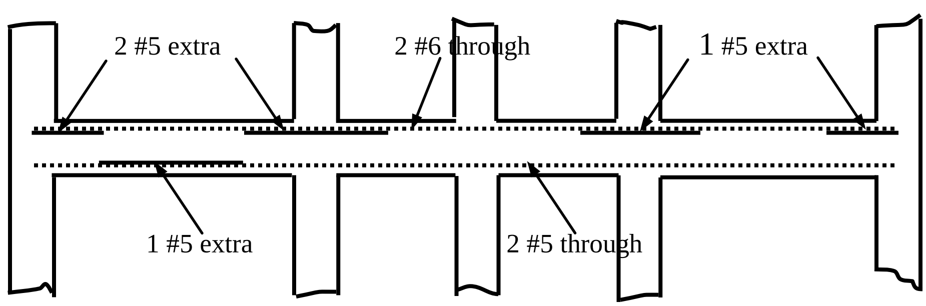
The design moments (k′) are



The flexural reinforcements (in^2) are



The reinforcements are arranged as follows



6. Design of Columns (USD)

The designed column should satisfy the five design conditions mentioned before (in the load combination for columns).

(1) Compressive Force = 253.2^k, Bending Moments $BM_{1x} = 4.1$ k′, $BM_{1y} = 2.8$ k′

(2) Compressive Force = 177.1^k, Bending Moments $BM_{1x} = 1.7$ k′, $BM_{1y} = 30.1$ k′

(3) Compressive Force = 202.8^k, Bending Moments $BM_{1x} = 1.7$ k′, $BM_{1y} = 25.9$ k′

(4) Compressive Force = 187.2^k, Bending Moments $BM_{1x} = 31.4$ k′, $BM_{1y} = 2.1$ k′

(5) Compressive Force = 192.6^k, Bending Moments $BM_{1x} = 28.2$ k′, $BM_{1y} = 2.1$ k′

To choose an assumed section, it will be designed only for an axial force slightly greater than the first of those conditions [since condition (1) has additional moments also]; and the design will be checked against the other conditions.

Assume the design axial load = 265^k

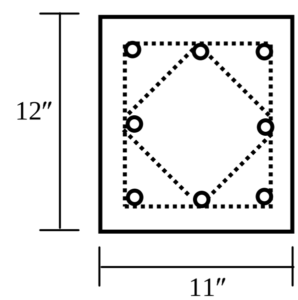
The following formula is valid for tied columns, using $f_c = 0.85\, f_c'$

$265 = 0.80\phi\ (0.85 f_c' A_c + A_s f_y) = 0.80\phi\ A_g\{f_c + p\,(f_y - f_c)\}$

$\therefore p = 0.03 \Rightarrow 265 = 0.80 \times 0.70\ A_g\ (0.85\times3 + 0.03\times37.45)$

$\Rightarrow A_g = 128.82$ in^2

$\therefore$ Choose (11″×12″) section with 8 #6 bars and #3 ties @11″ c/c.



$\therefore p = 3.52/(11\times12) = 0.027$, $\mu = f_y/0.85 f_c' = 40/2.55 = 15.69$

Use interaction diagram for rectangular columns with

$\gamma = 0.60$, and $p\mu = 0.027\times15.69 = 0.42$,

$\phi f_c' bh = 0.7\times3\times11\times12 = 277.2^k$, $\phi f_c' bh^2 = 3326.4$ k″ $= 277.2$ k′, $\phi f_c' hb^2 = 254.1$ k′

For condition (1)

$k = P/(\phi f_c' bh) = 253.2/277.2 = 0.91$

$k_0 = 1.18$

$ke_x/h = M_x/(\phi f_c' bh^2) = 4.1/277.2 = 0.015 \Rightarrow k_x = 1.15$

$ke_y/h = M_y/(\phi f_c' hb^2) = 2.8/254.1 = 0.011 \Rightarrow k_y = 1.16$

$\therefore$ Reciprocal method $\Rightarrow 1/k_{xy} = 1/k_x + 1/k_y - 1/k_0 = 1/1.15 + 1/1.16 - 1/1.18$

$\Rightarrow k_{xy} = 1.13$, which is $> k$ (= 0.91)   (OK)

$\therefore$ Condition (1) is satisfied.

For condition (2)

$k = P/(\phi f_c' bh) = 177.1/277.2 = 0.64$

$k_0 = 1.18$; $ke_x/h = 0.006 \Rightarrow k_x = 1.17$; $ke_y/h = 0.118 \Rightarrow k_y = 0.84$

$\therefore$ Reciprocal method $\Rightarrow 1/k_{xy} = 1/k_x + 1/k_y - 1/k_0 = 1/1.17 + 1/0.84 - 1/1.18$

$\Rightarrow k_{xy} = 0.83$, which is $> k$ (= 0.64) (OK)

$\therefore$ Condition (2) is satisfied.

For condition (3)

$k = P/(\phi f_c' bh) = 202.8/277.2 = 0.73$

$k_0 = 1.18$; $ke_x/h = 0.006 \Rightarrow k_x = 1.17$; $ke_y/h = 0.102 \Rightarrow k_y = 0.90$

$\therefore$ Reciprocal method $\Rightarrow 1/k_{xy} = 1/k_x + 1/k_y - 1/k_0 = 1/1.17 + 1/0.90 - 1/1.18$

$\Rightarrow k_{xy} = 0.89$, which is $> k$ (= 0.73) (OK)

$\therefore$ Condition (3) is satisfied.

For condition (4)

$k = P/(\phi f_c' bh) = 187.2/277.2 = 0.68$

$k_0 = 1.18$; $ke_x/h = 0.113 \Rightarrow k_x = 0.87$; $ke_y/h = 0.008 \Rightarrow k_y = 1.17$

$\therefore$ Reciprocal method $\Rightarrow 1/k_{xy} = 1/k_x + 1/k_y - 1/k_0 = 1/0.87 + 1/1.17 - 1/1.18$

$\Rightarrow k_{xy} = 0.86$, which is $> k$ (= 0.68) (OK)

$\therefore$ Condition (4) is satisfied.

For condition (5)

$k = P/(\phi f_c' bh) = 192.6/277.2 = 0.69$

$k_0 = 1.18$; $ke_x/h = 0.102 \Rightarrow k_x = 0.90$; $ke_y/h = 0.008 \Rightarrow k_y = 1.17$

$\therefore$ Reciprocal method $\Rightarrow 1/k_{xy} = 1/k_x + 1/k_y - 1/k_0 = 1/0.90 + 1/1.17 - 1/1.18$

$\Rightarrow k_{xy} = 0.89$, which is $> k$ (= 0.69) (OK)

$\therefore$ Condition (5) is satisfied.

$\therefore$ The assumed section is chosen for all the columns.

7. Design of Footings (USD)

The following sample footings will be designed.

1. Individual Footing: A footing under C_5 will be designed for column load 215.3^k (plus footing weight).
2. Combined Footing: A footing under C_6, C_7 and C_8 will be designed for column loads 210.3^k, 86.7^k and 253.2^k (plus footing weights) combined.

