Hybrid Masonry

Calculation Outline for Type I Hybrid Masonry Walls

1. Layout the building.
2. Determine fenestration.
3. Locate masonry control joints in wall panels and at columns.
4. Determine lateral stiffness for each wall panel.
5. Determine lateral stiffness of each wall.
7. Distribute lateral loads to walls and wall panels.
8. Design the steel framing.
9. Design wall panels for in-plane shear.
10. Design wall panels for out-of-flexure
11. Summarize the results.
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3. Locate masonry control joints in wall panels and at columns.  
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4. Determine lateral stiffness for each wall panel.  
   NORTH WALL AND SOUTH WALL ........................................... L1-L6  
   WEST WALL AND EAST WALL ............................................. S1-S2

5. Determine lateral stiffness of each wall.  
   NORTH WALL AND SOUTH WALL ........................................... L7-L8  
   WEST WALL AND EAST WALL ............................................. S3

   FOR THIS EXAMPLE, LOADS PROVIDED.

7. Distribute lateral loads to walls and wall panels.  
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10. Design wall panels for out-of-plane flexure. .......................... F1-F3

11. Summarize the results.  
    NORTH WALL AND SOUTH WALL ........................................... L21-L23  
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Ordinary (Type I) Hybrid

1/2 joints on columns & girders

Masonry Dimensions

\[ 14' - 16' - \frac{1}{2}'' = 14' - 1.37' = 12.63' \]
\[ - \frac{7.33}{5.33} \]

\[ 30' - 1' - 1'' = 28.92' \]

General Formulas to Develop Stiffness

\[ A_c = \frac{PL^3}{3EmI} + \frac{1.2PL}{EA} \]
\[ \Rightarrow \frac{PL}{Em} \left[ 4 \left( \frac{h}{L} \right)^3 + 3 \left( \frac{h}{L} \right) \right] \]
MOC 9.2-3

\[ AF = \frac{PL}{Em} \left[ \left( \frac{h}{L} \right)^3 + \frac{3}{2} \left( \frac{h}{L} \right) \right] \]

\[ K = \frac{1}{\Delta} \]
3 WINDOW PANEL

1. $\Delta_1$ as cantilever:
   
   $4 \left( \frac{h}{l} \right)^3 + 3 \left( \frac{h}{l} \right) = 4 \left( \frac{5.33}{2692} \right)^3 + 3 \left( \frac{5.33}{2692} \right) = 4 \left( \frac{.0062}{2692} \right) + 3 \left( .0184 \right) = .025 + .552 = .577$

2. $\Delta_2$ as fixed:
   
   $\left( \frac{h}{l} \right)^3 + 3 \left( \frac{h}{l} \right) = \left( \frac{4}{4} \right)^3 + 3 \left( \frac{4}{4} \right) = 1 + 3 = 4.0 \quad K = \frac{1}{4}$

3. $\Delta_3$ as fixed:
   
   $\left( \frac{3.33}{2692} \right)^3 + 3 \left( \frac{3.33}{2692} \right) = \left( \frac{3}{2692} \right)^3 + 3 \left( \frac{3}{2692} \right) = \left( .0015 \right) + 3 \left( .0015 \right) = .0045 + .0345 = .0391$

4. $\Delta_{wall} = \Delta_{solid(c)} - \Delta_{shy(c)} + \Delta_{1,1}(f)$

5. $\Delta_{shy(c)} = 4 \left( \frac{7.33}{2692} \right)^3 + 3 \left( \frac{7.33}{2692} \right) = .065 + 7.60 = .825$

6. $\Delta_{1,1}(f) = \frac{1}{4 + \frac{1}{4} + \frac{1}{4} + \frac{1}{4}} = 1.0$
\[ \Delta_2,5(f) = \Delta_{\text{solid}} 2,5(f) - \Delta_{\text{rip}}(f) + \Delta_2(f) \]

\[ \Delta_{\text{solid}} 2,5(f) = \left( \frac{7.33}{29.92} \right)^3 + 3 \left( \frac{7.33}{29.92} \right) = 0.016 + 0.760 = 0.776 \]

\[ \Delta_{\text{rip}}(f) = \left( \frac{4.0}{29.92} \right)^3 + 3 \left( \frac{4.0}{29.92} \right) = 0.003 + 0.415 = 0.418 \]

\[ \Delta_2(f) = 1.0 \text{ see p.2} \]

\[ \Delta_{2,5} = 0.776 - 0.418 + 1.0 = 1.358 \]

\[ \Delta_{\text{well}} = 1.649 - 0.825 + 1.358 = 2.182 \]

\[ k = \frac{1}{\Delta_{\text{well}}} = 0.458 \]

\[ \Delta_{\text{rip}}(f) = \left( \frac{12.66}{29.92} \right)^3 + 3 \left( \frac{12.66}{29.92} \right) = 0.334 + 1.313 = 1.649 \]
\[ \Delta_c = \frac{4 \left( \frac{1.466}{28.92} \right)^3}{(28.92)} \approx 1.649 \quad \text{[p. L3]} \]

\[ K_c = \frac{1}{\Delta_c} = 0.606 \]

**Panel w/o Window**

**Panel w/ Door Opening & Window**
\[ \Delta_{\text{wall}} = \Delta_{c} - \Delta_{\text{shp}}(c) + \Delta_{2a,2b,2c}(f) \]

\[ \Delta_{2a,2b,2c}(f) = \frac{1}{K_{2a,2b,2c}(f)} \]

\[ K_{2a,2b,2c}(f) = K_{2a}(f) + K_{2b,2c}(f) \]

\[ K_{2b,2c}(f) = \frac{1}{\Delta_{2b,2c}(f)} \]

\[ \Delta_{2a,2c}(f) = \Delta_{\text{solid}}(f) - \Delta_{\text{shp}}(c - \Delta_{2a,2b}(f) \approx 0.825 \quad (2.4) \]

\[ \Delta_{z}(f) = \left( \frac{7.93}{99.2} \right)^{3} + \left( \frac{7.93}{199.2} \right)^{2} \approx 1.050 + 1.104 = 1.154 \]

\[ \Delta_{2b,2c}(f) = \frac{1}{99.2} \approx 0.008 + 0.002 = 0.010 \]

\[ \Delta_{2a,2c}(f) = 1.154 - 0.010 + 1.147 = 1.693 \]

\[ K_{2a,2b,2c}(f) = \frac{1}{\Delta_{2a,2b,2c}(f)} = 0.591 \]

\[ K_{2a}(f) = \frac{1}{\Delta_{2a}(f)} = \left( \frac{7.93}{6} \right)^{3} + \left( \frac{7.93}{199.2} \right)^{2} \approx 1.823 + 3.665 = 5.488 = 0.182 \]

\[ K_{2b,2c}(f) = 0.591 + 0.182 = 0.773 \]

\[ \Delta_{\text{wall}} = 1.649 - 0.825 + 1.294 = 2.118 \]

