Design of a multi-storied steel 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 = 6.0 [i.e., R = 4.0]

Material Properties: $f'_c = 3$ ksi, $f_y = 40$ ksi, Allowable Bearing Capacity of soil = 2 ksf
1. Load Calculation for Slabs

**Slab (S₁) to Slab (S₅):**

Assumed slab thickness, \( t = 4.5'' \)

\[ \text{Self Weight of slab} = 4.5 \times 150/12 = 56.25 \text{ psf} \]

\[ DL = 56.25 + 20 + 20 = 96.25 \text{ psf} = 0.096 \text{ ksf} \]

\[ LL = 40 \text{ psf} = 0.04 \text{ ksf} \]

\[ \text{Total Wt./slab area} = 0.096 + 0.04 = 0.136 \text{ ksf} \]

**Slab (S₁₀):**

Assumed slab thickness, \( t = 5'' \)

\[ \text{Self Weight of slab} = 5 \times 150/12 = 62.5 \text{ psf} \]

FF = 20 psf, but there is no random wall and LL is less (i.e., assumed to be 20 psf)

\[ \text{Total Wt./slab area, } w = 62.5 + 20 + 20 = 102.5 \text{ psf} = 0.103 \text{ ksf} \]

**Slab (S₁₁):**

Assumed slab thickness, \( t = 7'' \)

\[ \text{Self Weight of slab} = 7 \times 150/12 = 87.5 \text{ psf} \]

FF = 20 psf, but there is no random wall and LL is high (i.e., assumed to be 100 psf)

\[ \text{Total Wt./slab area, } w = 87.5 + 20 + 100 = 207.5 \text{ psf} = 0.208 \text{ ksf} \]

Additional weight on flights due to 6'' high stairs = \( \frac{1}{2} \times (6/12) \times 150 \text{ psf} = 37.5 \text{ psf} \)

\[ \text{Total Wt./slab area, } w = 207.5 + 37.5 = 245 \text{ psf} = 0.245 \text{ ksf} \]
2. Vertical Load Analysis of Beams and Columns

Beams are assumed to have self-weights of about 30 lb/ft; i.e., 0.03 k/

Partition Walls (PW) are assumed to be 5" thick and Exterior Walls (EW) 10" thick

Weight of 5" PW = \(\frac{5''}{12}\times9\times120 = 450 \text{ lb/''} = 0.45 \text{ k/''}\)

Weight of 10" EW = \(\frac{10''}{12}\times9\times120 = 900 \text{ lb/''} = 0.90 \text{ k/''}\)
Frame (1) \([B_{8-9-10-11}]\):

Slab-load on B_8 = \([13/2 \times (16+3)/2 + 14/2 \times (16+2)/2] \times 0.136 = 16.97^k\)

\[\therefore \text{Equivalent UDL (} + \text{ Self Wt. and PW) } \approx 16.97/16 + 0.03 + 0.45 = 1.54^k\]

Slab-load on B_{9-10} = \([13/2 \times (14+1)/2 \times 0.136 + (14 \times 14 - 7 \times 7)/2 \times (0.208 + 0.245)/2 = 23.28^k\]

\[\therefore \text{Equivalent UDL (} + \text{ Self Wt. and PW) } \approx 23.28/14 + 0.03 + 0.45 = 2.14^k\]

Load from Slabs to B_{11} = \([13/2 \times (14+1)/2 + 14/2 \times (14)/2] \times 0.136 = 13.29^k\)

\[\therefore \text{Equivalent UDL (} + \text{ Self Wt. and PW) } \approx 13.29/14 + 0.03 + 0.45 = 1.43^k\]

\[
\begin{array}{|c|c|c|c|}
\hline
 & 1.54^k & 2.14^k & 2.14^k & 1.43^k \\
\hline
\hline
\hline
\hline
(-48.6, 5.8, -11.8) & (-55.2, -0.1, 0.0) & (-72.4, 1.8, -3.8) & (-39.6, 4.3, 8.3) & \\
\hline
C_{12} & C_{13} & C_{14} & C_{15} & C_{16}
\end{array}
\]

**Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k')) and Column (AF (k), BM_1, BM_2 (k')) in Frame (1) from Vertical Load Analysis**
Frame (2) [B12-13-14]:
Slab-load on B12 = \[13/2 \times (16+3)/2 \times 0.136 = 8.40^k\]
∴ Equivalent UDL (+ Self Wt. and EW) \(\approx 8.40/16 + 0.03 + 0.90 = 1.45^k\)
Slab-load on B13 = \[13/2 \times (14+1)/2 \times 0.136 = 6.63^k\]
∴ Equivalent UDL (+ Self Wt. and EW) \(\approx 6.63/14 + 0.03 + 0.90 = 1.40^k\)
Load from Slabs to B14 = \[13/2 \times (14+1)/2 \times 0.136 = 6.63^k\]
∴ Equivalent UDL (+ Self Wt. and EW) \(\approx 6.63/14 + 0.03 + 0.90 = 1.40^k\)

<table>
<thead>
<tr>
<th></th>
<th>1.45 k&quot;</th>
<th>1.40 k&quot;</th>
<th>1.40 k&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>B12</td>
<td>(11.2, -12.0, -25.8, 17.6, -31.8)</td>
<td>(9.9, -9.7, -24.3, 10.8, -22.7)</td>
<td></td>
</tr>
<tr>
<td>B13</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C17</td>
<td>(-45.0, 5.5, -10.8)</td>
<td>(-79.1, 0.1, -0.5)</td>
<td>(-38.0, -4.1, 7.9)</td>
</tr>
<tr>
<td>C18</td>
<td>(-87.4, -1.7, 3.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C19</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>C20</td>
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</tr>
</tbody>
</table>

Beam (SF1, SF2 (k), BM1, BM0, BM2 (k')) and Column (AF (k), BM1, BM2 (k')) in Frame (2) from Vertical Load Analysis
**Frame (3) [B_{16-20,21-26}]:**

Slab-load on B_{16} and B_{26} = \left[\frac{13}{2} \times \left(\frac{13}{2}\right) + \frac{13}{2} \times \left(\frac{13}{2}\right)\right] \times 0.136 = 11.49 \text{k}

\[\therefore\text{Equivalent UDL (+ Self Wt. and PW) } \cong 11.49 + 0.03 + 0.45 = 1.36 \text{k/}

Slab-load on B_{20-21} = \left[\frac{14}{2} \times \left(\frac{14}{2}\right)\right] \times 0.136 = 6.66 \text{k}

\[\therefore\text{Equivalent UDL (+ Self Wt. and PW) } \cong 6.66 + 0.03 + 0.45 = 0.96 \text{k/}