The allowable bearing capacity of the soil is 2.0 ksf

[This can be determined from field tests like SPT, CPT or from lab test for unconfined compression strength. The formula for bearing capacity has factors for soil cohesion (N_c), foundation width (N_γ) and depth (N_q). These are functions of the angle of friction ϕ. With a factor of safety 3.0, N_c approximately equals to 6.0 and neglecting N_γ and N_q, the unconfined compression strength equals to the allowable bearing capacity].

<u>Individual Footing under C_5:</u>

Column load = $215.3^k \Rightarrow$ Footing load $\cong 215.3^k \times 1.1 \cong 236.8^k$

$\therefore$ Footing area (based on working stresses) = $157.9^k/2.0$ ksf = 78.9 ft$^2 \cong 9.00' \times 9.00'$

$\therefore$ Effective bearing pressure = $215.3/(9.00)^2 = 2.66$ ksf

Column size = $11''\times12''$, Effective depth of footing = d

$\therefore$ Punching Shear area $A_p = 2 \times (11 + d + 12 + d)\ d = 4 \times (11.5 + d)\ d$

Punching Shear strength = $4\phi\sqrt{(f_c')} = 4 \times 0.85\sqrt{(3000)} = 186$ psi = 0.186 ksi

$\therefore 0.186\ \{4 \times (11.5 + d)\ d\} = 215.3 - 2.66 \times (11 + d) \times (12 + d)/(12)^2$

$\Rightarrow d^2 + 11.5\ d = 289.0 - (11 + d) \times (12 + d)/40.36 \Rightarrow d = 11.22''$

$\therefore$ Take footing thickness, $t = 15.5'' \Rightarrow d = 11.5''$

Flexural Shear strength = $2\phi\sqrt{(f_c')} = 2 \times 0.85\sqrt{(3000)} = 93.1$ psi = 13.41 ksf

Maximum flexural shear force = 2.66 ksf $\times\{(9.00\text{–}11/12)/2\text{–}d\}' = 2.66 \times (4.04\text{–}d)$ k/$'$

$\therefore 13.41\times1\times d = 2.66\times(4.04\text{–}d) \Rightarrow d = 2.66\times4.04/(13.41+2.66) = 0.67' = 8.02'' < 11.5''$, OK.

Total Maximum bending moment, $M = 2.66\times\{(9.00\text{-}11/12)/2\}^2/2 \times 9.00 = 195.34$ k$'$

$\therefore$ Depth required by M is = $\sqrt{(M/R_u b)} = \sqrt{\{195.34/(0.781\times9.00)\}} = 5.27'' < 11.5''$, OK.

$\therefore A_s = (f_c/f_y)[1\text{–}\sqrt{\{1\text{–}2M/(\phi f_c bd^2)\}}]bd = 5.88$ in^2

Minimum reinforcement = $(0.2/f_y)bd = (0.2/40)\times9.00\times12\times11.5 = 6.21$ in$^2 > A_s$

$\therefore$ Provide 11 #7 bars in each direction.

Combined Footing under C_6-C_7-C_8:

Column loads are 210.3^k, 86.7^k and 253.2^k

$\therefore$ Resultant = 210.3 + 86.7 + 253.2 = 550.2^k

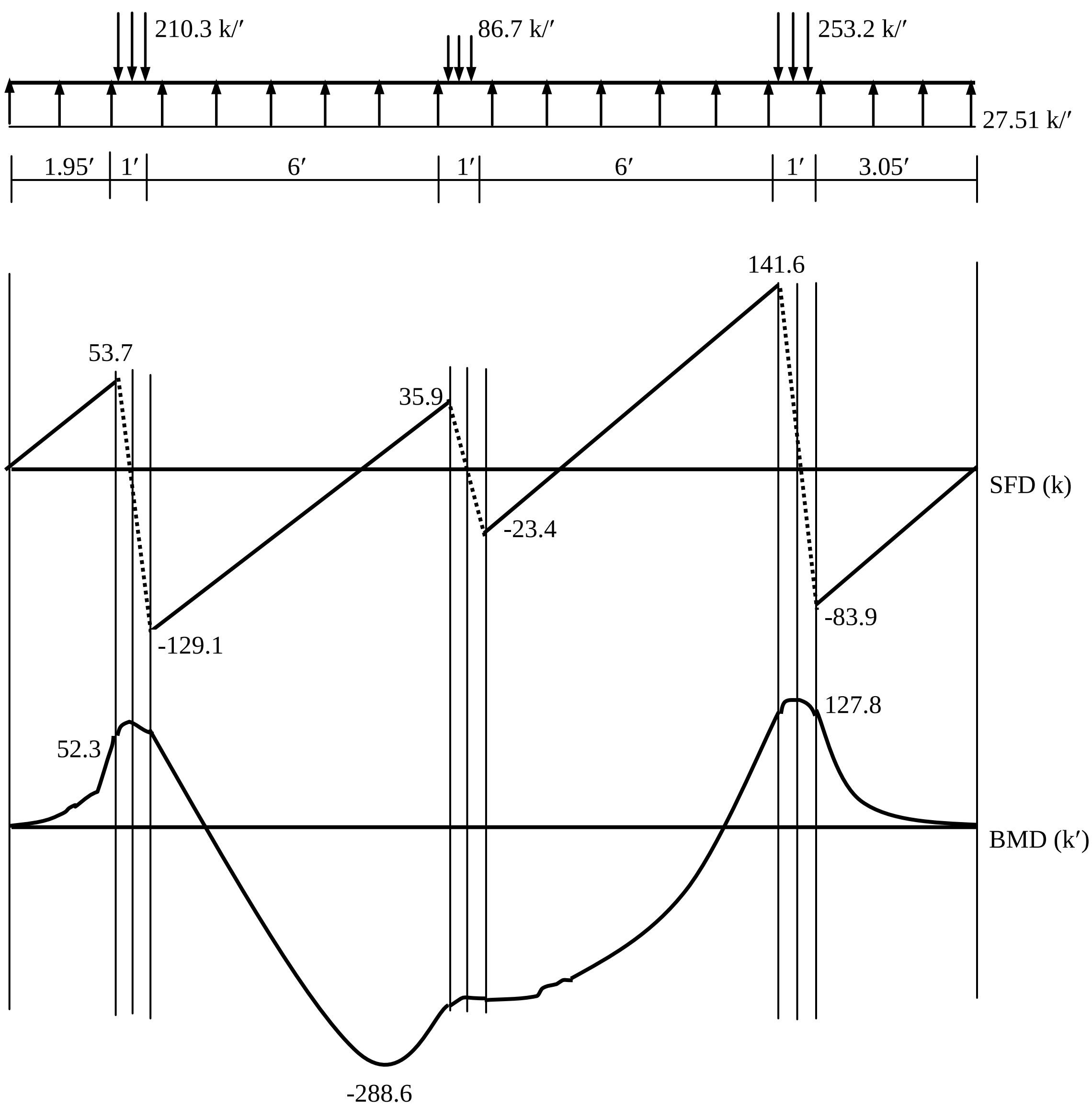
$\therefore$ Footing area (based on working stresses) = $366.8^k \times 1.1/2.0$ ksf = 201.74 ft$^2 \cong 20' \times 10'$

Distance of resultant from the first column

= $(210.3 \times 0 + 86.7 \times 7 + 253.2 \times 14)/550.2 = 7.55'$

$\therefore$ Effective bearing pressure = $550.2/(20\times10)$ = 2.751 ksf; i.e., 27.51 k/′ along the length.