\[ K_{\text{wall}} = 0.472 \]
\[
\Delta = \Delta_{\text{door}} - \Delta_{\text{strip}} + \Delta_{2a, 2b}
\]
\[
\Delta_{\text{strip}} = 1.649 \text{ (p14)}
\]
\[
\Delta_{\text{door}} = 0.825 \text{ (p14)}
\]
\[
\Delta_{2a, 2b}(f) = \frac{1}{K_{2a, 2b}(f)}
\]
\[
K_{2a, 2b}(f) = K_{2a}(f) + K_{2b}(f) = \frac{1}{\Delta_{2a}(f)} + \frac{1}{\Delta_{2b}(f)}
\]
\[
\Delta_{2a}(f) = \left( \frac{7.33}{6} \right)^3 + \left( \frac{7.33}{6} \right)^3 = 1.823 + 1.823 = 3.646
\]
\[
\Delta_{2b}(f) = \left( \frac{7.33}{19.92} \right)^3 + \left( \frac{7.33}{19.92} \right)^3 = 0.050 + 1.104 = 1.154
\]
\[
K_{2a, 2b}(f) = \frac{1}{3.646} + \frac{1}{1.154} = 0.162 + 0.867 = 1.029
\]
\[
\Delta_{\text{wall}} = 1.649 - 0.825 + 0.954 = 1.778
\]
\[
K_{\text{wall}} = 0.562
\]
Determine Maxmum Dimensions

12" glazing assumed
12" columns
1/2" joints

Type A

6.25 4.51 6.25 1/2"
17.15, ~ 17.10

Type B

6.25 4.51 6.25 1/2" 74.82, ~ 21.8
\[ \Delta_{\text{wall}} = \Delta_{\text{solid}}(c) - \Delta_{\text{ship}}(c) + \Delta_{2a,2b,3}(f) \]

\[ \Delta_{2a,2b,3}(f) = \Delta_{\text{ship}}(f) - \Delta_{2}(f) + \Delta_{2a,2b}(f) \]

\[ \Delta_{\text{solid}}(c) = 4 \left( \frac{12.66}{17.10} \right)^3 + 3 \left( \frac{12.66}{17.10} \right) \cdot 1.623 + 2.72 = 3.844 \]

\[ \Delta_{\text{ship}}(c) = 4 \left( \frac{7.22}{17.10} \right)^3 + 3 \left( \frac{7.22}{17.10} \right) \cdot 0.315 + 1.286 = 1.601 \]

\[ \Delta_{\text{d}}(c) = \left( \frac{7.33}{17.10} \right)^3 + 3 \left( \frac{7.33}{17.10} \right) = 0.019 + 1.286 = 1.305 \]

\[ \Delta_{2a}(f) = \left( \frac{4}{6.25} \right)^3 + 3 \left( \frac{4}{6.25} \right) = 0.762 + 1.920 = 2.182 \]

\[ \frac{1}{\Delta_{2a,2b,3}(f)} = \frac{1}{\Delta_{2a}} \cdot \frac{1}{\Delta_{2b}} \]

\[ \Delta_{2b}(f) = \left( \frac{4}{6.18} \right)^3 + 3 \left( \frac{4}{6.18} \right) = 0.271 + 1.942 = 2.213 \]

\[ \Delta_{2}(f) = \left( \frac{4}{5.10} \right)^3 + 3 \left( \frac{4}{5.10} \right) = 0.013 + 0.702 = 0.715 \]

\[ \Delta_{\text{wall}}(f) = (3.8 + 4) - (1.601) + \left[ 1.545 - 0.715 + 1.1 \right] = 3.993 \]

\[ K_{\text{wall}} = 0.250 \]
# Walls with Panel Stiffnesses

<table>
<thead>
<tr>
<th>Panel Stiffnesses</th>
<th>Stiffness/Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.458</td>
<td>2k = 1.374</td>
</tr>
<tr>
<td>0.458</td>
<td>2k = 1.388</td>
</tr>
</tbody>
</table>

**North Wall**

\[
\begin{align*}
V_r & \rightarrow V_r/3 \\
V_z & \rightarrow \frac{V_r \times 1.158}{458} \\
\end{align*}
\]

**Distribution of Load**

- 33\% \( V_r + V_z \)
- 34\% \( V_r + V_z \)
- 33\% \( V_r + V_z \)

_North Wall_
\[ \sqrt{r} \quad \sqrt{2} \]

- \( r = 0.458 \) with 30% shading
- \( r = 0.562 \) with 38% shading
- \( r = 0.458 \) with 31% shading
- \( r = 0.606 \) with 40% shading
- \( r = 0.488 \) with 30% shading

\[ 2k = 1.522 \]
\[ 3k = 1.478 \]

SOUTH WALL
TWO WINDOW PANEL

IDENTICAL TO (A)

Kwall = 0.250

\( i: 50\% \text{ distribution between A + B} \)

SINCE WINDOW ALIGNS W/ CJ, IGNORE STIFFNESS CONTRIBUTION OVER EDGE WINDOW
NORTH WALL

SOUTH WALL
Worst Cases

Design Loads

5.5 k

5.7 k

1st

2nd
WALL DESIGN LOADS

WEST WALL
(EAST WALL SIMILAR)
Steel Design Criteria -
Hybrid -Type I
Example Office Building

- Steel is designed for gravity loads only; the masonry shear walls will resist lateral loads.
- Floor design is composite.
- ASD method is used for the steel design to be consistent with the preferred masonry design method (ASD). Other methods are acceptable.
- See page SS-2 for loads including pre- and post-composite dead load breakdown.
- A construction live load of 20 psf is included in all steel designs.
**OFFICE BLD - 2 STORY (SEE PI)**

<table>
<thead>
<tr>
<th>FLOOR: LL = 50 PSF</th>
<th>LOAD ON STEEL</th>
<th>LOAD ON MASONRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL =</td>
<td>541 PSF</td>
<td>-</td>
</tr>
<tr>
<td>PRE-COMPOSITE</td>
<td>54 PSF</td>
<td>-</td>
</tr>
<tr>
<td>MASONRY WALL ABOVE</td>
<td>1.5 PSF</td>
<td>-</td>
</tr>
<tr>
<td>5½&quot; SLAB ON DECK</td>
<td>8 PSF</td>
<td>-</td>
</tr>
<tr>
<td>DECK (22 GA)</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>STEEL</td>
<td>2 PSF</td>
<td>-</td>
</tr>
</tbody>
</table>

**TOTAL** 86 PSF 0 PSF

<table>
<thead>
<tr>
<th>ROOF: SL = 40 PSF</th>
<th>LOAD ON STEEL</th>
<th>LOAD ON MASONRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL =</td>
<td>42 PSF</td>
<td>-</td>
</tr>
<tr>
<td>PRE-COMPOSITE</td>
<td>1.5 PSF</td>
<td>-</td>
</tr>
<tr>
<td>4½&quot; SLAB ON DECK</td>
<td>7 PSF</td>
<td>-</td>
</tr>
<tr>
<td>DECK (22 GA)</td>
<td>2 PSF</td>
<td>-</td>
</tr>
<tr>
<td>STEEL</td>
<td>5.5 PSF</td>
<td>-</td>
</tr>
<tr>
<td>MISC DL</td>
<td>10 PSF</td>
<td>-</td>
</tr>
</tbody>
</table>