<table>
<thead>
<tr>
<th></th>
<th>1.36 k/</th>
<th>0.96 k/</th>
<th>0.96 k/</th>
<th>1.36 k/</th>
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<tbody>
<tr>
<td>B_{16}</td>
<td></td>
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<tr>
<td>B_{20}</td>
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<tr>
<td>B_{21}</td>
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<tr>
<td>B_{26}</td>
<td></td>
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<tr>
<td>C_{2}</td>
<td></td>
<td></td>
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<tr>
<td>C_{6}</td>
<td></td>
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<tr>
<td>C_{10}</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>C_{13}</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>C_{18}</td>
<td></td>
<td></td>
<td></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 (3) from Vertical Load Analysis
Frame (4) \([B_{17,23,27}]\):

Slab-load on \(B_{17}\) and \(B_{27}\) = \([13/2 \times (13)/2 + 13/2 \times (13)/2] \times 0.136 = 11.49 \text{k}\) 

\(\therefore\) Equivalent UDL (+ Self Wt. and PW) \(\cong 11.49/13 + 0.03 + 0.45 = 1.36 \text{k/}^\prime\)

Slab-load on \(B_{23}\) = \([14/2 \times (14)/2] \times 0.136 = 6.66 \text{k}\)

\(\therefore\) Equivalent UDL (+ Self Wt., EW, \(S_9\)) \(\cong 6.66/14 + 0.03 + 0.90 + 3 \times 0.136 = 1.81 \text{k/}^\prime\)

[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></th>
<th>1.36 k/′</th>
<th>1.81 k/′</th>
<th>1.36 k/′</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{17})</td>
<td>((8.3,-9.4,-15.0,10.3,-22.0))</td>
<td>((12.7,-12.7,-28.6,15.7,-28.6))</td>
<td>((9.4,-8.3,-22.0,10.3,-15.0))</td>
</tr>
<tr>
<td>(B_{23})</td>
<td>((-33.7,3.2,-6.2))</td>
<td>((-87.7,1.4,-2.7))</td>
<td>((-87.7,-1.4,2.7))</td>
</tr>
<tr>
<td>(B_{27})</td>
<td>((-33.7,-3.2,6.2))</td>
<td>((-87.7,-1.4,-2.7))</td>
<td>((-33.7,3.2,6.2))</td>
</tr>
<tr>
<td>(C_3)</td>
<td>((-33.7,3.2,-6.2))</td>
<td>((-87.7,1.4,-2.7))</td>
<td>((-87.7,-1.4,2.7))</td>
</tr>
<tr>
<td>(C_8)</td>
<td>((-33.7,3.2,-6.2))</td>
<td>((-87.7,1.4,-2.7))</td>
<td>((-87.7,-1.4,2.7))</td>
</tr>
<tr>
<td>(C_{15})</td>
<td>((-33.7,3.2,-6.2))</td>
<td>((-87.7,1.4,-2.7))</td>
<td>((-87.7,-1.4,2.7))</td>
</tr>
<tr>
<td>(C_{19})</td>
<td>((-33.7,3.2,-6.2))</td>
<td>((-87.7,1.4,-2.7))</td>
<td>((-87.7,-1.4,2.7))</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) \([B_{8-9-10-11}]\).

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: $Z = 0.15$, $I = 1.0$, $S = 1.0$, $R = 6.0$, $C = 1.25 S/T^{2/3} \leq 2.75$

where $T = \text{Fundamental period of vibration} = 0.083 \times h^{3/4}$ [$h = \text{Building Height in meters}$]

For the moment resisting steel frame, $T = 0.083 \times (40/3.28)^{3/4} = 0.542 \text{ sec}$

$C = 1.25 S/T^{2/3} = 1.25 \times 1.0/(0.542)^{2/3} = 1.88$

$\therefore \text{Base Shear, } V = (ZIC/R) W = 0.15 \times 1.0 \times 1.88/6.0 \text{ W} = 0.0470W$

Since $T < 0.7 \text{ sec}$, $F_t = 0$; $\therefore$ For equally loaded stories without $F_t$, $F_i = (h/\Sigma h) V$

$\Rightarrow F_1 = 0.1V$, $F_2 = 0.2V$, $F_3 = 0.3V$, $F_4 = 0.4V$

Frame (1) [B_8-9-10-11]:

$W = 4 \times (1.54 \times 16 + 2.14 \times 7 + 2.14 \times 7 + 1.43 \times 14) = 298^k \Rightarrow V = 0.0470W = 14.03^k$

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}]:

\[ W = 4\times(1.45\times16 + 1.40\times14 + 1.40\times14) = 249.60^k \Rightarrow V = 0.0470W = 11.73^k \]

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

\[ W = 4 \times (1.36 \times 13 + 0.96 \times 7 + 0.96 \times 7 + 1.36 \times 13) = 195.20^k \quad \Rightarrow \quad V = 0.0470W = 9.17^k \]

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}]:

\[ W = 4 \times (1.36 \times 13 + 1.81 \times 14 + 1.36 \times 13) = 242.80^k \Rightarrow V = 0.0470W = 11.41^k \]

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_{4-5-6-7}]:
Similar to Frame (1) [B_{8-9-10-11}].

<table>
<thead>
<tr>
<th>B_{4}</th>
<th>B_{5}</th>
<th>B_{6}</th>
<th>B_{7}</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.5, 12.9, 10.9)</td>
<td>(4.9, 17.7, 16.4)</td>
<td>(4.8, -16.4, 17.4)</td>
<td>(1.9, -12.0, 14.3)</td>
</tr>
<tr>
<td>C_{5}</td>
<td>C_{6}</td>
<td>C_{7}</td>
<td>C_{8}</td>
</tr>
<tr>
<td>(-4.3, 14.6, -6.7)</td>
<td>(9.5, 17.9, -13.3)</td>
<td>(0.1, -8.4, 16.4)</td>
<td>(8.3, 18.1, -13.6)</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>B_{1}</th>
<th>B_{2}</th>
<th>B_{3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.9, 16.3, 14.6)</td>
<td>(2.1, -14.7, 14.6)</td>
<td>(2.4, -16.1, 18.0)</td>
</tr>
<tr>
<td>C_{1}</td>
<td>C_{2}</td>
<td>C_{3}</td>
</tr>
<tr>
<td>(-5.7, 17.2, -7.6)</td>
<td>(-0.6, 19.9, -13.0)</td>
<td>(-0.8, 20.2, -13.5)</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) \([B_{17-23-27}]\).