$\therefore$ Considering the column loads to be uniformly distributed over the column width = 1′, the footing loads and corresponding SFD and BMD are shown below



Column size = $11'' \times 12''$, Effective depth of footing = d

$\therefore$ Punching Shear area $A_p = 2 \times (11 + d + 12 + d)\, d = 4 \times (11.5 + d)\, d$

Punching Shear strength = $4\phi\sqrt{(f_c')} = 4 \times 0.85\sqrt{(3000)} = 186$ psi = 0.186 ksi

$\therefore 0.186\ \{4 \times (11.5 + d)\, d\} = 253.2 - 2.751 \times (11 + d) \times (12 + d)/(12)^2$

$\Rightarrow d^2 + 11.5\, d = 339.9 - (11 + d) \times (12 + d)/38.99 \Rightarrow d = 12.48''$

$\therefore$ Take footing thickness, $t = 16.5'' \Rightarrow d = 12.5''$

Flexural Shear strength = $2\phi\sqrt{(f_c')} = 2 \times 0.85\sqrt{(3000)} = 93.1$ psi = 13.41 ksf

Maximum shear force (according to SFD) = 141.6^k

$\therefore$ Maximum flexural shear force = $141.6 - 27.51d$ [d is in ft]

$\therefore 13.41 \times 10 \times d = 141.6 - 27.51d \Rightarrow d = 0.876' = 10.52'' < 12.5''$, OK.

Maximum bending moment (according to BMD) = 288.6 k′

$\therefore$ Depth required by M is = $\sqrt{(M/R_u b)} = \sqrt{\{288.6/(0.781 \times 10)\}} = 6.08'' < 12.5''$, OK.

$\therefore A_s^{(-)} = (f_c/f_y)[1 - \sqrt{\{1 - 2M^{(-)}/(\phi f_c bd^2)\}}]bd = 8.03\ in^2$

and $A_s^{(+)} = (f_c/f_y)[1 - \sqrt{\{1 - 2M^{(+)}/(\phi f_c bd^2)\}}]bd = 3.47\ in^2$ [using $M^{(+)} = 127.8$ k′]

$A_{s(min)} = (0.2/f_y)bd = (0.2/40) \times 10.00 \times 12 \times 12.5 = 7.50\ in^2$, which is $< A_s^{(-)}$ but $> A_s^{(+)}$

$\therefore$ Provide 13 #7 bars at top and bottom.

Width of the transverse beams under columns = 11 + 12.5 = 23.5″

Load per unit length under C_8 = 253.2/10 = 25.32 k/′

$\therefore$ Maximum bending moment = $25.32 \times [(10 - 12/12)/2]^2/2 = 256.4$ k′

$\therefore$ Depth required by M is = $\sqrt{(M/R_u b)} = \sqrt{\{256.4 \times 12/(0.781 \times 23.5)\}} = 12.95'' > 11.5''$.

$\therefore$ Provide d = 13.5″, increase the thickness to 17.5″ and put transverse rods at bottom

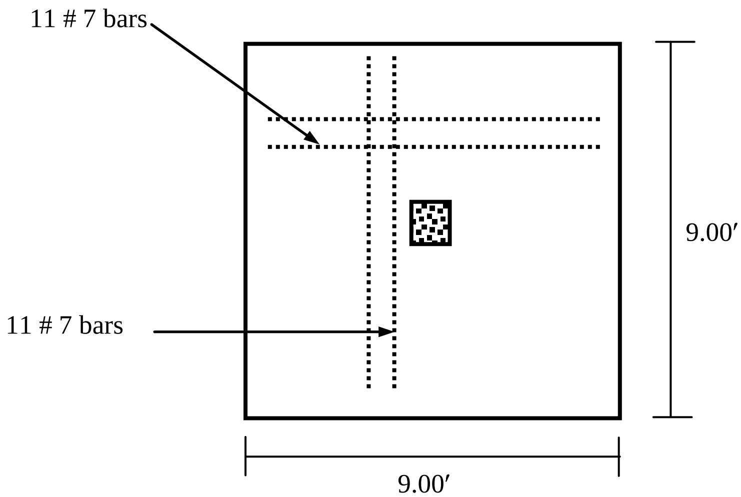
$\therefore A_s^{(+)} = (f_c/f_y)[1 - \sqrt{\{1 - 2M^{(+)}/(\phi f_c bd^2)\}}]bd = 7.86\ in^2$ [Note: d = 13.5″ here]

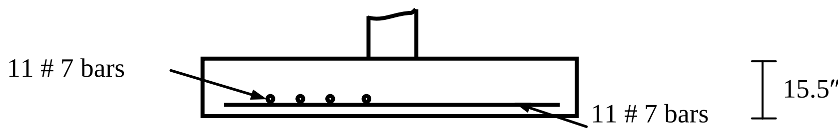
Similarly, $A_s^{(+)}$ under $C_6 = 6.21\ in^2$ and $A_s^{(+)}$ under $C_7 = 2.30\ in^2$

$\therefore A_{s(min)} = (0.2/f_y)bd = (0.2/40) \times 23.5 \times 13.5 = 1.59\ in^2 < A_s^{(+)}$ for all columns

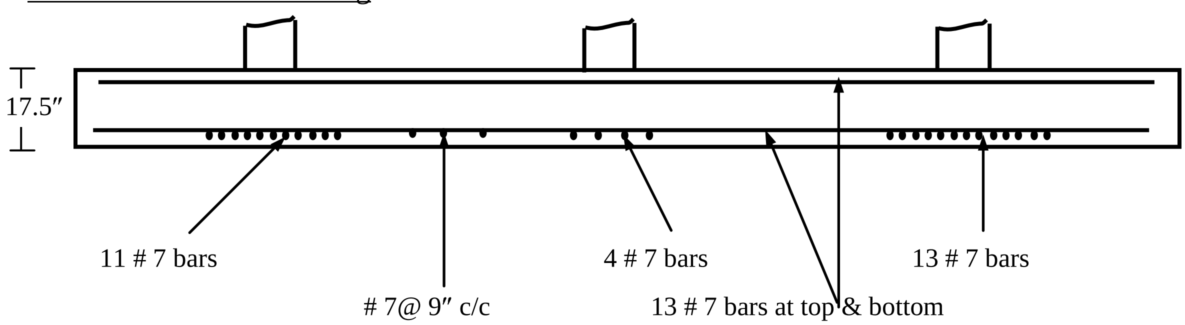
$\therefore$ Provide 11#7, 4#7 and 13#7 bars at bottom within the width of the transverse beams (23.5″) under each column (in placing 13 #7 bars in 23.5″, be careful about minimum spacing). Elsewhere, provide $(0.2/f_y)bd = 0.81\ in^2$/ft (i.e., #7 @ 9″ c/c).