**TOTAL** 68 PSF 0 PSF
Floor Map

Floor Type: Second

W30x90 (15) W24x55 (24)
W30x90 (15) W30x90 (15) W30x90 (15)
Floor Map

Floor Type: Roof

1. W24x76 (15)
2. W24x76 (15)
3. W24x76 (15)
4. W24x76 (15)

A
W24x55 (20)
W24x55 (20)
W24x55 (20)
W24x55 (20)
W24x55 (20)
W24x55 (20)
W24x55 (20)
W24x55 (20)
W21x44 (20)

B
W24x76 (15)
W24x76 (15)
W24x76 (15)
2ND FLOOR BEAM PLAN

ROOF BEAM PLAN
BEAM B1 DESIGN

- TYPICAL INTERIOR BAY, 2ND FLOOR
- LOAD COMBO ANALYZED: D + SDL + L

SPAN = 40'  
TRIB = 7.5'  
Fy = 50 ksi  
Lb = 0'-0"

ASD METHOD

USE: RAM SBEAM TO DESIGN

\[ D = D + SDL = (480 + 173) = 653 \text{ klf} \]

\[ \text{CONSTR. D} = 480 \text{ klf} \]

\[ L = 375 \text{ klf} \]

\[ \text{CONSTR. L} = 2.0 \text{ psf} \times 7.5' = 150 \text{ klf} \]

DEFLECTION: LIMIT PRE-COMPOSITE CROSS BAY Δ TO SMALLER OF

\[ \frac{L}{360} \text{ OR } 1'' \text{ PER RBA TECH NOTE } \Rightarrow \frac{L}{360} = \frac{50 \times 12}{360} = 1.67'' \Rightarrow \text{ USE } 1'' \]

DEFAULTS:

\begin{align*}
\text{L/d} & \quad \Delta \\
\text{Initial} & \quad 0.00 \quad 0.75'' \\
\text{Post LL} & \quad 360 \quad 0.00 \\
\text{Post Super} & \quad 360 \quad 0.00 \\
\text{Net Tot.} & \quad 240 \quad 0.00 \\
\end{align*}

OPTIMIZED BEAM IS: \( \mathbf{424 \times 55 [ 24 ]} \)

\[ \Delta = 1.1'' \quad \text{(TOTAL)} \]

\[ \Delta = 0.7'' \quad \text{(INITIAL)} \]

INITIAL Δ ALLOWABLE FOR GIRDER B3 = 1.0'' - 0.7'' = 0.3''
Gravity Beam Design

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (40.00,0.00)
- Beam Size (Optimum) = W24X55
- Total Beam Length (ft) = 40.00
- Distance to Adjacent Beam on Left (ft) = 7.5
- Distance to Adjacent Beam on Right (ft) = 7.5

COMPOSITE PROPERTIES (Not Shored):
- Concrete thickness (in) Left 3.50 Right 3.50
- Unit weight concrete (pcf) Left 150.00 Right 150.00
- f'c (ksi) Left 3.00 Right 3.00
- Decking Orientation perpendicular
- Decking type
  - USD 2" Lok-Floor
- beff (in) Left 90.00 Y bar (in) Right 22.51
- Seff (in3) Left 149.02 Str (in3) Right 182.02
- Ieff (in4) Left 2756.74 It (in4) Right 4111.41
- Stud length (in) Left 4.00 Stud diam (in) Right 0.75
- Stud Capacity (kips) q = 8.6
- # of studs: Full = 72 Partial = 24 Actual = 24
- Number of Stud Rows = 1 Percent of Full Composite Action = 25.77
- Top flange braced by decking.

LINE LOADS (k/ft):
- Load Dist (ft) DL CDL LL CLL
  - 1 0.000 0.653 0.480 0.375 0.150
  - 40.000 0.653 0.480 0.375 0.150

SHEAR: Max V (DL+LL) = 20.56 kips \( f_v = 2.30 \text{ ksi} \) \( f_v = 18.78 \text{ ksi} \)

MOMENTS:
- Span Cond Moment @ Lb Cb Tension Flange Compr Flange
  - kip-ft ft ft fb Fb fb Fb
  - Center PreCmp+ 126.0 20.0 0.0 1.00 13.15 33.00 13.15 33.00
  - Max + 205.6 20.0 --- ---
  - Mmax/Seff 16.56 33.00 --- ---
  - Mconst/Sx+Mpost/Seff 18.84 45.00 --- ---
  - Controlling 205.6 20.0 --- --- 16.56 33.00 --- ---
  - fc (ksi) = 0.24 Fc = 1.35

REACTIONS (kips):
- Initial reaction Left 12.60 Right 12.60
- DL reaction Left 13.06 Right 13.06
- Max +LL reaction Left 7.50 Right 7.50
- Max +total reaction Left 20.56 Right 20.56

DEFLECTIONS:
- Initial load (in) at 20.00 ft = -0.701 L/D = 685
<table>
<thead>
<tr>
<th>Description</th>
<th>Distance (ft)</th>
<th>Value</th>
<th>L/D</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load (in)</td>
<td>20.00</td>
<td>-0.270</td>
<td>L/D</td>
<td>1777</td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>20.00</td>
<td>-0.395</td>
<td>L/D</td>
<td>1216</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>20.00</td>
<td>-1.096</td>
<td>L/D</td>
<td>438</td>
</tr>
</tbody>
</table>
BEAM B2 DESIGN

- TYPICAL EDGE BEAMS ALONG COL LINES 1 & 4, 2ND FLOOR
- LOAD COMBO ANALYZED = D + SDL + L

SPAN = 39'

D = 56 PSF x 3.75' = 210 KLF

WD = 23 PSF x 3.75' = 86 KLF

Fy = 50 ksi

L = 50 PSF x 3.75' = 188 KLF

L_b = 0'-0"

WALL WEIGHT = 541 psf; ADD TO D

ASD METHOD

* Estimate for steel wt (8 PSF) subtracted; let RAM include in calc, since it will likely be more than 8 PSF

USE: RAM SBEAM TO DESIGN

D = D + SDL + WALL = (210 + 0.86 + 541) = 837 KLF

CONSTR. D = 210 KLF

L = 188 KLF

CONSTR. L = 20 PSF x 3.75' = 75 KLF

DEFORMATION: BEAM WILL SUPPORT A MASONRY WALL ⇒ ⅛" OR 0.3"

8/600 = 40 x 12 = 0.8"

USE 0.3" FOR POST-COMPOSITE SUPERIMPOSED DEFLEC.

* SEE BI FOR ADDITIONAL Δ REQUIREMENTS.