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) [B4-5-6-7] and [B8-9-10-11]:

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF1(V)</th>
<th>SF1(L)</th>
<th>SF1(D)</th>
<th>SF2(V)</th>
<th>SF2(L)</th>
<th>SF2(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B4, B8</td>
<td>12.1</td>
<td>±1.5</td>
<td>12.1</td>
<td>-12.6</td>
<td>±1.5</td>
<td>-12.6</td>
</tr>
<tr>
<td>B5, B9</td>
<td>8.6</td>
<td>±4.9</td>
<td>10.1</td>
<td>-6.4</td>
<td>±4.9</td>
<td>-8.5</td>
</tr>
<tr>
<td>B6, B10</td>
<td>6.7</td>
<td>±4.8</td>
<td>8.6</td>
<td>-8.3</td>
<td>±4.8</td>
<td>-9.8</td>
</tr>
<tr>
<td>B7, B11</td>
<td>10.2</td>
<td>±1.9</td>
<td>10.2</td>
<td>-9.8</td>
<td>±1.9</td>
<td>-9.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM1(V)</th>
<th>BM1(L)</th>
<th>BM1(D)</th>
<th>BM0(V=D)</th>
<th>BM2(V)</th>
<th>BM2(L)</th>
<th>BM2(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B4, B8</td>
<td>-28.1</td>
<td>±12.9</td>
<td>-30.8</td>
<td>19.3</td>
<td>-31.8</td>
<td>±10.9</td>
<td>-32.0</td>
</tr>
<tr>
<td>B5, B9</td>
<td>-15.1</td>
<td>±17.7</td>
<td>-24.6</td>
<td>2.2</td>
<td>-7.3</td>
<td>±16.4</td>
<td>-17.8</td>
</tr>
<tr>
<td>B6, B10</td>
<td>-7.2</td>
<td>±16.4</td>
<td>-17.7</td>
<td>3.2</td>
<td>-13.0</td>
<td>±17.4</td>
<td>-22.8</td>
</tr>
<tr>
<td>B7, B11</td>
<td>-22.4</td>
<td>±12.0</td>
<td>-25.8</td>
<td>13.8</td>
<td>-20.0</td>
<td>±14.3</td>
<td>-25.7</td>
</tr>
</tbody>
</table>

Frame (2) [B1-2-3] and [B12-13-14]:

<table>
<thead>
<tr>
<th>Beams</th>
<th>SF1(V)</th>
<th>SF1(L)</th>
<th>SF1(D)</th>
<th>SF2(V)</th>
<th>SF2(L)</th>
<th>SF2(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B12</td>
<td>11.2</td>
<td>±1.9</td>
<td>11.2</td>
<td>-12.0</td>
<td>±1.9</td>
<td>-12.0</td>
</tr>
<tr>
<td>B3, B14</td>
<td>10.2</td>
<td>±2.4</td>
<td>10.2</td>
<td>-9.4</td>
<td>±2.4</td>
<td>-9.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>BM1(V)</th>
<th>BM1(L)</th>
<th>BM1(D)</th>
<th>BM0(V=D)</th>
<th>BM2(V)</th>
<th>BM2(L)</th>
<th>BM2(D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B12</td>
<td>-25.8</td>
<td>±16.3</td>
<td>-31.6</td>
<td>17.6</td>
<td>-31.8</td>
<td>±14.6</td>
<td>-34.8</td>
</tr>
<tr>
<td>B2, B13</td>
<td>-24.3</td>
<td>±14.7</td>
<td>-29.3</td>
<td>10.8</td>
<td>-22.7</td>
<td>±14.6</td>
<td>-28.0</td>
</tr>
<tr>
<td>B3, B14</td>
<td>-23.8</td>
<td>±16.1</td>
<td>-29.9</td>
<td>12.9</td>
<td>-18.9</td>
<td>±18.0</td>
<td>-27.7</td>
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</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_2(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>8.7</td>
<td>(\pm 1.3)</td>
<td>8.7</td>
<td>-9.0</td>
<td>(\pm 1.3)</td>
<td>-9.0</td>
</tr>
<tr>
<td>(B_{20})</td>
<td>4.0</td>
<td>(\pm 3.0)</td>
<td>5.3</td>
<td>-2.8</td>
<td>(\pm 3.0)</td>
<td>-4.4</td>
</tr>
<tr>
<td>(B_{21})</td>
<td>2.8</td>
<td>(\pm 3.0)</td>
<td>4.4</td>
<td>-4.0</td>
<td>(\pm 3.0)</td>
<td>-5.3</td>
</tr>
<tr>
<td>(B_{26})</td>
<td>9.0</td>
<td>(\pm 1.3)</td>
<td>9.0</td>
<td>-8.7</td>
<td>(\pm 1.3)</td>
<td>-8.7</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>-16.1</td>
<td>(\pm 9.5)</td>
<td>-19.2</td>
<td>11.7</td>
<td>-18.0</td>
<td>(\pm 8.0)</td>
<td>-19.5</td>
</tr>
<tr>
<td>(B_{20})</td>
<td>-7.5</td>
<td>(\pm 10.9)</td>
<td>-13.8</td>
<td>0.6</td>
<td>-7.5</td>
<td>(\pm 10.4)</td>
<td>-13.8</td>
</tr>
<tr>
<td>(B_{21})</td>
<td>-3.4</td>
<td>(\pm 10.4)</td>
<td>-10.4</td>
<td>0.6</td>
<td>-7.5</td>
<td>(\pm 10.9)</td>
<td>-13.8</td>
</tr>
<tr>
<td>(B_{26})</td>
<td>-18.0</td>
<td>(\pm 8.0)</td>
<td>-19.5</td>
<td>11.7</td>
<td>-16.1</td>
<td>(\pm 9.5)</td>
<td>-19.2</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_2(D))</th>
<th>(SF_2(V))</th>
<th>(SF_2(L))</th>
<th>(SF_2(D))</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{15-17}, B_{18})</td>
<td>8.3</td>
<td>(\pm 2.6)</td>
<td>8.3</td>
<td>-9.4</td>
<td>(\pm 2.6)</td>
<td>-9.4</td>
</tr>
<tr>
<td>(B_{19-23}, B_{24})</td>
<td>12.7</td>
<td>(\pm 1.9)</td>
<td>12.7</td>
<td>-12.7</td>
<td>(\pm 1.9)</td>
<td>-12.7</td>
</tr>
<tr>
<td>(B_{25-27}, B_{28})</td>
<td>9.4</td>
<td>(\pm 2.6)</td>
<td>9.4</td>
<td>-8.3</td>
<td>(\pm 2.6)</td>
<td>-8.3</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_{15-17}, B_{18})</td>
<td>-15.0</td>
<td>(\pm 17.5)</td>
<td>-24.4</td>
<td>10.3</td>
<td>-22.0</td>
<td>(\pm 15.7)</td>
<td>-28.3</td>
</tr>
<tr>
<td>(B_{19-23}, B_{24})</td>
<td>-28.6</td>
<td>(\pm 13.2)</td>
<td>-31.4</td>
<td>15.7</td>
<td>-28.6</td>
<td>(\pm 13.2)</td>
<td>-31.4</td>
</tr>
<tr>
<td>(B_{25-27}, B_{28})</td>
<td>-22.0</td>
<td>(\pm 15.7)</td>
<td>-28.3</td>
<td>10.3</td>
<td>-15.0</td>
<td>(\pm 17.6)</td>
<td>-24.5</td>
</tr>
</tbody>
</table>