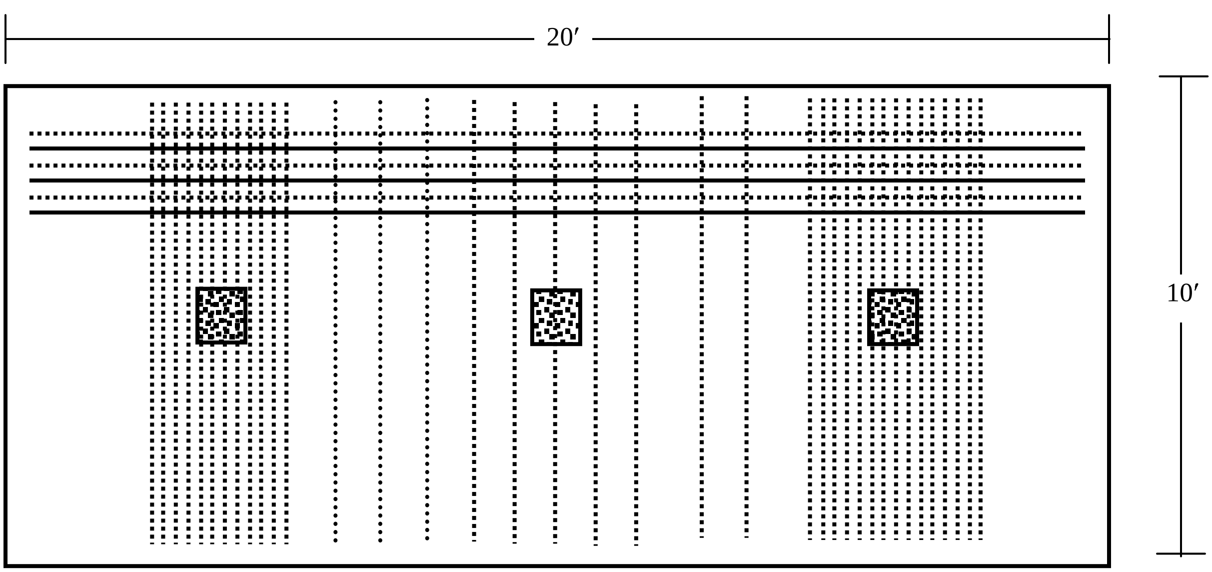
Details of Individual Footing





Details of Combined Footing





Seismic Detailing of RC Structures

1. Materials

	Specification	Possible Explanation
Concrete	$f_c' \geq 20$ Mpa ($\cong 3$ ksi) for 3-storied or taller buildings	Weak concretes have low shear and bong strengths and cannot take full advantage of subsequent design provisions
Steel	$f_y \leq 415$ Mpa ($\cong 60$ ksi), preferably ≤ 250 Mpa ($\cong$ 36 ksi)	Lower strength steels have (a) a long yield region, (b) greater ductility, (c) greater f_{ult}/f_y ratio

2. Flexural Members (members whose factored axial stress $\leq f_c'/10$)

	Specification	Possible Explanation
Size	$b/d \geq 0.3$	To ensure lateral stability and improve torsional resistance
	$b \geq 8''$	To (a) decrease geometric error, (b) facilitate rod placement
	$d \leq L_c/4$	Behavior and design of deeper members are significantly different
Longitudinal Reinforcement	$N_{s(top)}$ and $N_{s(bottom)} \geq 2$	Construction requirement
	$\rho \geq 0.1\sqrt{f_c'}/f_y$ (f_c', f_y in ksi) at both top and bottom	To avoid brittle failure upon cracking
	$\rho \leq 0.025$ at top or bottom	To (a) cause steel yielding before concrete crushing and (b) avoid steel congestion
	$A_{s(bottom)} \geq 0.5A_{s(top)}$ at joint and $A_{s(bottom)/(top)} \geq 0.25A_{s(top)\,(max)}$ at any section	To ensure (a) adequate ductility, (b) minimum reinforcement for moment reversal
	Both top and bottom bars at an external joint must be anchored $\geq L_d + 10d_b$ from inner face of column with 90° bends	To ensure (a) adequate bar anchorage, (b) joint ductility
	Lap splices are allowed for $\leq$ 50% of bars, only where stirrups are provided @$\leq d/4$ or $4''$ c/c	Closely spaced stirrups are necessary within lap lengths because of the possibility of loss of concrete cover
	Lap splice lengths $\geq L_d$ and are not allowed within distance of 2d from joints or near possible plastic hinges	Lap splices are not reliable under cyclic loading into the inelastic range

2. Flexural Members (continued)

	Specification	Possible Explanation
Web Reinforcement	Web reinforcements must consist of closed vertical stirrups with 135° hooks and $10d_t$ (≥ 3″) extensions	To provide lateral support and ensure strength development of longitudinal bars
	Design shear force is the maximum of (a) shear force from analysis, (b) shear force due to vertical loads plus as required for flexural yielding of joints	It is desirable that the beams should yield in flexure before failure in shear
	Spacing of hoops within 2d (beginning at ≤ 2″) at either end of a beam must be ≤ d/4, $8d_b$; elsewhere $S_t \le d/2$	To (a) provide resistance to shear, (b) confine concrete to improve ductility, (c) prevent buckling of longitudinal compression bars

3. Axial Members (members whose factored axial stress ≥ $f_c'/10$)

	Specification	Possible Explanation
Size	$b_c/h_c \ge 0.4$	To ensure lateral stability and improve torsional resistance
	$b_c \ge 12''$	To avoid (a) slender columns, (b) column failure before beams
Longitudinal Reinforcement	Lap splices are allowed only for ≤ 50% of bars, only where stirrups are provided @≤$b_c/4$ or 4″	Closely spaced stirrups are necessary within lap lengths because of the possibility of loss of concrete cover
	Lap splice lengths ≥ L_d and only allowed in the center half of columns	Lap splices are not reliable under cyclic loading into the inelastic range
	$0.01 \le \rho_g \le 0.06$	To (a) ensure effectiveness and (b) avoid congestion of longitudinal bars
	$\sum M_{c,ult} \ge 1.2 \sum M_{b,ult}$ at joint	To obtain 'strong column weak beam condition' to avoid column failure before beams
Transverse Reinforcement	Transverse reinforcement must consist of closed spirals or rectangular/ circular hoops with 135° hooks with $10d_t$ (≥ 3″) extensions	To provide lateral support and ensure strength development of longitudinal bars
	Parallel legs of rectangular hoops must be spaced @ ≤ 12″ c/c	To provide lateral support and ensure strength development of longitudinal bars
	Spacing of hoops within L_0 (≥ d_c, $h_c/6$, 18″) at each end of column must be ≤ $b_c/4$, 4″; else $S_t \le b_c/2$	To (a) provide resistance to shear, (b) confine concrete to improve ductility, (c) prevent buckling of longitudinal compression bars

3. Axial Members (continued)

	Specification	Possible Explanation
Transverse Reinforcement	Design shear force is the maximum of (a) shear force from analysis, (b) shear force required for flexural yielding of joints	It is desirable that the columns should yield in flexure before failure in shear
	Special confining reinforcement (i.e., $S_t \leq b_c/4$, 4″) should extend at least 12″ into any footing	To provide resistance to the very high axial loads and flexural demands at the base
	Special confining reinforcement (i.e., $S_t \leq b_c/4$, 4″) should be provided over the entire height of columns supporting discontinued stiff members and extend L_d into the member	Discontinued stiff members (e.g., shear walls, masonry walls, bracings, mezzanine floors) may develop significant forces and considerable inelastic response
	For special confinement, area of circular spirals $\geq 0.11\ S_t d\ (f_c'/f_y)(A_g/A_c-1)$, of rectangular hoops $\geq 0.3\ S_t d\ (f_c'/f_y)(A_g/A_c-1)$	To ensure load carrying capacity upto concrete spalling, taking into consideration the greater effectiveness of circular spirals compared to rectangular hoops. It also ensures toughness and ductility of columns

