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 S₄, with clear area (13’x15’).

\[ t = \frac{13 + 15}{2} \times 2/180 = 3.73” \text{; i.e., } 4” \Rightarrow d = 3” \text{ (or 2.5” for } M_{\text{min}}) \]

\[ \text{Self Wt.} = 50 \text{ psf} \Rightarrow \text{DL} = 50+20+20 = 90 \text{ psf} = 0.09 \text{ ksf} \]

\[ \text{LL} = 40 \text{ psf} = 0.04 \text{ ksf} \Rightarrow \text{Total Wt./slab area} = 0.09 + 0.04 = 0.13 \text{ ksf} \]

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

\[ R = \frac{1}{2} \times 1.35 \times 0.378 \times 0.874 = 0.223 \text{ ksi} \]

\[ A_s = \frac{M}{f_s} = \frac{M}{jd} = \frac{M}{12/(20 \times 0.874^3)} = \frac{M}{4.37} \text{ (or } M/3.64 \text{ for } M_{\text{min}}) \]

\[ A_s(\text{Temp}) = 0.0025 b t = 0.0025 \times 12 \times 4 = 0.12 \text{ in}^2/” \]

Slab (S₁):

Slab size (12’x15’), \( m = 12/15 = 0.80, \) Support condition Case 4.

\[ M_{A^+} = (0.039\times0.09+0.048\times0.04)\times(12)^2 = 0.782 \text{ k']/’} \Rightarrow A_{s(A)^+} = 0.782/4.37 = 0.18 \text{ in}^2/” \]

\[ M_{B^+} = (0.016\times0.09+0.020\times0.04)\times(15)^2 = 0.504 \text{ k']/’} \Rightarrow A_{s(B)^+} = 0.504/3.64 = 0.14 \text{ in}^2/” \]

\[ M_{A^-} = (0.071\times0.13)\times(12)^2 = 1.329 \text{ k']/’} \Rightarrow A_{s(A)^-} = 1.329/4.37 = 0.30 \text{ in}^2/” \]

\[ M_{B^-} = (0.029\times0.13)\times(15)^2 = 0.848 \text{ k']/’} \Rightarrow A_{s(B)^-} = 0.848/4.37 = 0.19 \text{ in}^2/” \]

Also, \( d_{req} = \sqrt{(M_{\text{max}}/R)} = \sqrt{(1.329/0.223)} = 2.44”, \) which is < 3”, OK.

Slab (S₂):

Slab size (12’x13’), \( m = 12/13 = 0.92, \) Support condition Case 3.

\[ M_{A^+} = (0.023\times0.09+0.033\times0.04)\times(12)^2 = 0.488 \text{ k']/’} \Rightarrow A_{s(A)^+} = 0.488/3.64 = 0.13 \text{ in}^2/” \]

\[ M_{B^+} = (0.025\times0.09+0.028\times0.04)\times(13)^2 = 0.570 \text{ k']/’} \Rightarrow A_{s(B)^+} = 0.570/4.37 = 0.13 \text{ in}^2/” \]

\[ M_{A^-} = 0 \Rightarrow A_{s(A)^-} = 0 \]

\[ M_{B^-} = (0.071\times0.13)\times(13)^2 = 1.560 \text{ k']/’} \Rightarrow A_{s(B)^-} = 1.560/4.37 = 0.36 \text{ in}^2/” \]

Also, \( d_{req} = \sqrt{(M_{\text{max}}/R)} = \sqrt{(1.560/0.223)} = 2.65”, \) which is < 3”, OK.

Slab (S₃):

Slab size (12’x13’), \( m = 12/13 = 0.92, \) Support condition Case 4.

\[ M_{A^+} = (0.032\times0.09+0.037\times0.04)\times(12)^2 = 0.628 \text{ k']/’} \Rightarrow A_{s(A)^+} = 0.628/4.37 = 0.14 \text{ in}^2/” \]

\[ M_{B^+} = (0.023\times0.09+0.028\times0.04)\times(13)^2 = 0.539 \text{ k']/’} \Rightarrow A_{s(B)^+} = 0.539/3.64 = 0.15 \text{ in}^2/” \]

\[ M_{A^-} = (0.058\times0.13)\times(12)^2 = 1.086 \text{ k']/’} \Rightarrow A_{s(A)^-} = 1.086/4.37 = 0.25 \text{ in}^2/” \]

\[ M_{B^-} = (0.042\times0.13)\times(13)^2 = 0.923 \text{ k']/’} \Rightarrow A_{s(B)^-} = 0.923/4.37 = 0.21 \text{ in}^2/” \]

Also, \( d_{req} = \sqrt{(M_{\text{max}}/R)} = \sqrt{(1.086/0.223)} = 2.21”, \) which is < 3”, OK.
Slab (S₅):  
Slab size (13′×15′), m =13/15 = 0.87, Support condition between Case 5 and Case 9.  
\[ M_A^+ = (0.029 \times 0.09 + 0.038 \times 0.04) \times (13)^2 = 0.698 \text{ kip} \Rightarrow A_{s(A)}^+ = 0.698/4.37 = 0.16 \text{ in}^2/\text{A} \]  
\[ M_B^+ = (0.013 \times 0.09 + 0.020 \times 0.04) \times (15)^2 = 0.443 \text{ kip} \Rightarrow A_{s(B)}^+ = 0.443/3.64 = 0.12 \text{ in}^2/\text{A} \]  
\[ M_A^- = (0.075 \times 0.13) \times (13)^2 = 1.648 \text{ kip} \Rightarrow A_{s(A)}^- = 1.648/4.37 = 0.38 \text{ in}^2/\text{A} \]  
\[ M_B^- = (0.011 \times 0.13) \times (15)^2 = 0.322 \text{ kip} \Rightarrow A_{s(B)}^- = 0.322/4.37 = 0.07 \text{ in}^2/\text{A} \]  
Also, \( d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.648/0.223)} = 2.72 \text{''}, which is > 3\text{''}, OK.

Slab (S₆):  
Slab size (13′×13′), m =13/13 = 1.00, Support condition between Case 5 and Case 9.  
\[ M_A^+ = (0.025 \times 0.09 + 0.031 \times 0.04) \times (13)^2 = 0.590 \text{ kip} \Rightarrow A_{s(A)}^+ = 0.590/4.37 = 0.13 \text{ in}^2/\text{A} \]  
\[ M_B^+ = (0.019 \times 0.09 + 0.028 \times 0.04) \times (13)^2 = 0.478 \text{ kip} \Rightarrow A_{s(B)}^+ = 0.478/3.64 = 0.13 \text{ in}^2/\text{A} \]  
\[ M_A^- = (0.068 \times 0.13) \times (13)^2 = 1.494 \text{ kip} \Rightarrow A_{s(A)}^- = 1.494/4.37 = 0.34 \text{ in}^2/\text{A} \]  
\[ M_B^- = (0.016 \times 0.13) \times (13)^2 = 0.352 \text{ kip} \Rightarrow A_{s(B)}^- = 0.352/3.64 = 0.08 \text{ in}^2/\text{A} \]  
Also, \( d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.494/0.223)} = 2.59 \text{''}, which is > 3\text{''}, OK.

Slab (S₇):  
Slab size (12′×15′), m =12/15 = 0.80, Support condition Case 4.Same design as S₅.  

Slab (S₈):  
Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 8.  
\[ M_A^+ = (0.024 \times 0.09 + 0.033 \times 0.04) \times (12)^2 = 0.501 \text{ kip} \Rightarrow A_{s(A)}^+ = 0.501/4.37 = 0.11 \text{ in}^2/\text{A} \]  
\[ M_B^+ = (0.020 \times 0.09 + 0.025 \times 0.04) \times (15)^2 = 0.473 \text{ kip} \Rightarrow A_{s(B)}^+ = 0.473/3.64 = 0.13 \text{ in}^2/\text{A} \]  
\[ M_A^- = (0.041 \times 0.13) \times (12)^2 = 0.767 \text{ kip} \Rightarrow A_{s(A)}^- = 0.767/4.37 = 0.18 \text{ in}^2/\text{A} \]  
\[ M_B^- = (0.054 \times 0.13) \times (13)^2 = 1.186 \text{ kip} \Rightarrow A_{s(B)}^- = 1.186/4.37 = 0.27 \text{ in}^2/\text{A} \]  
Also, \( d_{req} = \sqrt{(M_{max}/R)} = \sqrt{(1.186/0.223)} = 2.31 \text{''}, which is > 3\text{''}, OK.

Slab (S₉):  
Slab size (12′×13′), m =12/13 = 0.92, Support condition Case 8. Same design as S₇.

Slab (S₀):  
One-way cantilever slab with clear span = 2.5′  
\[ \therefore \text{Required thickness, } t = (L/10) \times (0.4+f_y/100) = (2.5\times12/10) \times (0.4+40/100) = 2.4'' < 4'' \text{, OK} \]  
\[ \therefore w = w_{DL} + w_{FF} + w_{LL} = 50 + 20 + 40 = 110.00 \text{ psf} = 0.110 \text{ ksf} \]  
\[ \therefore M^- = 0.11 \times (2.5)^2/2 = 0.344 \text{ kip} \Rightarrow A_{s^-} = 0.344/4.37 = 0.08 \text{ in}^2/\text{A} \]
Slab (S10):
One-way cantilever slab with clear span = 5.5'

\[ \text{Required thickness, } t = \left(\frac{5.5 \times 12}{10}\right) \times (0.4+40/100) = 5.28'' \Rightarrow 5.5'' \]

\[ \text{w} = w_{DL} + w_{FF} + w_{LL} = 68.75 + 20 + 20 = 108.75 \, \text{psf} = 0.109 \, \text{ksf} \]

\[ M^* = 0.109 \times 5.5^2/2 = 1.644 \, \text{k}/' \Rightarrow A_s^* = 1.644 \times 12/(20 \times 0.874 \times (5.5-1)) = 0.25 \, \text{in}^2/' \]

Slab (S11):
One-way simply supported slab with c/c span = 14' [two 3' landings and one 8' flight]

Assumed LL on stairs = 100 psf

\[ \text{Required thickness, } t = \left(\frac{14 \times 12}{20}\right) \times (0.4+40/100) = 6.72'' \Rightarrow 7'' \text{; Self weight = 87.5 psf.} \]

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

Additional weight on flights due to 6'' high stairs = \( \frac{1}{2} \times (6/12) \times 150 \) psf = 0.037 ksf

\[ \text{Weight on flight, } w_2 = 0.208 + 0.037 = 0.245 \, \text{ksf} \]

\[ M_{\text{max}} \approx 0.245 \times 14^2/8 = 6.003 \, \text{k}/' \]

\[ d_{\text{req}} = \sqrt{M_{\text{max}}/R} = \sqrt{6.003/0.223} = 5.19'', \text{ which is } < (7-1) = 6'', \text{ OK.} \]

\[ A_s^* = 6.003 \times 12/(20 \times 0.874 \times (7-1)) = 0.69 \, \text{in}^2/' \text{; i.e., } #5@5''c/c \]

\[ A_s(\text{Temp}) = 0.0025 \times 25 \times 12 \times 7 = 0.21 \, \text{in}^2/' \text{; i.e., } #3@6''c/c \]