Other Beams:

1. Beam \(B_{22}\) -

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

\[\text{Maximum SF} \cong 0.96 \times 7/2 = 3.36 \text{k} \]

and Maximum positive BM \(\cong 0.96 \times 7^2/8 = 5.88 \text{k'}\)

2. Edge Beam for \(S_{10}\) -

Uniformly distributed load on \(S_{10} = 0.103 \text{ksf}\)

Uniformly distributed load on Edge Beam = \(0.103 \times 5' = 0.51 \text{k'}\)

\[\text{Clear Span} = 13' \Rightarrow V_{\text{max}} \cong 0.51 \times (13)/2 = 3.3 \text{k}; M^2 \cong 0.51 \times (13)^2/10 = 8.6 \text{k'}\]
### 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>((L_{x}))</th>
<th>0.75((V+L_{x}))</th>
<th>0.75((V-L_{x}))</th>
<th>((L_{y}))</th>
<th>0.75((V+L_{y}))</th>
<th>0.75((V-L_{y}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>(C_1, C_{17})</td>
<td>2, 4</td>
<td>-78.7, 5.3, 3.2</td>
<td>-5.7, 17.2, 0</td>
<td>-63.3, 16.9, 2.4</td>
<td>-54.8, 8.9, 2.4</td>
<td>-7.4, 0, 16.7</td>
<td>-64.6, 4.0, 14.9</td>
<td>-53.5, 4.0, -10.1</td>
</tr>
<tr>
<td>(C_2, C_{18})</td>
<td>2, 3</td>
<td>-122.4, -1.7, 3.4</td>
<td>-0.6, 19.9, 0</td>
<td>-92.3, 13.7, 2.6</td>
<td>-91.4, -16.2, 2.6</td>
<td>-3.9, 0, 9.6</td>
<td>-97.4, -1.3, 15.1</td>
<td>-86.3, -1.3, 10.0</td>
</tr>
<tr>
<td>(C_3, C_{19})</td>
<td>2, 4</td>
<td>-112.8, 0.1, -3.2</td>
<td>-0.8, 20.2, 0</td>
<td>-85.2, 15.2, -2.4</td>
<td>-84.0, 15.1, -2.4</td>
<td>-7.4, 0, 16.8</td>
<td>-90.2, 0.1, 10.2</td>
<td>-79.1, 0.1, -15.0</td>
</tr>
<tr>
<td>(C_4, C_{20})</td>
<td>2, 4</td>
<td>-71.7, -4.1, -3.2</td>
<td>7.1, 17.6, 0</td>
<td>-7.4, 0, 16.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_5, C_{12})</td>
<td>1, 4</td>
<td>-126.3, 5.8, 1.4</td>
<td>-4.3, 14.6, 0</td>
<td>1.7, 0, 19.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_6, C_{13})</td>
<td>1, 3</td>
<td>-132.5, -3.5, -2.1</td>
<td>-9.5, 17.9, 0</td>
<td>-4.7, 0, 11.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_7, C_{14})</td>
<td>1</td>
<td>-55.2, -0.1, 0</td>
<td>0.1, 18.6, 0</td>
<td>0.0, 0.0, 0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_8, C_{15})</td>
<td>1, 4</td>
<td>-160.1, 1.8, 1.4</td>
<td>8.3, 18.1, 0</td>
<td>-113.9, 14.9, 1.1</td>
<td>-126.3, -12.2, 1.1</td>
<td>1.7, 0, 19.1</td>
<td>-118.8, 1.4, 15.4</td>
<td>-121.4, 1.4, -13.3</td>
</tr>
<tr>
<td>(C_9, C_{16})</td>
<td>1, 4</td>
<td>-127.3, -4.3, 1.4</td>
<td>5.4, 15.0, 0</td>
<td>1.7, 0, 19.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_{10})</td>
<td>3</td>
<td>-25.5, 0, 0</td>
<td>0.0, 0.0, 0.0</td>
<td>0.0, 0.12.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(C_{11})</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

In this work, only one size 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 = 160.1\(^k\), Bending Moments \(BM_{1x} = 1.4\ k'\), \(BM_{1y} = 1.8\ k'\).
2. Compressive Force = 113.9\(^k\), Bending Moments \(BM_{1x} = 1.1\ k'\), \(BM_{1y} = 14.9\ k'\).
3. Compressive Force = 118.8\(^k\), Bending Moments \(BM_{1x} = 15.4\ k'\), \(BM_{1y} = 1.4\ k'\).
5. Design of Beams

The figure below shows an I-section, which has been chosen here for all the beams.

The design of beams follows the following steps

1. Calculate the design moment \( M_d \) from vertical and lateral load analyses

2. Assume the allowable bending stress \( f_b \) and calculate \( Z_{req} = \frac{M_d}{f_b} \)
   If \( f_b = 0.6f_y = 24 \text{ ksi} \) and \( M_d \) is in k', then \( Z_{req} \) (in in\(^3\)) is = \( M_d \times 12/24 = M_d/2 \)

3. Choose a beam section with \( Z_{xx} \geq Z_{req} \)

4. Check the chosen beam section against
   (i) local buckling of flange; i.e., \( b_f/t_f \leq 190/\sqrt{f_y} \leq 30 \) (for \( f_y = 40 \text{ ksi} \))
   (ii) local buckling of web; i.e., \( d_w/t_w \leq 760/\sqrt{f_b} \leq 155 \) (for \( f_b = 24 \text{ ksi} \))
   and vertical buckling of flange; i.e., \( d_w/t_w \leq 2000/\sqrt{f_y} \leq 316 \) (for \( f_y = 40 \text{ ksi} \))
   (iii) lateral torsional buckling; i.e., \( kL/b_f \leq 76\sqrt{C_b/f_y} \)

5. Calculate the maximum shear stress \( f_s = \frac{VQ}{I_{xx} t_w} \) [where \( Q = t_w d_w^2/8 + A_f (d_w + t_f)/2 \)]

6. Check against shear buckling of web; i.e.,

\[
    f_s(\text{all}) = 0.4f_y \quad (= 16 \text{ ksi}) \quad \text{if } d_w/t_w \leq 369/\sqrt{f_y} \quad (= 58); \\
    = 148\sqrt{f_y}/(d_w/t_w) \quad \text{if } 369/\sqrt{f_y} \leq d_w/t_w \leq 532/\sqrt{f_y}; \\
    = 78000/(d_w/t_w)^2 \quad \text{if } d_w/t_w \geq 532/\sqrt{f_y}
\]
Frame (1) \([B_{4-5-6-7}]\) and \([B_{8-9-10-11}]\):