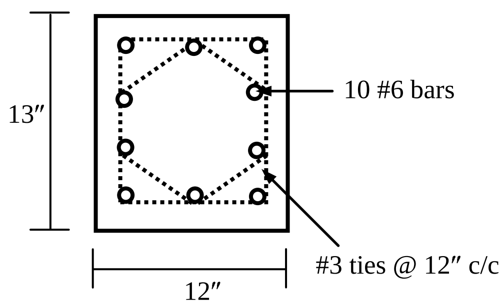
4. Joints of Frames

	Specification	Possible Explanation
Transverse Reinforcement	Special confining reinforcement (i.e., $S_t \leq b_c/4$, 4″) should extend through the joint	To provide resistance to the shear force transmitted by framing members and improve the bond between steel and concrete within the joint
	$S_t \leq b_c/2$, 6″ through joint with beams of width $b \geq 0.75b_c$	Some confinement is provided by the beams framing into the vertical faces of the joint

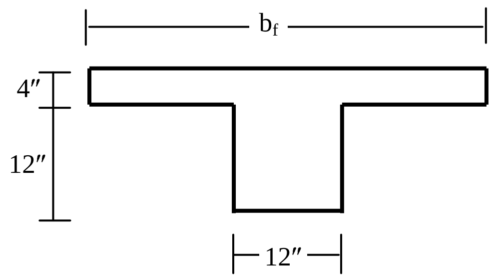
Seismic Detailing of a Typical Frame

<u>Original Design of Frame (3)</u>

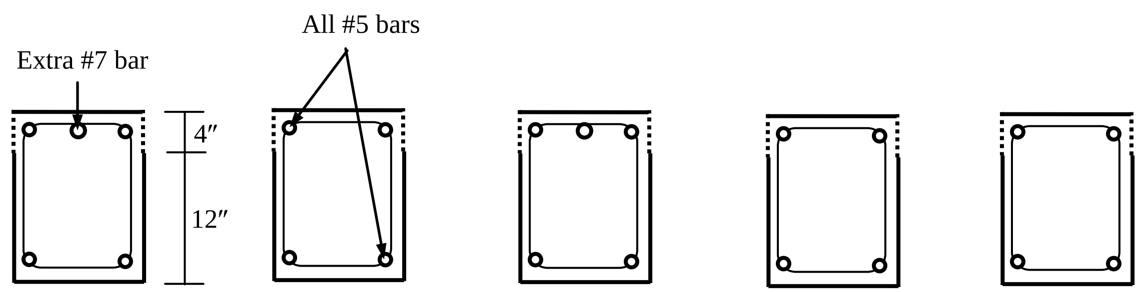
Beams	SF_1 (D)	SF_2 (D)	BM_1(D)	BM_0(V=D)	BM_2(D)
B_{16}	9.3	-9.6	-23.0	12.4	-22.9
B_{20}	6.5	-5.6	5.5, -17.6	0.7	8.2, -13.7



Column Section
(with reinforcements)



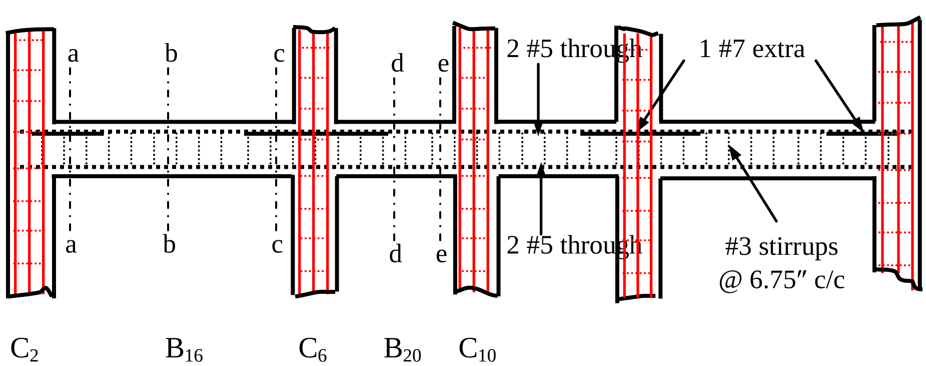
Beam + Slab Section
(without reinforcement)



Section a-a Section b-b Section c-c Section d-d Section e-e

Beam Sections
(with reinforcements)

The reinforcements are arranged as follows



1. Materials

	Specification	Design Condition	Comments
Concrete	$f_c' \geq 20$ Mpa ($\cong 3$ ksi) for 3-storied or taller buildings	$f_c' = 3$ ksi	OK
Steel	$f_y \leq 415$ Mpa ($\cong 60$ ksi), preferably ≤ 250 Mpa ($\cong$ 36 ksi)	$f_y = 40$ ksi	OK

2. Flexural Members (members whose factored axial stress $\leq f_c'/10$)

	Specification	Design Condition	Comments
Size	$b/d \geq 0.3$	$b/d = 12/13.5 = 0.89$	OK
	$b \geq 8''$	$b = 12''$	OK
	$d \leq L_c/4$	$d = 13.5''$, $L_c/4 = 3'$ and $1.5'$	OK
Longitudinal Reinforcement	$N_{s(top)}$ and $N_{s(bottom)} \geq 2$	$N_{s(top)} \geq 2$, $N_{s(bottom)} = 2$	OK
	$\rho \geq 0.1\sqrt{f_c'}/f_y$ (= 0.0043) at both top and bottom	$A_{s(top)} = 0.62, 1.22\ in^2$ $A_{s(bottom)} = 0.62\ in^2$ $\rho_{(top)} = 0.0038, 0.0075$ $\rho_{(bottom)} = 0.0038$	Both $\rho_{(top)}$ and $\rho_{(bottom)}$ are not OK
	$\rho \leq 0.025$ at top or bottom	Maximum $\rho = 0.0075$	OK
	$A_{s(bottom)} \geq 0.5A_{s(top)}$ at joint and $A_{s(bottom)/(top)} \geq 0.25A_{s(top)\ (max)}$ at any section	$A_{s(top)} = 0.62, 1.22\ in^2$ $A_{s(bottom)} = 0.62\ in^2$ (through)	OK
	Both top and bottom bars at an external joint must be anchored $\geq L_d + 10d_b$ from inner face of column with 90° bends	Not specified in design	Needs to be specified
	Lap splices are allowed for $\leq 50\%$ of bars, only where stirrups are provided @$\leq d/4$ (= 3.38″) or 4″ c/c	Not specified in design	Needs to be specified
	Lap splice lengths $\geq L_d$ and are not allowed within distance of 2d (= 27″) from joints or near possible plastic hinges	Not specified in design	Needs to be specified