**Loads on Staircase**

<table>
<thead>
<tr>
<th></th>
<th>0.208 ksf</th>
<th>0.245 ksf</th>
<th>0.208 ksf</th>
</tr>
</thead>
<tbody>
<tr>
<td>5'</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#3@6''c/c</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#5@5''c/c</td>
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</tbody>
</table>

**Staircase Reinforcements**
Required Slab Reinforcement (in $^2$/f) from Flexural Design
[Note: $A_{s(Temp)} = 0.12$ in $^2$/f 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): S10 Reinforcement – #3@5"c/c at Main (top), #3@10"c/c Temperature Rod
(J): S9 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.

\[ \text{Self-weight of Beams} = (12”\times12”) \times 150/144 = 150 \text{ lb/ft} = 0.15 \text{ k/ft} \]

Weight of 5” Partition Walls (PW) = \((5”/12)\times9\times120 = 450 \text{ lb/ft} = 0.45 \text{ k/ft} \)

\[ \text{Weight of 10” Exterior Walls (EW)} = 0.90 \text{ k/ft} \]

Load Distribution from Slab to Beam
Frame (1) $[B_{8,9,10,11}]$:

Slab-load on $B_8 = \left[\frac{13}{2} \times (16+3)/2 + \frac{14}{2} \times (16+2)/2\right] \times 0.13 = 16.22$ k

$\therefore$ Equivalent UDL (+ Self Wt. and PW) $\equiv \frac{16.22}{16} + 0.15 + 0.45 = 1.61$ k/

Slab-load on $B_{9,10} = \left[\frac{13}{2} \times (14+1)/2 + \frac{14}{2} \times (14)/2\right] \times 0.13 = 6.34 + 16.65 = 22.99$ k

$\therefore$ Equivalent UDL (+ Self Wt. and PW) $\equiv \frac{22.99}{14} + 0.15 + 0.45 = 2.24$ k/

Load from Slabs to $B_{11} = \left[\frac{13}{2} \times (14+1)/2 + \frac{14}{2} \times (14)/2\right] \times 0.13 = 12.71$ k

$\therefore$ Equivalent UDL (+ Self Wt. and PW) $\equiv \frac{12.71}{14} + 0.15 + 0.45 = 1.51$ k/

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

<table>
<thead>
<tr>
<th>C_12</th>
<th>C_13</th>
<th>C_14</th>
<th>C_15</th>
<th>C_16</th>
</tr>
</thead>
<tbody>
<tr>
<td>-50.8,-6.1,-12.3</td>
<td>(-86.5,-3.7,7.1)</td>
<td>(-76.1,1.9,-4.0)</td>
<td>(-41.8,-4.6,8.8)</td>
<td></td>
</tr>
<tr>
<td>1.61 k&quot;</td>
<td>2.24 k&quot;</td>
<td>2.24 k&quot;</td>
<td>1.51 k&quot;</td>
<td></td>
</tr>
</tbody>
</table>

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_{12,13,14}]:

Slab-load on B_{12} = \left[\frac{13}{2} \times \frac{(16+3)}{2}\right] \times 0.13 = 8.03 k

:. Equivalent UDL (+ Self Wt. and EW) \equiv \frac{8.03}{16} + 0.15 + 0.90 = 1.55 k/

Slab-load on B_{13} = \left[\frac{13}{2} \times \frac{(14+1)}{2}\right] \times 0.13 = 6.34 k

:. Equivalent UDL (+ Self Wt. and EW) \equiv \frac{6.34}{14} + 0.15 + 0.90 = 1.50 k/

Load from Slabs to B_{14} = \left[\frac{13}{2} \times \frac{(14+1)}{2}\right] \times 0.13 = 6.34 k

:. Equivalent UDL (+ Self Wt. and EW) \equiv \frac{6.34}{14} + 0.15 + 0.90 = 1.50 k/

Beam (SF_1, SF_2 (k), BM_1, BM_6, 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 \(B_{26}\) = \(\frac{13}{2} \times \frac{13}{2} + \frac{13}{2} \times \frac{13}{2}\) \(\times 0.13 = 10.99 \text{k}\)

\(:\) Equivalent UDL (+ Self Wt. and PW) \(\equiv 10.99/13 + 0.15 + 0.45 = 1.45 \text{k/}\)

Slab-load on \(B_{20,21}\) = \(\frac{14}{2} \times \frac{14}{2}\) \(\times 0.13 = 6.37 \text{k}\)

\(:\) Equivalent UDL (+ Self Wt. and PW) \(\equiv 6.37/14 + 0.15 + 0.45 = 1.06 \text{k/}\)

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

<table>
<thead>
<tr>
<th></th>
<th>1.45 k/</th>
<th>1.06 k/</th>
<th>1.06 k/</th>
<th>1.45 k/</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{16})</td>
<td>(4.3,-9.6,-17.2,12.4,-19.2)</td>
<td>(9.6,-9.3,-19.2,12.4,-17.2)</td>
<td>(3.1,-4.3,-3.7,0.7,-8.1)</td>
<td></td>
</tr>
<tr>
<td>(B_{20})</td>
<td>(-28.2,-0.0,0.0,0.0)</td>
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<tr>
<td>(B_{21})</td>
<td>(-28.2,-0.0,0.0,0.0)</td>
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<td></td>
</tr>
<tr>
<td>(B_{26})</td>
<td>(-37.3,-3.6,7.1)</td>
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</tbody>
</table>

\[B_{16,20,21,26}\]  
\(C_2\)  
\(C_6\)  
\(C_{10}\)  
\(C_{13}\)  
\(C_{18}\)

Beam (\(SF_1, SF_2 \text{ (k)}, BM_1, BM_0, BM_2 \text{ (k')}) and Column (\(AF \text{ (k)}, BM_1, BM_2 \text{ (k')} in Frame (3)

from Vertical Load Analysis
Frame (4) $[B_{17,23,27}]$:

Slab-load on $B_{17}$ and $B_{27} = [13/2 \times (13)/2 + 13/2 \times (13)/2] \times 0.13 = 10.99_k$

$\therefore$ Equivalent UDL (+ Self Wt. and PW) $\equiv 10.99/13 + 0.15 + 0.45 = 1.45_k$

Slab-load on $B_{23} = [14/2 \times (14)/2] \times 0.13 = 6.37_k$

$\therefore$ Equivalent UDL (+ Self Wt., EW, $S_9$) $\equiv 6.37/14 + 0.15 + 0.90 + 3 \times 0.13 = 1.90_k$

[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]

<table>
<thead>
<tr>
<th>C3</th>
<th>C8</th>
<th>C15</th>
<th>C19</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.45 k/</td>
<td>1.90 k/</td>
<td>1.45 k/</td>
<td></td>
</tr>
<tr>
<td>(35.9, -3.4, -6.6)</td>
<td>(92.7, 1.4, -2.7)</td>
<td>(92.7, -1.4, 2.7)</td>
<td>(35.9, -3.4, 6.6)</td>
</tr>
<tr>
<td>(9, -10.0, -16.0, 11.0, -23.3)</td>
<td>(10.0, -8.9, -23.3, 11.0, -16.0)</td>
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<td></td>
</tr>
</tbody>
</table>

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}].

<table>
<thead>
<tr>
<th></th>
<th>B_4</th>
<th>B_5</th>
<th>B_6</th>
<th>B_7</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(9.0,-6, 7,-15.8,2.3,-7.6)</td>
<td>(10.8,-10.4,-23.7,14.6,-21.1)</td>
<td>(7.0,-8.7,-7.5,3.3,-13.7)</td>
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<tr>
<td></td>
<td>(12.6,-13.1,-29.4,20.2,-33.3)</td>
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</tr>
<tr>
<td>C_5</td>
<td>(-50.8,6.1,-12.3)</td>
<td>(-57.8,-0.1,0.0)</td>
<td>(-41.8,-4.6,8.8)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(-86.5,3.7,7.1)</td>
<td>(-76.1,1.9,-4.0)</td>
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</tr>
</tbody>
</table>

Beam (SF_1, SF_2 (k), BM_1, BM_6, 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}].

<table>
<thead>
<tr>
<th></th>
<th>B_1</th>
<th>B_2</th>
<th>B_3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(10.6,-10.4,-26.1,11.6,-24.3)</td>
<td>(10.9,-10.1,-25.6,13.8,-20.3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(12.0,-12.8,-27.5,18.8,-34.0)</td>
<td></td>
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</tr>
<tr>
<td>C_1</td>
<td>(-48.1,5.7,-11.5)</td>
<td>(-93.5,-1.8,3.3)</td>
<td>(-84.8,0.1,-0.6)</td>
</tr>
<tr>
<td></td>
<td>(-40.8,-4.4,8.4)</td>
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</tbody>
</table>

Beam (SF_1, SF_2 (k), BM_1, BM_6, 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) \([B_{17,23,27}]\).

\[
\begin{array}{ccc}
& (13.3,-13.3,-30.1,16.5,-30.1) & \\
(8.9,-10.0,-16.0,11.0,-23.3) & (10.0,-8.9,-23.3,11.0,-16.0) \\
B_{15} & B_{19} & B_{25} \\
(-35.9,3.4,-6.6) & (-92.7,1.4,-2.7) & (-92.7,1.4,2.7) & (-35.9,3.4,6.6) \\
C_{1} & C_{5} & C_{12} & C_{17} \\
\end{array}
\]

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) \([B_{17,23,27}]\).

\[
\begin{array}{ccc}
& (13.3,-13.3,-30.1,16.5,-30.1) & \\
(8.9,-10.0,-16.0,11.0,-23.3) & (10.0,-8.9,-23.3,11.0,-16.0) \\
B_{18} & B_{24} & B_{28} \\
(-35.9,3.4,-6.6) & (-92.7,1.4,-2.7) & (-92.7,1.4,2.7) & (-35.9,3.4,6.6) \\
C_{4} & C_{9} & C_{16} & C_{20} \\
\end{array}
\]

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, \( T = 0.073 \times (40/3.28)^{1/4} = 0.476 \) sec, which is < 0.7 sec \( \Rightarrow V_t = 0 \).

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

\( \therefore \) Base Shear, \( V = (ZIC/R) \) \( W = 0.15 \times 1.0 \times 2.05/5.0 \) \( W = 0.0615 \) \( W \)

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

Frame (1) \( [B_{8-9-10-11}] \):
\( W = 4 \times (1.61 \times 16 + 2.24 \times 7 + 2.24 \times 7 + 1.51 \times 14) = 313.04^k \Rightarrow V = 0.0615W = 19.25^k \)

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) \([B_{12,13,14}]\):

\[
W = 4 \times (1.55 \times 16 + 1.50 \times 14 + 1.50 \times 14) = 267.20^k \Rightarrow V = 0.0615W = 16.43^k
\]

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) \([B_{16, 20, 21, 26}]\):

\[ W = 4(1.45 \times 13 + 1.06 \times 7 + 1.06 \times 7 + 1.45 \times 13) = 210.16^k \Rightarrow V = 0.0615W = 12.92^k \]
Frame (4) \([B_{17,23,27}]\):

\[ W = 4 \times (1.45 \times 13 + 1.90 \times 14 + 1.45 \times 13) = 257.20 \Rightarrow V = 0.0615W = 15.81k \]

Beam (SF(k), BM₁, BM₂ (k')) and Column (AF (k), BM₁, BM₂ (k')) in Frame (4) from Lateral Load Analysis
Frame \([B_{4-5-6-7}]\):
Similar to Frame (1) \([B_{8-9-10-11}]\).