The design moments (k') are

<table>
<thead>
<tr>
<th>Beam</th>
<th>(M_d) (k')</th>
<th>(Z_{req}) (in(^3))</th>
<th>(Z_{xs}) (in(^3))</th>
<th>(b_f) (in)</th>
<th>(t_f) (in)</th>
<th>(d_w) (in)</th>
<th>(t_w) (in)</th>
<th>(A_f) (in(^2))</th>
<th>(b/w/t_f)</th>
<th>(d_w/t_w)</th>
<th>(Ld_w/A_f)</th>
<th>(f_b) (ksi)</th>
<th>(Z_{req}) (in(^3))</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_4, B_8)</td>
<td>32.0</td>
<td>16.0</td>
<td>16.2</td>
<td>4.17</td>
<td>0.426</td>
<td>7.15</td>
<td>0.44</td>
<td>1.78</td>
<td>9.79</td>
<td>16.21</td>
<td>772</td>
<td>22.0</td>
<td>17.4</td>
<td>No</td>
</tr>
<tr>
<td>(B_5, B_9)</td>
<td>24.6</td>
<td>12.3</td>
<td>14.4</td>
<td>4.00</td>
<td>0.426</td>
<td>7.15</td>
<td>0.27</td>
<td>1.70</td>
<td>9.39</td>
<td>26.38</td>
<td>352</td>
<td>24.0</td>
<td>12.3</td>
<td>Yes</td>
</tr>
<tr>
<td>(B_6, B_{10})</td>
<td>22.8</td>
<td>11.4</td>
<td>14.4</td>
<td>4.00</td>
<td>0.426</td>
<td>7.15</td>
<td>0.27</td>
<td>1.70</td>
<td>9.39</td>
<td>26.38</td>
<td>352</td>
<td>24.0</td>
<td>11.4</td>
<td>Yes</td>
</tr>
<tr>
<td>(B_7, B_{11})</td>
<td>25.8</td>
<td>12.9</td>
<td>14.4</td>
<td>4.00</td>
<td>0.426</td>
<td>7.15</td>
<td>0.27</td>
<td>1.70</td>
<td>9.39</td>
<td>26.38</td>
<td>705</td>
<td>24.0</td>
<td>12.9</td>
<td>Yes</td>
</tr>
</tbody>
</table>

The design shear forces (k) are

Design Table for Beams (Moment)

Design Table for Beams (Shear)
Frame (2) [B1, B12] and [B13, B14]:

The design moments (k') are

\[
\begin{align*}
L = 16' & \\ 17.6 & -31.6 \\
10.8 & -29.3 \\
12.9 & -28.0
\end{align*}
\]

\[
\begin{align*}
L = 14' & \\ -34.8 & -29.3 \\
-28.0 & -29.9 \\
-27.7 & -28.0
\end{align*}
\]

The design shear forces (k) are

\[
\begin{align*}
B_1, B_{12} & \\ 11.2 & -12.0 \\
9.9 & -9.7 \\
10.2 & -9.4
\end{align*}
\]

\[
\begin{align*}
B_2, B_{13} & \\
B_3, B_{14} &
\end{align*}
\]

**Design Table for Beams (Moment)**

<table>
<thead>
<tr>
<th>Beam</th>
<th>$M_d$ (k')</th>
<th>$Z_{req}$ (in$^3$)</th>
<th>$Z_{xx}$ (in$^3$)</th>
<th>$b_f$ (in)</th>
<th>$t_f$ (in)</th>
<th>$d_w$ (in)</th>
<th>$t_w$ (in)</th>
<th>$A_f$ (in$^2$)</th>
<th>$b_f/t_f$</th>
<th>$d_w/t_w$</th>
<th>$Ld_w/A_f$</th>
<th>$f_b$ (ksi)</th>
<th>$Z_{req}$ (in$^3$)</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B12</td>
<td>34.8</td>
<td>17.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B2, B13</td>
<td>29.3</td>
<td>14.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B3, B14</td>
<td>29.9</td>
<td>15.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Design Table for Beams (Shear)**

<table>
<thead>
<tr>
<th>Beam</th>
<th>V (k)</th>
<th>$I_{xx}$ (in$^4$)</th>
<th>Q (in$^3$)</th>
<th>$f_s$ (ksi)</th>
<th>$f_{s(all)}$ (ksi)</th>
<th>OK</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B12</td>
<td>12.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B2, B13</td>
<td>9.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B3, B14</td>
<td>10.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Frame (3) \([B_{16,20,21,26}]\):  

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

\[
\begin{array}{cccccc}
11.7 & 0.6 & 0.6 & 11.7 \\
-19.2 & -19.5 & -10.4 & -19.5 & -19.2 \\
L = 13' & L = 7' & L = 7' & L = 13'
\end{array}
\]

The design shear forces \((k)\) are  

\[
\begin{array}{cccc}
8.7 & 5.3 & 4.4 & 9.0 \\
-9.0 & -4.4 & -5.3 & -8.7 \\
B_{16} & B_{20} & B_{21} & B_{26}
\end{array}
\]

Design Table for Beams (Moment)  

| Beam \(B\) | \(M_d\) \((k')\) | \(Z_{req}\) \((in^3)\) | \(Z_{xx}\) \((in^3)\) | \(b_f\) (in) | \(t_f\) (in) | \(d_w\) (in) | \(A_f\) \((in^2)\) | \(b_f/t_f\) | \(d_w/t_w\) | \(Ld_w/A_f\) | \(f_b\) (ksi) | \(Z_{req}\) \((in^3)\) | OK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| \(B_{16}\) | 19.5 | 9.8 |
| \(B_{20}\) | 13.8 | 6.9 |
| \(B_{21}\) | 13.8 | 6.9 |
| \(B_{26}\) | 19.5 | 9.8 |

Design Table for Beams (Shear)  

<table>
<thead>
<tr>
<th>Beam (B)</th>
<th>(V) ((k))</th>
<th>(I_{xx}) ((in^4))</th>
<th>(Q) ((in^3))</th>
<th>(f_s) (ksi)</th>
<th>(f_s(all)) (ksi)</th>
<th>OK</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_{16})</td>
<td>9.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(B_{20})</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(B_{21})</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(B_{26})</td>
<td>9.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Frame (4) [B_{15-17-18}], [B_{17-23-27}] and [B_{25-27-28}]:

The design moments (k') are

\[
\begin{array}{cccccc}
 & 10.3 & 15.7 & 10.3 & & \\
\end{array}
\]

L = 13' \hspace{2cm} L = 14' \hspace{2cm} L = 13'

The design shear forces (k) are

\[
\begin{array}{cccccc}
 & 8.3 & 12.7 & 9.4 & & \\
-9.4 & -12.7 & -8.3 & & \\
\end{array}
\]