BEAM DIAGRAM:

\[
\begin{align*}
D + S D L + L &= 1025 \text{ KLF} \\
39' \\
R_L &= R_R = 21.6 \text{ k} \\
M &= 210.9 \text{ k-ft}
\end{align*}
\]

OPTIMIZED BEAM: W27×84 [20]  SEE RAM OUTPUT
## Gravity Beam Design

**STEEL CODE:** ASD 9th Ed.  
**LC:** D+SDL+L  
**SPAN INFORMATION (ft):**  
-I-End (0.00,0.00)  
-J-End (39.00,0.00)  
-Beam Size (Optimum) = W27X84  
-Total Beam Length (ft) = 39.00  
-Distance to Adjacent Beam on Left (ft) = 7.5  
-Distance to Adjacent Edge on Right (ft) = 0.4

### COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th>Property</th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>3.50</td>
<td>3.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>fc (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>perpendicular</td>
<td>perpendicular</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>50.04</td>
<td>Y bar (in)</td>
</tr>
<tr>
<td>Seff (in3)</td>
<td>260.68</td>
<td>Str (in3)</td>
</tr>
<tr>
<td>Ieff (in4)</td>
<td>4877.72</td>
<td>Itr (in4)</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>4.00</td>
<td>Stud diam (in)</td>
</tr>
<tr>
<td>Stud Capacity (kips)</td>
<td>q = 8.6</td>
<td></td>
</tr>
<tr>
<td># of studs: Full</td>
<td>48</td>
<td>Partial = 20</td>
</tr>
<tr>
<td>Number of Stud Rows</td>
<td>1</td>
<td>Actual = 20</td>
</tr>
<tr>
<td>Percent of Full Composite Action = 38.62</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top flange braced by decking.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.084</td>
<td>0.084</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.837</td>
<td>0.210</td>
<td>0.188</td>
<td>0.075</td>
</tr>
<tr>
<td>39.000</td>
<td>0.837</td>
<td>0.210</td>
<td>0.188</td>
<td>0.075</td>
<td></td>
</tr>
</tbody>
</table>

### SHEAR: Max V (DL+LL) = 21.63 kips  
\( f_v = 1.85 \text{ ksi} \)  
\( F_v = 19.44 \text{ ksi} \)

### MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kip-ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td>fb</td>
<td>Fb</td>
</tr>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>70.2</td>
<td>19.5</td>
<td>0.0</td>
<td>1.00</td>
<td>3.96</td>
<td>33.00</td>
</tr>
<tr>
<td></td>
<td>Max +</td>
<td>210.9</td>
<td>19.5</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Mmax/Seff</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9.71</td>
<td>33.00</td>
</tr>
<tr>
<td>Mconst/Sx+Mpost/Seff</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10.29</td>
<td>45.00</td>
</tr>
</tbody>
</table>

Controlling  

| fc (ksi) | 0.38 | Fc = 1.35 |

### REACTIONS (kips):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>7.20</td>
<td>7.20</td>
</tr>
<tr>
<td>DL reaction</td>
<td>17.97</td>
<td>17.97</td>
</tr>
<tr>
<td>Max + LL reaction</td>
<td>3.67</td>
<td>3.67</td>
</tr>
<tr>
<td>Max + total reaction</td>
<td>21.63</td>
<td>21.63</td>
</tr>
</tbody>
</table>
## Gravity Beam Design

**B2 - HYBRID TYPE I**

**LC: D + SOL + L**

**DEFORMATIONS:**

<table>
<thead>
<tr>
<th>Description</th>
<th>Load Level</th>
<th>L/D Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial load (in)</td>
<td>at 19.50 ft</td>
<td>-0.185</td>
</tr>
<tr>
<td>Live load (in)</td>
<td>at 19.50 ft</td>
<td>-0.069</td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>at 19.50 ft</td>
<td>-0.300</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>at 19.50 ft</td>
<td>-0.485</td>
</tr>
</tbody>
</table>

**L/D Values:**

- **L/D = 2524**
- **L/D = 6765**
- **L/D = 1560**
- **L/D = 964**
BEAM B2 DESIGN cont't w/ DIFFERENT LC

- TYPICAL EDGE BEAMS ALONG COL LINES 1-4, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L + W

BEAM SUPPORTS 2 SHEAR WALLS OF EQUAL DIMENSION

\[ V = 6.3 \text{k} \]
\[ w = 541 \text{ plf} \]
\[ S = k d^2/6 = 8 (17.1 \times 12)^2/6 \]
\[ S = 56143 \text{ in}^3 \]
\[ M = 6.3 \text{k} \times 13' = 81.7 \text{k-ft} \]
\[ f_b = \frac{M}{S} = \frac{81.9 \times 12}{56143} = 0.175 \text{ ksi} \]
\[ 0.175 \text{ ksi} \times 8 \text{ in} \times 12 \text{ in} = 1.68 \text{ k/ft} \]

\[ w_1 = 541 - 1.68 = 1.14 \text{ k/ft} \]
\[ w_2 = 541 + 1.68 = 2.22 \text{ k/ft} \]

SPAN = 39'
TRIB = 3.75'
\[ f_y = 50 \text{ ksi} \]
\[ L_k = 0'-0'' \]
ASD METHOD

56 PSF \times 3.75' = 210 KLF
23 PSF \times 3.75' = 86 KLF
50 PSF \times 3.75' = 188 KLF
WALL WT. = 541 plf

*ESTIMATE FOR STEEL SUBTRACTED
BEAM B2 DESIGN  cont w/ DIFFERENT LC

USE: RAM SBEAM TO DESIGN

\[ D = D + 5DL = (0.210 + 0.086) = 0.296 \text{ klf} \]

\[ \text{CONST. } D = 0.210 \text{ klf} \]

\[ L = 0.183 \text{ klf} \]

\[ \text{CONST. } L = 0.075 \text{ klf} \]

ADD PARTIAL WALL TO D = 0.581 klf

\[ D + 5DL + L = 0.484 \]

\[ R_L = 17.2 \text{ k} \]

\[ R_R = 26.6 \text{ k} \]

\[ M = 215.3 \text{ k-ft} \]

OPTIMIZED BEAM: [W27 x 84] (24)  SEE RAM OUTPUT
Gravity Beam Design

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00)  J-End (39.00,0.00)
- Beam Size (Optimum) = W27X84
- Total Beam Length (ft) = 39.00
- Distance to Adjacent Beam on Left (ft) = 7.5
- Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>3.50</td>
<td>3.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>f'c (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>perpendicular</td>
<td>perpendicular</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>50.04</td>
<td>21.00</td>
</tr>
<tr>
<td>Seff (in3)</td>
<td>260.68</td>
<td>289.73</td>
</tr>
<tr>
<td>Ieff (in4)</td>
<td>4877.72</td>
<td>6112.69</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>4.00</td>
<td>0.75</td>
</tr>
</tbody>
</table>

- Stud Capacity (kips) q = 8.6
- # of studs: Full = 53 Partial = 24 Actual = 24
- Number of Stud Rows = 1 Percent of Full Composite Action = 36.81
- Top flange braced by decking.

LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.084</td>
<td>0.084</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>39.000</td>
<td>0.084</td>
<td>0.084</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.296</td>
<td>0.210</td>
<td>0.188</td>
<td>0.075</td>
</tr>
<tr>
<td></td>
<td>39.000</td>
<td>0.296</td>
<td>0.210</td>
<td>0.188</td>
<td>0.075</td>
</tr>
<tr>
<td>3</td>
<td>17.160</td>
<td>0.541</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>21.160</td>
<td>0.541</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4</td>
<td>0.000</td>
<td>-1.140</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>17.160</td>
<td>2.220</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>5</td>
<td>21.160</td>
<td>-1.140</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>39.000</td>
<td>2.220</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

SHEAR: Max V (DL+LL) = 26.01 kips  fv = 2.22 ksi  Fv = 19.44 ksi

MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>70.2</td>
<td>19.5</td>
<td>0.0</td>
<td>1.00</td>
<td>3.96</td>
<td>33.00</td>
</tr>
<tr>
<td></td>
<td>Max</td>
<td>215.3</td>
<td>16.5</td>
<td></td>
<td></td>
<td>9.91</td>
<td>33.00</td>
</tr>
<tr>
<td></td>
<td>Mmax/Seff</td>
<td></td>
<td>9.91</td>
<td></td>
<td></td>
<td>9.91</td>
<td>33.00</td>
</tr>
<tr>
<td></td>
<td>Mconst/Sx+Mpost/Seff</td>
<td>215.3</td>
<td>16.5</td>
<td></td>
<td></td>
<td>9.91</td>
<td>33.00</td>
</tr>
<tr>
<td></td>
<td>Controlling</td>
<td></td>
<td>215.3</td>
<td>16.5</td>
<td></td>
<td>9.91</td>
<td>33.00</td>
</tr>
</tbody>
</table>

fc (ksi) = 0.39  Fc = 1.35
### REACTIONS (kips):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>7.20</td>
<td>7.20</td>
</tr>
<tr>
<td>DL reaction</td>
<td>13.55</td>
<td>22.35</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>3.67</td>
<td>3.67</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>17.22</td>
<td>26.01</td>
</tr>
</tbody>
</table>

### DEFLECTIONS:

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial load (in)</td>
<td>at</td>
<td>19.11</td>
<td>-0.185</td>
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<tr>
<td>Live load (in)</td>
<td>at</td>
<td>19.11</td>
<td>-0.069</td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>at</td>
<td>19.11</td>
<td>-0.296</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>at</td>
<td>19.11</td>
<td>-0.481</td>
</tr>
</tbody>
</table>

L/D = 2525                  
L/D = 6768                  
L/D = 1582                  
L/D = 973
BEAM B3 DESIGN

- TYPICAL ALONG COL. LINES A-B, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L

SPAN = 30'

Fy = 50 ksi;

Lb = 7.5'

ASD METHOD

(3) POINT LOADS: 1@ 7.5', 1@ 15', 1@ 22.5'

D = 13.06 k (FROM RAM OUTPUT FOR B1)

L = 7.5 k (FROM RAM OUTPUT FOR B1)

CONSTR. DL = 64 psf x 7.5' x \frac{32}{2} = 9.6 k

CONSTR. LL = 20 psf x 7.5' x \frac{40}{2} = 3.0 k

UNIFORM LOAD FROM SUPPORTED WALL = 541 plf

\( \delta \) req'd = \( l/600 \) or 0.3', \( l/600 \) = 0.6':: USE 0.3'

* SEE B1 FOR ADDITIONAL \( \delta \) REQUIREMENTS

BEAM DIAGRAM:

\[ M = 377,000 \text{ k-ft} \]

OPTIMIZED BEAM IS: \[ W30 \times 90 [15'] \] SEE RAM OUTPUT
Gravity Beam Design

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft):  I-End (0.00,0.00)  J-End (30.00,0.00)
Beam Size (Optimum) = W30x90
Total Beam Length (ft) = 30.00
Distance to Adjacent Beam on Left (ft) = 40.0
Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>3.50</td>
<td>3.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>fc (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>parallel</td>
<td>parallel</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>50.04</td>
<td>Y bar(in)</td>
</tr>
<tr>
<td>Seff (in^3)</td>
<td>290.50</td>
<td>Str (in^3)</td>
</tr>
<tr>
<td>Ieff (in^4)</td>
<td>5851.38</td>
<td>Itr (in^4)</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>4.00</td>
<td>Stud diam (in)</td>
</tr>
<tr>
<td>Stud Capacity (kips) q</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td># of studs: Full</td>
<td>50</td>
<td>Partial</td>
</tr>
<tr>
<td>Number of Stud Rows</td>
<td>1</td>
<td>Actual</td>
</tr>
<tr>
<td>Percent of Full Composite Action</td>
<td>28.04</td>
<td></td>
</tr>
<tr>
<td>Top flange braced by decking.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

POINT LOADS (kips):

<table>
<thead>
<tr>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.500</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>15.000</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>22.500</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.090</td>
<td>0.090</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>30.000</td>
<td>0.090</td>
<td>0.090</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.541</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>30.000</td>
<td>0.541</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

SHEAR: Max V (DL+LL) = 40.30 kips  fv = 3.03 ksi  Fv = 17.85 ksi

MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment @</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange fb</th>
<th>Compr Flange fb</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kip-ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>199.1</td>
<td>15.0</td>
<td>7.5</td>
<td>1.13</td>
<td>9.75</td>
</tr>
<tr>
<td></td>
<td>Max +</td>
<td>379.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>9.75</td>
</tr>
<tr>
<td></td>
<td>Mmax/Seff</td>
<td>15.67</td>
<td>33.00</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Mconst/Sx+Mpost/Seff</td>
<td>16.85</td>
<td>45.00</td>
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<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
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<td>379.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>15.67</td>
</tr>
<tr>
<td></td>
<td>fc (ksi)</td>
<td>0.44</td>
<td>1.35</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

f_c = 0.44  f_c = 1.35
# Gravity Beam Design

REACTIONS (kips):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>20.25</td>
<td>20.25</td>
</tr>
<tr>
<td>DL reaction</td>
<td>29.05</td>
<td>29.05</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>11.25</td>
<td>11.25</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>40.30</td>
<td>40.30</td>
</tr>
</tbody>
</table>

DEFLECTIONS:

<table>
<thead>
<tr>
<th></th>
<th>at</th>
<th></th>
<th></th>
<th>L/D</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial load (in)</td>
<td>15.00</td>
<td>-0.227</td>
<td>L/D = 1584</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Live load (in)</td>
<td>15.00</td>
<td>-0.102</td>
<td>L/D = 3528</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>15.00</td>
<td>-0.207</td>
<td>L/D = 1737</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>15.00</td>
<td>-0.435</td>
<td>L/D = 828</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
BEAM B3 DESIGN con't w/ DIFFERENT LC

- TYPICAL EDGE BEAM ALONG COL LINES 1-4, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L + W