<table>
<thead>
<tr>
<th></th>
<th>B_8</th>
<th>B_9</th>
<th>B_10</th>
<th>B_11</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(2.0,-17.7,14.9)</td>
<td>(6.7,-24.3,22.6)</td>
<td>(5.6,-22.5,23.8)</td>
<td>(2.6,-16.5,19.6)</td>
</tr>
<tr>
<td></td>
<td>B_5</td>
<td>B_6</td>
<td>B_7</td>
<td>B_8</td>
</tr>
<tr>
<td>2</td>
<td>(-5.9,20.0,-8.8)</td>
<td>(-13.0,24.6,6,-18.2)</td>
<td>(11.4,24.9,9,-18.7)</td>
<td>(7.4,20.6,-10.0)</td>
</tr>
</tbody>
</table>

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

Frame \([B_{1-2-3}]\):
Similar to Frame (2) \([B_{12-13-14}]\).

<table>
<thead>
<tr>
<th></th>
<th>B_12</th>
<th>B_13</th>
<th>B_14</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(2.7,-22.9,20.5)</td>
<td>(3.0,-20.6,20.4)</td>
<td>(3.5,-22.5,25.2)</td>
</tr>
<tr>
<td></td>
<td>C_17</td>
<td>C_18</td>
<td>C_19</td>
</tr>
<tr>
<td>2</td>
<td>(-7.9,24.1,-10.6)</td>
<td>(-0.8,27.9,-18.2)</td>
<td>(-1.2,28.3,-19.0)</td>
</tr>
</tbody>
</table>

**Beam (SF (k), BM_1, BM_2 (k')) and Column (AF (k), BM_1, BM_2 (k')) in Frame \([B_{1-2-3}]\) from Lateral Load Analysis**
Frame $[B_{15,19,25}]$:
Similar to Frame (4) $[B_{17,23,27}]$.

\[
\begin{array}{|c|c|c|}
\hline
& (3.5,-24.2,21.8) & (2.6,-18.3,18.3) & (3.5,-21.8,24.3) \\
\hline
B_{15} & B_{19} & B_{25} \\
\hline
(-10.2,23.1,-11.4) & (2.4,26.5,-18.0) & (-2.4,26.5,-18.0) & (10.2,23.3,-11.4) \\
\hline
C_{1} & C_{5} & C_{12} & C_{17} \\
\hline
\end{array}
\]

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) $[B_{17,23,27}]$.

\[
\begin{array}{|c|c|c|}
\hline
& (3.5,-24.2,21.8) & (2.6,-18.3,18.3) & (3.5,-21.8,24.3) \\
\hline
B_{18} & B_{24} & B_{28} \\
\hline
(-10.2,23.1,-11.4) & (2.4,26.5,-18.0) & (-2.4,26.5,-18.0) & (10.2,23.3,-11.4) \\
\hline
C_{4} & C_{9} & C_{16} & C_{20} \\
\hline
\end{array}
\]

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 \([B_{8, 9, 10, 11}]\):

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF(_1)(V)</th>
<th>SF(_1)(L)</th>
<th>SF(_2)(D)</th>
<th>SF(_2)(V)</th>
<th>SF(_2)(L)</th>
<th>SF(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(_4), B(_8)</td>
<td>12.6</td>
<td>±2.0</td>
<td>12.6</td>
<td>-13.1</td>
<td>±2.0</td>
<td>-13.1</td>
</tr>
<tr>
<td>B(_5), B(_9)</td>
<td>9.0</td>
<td>±6.7</td>
<td>11.8</td>
<td>-6.7</td>
<td>±6.7</td>
<td>-10.1</td>
</tr>
<tr>
<td>B(_6), B(_10)</td>
<td>7.0</td>
<td>±6.6</td>
<td>10.2</td>
<td>-8.7</td>
<td>±6.6</td>
<td>-11.5</td>
</tr>
<tr>
<td>B(_7), B(_11)</td>
<td>10.8</td>
<td>±2.0</td>
<td>10.8</td>
<td>-10.4</td>
<td>±2.0</td>
<td>-10.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM(_1)(V)</th>
<th>BM(_1)(L)</th>
<th>BM(_1)(D)</th>
<th>BM(_0)(V=D)</th>
<th>BM(_2)(V)</th>
<th>BM(_2)(L)</th>
<th>BM(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(_4), B(_8)</td>
<td>-29.4</td>
<td>±17.7</td>
<td>-35.3</td>
<td>20.2</td>
<td>-33.3</td>
<td>±14.9</td>
<td>-36.2</td>
</tr>
<tr>
<td>B(_5), B(_9)</td>
<td>-15.8</td>
<td>±24.3</td>
<td>6.4</td>
<td>-30.1</td>
<td>2.3</td>
<td>-7.6</td>
<td>±22.6</td>
</tr>
<tr>
<td>B(_6), B(_10)</td>
<td>-7.5</td>
<td>±22.5</td>
<td>11.3, -22.5</td>
<td>3.3</td>
<td>-13.7</td>
<td>±23.8</td>
<td>7.6, -28.1</td>
</tr>
<tr>
<td>B(_7), B(_11)</td>
<td>-23.7</td>
<td>±16.5</td>
<td>-30.2</td>
<td>14.6</td>
<td>-21.1</td>
<td>±19.6</td>
<td>-30.5</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF(_1)(V)</th>
<th>SF(_1)(L)</th>
<th>SF(_2)(D)</th>
<th>SF(_2)(V)</th>
<th>SF(_2)(L)</th>
<th>SF(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(_1), B(_12)</td>
<td>12.0</td>
<td>±2.3</td>
<td>12.0</td>
<td>-12.8</td>
<td>±2.3</td>
<td>-12.8</td>
</tr>
<tr>
<td>B(_2), B(_13)</td>
<td>10.6</td>
<td>±3.0</td>
<td>10.6</td>
<td>-10.4</td>
<td>±3.0</td>
<td>-10.4</td>
</tr>
<tr>
<td>B(_3), B(_14)</td>
<td>10.9</td>
<td>±3.5</td>
<td>10.9</td>
<td>-10.1</td>
<td>±3.5</td>
<td>-10.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM(_1)(V)</th>
<th>BM(_1)(L)</th>
<th>BM(_1)(D)</th>
<th>BM(_0)(V=D)</th>
<th>BM(_2)(V)</th>
<th>BM(_2)(L)</th>
<th>BM(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(_1), B(_12)</td>
<td>-27.5</td>
<td>±22.9</td>
<td>-37.8</td>
<td>18.8</td>
<td>-34.0</td>
<td>±20.5</td>
<td>-40.9</td>
</tr>
<tr>
<td>B(_2), B(_13)</td>
<td>-26.1</td>
<td>±20.6</td>
<td>-35.0</td>
<td>11.6</td>
<td>-24.3</td>
<td>±20.4</td>
<td>-33.5</td>
</tr>
<tr>
<td>B(_3), B(_14)</td>
<td>-25.6</td>
<td>±22.5</td>
<td>-36.1</td>
<td>13.8</td>
<td>-20.3</td>
<td>±25.2</td>
<td>-34.1</td>
</tr>
</tbody>
</table>
Frame (3) \([B_{16,20,21,26}]\):

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF(_1)(V)</th>
<th>SF(_1)(L)</th>
<th>SF(_1)(D)</th>
<th>SF(_2)(V)</th>
<th>SF(_2)(L)</th>
<th>SF(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{16})</td>
<td>9.3</td>
<td>±1.9</td>
<td>9.3</td>
<td>-9.6</td>
<td>±1.9</td>
<td>-9.6</td>
</tr>
<tr>
<td>(B_{20})</td>
<td>4.3</td>
<td>±4.3</td>
<td>6.5</td>
<td>-3.1</td>
<td>±4.3</td>
<td>-5.6</td>
</tr>
<tr>
<td>(B_{21})</td>
<td>3.1</td>
<td>±4.3</td>
<td>5.6</td>
<td>-4.3</td>
<td>±4.3</td>
<td>-6.5</td>
</tr>
<tr>
<td>(B_{26})</td>
<td>9.6</td>
<td>±1.9</td>
<td>9.6</td>
<td>-9.3</td>
<td>±1.9</td>
<td>-9.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM(_1)(V)</th>
<th>BM(_1)(L)</th>
<th>BM(_1)(D)</th>
<th>BM(_0)(V=D)</th>
<th>BM(_2)(V)</th>
<th>BM(_2)(L)</th>
<th>BM(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{16})</td>
<td>-20.5</td>
<td>±13.4</td>
<td>-23.0</td>
<td>12.4</td>
<td>-22.9</td>
<td>±11.3</td>
<td>-22.9</td>
</tr>
<tr>
<td>(B_{20})</td>
<td>-9.5</td>
<td>±15.4</td>
<td>5.5, -17.6</td>
<td>0.7</td>
<td>-4.2</td>
<td>±14.6</td>
<td>8.2, -13.7</td>
</tr>
<tr>
<td>(B_{21})</td>
<td>-4.2</td>
<td>±14.6</td>
<td>8.2, -13.7</td>
<td>0.7</td>
<td>-9.5</td>
<td>±15.4</td>
<td>5.5, -17.6</td>
</tr>
<tr>
<td>(B_{26})</td>
<td>-22.9</td>
<td>±11.3</td>
<td>-22.9</td>
<td>12.4</td>
<td>-20.5</td>
<td>±13.4</td>
<td>-23.0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF(_1)(V)</th>
<th>SF(_1)(L)</th>
<th>SF(_1)(D)</th>
<th>SF(_2)(V)</th>
<th>SF(_2)(L)</th>
<th>SF(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{15}, B_{17}, B_{18})</td>
<td>8.9</td>
<td>±3.5</td>
<td>9.3</td>
<td>-10.0</td>
<td>±3.5</td>
<td>-10.1</td>
</tr>
<tr>
<td>(B_{19}, B_{23}, B_{24})</td>
<td>13.3</td>
<td>±2.6</td>
<td>13.3</td>
<td>-13.3</td>
<td>±2.6</td>
<td>-13.3</td>
</tr>
<tr>
<td>(B_{25}, B_{27}, B_{28})</td>
<td>10.0</td>
<td>±3.5</td>
<td>10.1</td>
<td>-8.9</td>
<td>±3.5</td>
<td>-9.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM(_1)(V)</th>
<th>BM(_1)(L)</th>
<th>BM(_1)(D)</th>
<th>BM(_0)(V=D)</th>
<th>BM(_2)(V)</th>
<th>BM(_2)(L)</th>
<th>BM(_2)(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{15}, B_{17}, B_{18})</td>
<td>-16.0</td>
<td>±24.2</td>
<td>6.2, -30.2</td>
<td>11.0</td>
<td>-23.3</td>
<td>±21.8</td>
<td>-33.8</td>
</tr>
<tr>
<td>(B_{19}, B_{23}, B_{24})</td>
<td>-30.1</td>
<td>±18.3</td>
<td>-36.3</td>
<td>16.5</td>
<td>-30.1</td>
<td>±18.3</td>
<td>-36.3</td>
</tr>
<tr>
<td>(B_{25}, B_{27}, B_{28})</td>
<td>-23.3</td>
<td>±21.8</td>
<td>-33.8</td>
<td>11.0</td>
<td>-16.0</td>
<td>±24.2</td>
<td>6.2, -30.2</td>
</tr>
</tbody>
</table>