B_{15}, B_{17}, B_{18} \hspace{2cm} B_{19}, B_{23}, B_{24} \hspace{2cm} B_{25}, B_{27}, B_{28}

### Design Table for Beams (Moment)

<table>
<thead>
<tr>
<th>Beam</th>
<th>(M_d) (k')</th>
<th>(Z_{req}) (in^3)</th>
<th>(Z_{xx}) (in^3)</th>
<th>(b_f) (in)</th>
<th>(t_f) (in)</th>
<th>(d_w) (in)</th>
<th>(t_w) (in)</th>
<th>(A_f) (in^2)</th>
<th>(b_f/t_f)</th>
<th>(d_w/t_w)</th>
<th>(Ld_w/A_f)</th>
<th>(f_b) (ksi)</th>
<th>(Z_{req}) (in^3)</th>
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<tbody>
<tr>
<td>B_{15}, B_{17}, B_{18}</td>
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<tr>
<td>B_{25}, B_{27}, B_{28}</td>
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</table>

### Design Table for Beams (Shear)

<table>
<thead>
<tr>
<th>Beam</th>
<th>(V) (k)</th>
<th>(I_{xx}) (in^4)</th>
<th>(Q) (in^3)</th>
<th>(f_s) (ksi)</th>
<th>(f_{s(all)}) (ksi)</th>
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<td>B_{25}, B_{27}, B_{28}</td>
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<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
6. Design of Columns

Design Concept of Compression Members:
The design of steel compression members is carried out using the following equations.

(1) Members under pure compression –
   - If \( F_c = \) Compressive force, \( E = \) Modulus of elasticity
   - \( f_c = \) Compressive stress, \( f_{all(c)} = \) Allowable compressive stress
   - \( A = \) Cross-sectional area, \( L_e = \) Effective length of member = \( kL \) (\( k \) is determined from alignment chart based on column end conditions), \( r_{min} = \) Minimum radius of gyration

\[
f_c = \frac{F_c}{A}
\]

Slenderness Ratio, \( \lambda = \frac{L_e}{r_{min}} \), and \( \lambda_c = \pi \sqrt{\frac{2E}{f_y}} \)

If \( \lambda \leq \lambda_c, f_{all(c)} = f_y \frac{[1-0.5(\frac{\lambda}{\lambda_c})^2]([5/3+3/8(\frac{\lambda}{\lambda_c})-1/8(\frac{\lambda}{\lambda_c})^3]}{[5/3+3/8(\frac{\lambda}{\lambda_c})-1/8(\frac{\lambda}{\lambda_c})^3]}

If \( \lambda > \lambda_c, f_{all(c)} = 0.52 \left( \frac{\pi^2 E}{\lambda_c^2} \right) \)

The acceptable design condition is \( f_c \leq f_{all(c)} \); i.e., \( f_c/f_{all(c)} \leq 1 \)

For the material properties used for design; i.e., \( E = 29000 \) ksi, \( f_y = 40 \) ksi

\[
\lambda_c = \pi \sqrt{\frac{2E}{f_y}} = 119.63, \ \rho = \frac{\lambda}{\lambda_c} = \frac{\lambda}{119.63}
\]

If \( \rho \leq 1, f_{all(c)} = 40 \left(1-0.5\rho^2\right)/\left(5/3+3\rho/8 -\rho^3/8\right)\)

If \( \rho > 1, f_{all(c)} = 149000/\rho^2 = 10.4/\rho^2 \)

The stresses \( f_c, f_{all(c)} \) are obtained from Eqs. (6.1) and (6.2)

\( f_{bx} = \) Bending stress about x-axis = \( M_x(mag)/Z_{xx} \)

Here \( M_x(mag) \) is the bending moment about x-axis magnified by the axial force, and it is given by \( M_x(mag) = M_x \left[ C_{mx}/(1-f_c/f_{all(ex)}) \right] \)

where \( C_{mx} = 1.0 \) for unrestrained members, and

\( f_{all(ex)} \) is the allowable Euler stress about x-axis; i.e., \( f_{all(ex)} = 0.52 \left( \pi^2 E/\lambda_x^2 \right) \)

\( f_{all(bx)} = \) Allowable bending stress about x-axis = \( 0.6 f_y \) ………………. (6.4(a)~6.4(d))

\( \therefore f_{all(bx)} = 24 \) ksi, in this case

Similar notations are used for \( f_{by} \) and \( f_{all(by)} \); i.e.,

\( f_{by} = M_y \left[ C_{my}/(1-f_c/f_{all(ey)}) \right]/Z_{yy}, f_{all(by)} = 0.6 f_y \) ………………. (6.5(a)~6.5(b))
Design Forces and Assumed Section:

The designed column (C_8, C_{15}) should satisfy the three design conditions mentioned before (in the load combination for columns); i.e.,

(i) $F_c = 160.1^k, M_x = 1.4 \, k', M_y = 1.8 \, k'$
(ii) $F_c = 113.9^k, M_x = 1.1 \, k', M_y = 14.9 \, k'$
(iii) $F_c = 118.8^k, M_x = 15.4 \, k', M_y = 1.4 \, k'$

To ensure larger moments of inertia and better flexural/buckling behavior, several built-up and closed sections can be used as steel columns. But as was done for all the beams, the I-section is chosen here for all the columns. Moreover the wide-flanged W-sections are chosen because they are more compact and therefore suitable as buckling members.

Preliminary choice of column section –

The bending moments are very small for condition (i), so that the section can be chosen based on axial force only. Since $F_{all(c)} = 0.52(\pi^2E I_{min}/L_e)^2$, one needs to know $L_e$ in order to calculate $I_{min}$ for a given load.

Assuming (a) unrestrained column, (b) critical axis is y-axis (consider Frame1), (c) column base is fixed, (d) two similar beam and column sections at the other joint; i.e.,

$G_A = 0, G_B = (1/10 + 1/10)/(1/7 + 1/14) = 0.93 \Rightarrow k = 1.12$

$\therefore 160.1 = 0.52 \times \pi^2 \times 29000 \times I_{min}/(1.12 \times 1 \times 12)^2 \Rightarrow I_{min} = 19.43 \, in^4$

$\therefore$ Choose the smallest section with the required $I_{min}$, i.e., W $8 \times 28$
Design Table for Columns

Design condition (i); i.e., \( F_c = 160.1^k \), \( M_x = 1.4 \; k' = 16.8 \; k'' \), \( M_y = 1.8 \; k' = 21.6 \; k'' \)