BEAM SUPPORTS SHEAR WALL

\[ V = 2.2k \]
\[ W = 541 \text{ PLF} \]
\[ S = \frac{1}{6}d^2/6 = 8(29 \times 12)^2/6 \]
\[ S = 161472 \text{ in}^3 \]
\[ M = 2.2 \times 13' = 28.6 \text{ k-ft} \]
\[ F_b = \frac{M}{3} = \frac{28.6(12)}{161472} = .0021 \text{ ksf} \]
\[ .0021 \text{ ksf} \times 8' \times 12'' = .20 \text{ k/ft} \]
\[ W_1 = .541 -.20 \text{ k/ft} = .341 \]
\[ W_2 = .541 + .30 \text{ k/ft} = .741 \]

\[ .341 \text{ k/ft} \]
\[ .741 \text{ k/ft} \]

\[ R_1 = \]
\[ M = \]

OPTIMIZED BEAM IS: W30 x 90 [15] SEE RAM OUTPUT
Gravity Beam Design

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft):  I-End (0.00,0.00)  J-End (30.00,0.00)
- Beam Size (Optimum) = W30X90
- Total Beam Length (ft) = 30.00
- Distance to Adjacent Beam on Left (ft) = 40.0
- Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>3.50</td>
<td>3.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>f'c (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>parallel</td>
<td>parallel</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>50.04</td>
<td>Y bar(in)</td>
</tr>
<tr>
<td>Seff (in3)</td>
<td>290.50</td>
<td>Str (in3)</td>
</tr>
<tr>
<td>leff (in4)</td>
<td>5851.38</td>
<td>Itr (in4)</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>4.00</td>
<td>Stud diam (in)</td>
</tr>
<tr>
<td>Stud Capacity (kips)</td>
<td>q = 11.5</td>
<td></td>
</tr>
<tr>
<td># of studs:</td>
<td>Full = 50</td>
<td>Partial = 15</td>
</tr>
<tr>
<td>Number of Stud Rows</td>
<td>1</td>
<td>Actual = 15</td>
</tr>
<tr>
<td>Percent of Full Composite Action</td>
<td>28.04</td>
<td></td>
</tr>
<tr>
<td>Top flange braced by decking.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

POINT LOADS (kips):

<table>
<thead>
<tr>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.500</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
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<tr>
<td>15.000</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>22.500</td>
<td>13.06</td>
<td>9.60</td>
<td>7.50</td>
<td>3.00</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.090</td>
<td>0.090</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>30.000</td>
<td>0.090</td>
<td>0.090</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.341</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>30.000</td>
<td>0.741</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

SHEAR: Max V (DL+LL) = 41.30 kips  \( f_v = 3.11 \text{ ksi} \)  \( F_v = 17.85 \text{ ksi} \)

MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kip-ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td>fb</td>
<td>Fb</td>
</tr>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>199.1</td>
<td>15.0</td>
<td>7.5</td>
<td>1.13</td>
<td>9.75</td>
<td>30.00</td>
</tr>
<tr>
<td></td>
<td>Max +</td>
<td>379.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>15.67</td>
<td>33.00</td>
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<tr>
<td></td>
<td>Mmax/Seff</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16.85</td>
<td>45.00</td>
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<tr>
<td></td>
<td>Mconst/Sx+Mpost/Seff</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15.67</td>
<td>33.00</td>
</tr>
<tr>
<td>Controlling</td>
<td>379.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>15.67</td>
<td>33.00</td>
<td>---</td>
</tr>
<tr>
<td>fc (ksi)</td>
<td>0.44</td>
<td>Fc</td>
<td>1.35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( f_c (\text{ksi}) = 0.44 \)  \( F_c = 1.35 \)
Gravity Beam Design

B3 - HYBRID TYPE I
LC: D + 5DL + L + W

REATIONS (kips):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>20.25</td>
<td>20.25</td>
</tr>
<tr>
<td>DL reaction</td>
<td>28.05</td>
<td>30.05</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>11.25</td>
<td>11.25</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>39.30</td>
<td>41.30</td>
</tr>
</tbody>
</table>

DEFLECTIONS:

<table>
<thead>
<tr>
<th></th>
<th>at 15.00 ft</th>
<th>L/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial load (in)</td>
<td>= -0.227</td>
<td>1584</td>
</tr>
<tr>
<td>Live load (in)</td>
<td>= -0.102</td>
<td>3528</td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>= -0.207</td>
<td>1737</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>= -0.435</td>
<td>828</td>
</tr>
</tbody>
</table>
BEAM R1 DESIGN

- TYPICAL INTERIOR BAY
- LOAD COMBO ANALYZED = D + SDL + SL

SPAN = 40'
TRIB = 7.5'
F_y = 50 ksi
L_b = 0' - 0''

* D = 44 PSF
SDL = 18 PSF
SL = 40 PSF

* SUBTRACT 7 PSF ESTIMATE OF STRUT - LET RAM CALCULATE.

USE: RAM STRUCTURAL SYSTEM
D = D + SDL = 62 PSF
CONSTR. D = 44 PSF
SL = 40 PSF
CONSTR. L = 20 PSF

DEFLECTION: LIMIT PRE-COMPOSITE CROSS BAY A TO SMALLER OF 2/360 OR 1'' PER RBA TECH NOTE = 4/360 = 50' x 12 = 1.67'' :: USE 1.0''

DEFAULTS: L/l Δ
Initial 0.00 0.75''
Post LL 360 0.00
Post Super. 360 0.00
Net Total 240 0.00

OPTIMIZED BEAM = [U24 x 55 [20]] , Δ = .9'' (TOTAL)
Δ = .56'' (INITIAL)

INITIAL Δ ALLOWABLE FOR GIRDER R3 = 1.0 - .56'' = .44''
Gravity Beam Design

BM R1 - HYBRID TYPE I
LC: D + SDL + SL

Floor Type: Roof
Beam Number = X

SPAN INFORMATION (ft):

- I-End (15.00,0.00)
- J-End (15.00,40.00)

Beam Size (Optimum) = W24X55
Total Beam Length (ft) = 40.00

Fy = 50.0 ksi

COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th>Concrete thickness (in)</th>
<th>2.50</th>
<th>2.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>f'c (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
</tbody>
</table>

Decking Orientation: perpendicular
Decking type: USD 2" Lok-Floor
bEff (in) = 90.00
Seff (in3) = 146.40
Ieff (in4) = 2607.27
Stud length (in) = 3.50

Stud Capacity (kips) q = 8.6
# of studs: Full = 56   Partial = 20   Actual = 20
Number of Stud Rows = 1
Percent of Full Composite Action = 30.06

LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>Red%</th>
<th>Type</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.465</td>
<td>0.330</td>
<td>0.300</td>
<td>---</td>
<td>NonR</td>
<td>0.150</td>
</tr>
<tr>
<td>40.000</td>
<td>0.465</td>
<td>0.330</td>
<td>0.300</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.055</td>
<td>0.055</td>
<td>0.000</td>
<td>---</td>
<td>NonR</td>
<td>0.000</td>
</tr>
<tr>
<td>40.000</td>
<td>0.055</td>
<td>0.055</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