Other Beams:

1. Beam \(B_{22}\) -
   Approximately designed as a simply supported beam under similar load as \(B_{20}\).
   \[\therefore \text{Maximum SF} \equiv 1.06 \times 7/2 = 3.71 \text{ k}\]
   and Maximum positive BM \[\equiv 1.06 \times 7^2/8 = 6.49 \text{ k'}\]

2. Edge Beam for \(S_{10}\) -
   Uniformly distributed load on \(S_{10} = 0.109 \text{ ksf}\)
   Uniformly distributed load on Edge Beam = 0.109\( \times 5'' = 0.55 \text{ k''}\)
   \[\therefore \text{Clear Span} = 13'' \Rightarrow V_{\text{max}} \equiv 0.55 \times (13)/2 = 3.6 \text{ k}; M^2 \equiv 0.55 \times (13)^2/10 = 9.3 \text{ k'}\]
4.2 Load Combination for Columns

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

<table>
<thead>
<tr>
<th>Columns</th>
<th>Frame</th>
<th>(V)</th>
<th>(L₃)</th>
<th>0.75(V+L₃)</th>
<th>0.75(V-L₃)</th>
<th>(L₃)</th>
<th>0.75(V+L₃)</th>
<th>0.75(V-L₃)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₁, C₁₇</td>
<td>2, 4</td>
<td>-84.0</td>
<td>-7.9</td>
<td>-68.9</td>
<td>0.0</td>
<td>-57.1</td>
<td>-10.2</td>
<td>-70.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7</td>
<td>24.1</td>
<td>22.4</td>
<td>2.6</td>
<td>13.8</td>
<td>2.6</td>
<td>4.3</td>
</tr>
<tr>
<td>C₂, C₁₈</td>
<td>2, 3</td>
<td>-130.8</td>
<td>-0.8</td>
<td>-98.7</td>
<td>0.0</td>
<td>-97.5</td>
<td>-5.4</td>
<td>-102.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-1.8</td>
<td>27.9</td>
<td>19.6</td>
<td>2.7</td>
<td>22.3</td>
<td>2.7</td>
<td>1.4</td>
</tr>
<tr>
<td>C₃, C₁₉</td>
<td>2, 4</td>
<td>-120.7</td>
<td>-1.2</td>
<td>-91.4</td>
<td>0.0</td>
<td>-89.6</td>
<td>-10.2</td>
<td>-98.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.1</td>
<td>28.3</td>
<td>21.6</td>
<td>2.6</td>
<td>21.2</td>
<td>2.6</td>
<td>0.1</td>
</tr>
<tr>
<td>C₄, C₂₀</td>
<td>2, 4</td>
<td>-76.7</td>
<td>9.9</td>
<td>-50.1</td>
<td>0.0</td>
<td>-65.0</td>
<td>-10.2</td>
<td>-65.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-4.4</td>
<td>24.7</td>
<td>15.2</td>
<td>2.6</td>
<td>21.8</td>
<td>2.6</td>
<td>3.3</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C₅, C₂₁</td>
<td>1, 4</td>
<td>-143.5</td>
<td>-5.9</td>
<td>-112.1</td>
<td>0.0</td>
<td>-103.2</td>
<td>2.4</td>
<td>-105.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.1</td>
<td>20.0</td>
<td>19.6</td>
<td>1.1</td>
<td>10.4</td>
<td>1.1</td>
<td>4.6</td>
</tr>
<tr>
<td>C₆, C₂₂</td>
<td>1, 3</td>
<td>-140.2</td>
<td>-13.0</td>
<td>-114.9</td>
<td>0.0</td>
<td>-95.4</td>
<td>-6.7</td>
<td>-110.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-3.7</td>
<td>24.6</td>
<td>15.7</td>
<td>-1.7</td>
<td>21.2</td>
<td>1.7</td>
<td>-2.8</td>
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<tr>
<td>C₇, C₂₃</td>
<td>1</td>
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<td>0.1</td>
<td>-43.3</td>
<td>0.0</td>
<td>-43.4</td>
<td>0.0</td>
<td>-43.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-0.1</td>
<td>24.2</td>
<td>18.1</td>
<td>0.0</td>
<td>18.1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>C₈, C₂₄</td>
<td>1, 4</td>
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<td>11.4</td>
<td>-118.1</td>
<td>0.0</td>
<td>-135.2</td>
<td>2.4</td>
<td>-124.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.9</td>
<td>24.9</td>
<td>20.1</td>
<td>1.1</td>
<td>17.3</td>
<td>1.1</td>
<td>1.4</td>
</tr>
<tr>
<td>C₉, C₂₅</td>
<td>1, 4</td>
<td>-134.5</td>
<td>7.4</td>
<td>-95.3</td>
<td>0.0</td>
<td>-106.4</td>
<td>1.7</td>
<td>-99.6</td>
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<tr>
<td></td>
<td></td>
<td>-4.6</td>
<td>20.6</td>
<td>12.0</td>
<td>-2.0</td>
<td>18.9</td>
<td>-2.0</td>
<td>-3.5</td>
</tr>
<tr>
<td>C₁₀</td>
<td>3</td>
<td>-28.2</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Besides, the design force on C₁₁ is assumed to be 3.71⁴; i.e., the SF at support of B₂₂.

In this work, only one size and reinforcements will be chosen for all the columns. For this purpose, the columns (C₈, C₁₅) 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 = 168.8⁴, Bending Moments BM₁ₓ = 2.7 k’, BM₁ᵧ = 1.9 k’.
2. Compressive Force = 118.1⁴, Bending Moments BM₁ₓ = 1.1 k’, BM₁ᵧ = 20.1 k’.
3. Compressive Force = 135.2⁴, Bending Moments BM₁ₓ = 1.1 k’, BM₁ᵧ = 17.3 k’.
4. Compressive Force = 124.8⁴, Bending Moments BM₁ₓ = 20.9 k’, BM₁ᵧ = 1.4 k’.
5. Compressive Force = 128.4⁴, Bending Moments BM₁ₓ = 18.8 k’, BM₁ᵧ = 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.

\[
\therefore \text{For 1 layer of rods, } d = 16 - 2.5 = 13.5'' \text{, } d' = 2.5''
\]

\[
M_c = Rbd'^2 = 0.223 \times 12 \times (13.5)^2/12 = 40.6 \text{ k}'
\]

The \(M_{\text{max}}\) 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 \((40.9 - 40.6 = 0.3 \text{ k}')\) is negligible and expected to be absorbed within the necessary reinforcements on the other side.

\[
\therefore A_s = \frac{M}{f_s j d} = \frac{M}{19.67}
\]

For T-beams (possible for positive moments),

\[
A_s = \frac{M}{f_s (d - t/2)} = \frac{M}{19.17}
\]

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

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

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

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

\[
\therefore S_{\text{max}} = \frac{d/2}{6.75^\prime}, \quad 12'' \text{ or } A_s/(0.0015b_w) = 0.22/(0.0015 \times 12) = 12.2^\prime \Rightarrow S_{\text{max}} = 6.75^\prime
\]

Spacing of #3 Stirrups, \(S = A_s f_s (V-V_c) = 0.22 \times 20 \times 13.5/(V-9.8) = 59.4/(V-9.8)
\]

\[= 143.5'', \text{ when } V = 10.2^k\]

\(\therefore\) The design is governed by \(S_{\text{max}} = 6.75''\)

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

\[
\begin{array}{cccccccc}
-35.3 & -36.2 & -30.1 & -22.7 & -22.5 & -28.1 & -30.2 & -30.5 \\
20.2 & 6.4 & 2.3 & 11.3 & 3.3 & 7.6 & 14.6 \\
\end{array}
\]

\(B_4, B_8\)

\(B_5, B_9\)

\(B_6, B_{10}\)

\(B_7, B_{11}\)

The flexural reinforcements (in\(^2\)) are

\[
\begin{array}{cccccccc}
-1.80 & -1.84 & -1.53 & -1.15 & -1.14 & -1.43 & -1.54 & -1.55 \\
1.05 & 0.33 & 0.12 & 0.59 & 0.59 & 0.17 & 0.40 & 0.76 \\
\end{array}
\]

\(B_4, B_8\)

\(B_5, B_9\)

\(B_6, B_{10}\)

\(B_7, B_{11}\)

The reinforcements are arranged as follows

1 #7 extra

2 #7 through

1 #5 extra

1 #7 extra

2 #5 through

1 #5 extra
Frame (2) \([B_{1,2,3}]\) and \([B_{12,13,14}]\):

The design moments \((k')\) are

\[
\begin{array}{cccc}
-37.8 & -40.9 & -35.0 & -33.5 \\
18.8 & 11.8 & 13.8 & \\
B_1, B_{12} & B_2, B_{13} & B_3, B_{14} & \\
\end{array}
\]

The flexural reinforcements \((in^2)\) are

\[
\begin{array}{cccc}
-1.92 & -2.08 & -1.78 & -1.70 \\
0.98 & 0.61 & 0.72 & 0.19 \\
B_1, B_{12} & B_2, B_{13} & B_3, B_{14} & \\
\end{array}
\]

The reinforcements are arranged as follows

2 #6 extra

2 #7 through

2 #6 extra

2 #6 through
Frame (3) \([B_{16,20,21,26}]\):
The design moments \((k')\) are

\[
\begin{array}{cccccccc}
-23.0 & -22.9 & -17.6 & -13.7 & -17.6 & -22.9 & -23.0 \\
12.4 & 5.5 & 0.7 & 8.2 & 0.7 & 5.5 & 12.4 \\
\end{array}
\]

\(B_{16}\) \(B_{20}\) \(B_{21}\) \(B_{26}\)

The flexural reinforcements \((\text{in}^3)\) are

\[
\begin{array}{cccccccc}
-1.17 & -1.16 & -0.89 & -0.70 & -0.70 & -0.89 & -1.16 & -1.17 \\
0.65 & 0.29 & 0.40 & 0.43 & 0.43 & 0.40 & 0.29 & 0.65 \\
\end{array}
\]

\(B_{16}\) \(B_{20}\) \(B_{21}\) \(B_{26}\)

The reinforcements are arranged as follows

1 #7 extra 2 #5 through 1 #7 extra 2 #5 through
Frame (4) \([B_{15,17,18}], [B_{17,23,27}]\) and \([B_{25,27,28}]\):

The design moments (\(k'\)) are

\[
\begin{array}{cccccc}
-30.2 & -33.8 & -36.3 & -36.3 & -33.8 & -30.2 \\
6.2 & 11.0 & 16.5 & 11.0 & 6.2 & \\
\end{array}
\]

\(B_{15}, B_{17}, B_{25} \quad B_{17}, B_{23}, B_{27} \quad B_{18}, B_{27}, B_{28}\)