Assumed section \( W \times 28 \rightarrow A = 8.25 \; in^2 \), \( I_{xx} = 98.0 \; in^4 \), \( Z_{xx} = 24.3 \; in^3 \), \( r_{xx} = 3.45'' \)

\[ I_{\min} = I_{yy} = 21.7 \; in^4 \], \( Z_{yy} = 6.63 \; in^3 \), \( r_{yy} = 1.62'' \)

\[ f_c = F_c / A = 160.1 / 8.25 = 19.41 \; ksi, \quad \rho = (10 \times 12 / 1.62) / 119.63 = 77.78 / 119.63 = 0.65 \]

\[ \Rightarrow f_{all(c)} = 40 (1 - 0.5 \rho^2) / (5/3 + 3 \rho / 8 - \rho^3 / 8) = 16.81 \; ksi \]

About x-axis (axis of \( I_{\max} \)); i.e., in the y-direction,

\[ G_{Ax} = 0, \quad G_{Bx} = (98.0 / 10 + 98.0 / 10) / (57.6 / 13 + 57.6 / 14) = 2.29 \rightarrow k_x = 1.30 \]

\[ L_{ey} = 1.30 \times 120 = 156'', \quad \lambda_x = L_{ex} / r_{xx} = 156 / 3.45 = 45.22 \]

\[ f_{all(ex)} = 0.52 (\pi^2 E / \lambda_x^2) = 0.52 (\pi^2 \times 29000 / 45.22^2) = 72.79 \; ksi \]

\[ f_{bx} = M_x [C_{mx} / (1 - f_c / f_{all(ex))}] / Z_{xx} = 16.8 \times [1.0 / (1 - 19.41 / 72.79)] / 24.3 = 0.94 \; ksi \]

About y-axis (axis of \( I_{\min} \)); i.e., in the x-direction,

\[ G_{Ay} = 0, \quad G_{By} = (21.7 / 10 + 21.7 / 10) / (57.6 / 7 + 57.6 / 14) = 0.35 \rightarrow k_y = 1.05 \]

\[ L_{ey} = 1.05 \times 120 = 126'', \quad \lambda_y = L_{ey} / r_{yy} = 126 / 1.62 = 77.78 \]

\[ f_{all(ey)} = 0.52 (\pi^2 E / \lambda_y^2) = 0.52 (\pi^2 \times 29000 / 77.78^2) = 24.60 \; ksi \]

\[ f_{by} = M_y [C_{my} / (1 - f_c / f_{all(ey))}] / Z_{yy} = 21.6 \times [1.0 / (1 - 19.41 / 24.60)] / 6.63 = 15.42 \; ksi \]

\[ \therefore f_c / f_{all(c)} + f_{bx} / f_{all(bx)} + f_{by} / f_{all(by)} = 19.41 / 16.81 + 0.94 / 24 + 15.42 / 24 = 1.16 + 0.04 + 0.64 \]

\[ = 1.84 > 1, \; i.e., \; not \; OK \]

<table>
<thead>
<tr>
<th>Section</th>
<th>( k_x )</th>
<th>( k_y )</th>
<th>( L_{ex} )(in)</th>
<th>( L_{ey} )(in)</th>
<th>( \lambda_x )</th>
<th>( \lambda_y )</th>
<th>( f_c )(ksi)</th>
<th>( f_{all(c)})(ksi)</th>
<th>( f_{all(ex)})(ksi)</th>
<th>( f_{by} )(ksi)</th>
<th>( f_{all(by)})(ksi)</th>
<th>( f_{bx} )(ksi)</th>
<th>( f_{all(bx)})(ksi)</th>
<th>( LS )</th>
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</thead>
<tbody>
<tr>
<td>W 8 \times 28</td>
<td>1.30</td>
<td>1.05</td>
<td>156.0</td>
<td>126.0</td>
<td>45.22</td>
<td>77.78</td>
<td>19.41</td>
<td>16.81</td>
<td>72.79</td>
<td>0.94</td>
<td>24.60</td>
<td>15.42</td>
<td>1.84</td>
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<td>W 8 \times 35</td>
<td>1.38</td>
<td>1.10</td>
<td>165.6</td>
<td>132.0</td>
<td>47.18</td>
<td>65.02</td>
<td>15.54</td>
<td>18.42</td>
<td>66.86</td>
<td>0.70</td>
<td>35.20</td>
<td>3.65</td>
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<tr>
<td>W 8 \times 40</td>
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<td>1.13</td>
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<td>47.93</td>
<td>66.47</td>
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<td>33.69</td>
<td>2.98</td>
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</table>

\[ \therefore \; \text{The section W 8 \times 40 is OK for design condition (i).} \]
Design condition (ii); i.e., $F_c = 113.9^k$, $M_x = 1.1 k' = 13.2 k''$, $M_y = 14.9 k' = 178.8 k''$

<table>
<thead>
<tr>
<th>Section</th>
<th>$k_x$</th>
<th>$k_y$</th>
<th>$L_{ex}$ (in)</th>
<th>$L_{ey}$ (in)</th>
<th>$\lambda_x$</th>
<th>$\lambda_y$</th>
<th>$f_c$ (ksi)</th>
<th>$f_{all(c)}$ (ksi)</th>
<th>$f_{all(ex)}$ (ksi)</th>
<th>$f_{all(ey)}$ (ksi)</th>
<th>$f_{bx}$ (ksi)</th>
<th>$f_{by}$ (ksi)</th>
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<td>47.93</td>
<td>66.47</td>
<td>9.74</td>
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<td>64.78</td>
<td>0.44</td>
<td>33.69</td>
<td>20.61</td>
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<tr>
<td>W 8 x 58</td>
<td>1.52</td>
<td>1.18</td>
<td>182.4</td>
<td>141.6</td>
<td>49.97</td>
<td>67.43</td>
<td>6.66</td>
<td>18.13</td>
<td>59.60</td>
<td>0.29</td>
<td>32.74</td>
<td>12.27</td>
<td>0.89</td>
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</tbody>
</table>

The section W 8 x 58 is OK for design condition (ii)

Design condition (iii); i.e., $F_c = 118.8^k$, $M_x = 15.4 k' = 184.8 k''$, $M_y = 1.4 k' = 16.8 k''$

<table>
<thead>
<tr>
<th>Section</th>
<th>$k_x$</th>
<th>$k_y$</th>
<th>$L_{ex}$ (in)</th>
<th>$L_{ey}$ (in)</th>
<th>$\lambda_x$</th>
<th>$\lambda_y$</th>
<th>$f_c$ (ksi)</th>
<th>$f_{all(c)}$ (ksi)</th>
<th>$f_{all(ex)}$ (ksi)</th>
<th>$f_{all(ey)}$ (ksi)</th>
<th>$f_{bx}$ (ksi)</th>
<th>$f_{by}$ (ksi)</th>
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<td>6.95</td>
<td>18.13</td>
<td>59.60</td>
<td>4.02</td>
<td>32.74</td>
<td>1.17</td>
<td>0.60</td>
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</table>

The section W 8 x 58 is also OK for design condition (iii)

The section is the strongest about x-axis and weakest about y-axis; therefore case (ii) provides the most critical design condition here [in this case, it is even more critical than case (i)].