2. Flexural Members (continued)

	Specification	Design Condition	Comments
Web Reinforcement	Web reinforcements must consist of closed vertical stirrups with 135° hooks, $10d_t \geq 3''$ extensions	Not specified in design	Needs to be specified
	Design shear force is the maximum of (a) shear force from analysis, (b) shear force due to vertical loads plus as required for flexural yielding of joints	Design shear force is taken only from analysis	Needs to be checked
	Spacing of hoops within 2d (= 27″), beginning at ≤ 2″, at either end of a beam must be ≤ d/4, $8d_b$ (= 3.38″, 5″) elsewhere $S_t \leq d/2$ (= 6.75″)	S_t = 6.75″ c/c (= d/2) throughout the beams	Not OK

3. Axial Members (members whose factored axial stress ≥ $f_c'/10$)

	Specification	Design Condition	Comments
Size	$b_c/h_c \geq 0.4$	$b_c/h_c = 12/13 = 0.92$	OK
	$b_c \geq 12''$	$b_c = 12''$	OK
Longitudinal Reinforcement	Lap splices are allowed only for ≤ 50% of bars, only where stirrups are provided @≤ $b_c/4$ or 4″	Not specified in design	Needs to be specified
	Lap splice lengths ≥ L_d and only allowed in the center half of columns	Not specified in design	Needs to be specified
	$0.01 \leq \rho_g \leq 0.06$	$A_s = 4.40$ in^2 $\rho_g = 0.029$	OK
	$\sum M_{c,ult} \geq 1.2 \sum M_{b,ult}$ at joint	Not specified in design	Needs to be checked
Transverse Reinforcement	Stirrups must be closed rectangular/ circular hoops with 135° hooks with $10d_t$ (≥ 3.75″) extensions	Not specified in design	Needs to be specified
	Parallel legs of rectangular hoops must be spaced @ ≤ 12″ c/c	Parallel legs of rectangular hoops spaced @ ≤ 10″ c/c	OK
	Hoop spacing within $L_0 \geq d_c$, $H_c/6$, 18″ (= 10″, 18″, 18″) at each end of column ≤ $b_c/4$ (= 3″), 4″; else $S_t \leq b_c/2$ (= 6″)	S_t = 12″ c/c throughout the columns	Not OK

3. Axial Members (continued)

	Specification	Design Condition	Comments
Transverse Reinforcement	Design shear force is the maximum of (a) shear force from analysis, (b) shear force required for flexural yielding of joints	Design shear force is taken only from analysis	Needs to be checked
	Special confining reinforcement (i.e., $S_t \leq 3''$) should extend at least 12″ into any footing	Not specified in design	Needs to be specified
	Special confining reinforcement (i.e., $S_t \leq 3''$) should be provided over the entire height of columns supporting discontinued stiff members and extend L_d into the member	Column supports no particular stiff member, but soft first storey	Special confining reinforcement (i.e., $S_t \leq 3''$) over the entire height of ground floor columns
	For special confinement, area of circular spirals ≥ $0.11\ S_t d\ (f_c'/f_y)(A_g/A_c-1)$, of rectangular hoops ≥ $0.3\ S_t d\ (f_c'/f_y)(A_g/A_c-1)$ [$= 0.3 \times 3 \times 9.5\ (3/40)\ (156/90-1) = 0.47\ in^2$]	No special confinement provided and $0.22\ in^2$ hoop area provided @ 12″ c/c	Atleast 2-legged #4 or 4-legged #3 bars needed as special confinement

4. Joints of Frames

	Specification	Design Condition	Comments
Transverse Reinforcement	Special confining reinforcement (i.e., $S_t \leq 3''$) should extend through the joint	Not specified in design	Needs to be specified
	$S_t \leq b_c/2$ (= 6″) and 6″ through joint with beams of width $b \geq 0.75b_c$	b = 12″, b_c = 12″, but no stirrups specified within joints	Since $b \geq 0.75b_c$ ∴ $S_t \leq 6''$ through joint

Correct $A_{s(min)}$ for Beams

The longitudinal steel ratio ρ is < $\rho_{(min)} = 0.1\sqrt{f_c'}/f_y$ (= 0.0043) in some cases

$\therefore$ If all the #5 bars are changed to #6 bars
$A_{s(top)} = 0.88, 1.48\ in^2$, $A_{s(bottom)} = 0.88\ in^2$
$\rho_{(top)} = 0.0054, 0.0091$, $\rho_{(bottom)} = 0.0054$, which are all > $\rho_{(min)}$ and < $\rho_{(max)}$ (= 0.025)

Check Shear Capacity of Beams

Beams	SF (V)	SF (D)	SF (USD)
B_{16}	9.6	9.6	14.4
B_{20}	4.3	6.5	9.8

For Beam B_{16}, $a = A_sf_y/0.85f_c'b = 1.48 \times 40/(0.85 \times 3 \times 12) = 1.93''$

M_{ult} at joint1 $= A_sf_y\,(d–a/2) = 1.48 \times 40 \times (13.5–1.93/2)/12 = 61.83\ k'$

Similarly, $A_s = 0.88\ in^2 \Rightarrow M_{ult}$ at joint2 $= 37.91\ k'$

$V_{Design} = 1.4\ (61.83 + 37.91)/12 + 9.6 \times 1.5 = 26.04\ k$

For Beam B_{20}, $V_{Design} = 1.4\ (61.83 + 37.91)/6 + 4.3 \times 1.5 = 29.72\ k$

$V_c = 2\sqrt{f_c'}bd = 2\sqrt{(3/1000)} \times 12 \times 13.5 = 17.75\ k$, $V_{c1} = 6\sqrt{f_c'}bd = 53.24\ k$
$\therefore S_{max} = A_sf_yd/(V_n–V_c) = 0.22 \times 40 \times 13.5/(29.72/0.85–17.75) = 6.89''$

Stirrup spacing provided @6.75″ is just adequate, but special transverse reinforcements are spaced @3″ closer to the joints.

Check Shear Capacity of Columns

Since the bending moments from analysis are small, shear forces are also assumed small; therefore the shear force is checked for flexural yielding of joints.
Also the column size is 13″×12″; i.e., the dimension 12″ works as h.

For Column C_2 at ground floor, $P = 130.8 \times 1.4 = 183.12\ k$

$P/(\phi A_gf_c') = 183.12/(0.7\times13\times12\times3) = 0.56$, $\gamma = 7/12 \cong 0.6$, $\mu\rho = (40/2.55)\times0.023 = 0.36$

$\therefore M/(\phi b_ch_c^2f_c') = 0.18 \Rightarrow M = 0.18\ (0.9\times13\times12^2\times3) = 909.79\ k'' = 75.82\ k'$

$V_{Design} = 1.4\ (75.82 + 75.82)/9 = 23.59\ k$

For Column C_6, $P = 140.2 \times 1.4 = 196.28\ k$

$P/(\phi A_gf_c') = 196.28/(0.7\times13\times12\times3) = 0.60$, with $\gamma = 0.6$, $\mu\rho = 0.36$

$\therefore M/(\phi b_ch_c^2f_c') = 0.17 \Rightarrow M = 0.17\ (0.9\times13\times12^2\times3) = 859.25\ k'' = 71.60\ k'$

$V_{Design} = 1.4\ (71.60 + 71.60)/9 = 22.28\ k$

$V_c = 2\sqrt{f_c'}\ bd = 2\sqrt{(3/1000)} \times 13 \times 9.5 = 13.53\ k$, $V_{c1} = 6\sqrt{f_c'}bd = 40.59\ k$
$\therefore S_{max} = A_sf_yd/(V_n–V_c) = 0.22 \times 40 \times 9.5/(23.59/0.85–13.53) = 5.88''$

Stirrup spacing provided @12″ throughout the columns is not adequate; i.e., provide #3 ties @5″ c/c, moreover special transverse reinforcements are spaced @3″ closer to the joints.