The flexural reinforcements (in\(^2\)) are

\[
\begin{array}{cccccc}
-1.54 & -1.72 & -1.85 & -1.85 & -1.72 & -1.54 \\
0.32 & 0.57 & 0.86 & 0.57 & 0.32 & \\
\end{array}
\]

\(B_{15}, B_{17}, B_{25} \quad B_{17}, B_{23}, B_{27} \quad B_{18}, B_{27}, B_{28}\)

The reinforcements are arranged as follows

1 #7 extra \quad 2 #7 through \quad 1 #7 extra

1 #5 extra \quad 2 #5 through
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

\[ 175 = 0.85 (0.25f'_c A_g + A_s f_s) = 0.85 A_g (0.25 f'_c + p_g f_s) \]

\[ \therefore p_g = 0.03 \Rightarrow 175 = 0.85 A_g (0.25 \times 3 + 0.03 \times 20) \]

\[ \Rightarrow A_g = 152.21 \text{ in}^2 \]

\[ \therefore \text{Choose (12"x13") section with 10 #6 bars and #3 ties @12" c/c.} \]

\[ \therefore p_g = 4.4/(12\times13) = 0.028, m = f\text{/0.85f'} = 40/2.55 = 15.69 \]

\[ \therefore \text{The column has two axes with different dimensions and steel arrangements} \]

For the strong axis, load eccentricity for balanced condition,

\[ e_{bx} = (0.67 p_g m + 0.17) d = (0.67 \times 0.028 \times 15.69 + 0.17) \times (13-2.5) = 4.90'' = 0.41' \]

For the weak axis, load eccentricity for balanced condition,

\[ e_{by} = (0.67 p_g m + 0.17) d = (0.67 \times 0.028 \times 15.69 + 0.17) \times (12-2.5) = 4.43'' = 0.37' \]

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

\[ S_x = (1/c) [bh^3/12 + (2n-1)\Sigma A_n (h_n-h/2)^3] \]

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

\[ S_y = (1/6.0) [13\times12^3/12 + (2\times9-1)\times2 \{1.76\times(3.5)^2 + 0.44\times(0)^2\}] = 434.2 \text{ in}^3 \]
For condition (1)
\[ f_a = \frac{P}{A_e} = 168.8/156 = 1.08 \text{ ksi}, \quad F_a = 0.34(1+p_{gm})f'_c = 0.34(1+0.028\times15.69) \times 3 = 1.47 \text{ ksi}, \]
\[ f_{bx} = \frac{M_x}{S_x}, \quad F_{bx} = 0.45f'_c = 1.35 \text{ ksi}, \quad f_{by} = \frac{M_y}{S_y}, \quad F_{by} = 0.45f'_c = 1.35 \text{ ksi} \]
\[ \therefore \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} = \frac{1.08}{1.47} + \frac{2.7\times12/453.1}{1.35} + \frac{1.9\times12/434.2}{1.35} = 0.74 + 0.05 + 0.04 = 0.83 < 1 \text{ (OK)} \]
\[ \therefore \text{Condition (1) is satisfied.} \]

For condition (2)
\[ (118.1/156)/1.47 + (1.1\times12/453.1)/1.35 + (20.1\times12/434.2)/1.35 = 0.51 + 0.02 + 0.41 = 0.95 < 1 \text{ (OK)} \]
\[ \therefore \text{Condition (2) is satisfied.} \]

For condition (3)
\[ (135.2/156)/1.47 + (1.1\times12/453.1)/1.35 + (17.3\times12/434.2)/1.35 = 0.59 + 0.02 + 0.35 = 0.96 < 1 \text{ (OK)} \]
\[ \therefore \text{Condition (3) is satisfied.} \]

For condition (4)
\[ (124.8/156)/1.47 + (20.9\times12/453.1)/1.35 + (1.4\times12/434.2)/1.35 = 0.54 + 0.41 + 0.03 = 0.98 < 1 \text{ (OK)} \]
\[ \therefore \text{Condition (4) is satisfied.} \]

For condition (5)
\[ (128.4/156)/1.47 + (18.8\times12/453.1)/1.35 + (1.4\times12/434.2)/1.35 = 0.56 + 0.37 + 0.03 = 0.96 < 1 \text{ (OK)} \]
\[ \therefore \text{Condition (5) is satisfied.} \]

\[ \therefore \text{The assumed section is chosen for all the columns.} \]
7. Design of Footings

The following sample footings will be designed.

1. Individual Footing: A footing under C₅ will be designed for column load 143.5 k (plus footing weight).
2. Combined Footing: A footing under C₆, C₇ and C₈ 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 ($N_b$) and depth ($N_q$). These are functions of the angle of friction $\phi$. With a factor of safety 3.0, $N_c$ approximately equals to 6.0 and neglecting $N_b$ and $N_q$, the unconfined compression strength equals to the allowable bearing capacity].

Individual Footing under C₅:

Column load = 143.5 k ⇒ Footing load 143.5 k \times 1.1 = 157.9 k

∴ Footing area = 157.9 k / 2.0 ksf = 78.9 ft² \approx 9.00' \times 9.00'

∴ Effective bearing pressure = 143.5 / (9.00)² = 1.77 ksf

Column size = 12" x 13", Effective depth of footing = d

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

Punching Shear strength = $2\sqrt{f'_c} = 2\sqrt{3000} = 110$ psi = 0.110 ksi

∴ $0.110 \times 4 \times (12.5 + d) \times (13 + d)/(12)²$

⇒ $d² + 12.5 d = 327.5 - (12 + d) \times (13 + d)/35.62 \Rightarrow d = 12.09''$

∴ 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 ksf = 8.68 ksf

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

∴ $8.68 \times 1 \times d = 1.77 \times (4.0 - d) \Rightarrow d = 1.77 \times 4.0/(8.68 + 1.77) = 0.68' = 8.14'' < 12.5''$, OK.

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

∴ Depth required by $M$ is = $\sqrt{M/R_b} = \sqrt{(127.56/(0.223 \times 9.00))} = 7.97'' < 12.5''$, OK.

∴ $A_s = M/f_yd = 127.56 \times 12/(20 \times 0.874 \times 12.5) = 7.01$ in²

Minimum reinforcement = $(0.2/f_y)bd = (0.2/40) \times 9.00 \times 12 \times 12.5 = 6.75$ in² < $A_s$

∴ Provide 12 #7 bars in each direction.
Combined Footing under $C_6-C_7-C_8$:

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

\[ \text{Resultant} = 140.2 + 57.8 + 168.8 = 366.8^k \]

\[ \text{Footing area} = 366.8^k \times 1.1/2.0 \text{ ksf} = 201.74 \text{ ft}^2 = 20' \times 10' \]

Distance of resultant from the first column

\[ = (140.2 \times 0 + 57.8 \times 7 + 168.8 \times 14)/366.8 = 7.55' \]

\[ \text{Effective bearing pressure} = 366.8/(20 \times 10) = 1.834 \text{ ksf}; \text{ i.e., } 18.34 \text{ k/} \text{f} \text{ along the length.} \]

\[ \text{Considering the column loads to be uniformly distributed over the column width} = 1' \text{, the footing loads and corresponding SFD and BMD are shown below} \]
Column size = 12"×13", Effective depth of footing = d

\[ \text{Punching Shear area } A_p = 2 \times (12 + d + 13 + d) \, d = 4 \times (12.5 + d) \, d \]

Punching Shear strength = \(2\sqrt{f_{ct}} = 110 \, \text{psi} = 0.110 \, \text{ksi}\)

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

\[ \Rightarrow d^2 + 12.5 \, d = 445.5 - (12 + d)(13 + d)/34.57 \Rightarrow d = 15.55" \]

\[ \therefore \text{Take footing thickness, } t = 20" \Rightarrow d = 16" \]

Flexural Shear strength = \(1.1 \sqrt{f_{ct}} = 60.2 \, \text{psi} = 0.0602 \, \text{ksi} = 8.68 \, \text{ksf}\)

Maximum shear force (according to SFD) = 94.4 k

\[ \therefore \text{Maximum flexural shear force} = 94.4 - 18.25d \, [d \text{ is in ft}] \]

\[ \therefore 8.68 \times 10 \times d = 94.4 - 18.25d \Rightarrow d = 0.899' = 10.78'' < 16", \text{ OK}. \]

Maximum bending moment (according to BMD) = 192.4 k

\[ \therefore \text{Depth required by } M \text{ is } = \sqrt{(M/Rb)} = \sqrt{192.4/(0.223 \times 10)} = 9.29'' < 16", \text{ OK} \]

\[ \therefore A_s^{(+)} = M/f_{yd} = 192.4 \times 12/(20 \times 0.874 \times 16) = 192.4/23.3 = 8.26 \, \text{in}^2, \text{ at top} \]

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

Minimum reinforcement = \((0.2/f_y)bd = (0.2/40) \times 10 \times 12 \times 16 = 9.60 \, \text{in}^2 > A_s \)

\[ \therefore \text{Provide 16 #7 bars at top and bottom}. \]

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

Load per unit length under \(C_8 = 168.8/10 = 16.88 \, \text{k/ft} \)

\[ \therefore \text{Maximum bending moment} = 16.88 \times [(10 - 13/12)/2] = 167.76 \, \text{k/ft} \]

\[ \therefore \text{Depth required by } M \text{ is } = \sqrt{(M/Rb)} = \sqrt{167.76 \times 12/(0.223 \times 28)} = 17.96'' < 16". \]

\[ \therefore \text{Provide } d = 18", \text{ increase the thickness to 22" and put transverse rods at bottom} \]

\[ \therefore A_s^{(+)} = 167.76 \times 12/(20 \times 0.874 \times 18) = 6.40 \, \text{in}^2, \text{ at bottom} [\text{Note: } d = 18" \text{ here}] \]

Similarly, \(A_s^{(+)} \text{ under } C_6 = 6.40 \times 140.2/168.8 = 5.31 \, \text{in}^2 \)

and \(A_s^{(-)} \text{ under } C_7 = 6.40 \times 57.8/168.8 = 2.19 \, \text{in}^2 \)

\[ \therefore A_{s(min)} = (0.2/f_y)bd = (0.2/40) \times 28 \times 18 = 2.52 \, \text{in}^2 < A_s^{(+)} \text{ for all columns except } C_7 \]

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

Elsewhere, provide \((0.2/f_y)bd = 1.08 \, \text{in}^2/\text{ft} \text{ (i.e., } \#7 \text{ } @ \text{ 6.5" c/c}) \).
Details of Individual Footing

12 # 7 bars

Details of Combined Footing

9 # 7 bars

5 # 7 bars

11 # 7 bars

# 7@ 6.5" c/c

16 # 7 bars at top & bottom
5. Design of Beams (USD)

Material Properties: \( f'_c = 3 \text{ ksi}, f_y = 40 \text{ ksi}, \ldots f_c = 0.85f'_c = 2.55 \text{ ksi} \)

**Flexural Design**

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

The section shown is chosen for all the beams.

\[
\therefore \ 	ext{For 1 layer of rods, } d = 16 - 2.5 = 13.5'' \text{, } d' = 2.5''
\]

\[
M_c = R_u b d'^2 = 0.781 \times 12 \times (13.5)^2/12 = 142.3 \text{ k}'
\]