This needs to be kept in consideration while choosing the orientation of the column section in the building plan, so that bending moment about the weakest axis is kept as small as possible. In case of the building designed, with an almost square-shaped plan, this does not affect the design significantly. But for highly elongated plans (high L/B ratio), weakest axis of the section should not coincide with the weakest axis of the plan.
7. Design of Connections

The following sample connections will be designed for illustration.

1. A moment-resisting connection between column C₈ and beam B₇ will be designed for shear force 10.2 k and negative bending moment of 25.8 k.'

2. A base slab for column C₈ and the footing underneath will be designed for column axial force of 160.1 k

**Moment-resisting connection between column C₈ and beam B₇:**

Column C₈ is a W 8 × 58 section and beam B₇ is an S 8 × 18.4 section

(i) Tension Connection

Tensile Force (T) in cover plate due to moment = 25.8 × 12 k"/8" = 38.7k

Allowable bending stress = 24 ksi

∴ Area of cover plate required = 38.7k/24 ksi = 1.61 in²

For beam B₇, width of the flange b₇ = 4.00'

∴ Width of cover plate = 3", in order to allow for welding and clearance on both sides

∴ Thickness of cover plate = 1.61 in²/3" = 0.54"

Use cover plate of size 3" × 5/8", to be butt-welded to column flange and fillet-welded to tension flange of the beam.

Also, 5/8" thick stiffeners may be used between column flanges on both sides of the web

Connect cover plate to tension flange of beam by 0.25" fillet welds

Strength of weld = fᵥ × 0.707t = 12 × 0.707 × 0.25 = 2.12 k/

∴ Total length of weld required, Lₜ = T/2.12 = 38.7/2.12 = 18.25"

∴ Provide (8" + 3" + 8" = 19") long 0.25" thick weld around the cover plate.

Also keep an additional 4" unwelded on both sides of the cover plate

∴ Total length of cover plate = 8" + 4" = 12"
(ii) Compression Connection

Compressive Force (C) in cover plate due to moment is also = 38.7k

To transfer this compressive force to the column, the compressive flange of the beam is fillet-welded to a horizontal seat plate, dividing the weld length (thickness = 0.25\"”) into two 9.5\” parts on both sides of the beam. Providing an additional 0.5\” as clearance between beam and column, the length of the plate = 9.5\” + 0.5\” = 10\”.

If the width of the plate is chosen to be = 5\” (considering beam flange width of 4\”), its thickness = 1.61/5 = 0.32\”; i.e., provide 3/8\” thickness.

Provide 10\” × 5\” × 3/8\” horizontal seat plate, which is butt-welded to the column flange.

For the shear force of 10.2k, which is not considered too high, an unstiffened seat connection is designed at the bottom of the beam.

The length of web (B) required to transfer the force without web crippling of beam is

\[ B = \frac{R}{\sigma_p t_w} = \frac{10.2}{(0.75 \times 40 \times 0.27)} = 1.26" \]

Thickness of beam flange, \( t_f = 0.426" \).

Effective bearing width is the greater of (B – h₂ cot 30°) and B/2

Assuming a fillet of similar diameter, depth below root of fillet \( h_2 = 0.852" \)

\( \Rightarrow b = B - h_2 \cot 30° \) is negative

However \( b \geq B/2 \Rightarrow b = 0.63" \)

\( \therefore \) Assuming 0.5\” clearance, the minimum width of the horizontal leg of angle

\( = 0.63 + 0.5 = 1.13" \)

Selecting equal angle section of L 4 × 4 × 1/4, \( t_a = 0.25" \), assumed \( r_1 = 0.25" \)

\( \Rightarrow \) Overturning moment \( M = R \left( \frac{b}{2} + 0.5 - t_a - r_1 \right) \)

\( = 10.2 \times (0.31 + 0.5 - 0.25 - 0.25) = 3.16 \text{ k"} \)

\( \therefore \) Length of seating angle across the flange-width, \( L_a = 5" \) (in order to accommodate beam-flange width and welding).

\( \therefore \) Thickness \( t_a \) (required) = \( \sqrt{6M/\sigma_p L_a} = \sqrt{6 \times 3.16/(24 \times 5)} = 0.40" \)

Selecting equal angle section of L 4 × 4 × 5/16, \( t_a = 0.31" \), assumed \( r_1 = 0.31" \)
Overturning moment $M = R (b/2 + 0.5 - t_a - r_1) = 1.94 \text{k}''$

\[ \therefore \text{Thickness } t_a \text{ (required)} = \sqrt{(6M/\sigma_b L_a)} = \sqrt{(6 \times 1.94/(24 \times 5))} = 0.31'' \]

Select equal angle section of L 4 × 4 × 5/16

\[ \therefore \text{Moment at welds } M_w = R (b/2 + 0.5) = 10.2 \times (0.31 + 0.5) = 8.26 \text{k}'' \]

\[ \therefore \text{Horizontal shear force per length } V_h = 3M/d^2 = 3 \times 8.26/4^2 = 1.56 \text{k}'' \]

\[ \therefore \text{Vertical shear force per length } V_v = V/2d = 10.2/8 = 1.28 \text{k}'' \]

\[ \therefore \text{Resultant shear force per unit length, } V_r = \sqrt{V_h^2 + V_v^2} = 2.02 \text{k}'' \]

Strength of weld = $f_v \times 0.707t = 12 \times 0.707t = 8.48 t$

\[ \therefore \text{Required thickness of weld, } t = 2.02/8.48 = 0.24'' \Rightarrow \text{Provide } t = 1/4'' \]

A base slab for column C₈ and the footing underneath:

Axial load on column = 160.1 k and the additional moments are small enough to be neglected for the design of the base plate.

Assuming the base plate area = $A_p$ and allowable bearing pressure = 0.35 $f_c' = 1.05 \text{ ksi}$

\[ 1.05A_p = 160.1 \Rightarrow A_p = 160.1/1.05 = 152.5 \text{ in}^2 \]

\[ \therefore \text{Provide } 12'' \times 13'' \text{ base plate} \]

\[ \therefore \text{Bearing pressure } q = 160.1/(12 \times 13) = 1.03 \text{ ksi} \]

Thickness of the plate is given by the

\[ t = \sqrt{3q(a^2 - vb^2)/f_b} \]

\[ = \sqrt{3 \times 1.03 \times (2.39^2 - 0.25 	imes 2^2)/24} = 0.78'' \]

\[ \therefore \text{Provide a 7/8'' thick base plate} \]

Although fastenings are not required to transmit any load, connect each flange of the column to the base plate by 0.5'' welds.

Also connect the base plate to the footing below using 4 #6 bolts and anchor them their development lengths.