SHEAR: Max V (DL+LL) = 16.41 kips  \( f_v = 1.84 \) ksi  \( F_v = 18.78 \) ksi

MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kip-ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td>fb</td>
<td>Fb</td>
</tr>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>107.1</td>
<td>20.0</td>
<td>0.0</td>
<td>1.00</td>
<td>11.17</td>
<td>33.00</td>
</tr>
<tr>
<td>Max +</td>
<td>164.1</td>
<td>20.0</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Mmax/Seff</td>
<td>13.45</td>
<td>33.00</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>Mconst/Sx+Mpost/Seff</td>
<td>15.18</td>
<td>45.00</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>Controlling</td>
<td>164.1</td>
<td>20.0</td>
<td>---</td>
<td>---</td>
<td>13.45</td>
<td>33.00</td>
<td>---</td>
</tr>
</tbody>
</table>

fc (ksi) = 0.23  Fc = 1.35

REATIONS (kips):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>10.71</td>
<td>10.71</td>
</tr>
<tr>
<td>DL reaction</td>
<td>10.41</td>
<td>10.41</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>6.00</td>
<td>6.00</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>16.41</td>
<td>16.41</td>
</tr>
</tbody>
</table>

DEFLECTIONS:

<p>| Initial load (in) | at | 20.00 ft | -0.563 | L/D = 853 |
| Live load (in) | at | 20.00 ft | -0.229 | L/D = 2100 |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Post Comp load (in)</td>
<td>at 20.00 ft</td>
<td>= -0.331</td>
<td>L/D = 1448</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>at 20.00 ft</td>
<td>= -0.894</td>
<td>L/D = 537</td>
</tr>
</tbody>
</table>
BEAM R2 DESIGN

- TYPICAL EDGE BEAMS ALONG COL LINES 1'4" ROOF
- LOAD COMBO ANALYZED = D + SDL + SL

SAME DESCRIPTION AS R1 EXCEPT TRIB WILL BE ABOUT 1/2 (7.5"

USE: RAM STRUCTURAL SYSTEM
SAME INPUTS AS R1

DEFORMATION:
SAME AS DEFAULTS FOR R1

OPTIMIZED BEAM = W21 x 44 [20]  
SEE RAM OUTPUT
Gravity Beam Design

BM R2 - HYBRID TYPE I
LC: O + SOL + SL

12/04/07 18:42:02
Steel Code: ASD 9th Ed.

Floor Type: Roof
Beam Number = X

SPAN INFORMATION (ft):
- I-End (0.00,0.00)
- J-End (0.00,40.00)
  Beam Size (Optimum) = W21X44
  Total Beam Length (ft) = 40.00
  Fy = 50.0 ksi

COMPOSITE PROPERTIES (Not Shored):

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>2.50</td>
<td>2.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>f'(ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>perpendicular</td>
<td>perpendicular</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>51.00</td>
<td>Y bar (in)</td>
</tr>
<tr>
<td>Seff (in3)</td>
<td>109.87</td>
<td>Str (in3)</td>
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<tr>
<td>leff (in4)</td>
<td>1774.99</td>
<td>Itr (in4)</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>3.50</td>
<td>Stud diam (in)</td>
</tr>
</tbody>
</table>

Stud Capacity (kips) q = 8.6
# of studs: Full = 38 Partial = 20 Actual = 20
Number of Stud Rows = 1 Percent of Full Composite Action = 53.05

LINE LOADS (k/ft):

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>Red%</th>
<th>Type</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.031</td>
<td>0.022</td>
<td>0.020</td>
<td>---</td>
<td>NonR</td>
<td>0.010</td>
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<tr>
<td>2</td>
<td>0.00</td>
<td>0.232</td>
<td>0.165</td>
<td>0.150</td>
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<td>NonR</td>
<td>0.075</td>
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<td>0.044</td>
<td>0.044</td>
<td>0.000</td>
<td>---</td>
<td>NonR</td>
<td>0.000</td>
</tr>
</tbody>
</table>

SHEAR: Max V (DL+LL) = 9.55 kips  \( f_v = 1.38 \text{ ksi} \)  \( F_v = 18.99 \text{ ksi} \)

MOMENTS:

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>ft</td>
<td>ft</td>
<td>ft</td>
<td>fb</td>
<td>Fb</td>
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<tr>
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<td>kip-ft</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Center</td>
<td></td>
<td>PreCmp+</td>
<td>63.2</td>
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<td>Max</td>
<td>95.5</td>
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<td>---</td>
</tr>
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<td>10.44</td>
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<td>12.19</td>
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<tr>
<td>Controlling</td>
<td></td>
<td>95.5</td>
<td>20.0</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>10.44</td>
</tr>
</tbody>
</table>

fc (ksi) = 0.24  \( F_c = 1.35 \)

REACTIONS (kips):

<table>
<thead>
<tr>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>6.32</td>
</tr>
<tr>
<td>DL reaction</td>
<td>6.15</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>3.40</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>9.55</td>
</tr>
<tr>
<td>Deflection</td>
<td>Load Condition</td>
</tr>
<tr>
<td>----------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>Initial Load (in)</td>
<td></td>
</tr>
<tr>
<td>Live Load (in)</td>
<td></td>
</tr>
<tr>
<td>Post Comp Load (in)</td>
<td></td>
</tr>
<tr>
<td>Net Total Load (in)</td>
<td></td>
</tr>
</tbody>
</table>
BEAM A3 DESIGN

- TYPICAL ALONG COL LINES A & B, ROOF
- LOAD COMBO ANALYZED = D + SD + LL

SPAN = 30'
F_y = 50 ksi
L_b = 7.5'

REA METHOD

SAME LOADS AS R1
USE RAM STRUCTURAL SYSTEM

DEFLECTION:
INITIAL ALLOWABLE Δ = .94" (SEE R1 FOR EXPLANATION)

OPTIMIZED BEAM = W24x76 [15]
**Gravity Beam Design**

**BM R3 - HYBRID TYPE I**

**LC: D + SDF + SC**

**Floor Type: Roof**

**Beam Number = 8**

**SPAN INFORMATION (ft):**

- I-End (0.00,40.00)
- J-End (30.00,40.00)

- Beam Size (User Selected) = W24X76
- Total Beam Length (ft) = 30.00
- Fy = 50.0 ksi

**COMPOSITE PROPERTIES (Not Shored):**

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thickness (in)</td>
<td>2.50</td>
<td>2.50</td>
</tr>
<tr>
<td>Unit weight concrete (pcf)</td>
<td>150.00</td>
<td>150.00</td>
</tr>
<tr>
<td>fc (ksi)</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Decking Orientation</td>
<td>parallel</td>
<td>parallel</td>
</tr>
<tr>
<td>Decking type</td>
<td>USD 2&quot; Lok-Floor</td>
<td>USD 2&quot; Lok-Floor</td>
</tr>
<tr>
<td>beff (in)</td>
<td>51.00</td>
<td>18.90</td>
</tr>
<tr>
<td>Seff (in3)</td>
<td>209.09</td>
<td>231.63</td>
</tr>
<tr>
<td>Ieff (in4)</td>
<td>3466.34</td>
<td>4397.39</td>
</tr>
<tr>
<td>Stud length (in)</td>
<td>3.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Stud Capacity (kips)</td>
<td>q = 11.5</td>
<td></td>
</tr>
<tr>
<td># of studs:</td>
<td>Full = 40</td>
<td>Partial = 15</td>
</tr>
<tr>
<td>Number of Stud Rows = 1</td>
<td>Percent of Full Composite Action = 35.37</td>
<td></td>
</tr>
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**POINT LOADS (kips):**