The \( M_{\text{max}} \) is 61.4 k' (in \( B_1 \) and \( B_{12} \))

\[
\Rightarrow \ 	ext{All the beams are Singly Reinforced.}
\]

\[
A_r = (f'_c/f_y) \left[ 1 - \sqrt{\left[ 1 - 2M/(f'_c b d'^2) \right]} \right] b d
\]

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

\[
= 10.33 \times \left[ 1 - \sqrt{1 - M/209.1} \right]
\]

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

(i) \( 16t + b_w = 76'' \), (ii) Simple Span/4 \( \geq 0.6L \times 12/4 = 1.8L('') \), (iii) \( c/c \)

\[
\Rightarrow \ 	ext{Since } L \text{ varies between } 7'\sim18' \text{ and } c/c \text{ between } 16'\sim18' , \text{ it is conservative and simplified to assume } b_f = 12'' ; \text{ i.e., calculations are similar to rectangular beam.}
\]

**Shear Design**

\[
V_c = 2\sqrt{(f'_c)b_w d} = 2\sqrt{(3000)\times 12 \times 13.5/1000} = 17.7^k
\]

\[
V_{c1} = 6\sqrt{(f'_c)b_w d} = 53.2^k, \quad V_{c2} = 10\sqrt{(f'_c) b_w d} = 88.7^k
\]

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

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

\[
\therefore V_n > V_c , \text{ but } < V_{c1} \text{ and } V_{c2}.
\]

\[
\therefore S_{\text{max}} = d/2 = 6.75'' , \ 24'' \text{ or } A_r f_y/50b_w = 0.22\times40000/(50\times12) = 14.7'' \Rightarrow S_{\text{max}} = 6.75''
\]

Spacing of #3 Stirrups, \( S = A_r f_y/(V_n - V_c) = 0.22\times40\times13.5/(V_n - 17.7) = 118.8/(V_n - 17.7)
\]

\[
= 169.7'', \text{ when } V = 18.4^k
\]

\[
\therefore \ 	ext{The design is governed by } S_{\text{max}} = 6.75''
\]

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

\[
\begin{array}{cccccccc}
-53.0 & -54.3 & -45.1 & -34.1 & -33.8 & -42.1 & -45.3 & -45.8 \\
30.3 & 9.6 & 3.5 & 33.5 & 16.9 & 5.0 & 11.4 & 21.9 \\
\end{array}
\]

\(B_4, B_8\) \(B_5, B_9\) \(B_6, B_{10}\) \(B_7, B_{11}\)

The flexural reinforcements \((in^2)\) are

\[
\begin{array}{cccccccc}
-1.40 & -1.44 & -1.18 & -0.88 & -0.87 & -1.10 & -1.19 & -1.20 \\
0.78 & 0.24 & 0.09 & 0.43 & 0.43 & 0.12 & 0.29 & 0.56 \\
\end{array}
\]

\(B_4, B_8\) \(B_5, B_9\) \(B_6, B_{10}\) \(B_7, B_{11}\)

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 \(f_c = 0.85 f_{c}'\)

\[
265 = 0.80\phi (0.85f_{c}'A_c + A_s f_y) = 0.80\phi A_g (f_c + p (f_y - f_c))
\]

\[
\therefore p = 0.03 \Rightarrow 265 = 0.80 \times 0.70 A_g (0.85 \times 3 + 0.03 \times 37.45)
\]

\[
\Rightarrow A_g = 128.82 \text{ in}^2
\]

\[
\therefore \text{Choose (11"x12") section with 8 #6 bars and 3 ties @11" c/c.}
\]

\[
\therefore p = 3.52/(11 \times 12) = 0.027, \mu = f_y/0.85f_{c}' = 40/2.55 = 15.69
\]

Use interaction diagram for rectangular columns with

\(\gamma = 0.60, \text{ and } p_\mu = 0.027 \times 15.69 = 0.42,\)

\(\phi f_{c}'bh = 0.7 \times 3 \times 11 \times 12 = 277.2 k, \phi f_{c}'hb^2 = 3326.4 k'' = 277.2 k', \phi f_{c}'hb^2 = 254.1 k'\)

For condition (1)

\(k = P/(\phi f_{c}'bh) = 253.2/277.2 = 0.91\)

\(k_0 = 1.18\)

\(ke_x/h = M_{y}/(\phi f_{c}'bh^2) = 4.1/277.2 = 0.015 \Rightarrow k_x = 1.15\)

\(ke_y/h = M_{x}/(\phi f_{c}'hb^2) = 2.8/254.1 = 0.011 \Rightarrow k_y = 1.16\)

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

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

\(\therefore \text{Condition (1) is satisfied.}\)
For condition (2)
\[ k = \frac{P}{(\phi f_c' bh)} = \frac{177.1}{277.2} = 0.64 \]
\[ k_0 = 1.18; \text{ke}_x/h = 0.006 \Rightarrow k_x = 1.17; \text{ke}_y/h = 0.118 \Rightarrow k_y = 0.84 \]
\[ \therefore \text{Reciprocal method} \Rightarrow \frac{1}{k_{xy}} = \frac{1}{k_x} + \frac{1}{k_y} - \frac{1}{k_0} = 1/1.17 + 1/0.84 - 1/1.18 \]
\[ \Rightarrow k_{xy} = 0.83, \text{ which is } > k (= 0.64) \text{ (OK)} \]
\[ \therefore \text{Condition (2) is satisfied.} \]

For condition (3)
\[ k = \frac{P}{(\phi f_c' bh)} = \frac{202.8}{277.2} = 0.73 \]
\[ k_0 = 1.18; \text{ke}_x/h = 0.006 \Rightarrow k_x = 1.17; \text{ke}_y/h = 0.102 \Rightarrow k_y = 0.90 \]
\[ \therefore \text{Reciprocal method} \Rightarrow \frac{1}{k_{xy}} = \frac{1}{k_x} + \frac{1}{k_y} - \frac{1}{k_0} = 1/1.17 + 1/0.90 - 1/1.18 \]
\[ \Rightarrow k_{xy} = 0.89, \text{ which is } > k (= 0.73) \text{ (OK)} \]
\[ \therefore \text{Condition (3) is satisfied.} \]

For condition (4)
\[ k = \frac{P}{(\phi f_c' bh)} = \frac{187.2}{277.2} = 0.68 \]
\[ k_0 = 1.18; \text{ke}_x/h = 0.113 \Rightarrow k_x = 0.87; \text{ke}_y/h = 0.008 \Rightarrow k_y = 1.17 \]
\[ \therefore \text{Reciprocal method} \Rightarrow \frac{1}{k_{xy}} = \frac{1}{k_x} + \frac{1}{k_y} - \frac{1}{k_0} = 1/0.87 + 1/1.17 - 1/1.18 \]
\[ \Rightarrow k_{xy} = 0.86, \text{ which is } > k (= 0.68) \text{ (OK)} \]
\[ \therefore \text{Condition (4) is satisfied.} \]

For condition (5)
\[ k = \frac{P}{(\phi f_c' bh)} = \frac{192.6}{277.2} = 0.69 \]
\[ k_0 = 1.18; \text{ke}_x/h = 0.102 \Rightarrow k_x = 0.90; \text{ke}_y/h = 0.008 \Rightarrow k_y = 1.17 \]
\[ \therefore \text{Reciprocal method} \Rightarrow \frac{1}{k_{xy}} = \frac{1}{k_x} + \frac{1}{k_y} - \frac{1}{k_0} = 1/0.90 + 1/1.17 - 1/1.18 \]
\[ \Rightarrow k_{xy} = 0.89, \text{ which is } > k (= 0.69) \text{ (OK)} \]
\[ \therefore \text{Condition (5) is satisfied.} \]

\[ \therefore \text{The assumed section is chosen for all the columns.} \]
7. Design of Footings (USD)

The following sample footings will be designed.

1. Individual Footing: A footing under C₅ will be designed for column load 215.3 k (plus footing weight).

2. Combined Footing: A footing under C₆, C₇ and C₈ 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 (Nᵥ), foundation width (Nₜ) and depth (N₉). These are functions of the angle of friction φ. With a factor of safety 3.0, Nᵥ approximately equals to 6.0 and neglecting Nₜ and N₉, the unconfined compression strength equals to the allowable bearing capacity].

Individual Footing under C₅:

Column load = 215.3 k \implies \text{Footing load} = 215.3 k \times 1.1 = 236.8 k

\therefore \text{Effective area (based on working stresses)} = 157.9 k/2.0 \text{ ksf} = 78.9 \text{ ft}^2 \approx 9.00' \times 9.00'

\therefore \text{Effective bearing pressure} = 215.3/(9.00)^2 = 2.66 \text{ ksf}

Column size = 11''\times 12'', \text{Effective depth of footing} = d

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

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

\therefore 0.186 \left[ 4 \times (11.5 + d) \times d \right] = 215.3 - 2.66 \times (11 + d) \times (12 + d)/(12)^2

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

\therefore \text{Take footing thickness, } t = 15.5'' \Rightarrow d = 11.5''

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

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

\therefore 13.41 \times 1 \times d = 2.66 \times (4.04-d) \Rightarrow d = 2.66 \times 0.04/(13.41+2.66) = 0.67'' < 11.5'', \text{ OK.}

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

\therefore \text{Depth required by } M \text{ is } = \sqrt{(M/R_u b)} = \sqrt{[195.34/(0.781 \times 9.00)]} = 5.27'' < 11.5'', \text{ OK.}

\therefore A_s = (f_y/f_c)(1-\sqrt{[1-2M/(\phi f_c b d^2)]})b d = 5.88 \text{ in}^2

\text{Minimum reinforcement} = (0.2/f_c)b d = (0.2/40) \times 9.00 \times 12 \times 11.5 = 6.21 \text{ in}^2 > A_s

\therefore \text{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\(^k\)

\[ \text{:. Resultant} = 210.3 + 86.7 + 253.2 = 550.2^k \]

\[ \text{:. Footing area (based on working stresses)} = 366.8^k \times 1.1/2.0 \text{ ksf} = 201.74 \text{ ft}^2 \approx 20' \times 10' \]

Distance of resultant from the first column

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

\[ \text{:. Effective bearing pressure} = 550.2/(20 \times 10) = 2.751 \text{ ksf}; \text{ i.e.}, 27.51 \text{ k/} \text{ along the length}. \]

\[ \text{:. Considering the column loads to be uniformly distributed over the column width = 1'}, \text{ the footing loads and corresponding SFD and BMD are shown below} \]

![Diagram showing footing loads and corresponding SFD and BMD](image-url)
Column size = 11”x12”, Effective depth of footing = d

\[ A_p = 2 \times (11 + d + 12 + d) d = 4 \times (11.5 + d) d \]

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

\[ P = 2 \times 0.85\sqrt{(3000)} = 186 \text{ psi} = 0.186 \text{ ksi} \]

\[ 0.186 \times 4 \times (11.5 + d) d = 253.2 - 2.751 \times (11 + d) \times (12 + d)/(12)^2 \]

\[ d^2 + 11.5 d = 339.9 - (11 + d) \times (12 + d)/38.99 \Rightarrow d = 12.48” \]

\[ \Rightarrow \text{Take footing thickness, } t = 16.5” \Rightarrow d = 12.5” \]