Check Moment Capacity of Joints (for Weak Beam Strong Column)

Since the first joint (between C_2 at two floors and B_{16}) consists of one beam only with two columns, the second joint (between C_6 at two floors and B_{16}, B_{20}) is checked first for the Weak Beam Strong Column.

For ground floor Column C_6, $P = 140.2 \times 1.4 = 196.28$ k

$P/(\phi A_g f_c') = 0.60$, with $\gamma = 0.6$, $\mu\rho = 0.36 \Rightarrow M = 71.60$ k′

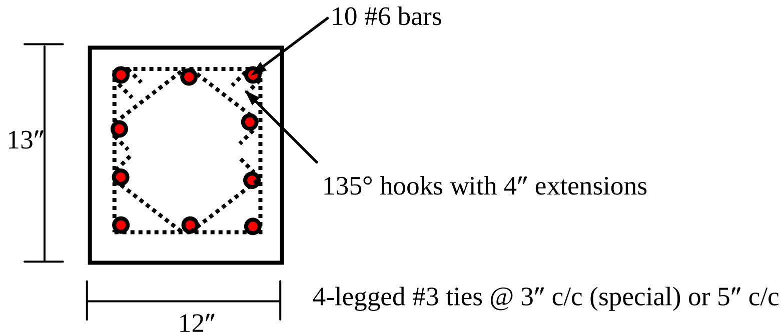
For first floor Column C_6, $P/(\phi A_g f_c') \cong 3/4 \times 0.60 = 0.45$, with $\gamma = 0.6$, $\mu\rho = 0.36$

$\therefore M/(\phi b_c h_c^2 f_c') = 0.19 \Rightarrow M = 0.19\,(0.9\times13\times12^2\times3) = 960.34$ k″ $= 80.03$ k′

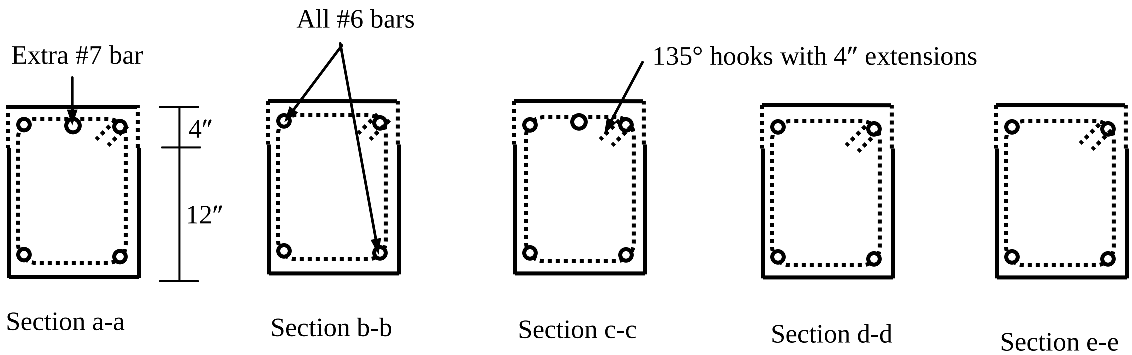
$\therefore \sum M_{c,ult} = 71.60 + 80.03 = 151.63$ k′, $1.2 \sum M_{b,ult} = 1.2\,(37.91 + 37.91) = 90.98$ k′

$\therefore \sum M_{c,ult} > 1.2 \sum M_{b,ult}$; i.e., the Weak Beam Strong Column condition is satisfied.

The following sections are chosen as columns and beams

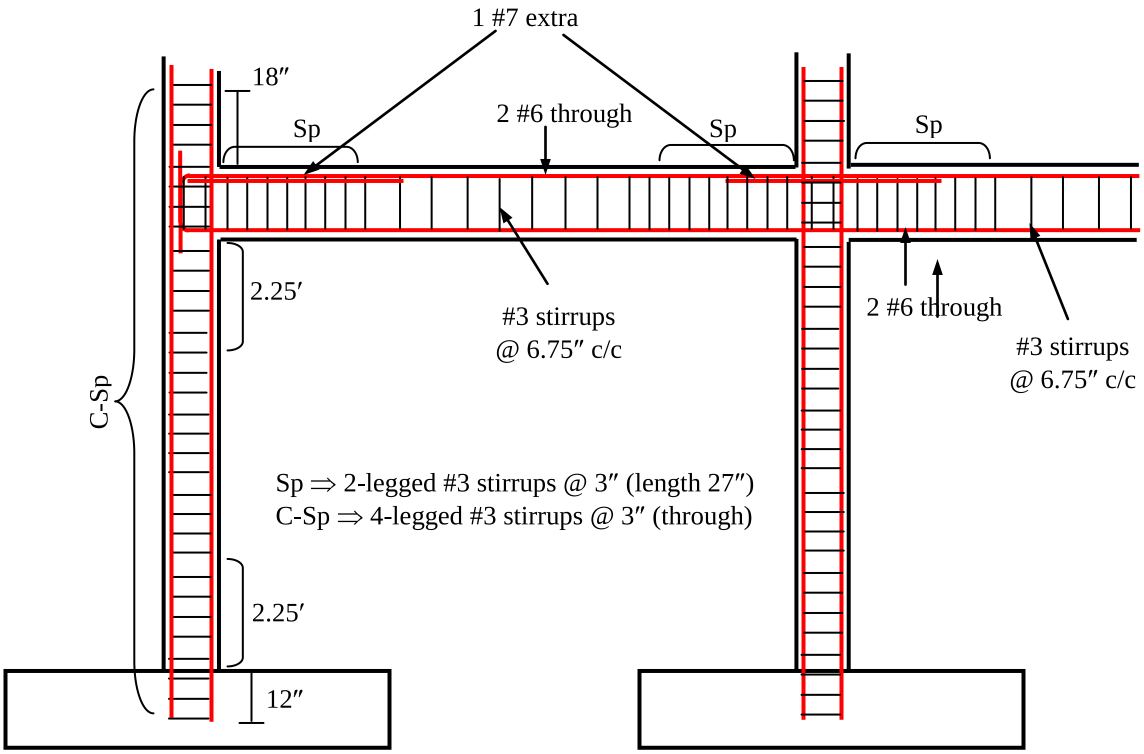


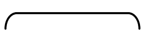
Column Section
(with reinforcements)



Beam Sections
(with reinforcements)

The reinforcements are arranged as follows



 Lap-splices not allowed here

Elsewhere, it is only allowed for 50% bars with special confinement

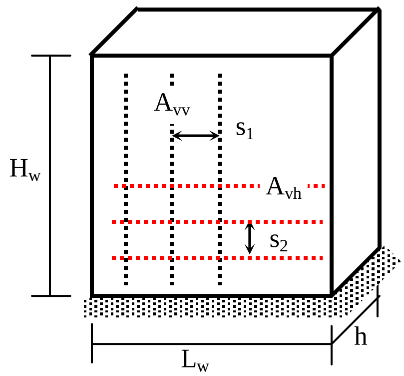
Anchorage at end joints $L_{anch} = L_d + 10\ d_b$