Flexural Shear strength = \( 2 \times 0.85\sqrt{(3000)} = 93.1 \text{ psi} = 13.41 \text{ ksf} \)

Maximum shear force (according to SFD) = 141.6 \( k' \)

\[ \Rightarrow \text{Maximum flexural shear force } = 141.6 - 27.51d \quad [d \text{ is in ft}] \]

\[ 13.41 \times d = 141.6 - 27.51d \Rightarrow d = 0.876” < 12.5” \text{, OK.} \]

Maximum bending moment (according to BMD) = 288.6 \( k' \)

\[ \Rightarrow \text{Depth required by } M \text{ is } = \sqrt{M/R_{ub}} = \sqrt{288.6/(0.781 \times 10)} = 6.08” < 12.5” \text{, OK.} \]

\[ A_{s(-)} = (f_c/f_y)[1-\sqrt{1-2M'/bf_cbd^2}]bd = 8.03 \text{ in}^2 \]

\[ A_{s(+)} = (f_c/f_y)[1-\sqrt{1-2M'/bf_cbd^2}]bd = 3.47 \text{ in}^2 \text{[using } M' = 127.8 \text{ k']} \]

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

\[ \Rightarrow \text{Provide 13 #7 bars at top and bottom.} \]

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

Load per unit length under C_6 = 253.2/10 = 25.32 \( k' \)

\[ \Rightarrow \text{Maximum bending moment } = 25.32 \times [(10-12/12)/2]^2/2 = 256.4 \text{ k’} \]

\[ \Rightarrow \text{Depth required by } M \text{ is } = \sqrt{M/R_{ub}} = \sqrt{256.4 \times 12/(0.781 \times 23.5)} = 12.95” > 11.5”. \]

\[ \Rightarrow \text{Provide } d = 13.5”, \text{ increase the thickness to 17.5” and put transverse rods at bottom} \]

\[ A_{s(+)} = (f_c/f_y)[1-\sqrt{1-2M'/bf_cbd^2}]bd = 7.86 \text{ in}^2 \quad [\text{Note: } d = 13.5” \text{ here}] \]

Similarly, \( A_{s(+)} \) under C_6 = 6.21 \text{ in}^2 \text{ and } A_{s(+)} \text{ under C_7} = 2.30 \text{ in}^2 \]

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

\[ \Rightarrow \text{Provide 11#7, 4#7 and 13#7 bars at bottom within the width of the transverse beams (23.5”) under each column (in placing 13 #7 bars in 23.5”, be careful about minimum spacing).} \]

Elsewhere, provide (0.2/f_y)bd = 0.81 \text{ in}^2/\text{ft} (i.e., #7 @ 9” c/c).
Details of Individual Footing

11 # 7 bars

9.00'

9.00'

Details of Combined Footing

11 # 7 bars

# 7@ 9” c/c

13 # 7 bars at top & bottom

15.5”

20’
# Seismic Detailing of RC Structures

## 1. Materials

<table>
<thead>
<tr>
<th></th>
<th>Specification</th>
<th>Possible Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td><strong>$f'_{c} \geq 20$ Mpa ($\approx 3$ ksi) for 3-storied or taller buildings</strong></td>
<td>Weak concretes have low shear and bond strengths and cannot take full advantage of subsequent design provisions</td>
</tr>
<tr>
<td>Steel</td>
<td><strong>$f_{y} \leq 415$ Mpa ($\approx 60$ ksi), preferably $\leq 250$ Mpa ($\approx 36$ ksi)</strong></td>
<td>Lower strength steels have (a) a long yield region, (b) greater ductility, (c) greater $f_{ult}/f_{y}$ ratio</td>
</tr>
</tbody>
</table>

## 2. Flexural Members (members whose factored axial stress $\leq f'_{c}/10$)

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

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

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

<table>
<thead>
<tr>
<th>Specification</th>
<th>Possible Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td></td>
</tr>
<tr>
<td>b_c/h_c ≥ 0.4</td>
<td>To ensure lateral stability and improve torsional resistance</td>
</tr>
<tr>
<td>b_c ≥ 12&quot;</td>
<td>To avoid (a) slender columns, (b) column failure before beams</td>
</tr>
<tr>
<td>Lap splices are allowed only for ≤ 50% of bars, only where stirrups are</td>
<td>Closely spaced stirrups are necessary within lap lengths because of the possibility of loss of concrete cover</td>
</tr>
<tr>
<td>provided @≤ b_c/4 or 4&quot;</td>
<td></td>
</tr>
<tr>
<td>Lap splice lengths ≥ L_d</td>
<td></td>
</tr>
<tr>
<td>and only allowed in the center half of columns</td>
<td>Lap splices are not reliable under cyclic loading into the inelastic range</td>
</tr>
<tr>
<td>0.01 ≤ η ≤ 0.06</td>
<td></td>
</tr>
<tr>
<td>∑M_c,ult ≥ 1.2 ∑M_b,ult at joint</td>
<td>To (a) ensure effectiveness and (b) avoid congestion of longitudinal bars</td>
</tr>
<tr>
<td>Transverse reinforcement</td>
<td>To obtain “strong column weak beam condition” to avoid column failure before beams</td>
</tr>
<tr>
<td>Transverse reinforcement must consist of closed spirals or rectangular/</td>
<td></td>
</tr>
<tr>
<td>circular hoops with 135° hooks with 10d, (≥ 3&quot;) extensions</td>
<td></td>
</tr>
<tr>
<td>Parallel legs of rectangular hoops must be spaced @ ≤ 12&quot; c/c</td>
<td></td>
</tr>
<tr>
<td>Spacing of hoops within L_0 (≥ d_c, h_c/6, 18&quot;) at each end of column must</td>
<td>To (a) provide resistance to shear, (b) confine concrete to improve ductility, (c) prevent buckling of longitudinal compression bars</td>
</tr>
</tbody>
</table>
### 3. Axial Members (continued)

<table>
<thead>
<tr>
<th>Transverse Reinforcement</th>
<th>Specification</th>
<th>Possible Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear force is the maximum of (a) shear force from analysis, (b) shear force required for flexural yielding of joints</td>
<td>It is desirable that the columns should yield in flexure before failure in shear</td>
</tr>
<tr>
<td></td>
<td>Special confining reinforcement (i.e., , ) should extend at least into any footing</td>
<td>To provide resistance to the very high axial loads and flexural demands at the base</td>
</tr>
<tr>
<td></td>
<td>Special confining reinforcement (i.e., , ) should be provided over the entire height of columns supporting discontinued stiff members and extend into the member</td>
<td>Discontinued stiff members (e.g., shear walls, masonry walls, bracings, mezzanine floors) may develop significant forces and considerable inelastic response</td>
</tr>
<tr>
<td></td>
<td>For special confinement, area of circular spirals , of rectangular hoops ,</td>
<td>To ensure load carrying capacity up to concrete spalling, taking into consideration the greater effectiveness of circular spirals compared to rectangular hoops. It also ensures toughness and ductility of columns</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Possible Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 4. Joints of Frames

<table>
<thead>
<tr>
<th>Transverse Reinforcement</th>
<th>Specification</th>
<th>Possible Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Special confining reinforcement (i.e., , ) should extend through the joint</td>
<td>To provide resistance to the shear force transmitted by framing members and improve the bond between steel and concrete within the joint</td>
</tr>
<tr>
<td></td>
<td>, through joint with beams of width</td>
<td>Some confinement is provided by the beams framing into the vertical faces of the joint</td>
</tr>
</tbody>
</table>
Seismic Detailing of a Typical Frame

Original Design of Frame (3)

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF&lt;sub&gt;1&lt;/sub&gt; (D)</th>
<th>SF&lt;sub&gt;2&lt;/sub&gt; (D)</th>
<th>BM&lt;sub&gt;i&lt;/sub&gt;(D)</th>
<th>BM&lt;sub&gt;0&lt;/sub&gt;(V=D)</th>
<th>BM&lt;sub&gt;t&lt;/sub&gt;(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B&lt;sub&gt;16&lt;/sub&gt;</td>
<td>9.3</td>
<td>-9.6</td>
<td>-23.0</td>
<td>12.4</td>
<td>-22.9</td>
</tr>
<tr>
<td>B&lt;sub&gt;20&lt;/sub&gt;</td>
<td>6.5</td>
<td>-5.6</td>
<td>5.5, -17.6</td>
<td>0.7</td>
<td>8.2, -13.7</td>
</tr>
</tbody>
</table>

The reinforcements are arranged as follows

- **Section a-a**: All #5 bars
- **Section b-b**: Extra #7 bar
- **Section c-c**: 10 #6 bars
- **Section d-d**: #3 ties @ 12" c/c
- **Section e-e**: 4"

The column section (with reinforcements) and the beam + slab section (without reinforcement) are shown with appropriate dimensions and notations.

Beam Sections (with reinforcements) with labels a, b, c, d, e.
1. Materials

<table>
<thead>
<tr>
<th></th>
<th>Specification</th>
<th>Design Condition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$f'_c \geq 20$ Mpa ($\approx 3$ ksi) for 3-storied or taller buildings</td>
<td>$f'_c = 3$ ksi</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>$f_y \leq 415$ Mpa ($\approx 60$ ksi), preferably $\leq 250$ Mpa ($\approx 36$ ksi)</td>
<td>$f_y = 40$ ksi</td>
<td>OK</td>
</tr>
</tbody>
</table>

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

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

<table>
<thead>
<tr>
<th>Specification</th>
<th>Design Condition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web reinforcements must consist of closed vertical stirrups with 135° hooks, 10dᵣ ≥ 3” extensions</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>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</td>
<td>Design shear force is taken only from analysis</td>
<td>Needs to be checked</td>
</tr>
<tr>
<td>Spacing of hoops within 2d (= 27”), beginning at ≤ 2”, at either end of a beam must be ≤ d/4, 8dᵣ (= 3.38”, 5”) elsewhere Sᵣ ≤ d/2 (= 6.75”)</td>
<td>Sᵣ = 6.75” c/c (= d/2) throughout the beams</td>
<td>Not OK</td>
</tr>
</tbody>
</table>