L_d for #7 bars $= 0.04\ A_s\ f_y/\sqrt{f_c'} = 0.04 \times 0.60 \times 40/\sqrt{(3/1000)} \times 1.4 = 24.53''$

$L_{anch} = 24.53 + 10 \times 7/8 = 33.29''$; i.e., 34″

Design of Shear Walls

ACI Provisions for Shear Wall Design



1. The thickness of the shear wall must be atleast 8 inch; i.e., $h \geq 8''$

2. Sections taken at $L_w/2$ or $H_w/2$ (whichever is less) from base is considered as critical for shear

3. In designing the horizontal shear forces or bending moments, the depth of the section is taken as $d = 0.8L_w$

4. The nominal shear force, i.e., the permissible shear force in the section is
(i) greater the design shear force, i.e., $V_n = V_{design}/\phi$ [$V = V_{design}$ in WSD]
(ii) summation of the shear force capacities of concrete and steel , i.e., $V_n = V_c + V_s$
(iii) cannot be greater than $10\sqrt{f_c'}\,hd$; i.e., $V_n \leq 10\sqrt{f_c'}\,hd$ [$V \leq 5\sqrt{f_c'}\,hd$ in WSD]

5. If N_u is taken as negative for tensile forces (lbs)
V_c can be taken as $\leq 2(1 + N_u/500A_g)\sqrt{f_c'}\,hd$ [$V_c \leq 1.1(1 + N_u/500A_g)\sqrt{f_c'}\,hd$ in WSD]
Using a more detailed analysis,
$V_c \leq 3.3\sqrt{f_c'}\,hd + N_u d/(4\,L_w)$
and $\leq [0.6\sqrt{f_c'} + \{1.25\sqrt{f_c'} + 0.2\,N_u/(h\,L_w)\}/\{M_u/(V_u L_w) - 0.5\}]\,hd$
[In WSD, take V_c to be about half of these]

6. If $V_n \leq V_c/2$, then the minimum horizontal reinforcements are provided

7. If $V_n \geq V_c$, then the horizontal reinforcements are spaced at
$s_2 = A_{vh} f_y\, d/(V_n - V_c)$ [$s_2 = A_{vh} f_s d/(V - V_c)$ in WSD] or ; where $s_2 \leq L_w/5$, $3h$, $18''$
However, $\rho_h = A_{vh}/(hs_2) \geq 0.0025$; i.e., $s_2 \leq A_{vh}/(0.0025\,h)$

8. The vertical reinforcements are spaced at
$s_1 \leq L_w/3$, $3h$, $18''$
$\rho_v = A_{vv}/(hs_1) = 0.0025 + 0.5\,(2.5 - H_w/L_w)\,(\rho_h - 0.0025)$
However, $\rho_v \geq 0.0025$ and $\leq \rho_h$

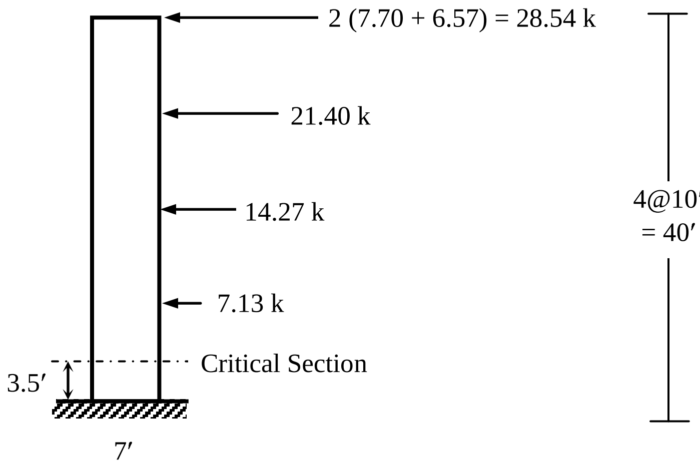
9. Flexural reinforcements are provided like normal beams

Shear Wall Design in the Long Direction of the Building

The long direction includes two Frame(1)'s and two Frame (2)'s.

Here, the height of the wall $H_w = 40'$, the length of the wall $L_w = 7'$

Assuming only the rear wall to take all the loads (due to the large opening in the front wall), with similar seismic coefficients as taken initially for the frame analysis, the following lateral forces are calculated in the long direction of the building.



1. Assumed thickness of the wall, $h = 8''$

2. Sections taken at $L_w/2$ (= 3.5′which is < $H_w/2 = 20'$) from base can be considered as critical
   $\therefore$ The nominal shear force, $V = 28.54 + 21.40 + 14.27 + 7.13 = 71.24$ k

3. The depth chosen for design is, $d \cong 0.8L_w = 0.8 \times 7 \times 12 = 67.2''$

4. $V = 71.24$ k, while $5\sqrt{f_c'}\ hd = 5\sqrt{(3/1000)} \times 8 \times 67.2 = 147.23$ k
   $\therefore V \leq 5\sqrt{f_c'}\ hd \Rightarrow$ OK

5. Since there is no net tensile force on the section, V_c can be taken as $= 1.1\sqrt{f_c'}\ hd = 32.39$ k

6. Here V is not $\leq V_c/2$, then the minimum horizontal reinforcements are not sufficient

7. Using 2-legged #3 bars, the horizontal reinforcements are spaced at
   $s_2 = 0.22 \times 20 \times 67.2/(71.24 - 32.39) = 7.61''$
   Using 2-legged #4 bars, $s_2 = 13.84''$
   Also $s_2 \leq (L_w/5 =)\ 16.8''$, $(3h =)\ 24''$, $18''$, $(A_{vh}/0.0025\ h =)\ 20''$
   $\Rightarrow$ Use 2-legged #4 bars @ 14″ c/c
   $\therefore \rho_h = A_{vh}/(hs_2) = 0.40/(8\times14) = 0.0036$

8. Using 2-legged #4 bars as vertical reinforcements
   $\rho_v = A_{vv}/(hs_1) = 0.0025 + 0.5\ (2.5 - H_w/L_w)\ (\rho_h - 0.0025)$
   $\Rightarrow 0.40/(8s_1) = 0.0025 + 0.5\ (2.5 - 40/7)\ (0.0036 - 0.0025) = 0.0008 \Rightarrow s_1 = 64.26''$
   However, $s_1 \leq (L_w/3 =)\ 28''$, $(3h =)\ 24''$, $18''$, $(A_{vv}/0.0025\ h =)\ 20''$
   $\Rightarrow$ Use 2-legged #4 bars @ 18″ c/c
   $\therefore \rho_h = A_{vv}/(hs_a) = 0.40/(8\times18) = 0.0028$, which is ≥ 0.0025 and $\leq \rho_h$

9. Flexural reinforcements are provided like normal beams
   M_{max} = 7.13 × 10 + 14.27 × 20 + 21.40 × 30 + 28.54 × 40 = 2140 k-ft
   ∴ $A_s = M/(f_s jd)$ = 2140×12/(20×0.87×67.2) = 21.97 in^2,
   ∴ Use 18 #10 bars on each side, to be curtailed over the height