3. Axial Members (members whose factored axial stress ≥ f’c/10)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Design Condition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td></td>
<td></td>
</tr>
<tr>
<td>bₑ/hₑ ≥ 0.4</td>
<td>bₑ/hₑ = 12/13 = 0.92</td>
<td>OK</td>
</tr>
<tr>
<td>bₑ ≥ 12”</td>
<td>bₑ = 12”</td>
<td>OK</td>
</tr>
<tr>
<td>Lap splices are allowed only for ≤ 50% of bars, only where stirrups are provided @ bᵣ/4 or 4”</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>Lap splice lengths ≥ Lₑ and only allowed in the center half of columns</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>0.01 ≤ ρₑ ≤ 0.06</td>
<td>Aᵣ = 4.40 in²</td>
<td>OK</td>
</tr>
<tr>
<td>∑Mₑ,alt ≥ 1.2 ∑Mₐ,alt at joint</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stirrups must be closed rectangular/ circular hoops with 135° hooks with 10dᵣ, 4” extensions</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>Parallel legs of rectangular hoops must be spaced @ ≤ 12” c/c</td>
<td>Parallel legs of rectangular hoops spaced @ ≤ 10” c/c</td>
<td>OK</td>
</tr>
<tr>
<td>Hoop spacing within Lₑ ≥ dₑ, Hₑ/6, 18” (= 10”, 18”, 18”) at each end of column ≤ bᵣ/4 (= 3”), 4”; else Sᵣ ≤ bᵣ/2 (= 6”)</td>
<td>Sᵣ = 12” c/c throughout the columns</td>
<td>Not OK</td>
</tr>
</tbody>
</table>
3. Axial Members (continued)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Design Condition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Reinforcement&lt;br&gt;Design shear force is the maximum of (a) shear force from analysis, (b) shear force required for flexural yielding of joints</td>
<td>Design shear force is taken only from analysis</td>
<td>Needs to be checked</td>
</tr>
<tr>
<td>Special confining reinforcement (i.e., $S_t \leq 3''$) should extend at least 12” into any footing</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>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</td>
<td>Column supports no particular stiff member, but soft first storey</td>
<td>Special confining reinforcement (i.e., $S_t \leq 3''$) over the entire height of ground floor columns</td>
</tr>
<tr>
<td>For special confinement, area of circular spirals $\geq 0.11 S_d (f_c/f_y)(A_g/A_c - 1)$, of rectangular hoops $\geq 0.3 S_d (f_c/f_y)(A_g/A_c - 1)$ $[= 0.3 \times 3 \times 9.5 \times 0.25 (3/40) (156/90 - 1) = 0.47 \text{ in}^2]$</td>
<td>No special confinement provided and 0.22 in$^2$ hoop area provided @ 12” c/c</td>
<td>Atleast 2-legged #4 or 4-legged #3 bars needed as special confinement</td>
</tr>
</tbody>
</table>

4. Joints of Frames

<table>
<thead>
<tr>
<th>Specification</th>
<th>Design Condition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Reinforcement&lt;br&gt;Special confining reinforcement (i.e., $S_t \leq 3''$) should extend through the joint</td>
<td>Not specified in design</td>
<td>Needs to be specified</td>
</tr>
<tr>
<td>$S_t \leq b_c/2$ ( = 6”) and 6” through joint with beams of width $b \geq 0.75b_c$</td>
<td>$b = 12''$, $b_c = 12''$, but no stirrups specified within joints</td>
<td>Since $b \geq 0.75b_c$ : $S_t \leq 6''$ through joint</td>
</tr>
</tbody>
</table>
Correct $A_{s\text{min}}$ for Beams
The longitudinal steel ratio $\rho$ is < $\rho_{\text{min}} = 0.1 \sqrt{f'_c/f_y} (= 0.0043)$ in some cases

\[ \therefore \text{If all the #5 bars are changed to #6 bars} \]

\[ A_{s\text{top}} = 0.88, 1.48 \text{ in}^2, A_{s\text{bottom}} = 0.88 \text{ in}^2 \]

\[ \rho_{\text{top}} = 0.0054, 0.0091, \rho_{\text{bottom}} = 0.0054, \text{which are all } > \rho_{\text{min}} \text{ and } < \rho_{\text{max}} (= 0.025) \]

Check Shear Capacity of Beams

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF (V)</th>
<th>SF (D)</th>
<th>SF (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B_{16}</td>
<td>9.6</td>
<td>9.6</td>
<td>14.4</td>
</tr>
<tr>
<td>B_{20}</td>
<td>4.3</td>
<td>6.5</td>
<td>9.8</td>
</tr>
</tbody>
</table>

For Beam $B_{16}$, $a = A_{f_y}/0.85f'_c = 1.48 \times 40/(0.85 \times 3 \times 12) = 1.93''$

\[ M_{\text{ult}} \text{ at joint1} = A_{f_y} (d-a/2) = 1.48 \times 40 \times (13.5-1.93/2)/12 = 61.83 \text{ k} \]

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

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

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

\[ V_c = 2\sqrt{f'_c'/bd} = 2\sqrt{3/1000} \times 12 \times 13.5 = 17.75 \text{ k} \]

\[ V_{c1} = 6\sqrt{f'_c'bd} = 53.24 \text{ k} \]

\[ \therefore S_{\text{max}} = A_{f_y}d/(V_n-V_c) = 0.22 \times 40 \times 13.5/(29.72/0.85-17.75) = 6.89'' \]

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”x12”; i.e., the dimension 12” works as h.

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

\[ P/(\phi A_{f_y}f'_c) = 183.12/(0.7 \times 13 \times 12 \times 3) = 0.56, \gamma = 7/12 \geq 0.6, \mu = (40/2.55) \times 0.023 = 0.36 \]

\[ \therefore M/(\phi h_c^2f'_c) = 0.18 \Rightarrow M = 0.18 \times (0.9 \times 13 \times 12^2 \times 3) = 909.79 \text{ k}'' = 75.82 \text{ k} \]

$V_{\text{Design}} = 1.4 \times (75.82 + 75.82)/9 = 32.59 \text{ k}$

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

\[ P/(\phi A_{f_y}f'_c) = 196.28/(0.7 \times 13 \times 12 \times 3) = 0.60, \text{ with } \gamma = 0.6, \mu = 0.36 \]

\[ \therefore M/(\phi h_c^2f'_c) = 0.17 \Rightarrow M = 0.17 \times (0.9 \times 13 \times 12^2 \times 3) = 859.25 \text{ k}'' = 71.60 \text{ k} \]

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

\[ V_c = 2\sqrt{f'_c'bd} = 2\sqrt{3/1000} \times 13 \times 9.5 = 13.53 \text{ k} \]

\[ V_{c1} = 6\sqrt{f'_c'bd} = 40.59 \text{ k} \]

\[ \therefore S_{\text{max}} = A_{f_y}d/(V_n-V_c) = 0.22 \times 40 \times 9.5/(23.59/0.85-13.53) = 5.88'' \]

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 C₂ at two floors and B₁₆) consists of one beam only with two columns, the second joint (between C₆ at two floors and B₁₆, B₂₀) is checked first for the Weak Beam Strong Column.

For ground floor Column C₆, \( P = 140.2 \times 1.4 = 196.28 \text{ k} \)
\( P/(\phi A_{g,c'}) = 0.60, \text{ with } \gamma = 0.6, \mu p = 0.36 \Rightarrow M = 71.60 \text{ k}' \)

For first floor Column C₆, \( P/(\phi A_{g,c'}) \approx 3/4 \times 0.60 = 0.45, \text{ with } \gamma = 0.6, \mu p = 0.36 \)
\( \therefore M/(\phi b_{h,c'}^2 f_{c'}) = 0.19 \Rightarrow M = 0.19 (0.9 \times 13 \times 12^2 \times 3) = 960.34 \text{ k}'' = 80.03 \text{ k}' \)

\( \therefore \Sigma M_{c,ult} = 71.60 + 80.03 = 151.63 \text{ k}', 1.2 \Sigma M_{b,ult} = 1.2 (37.91 + 37.91) = 90.98 \text{ k}' \)
\( \therefore \Sigma M_{c,ult} > 1.2 \Sigma M_{b,ult}; \text{ i.e., the Weak Beam Strong Column condition is satisfied.} \)

The following sections are chosen as columns and beams

[Diagram of column section with 10 #6 bars, 135° hooks with 4" extensions, 4-legged #3 ties @ 3" c/c (special) or 5" c/c]

[Diagram of beam sections: Section a-a (Extra #7 bar), Section b-b (All #6 bars), Section c-c (135° hooks with 4" extensions), Section d-d, Section e-e]
The reinforcements are arranged as follows

Sp 2
Sp 2
Sp 2
Sp 2
Sp

1 #7 extra

2 #6 through

#3 stirrups
@ 6.75" c/c

#3 stirrups
@ 6.75" c/c

C-Sp 4

Sp ⇒ 2-legged #3 stirrups @ 3" (length 27")

C-Sp ⇒ 4-legged #3 stirrups @ 3" (through)

1 #7 extra

2.25'

18"

Lap-splices not allowed here

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

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

\( L_d \) for #7 bars = 0.04 \( A \), \( f_p/f_{p,cr} = 0.04 \times 0.60 \times 40/\sqrt{3/1000} \times 1.4 = 24.53" \)

\( L_{anch} = 24.53 + 10 \times 7/8 = 33.29" \); i.e., 34"
Design of Shear Walls

ACI Provisions for Shear Wall Design

1. The thickness of the shear wall must be at least 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., $V_n = V_{\text{design}}/\phi$ [\( V = V_{\text{design}} \) 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, $V_c \leq 3.3\sqrt{f_c}hd + N_u(4L_w)$
   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 $s_2 = A_{vh}f_y d/(V_n – V_c)$ [\( s_2 = A_{vh}f_y d/(V – V_c) \) in WSD] or where $s_2 \leq L_u/5, 3h, 18''$
   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 $s_1 \leq L_u/3, 3h, 18''$
   $\rho_v = A_{vh}/(hs_1) = 0.0025 + 0.5 (2.5 – H_w/L_w) (\rho_h – 0.0025)$
   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, $d \geq 0.8L_w = 0.8 \times 7 \times 12 = 67.2''$

4. $V = 71.24$ k, while $5\sqrt{f'c} \cdot h \cdot d = 5\sqrt{3/1000} \times 8 \times 67.2 = 147.23$ k

   $\therefore V \leq 5\sqrt{f'c} \cdot h \cdot d \Rightarrow OK$

5. Since there is no net tensile force on the section, $V_c$ can be taken as $= 1.1 \sqrt{f'c} \cdot h \cdot d = 32.39$ 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'', $A_{cd}/0.0025 \cdot h =) 20''$

   $\Rightarrow$ Use 2-legged #4 bars @ 14'' c/c

   $\therefore \rho_h = A_{cd}/(h \cdot s_2) = 0.40/(8 \times 14) = 0.0036$

8. Using 2-legged #4 bars as vertical reinforcements

   $\rho_v = A_{cv}/(h \cdot s_1) = 0.0025 + 0.5 \cdot (2.5 - H_w/L_w) \cdot (\rho_h - 0.0025)$

   $\Rightarrow 0.40/(8s_1) = 0.0025 + 0.5 \cdot (2.5 - 40/7) (0.0036 - 0.0025) = 0.0008 \Rightarrow s_1 = 64.26''$

   However, $s_1 \leq L_w/3 = 28''$, $(3h =) 24''$, 18'', $A_{cv}/0.0025 \cdot h =) 20''$

   $\Rightarrow$ Use 2-legged #4 bars @ 18'' c/c

   $\therefore \rho_h = A_{cv}/(h \cdot s_a) = 0.40/(8 \times 18) = 0.0028$, which is $\geq 0.0025$ and $\leq \rho_h$
9. Flexural reinforcements are provided like normal beams

\[ M_{\text{max}} = 7.13 \times 10 + 14.27 \times 20 + 21.40 \times 30 + 28.54 \times 40 = 2140 \text{ k-ft} \]

\[ A_s = \frac{M}{f_d} = \frac{2140 \times 12}{20 \times 0.87 \times 67.2} = 21.97 \text{ in}^2, \]

\[ \therefore \] Use 18 #10 bars on each side, to be curtailed over the height.