<table>
<thead>
<tr>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>RedLL</th>
<th>Red%</th>
<th>NonRLL</th>
<th>StorLL</th>
<th>Red%</th>
<th>RoofLL</th>
<th>Red%</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.500</td>
<td>10.41</td>
<td>7.71</td>
<td>0.00</td>
<td>0.0</td>
<td>6.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.00</td>
<td>0.00</td>
<td>3.00</td>
</tr>
<tr>
<td>15.00</td>
<td>10.41</td>
<td>7.71</td>
<td>0.00</td>
<td>0.0</td>
<td>6.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.00</td>
<td>0.00</td>
<td>3.00</td>
</tr>
<tr>
<td>22.500</td>
<td>10.41</td>
<td>7.71</td>
<td>0.00</td>
<td>0.0</td>
<td>6.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.00</td>
<td>0.00</td>
<td>3.00</td>
</tr>
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**LINE LOADS (k/ft):**

<table>
<thead>
<tr>
<th>Load</th>
<th>Dist (ft)</th>
<th>DL</th>
<th>CDL</th>
<th>LL</th>
<th>Red%</th>
<th>Type</th>
<th>CLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>0.031</td>
<td>0.022</td>
<td>0.020</td>
<td>---</td>
<td>NonR</td>
<td>0.010</td>
</tr>
<tr>
<td>30.000</td>
<td>0.031</td>
<td>0.022</td>
<td>0.020</td>
<td></td>
<td>0.010</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>0.076</td>
<td>0.076</td>
<td>0.000</td>
<td>---</td>
<td>NonR</td>
<td>0.000</td>
</tr>
<tr>
<td>30.000</td>
<td>0.076</td>
<td>0.076</td>
<td>0.000</td>
<td></td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SHEAR:** Max V (DL+LL) = 26.52 kips  \( f_v = 2.52 \text{ ksi} \)  \( f_v = 20.00 \text{ ksi} \)

**MOMENTS:**

<table>
<thead>
<tr>
<th>Span</th>
<th>Cond</th>
<th>Moment</th>
<th>@</th>
<th>Lb</th>
<th>Cb</th>
<th>Tension Flange</th>
<th>Compr Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kip-ft</td>
<td>ft</td>
<td>ft</td>
<td></td>
<td>fb</td>
<td>Fb</td>
</tr>
<tr>
<td>Center</td>
<td>PreCmp+</td>
<td>172.8</td>
<td>15.0</td>
<td>7.5</td>
<td>1.13</td>
<td>11.78</td>
<td>33.00</td>
</tr>
<tr>
<td>Max+</td>
<td></td>
<td>260.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>14.95</td>
<td>33.00</td>
</tr>
<tr>
<td>Mmax/Seff</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16.31</td>
<td>45.00</td>
</tr>
<tr>
<td>Mconst/Sx+Mpost/Seff</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>14.95</td>
<td>33.00</td>
</tr>
<tr>
<td>Controlling</td>
<td></td>
<td>260.4</td>
<td>15.0</td>
<td>---</td>
<td>---</td>
<td>14.95</td>
<td>33.00</td>
</tr>
<tr>
<td>fc (ksi) = 0.39</td>
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<td></td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Fc = 1.35</td>
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<td></td>
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</table>

**REACTIONS (kips):**

<table>
<thead>
<tr>
<th></th>
<th>Left</th>
<th>Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial reaction</td>
<td>17.69</td>
<td>17.69</td>
</tr>
<tr>
<td>DL reaction</td>
<td>17.22</td>
<td>17.22</td>
</tr>
<tr>
<td></td>
<td>Left</td>
<td>Right</td>
</tr>
<tr>
<td>--------------------------</td>
<td>------</td>
<td>-------</td>
</tr>
<tr>
<td>Max +LL reaction</td>
<td>9.30</td>
<td>9.30</td>
</tr>
<tr>
<td>Max +total reaction</td>
<td>26.52</td>
<td>26.52</td>
</tr>
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</table>

**DEFLECTIONS:**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>At</th>
<th>Value</th>
<th>L/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial load (in)</td>
<td>15.00 ft</td>
<td>-0.322</td>
<td>1119</td>
</tr>
<tr>
<td>Live load (in)</td>
<td>15.00 ft</td>
<td>-0.141</td>
<td>2546</td>
</tr>
<tr>
<td>Post Comp load (in)</td>
<td>15.00 ft</td>
<td>-0.205</td>
<td>1756</td>
</tr>
<tr>
<td>Net Total load (in)</td>
<td>15.00 ft</td>
<td>-0.527</td>
<td>684</td>
</tr>
</tbody>
</table>
From p. L9 - North Wall

2nd Floor Wall

Roof

19k

19k

6.33'

4'

3.33'

28.92'


\[ f = \frac{p}{a} + \frac{m}{3} \]

\[ p = \text{Wall} = 44 \text{ psf} \times 28.92'(5.33') = 6.8k \]

\[ a = 28.92' \times (40.7 \text{ in}^2/ft) = 1171 \text{ in}^2 \]

\[ A = 101.4k \]

\[ b = \text{Net Width} = \frac{40.7 \text{ in}^2/ft}{12} = 3.39 \text{ in} \]

\[ S = \frac{6b^2}{6} = \frac{3.39 \times (28.92 \times 12)^2}{6} = 68047 \text{ in}^3 \]

\[ (D+W) \]

\[ f = \frac{6.8k}{1171} + \frac{101.4k \times 12}{68047} = 5.78 \pm 1.78 = 7.6 \text{ psi (max)} \]

[4.0 psi (min)]

\[ (60+W) \]

\[ f = 3.5 \pm 1.8 = \text{ Still no tension} \]

\[ \sigma = \frac{1900}{1171 \text{ in}^2} = 1.6 \text{ psi} \]

No shear reinforcement needed.

No steel req'd.
assume equal

\[ P_A(12.23) + P_B(4') - P_C(4') - P_D(12.23) = M = 1.9k \cdot (5.33 + 4) = 17.1k \]

\[ \Sigma P = 6.8k + 4.46' \cdot 4' \cdot 2(4' \text{psf}) \cdot 4' \cdot 14k = 98k \]

\[ 12.23' (P_A - P_D) + 4'(P_B - P_C) = M \]

\[ P = \frac{\Sigma P}{n} = \frac{9.8k + 17.1k}{3} = 2.45 \pm \frac{17.1(12.23)}{331} \]

\[ 2 \Delta d^2 = 2(12.23)^2 + 2(4)^2 = 331 \]

\[ 2.45 \pm \frac{17.7(4)}{331} \]

\[ \frac{2.66P_B}{2.24P_C} \]

check \( M = (3.1 - 1.8)12.23 + (3.16 - 2.24) = 17.6k \)

\[ V = 1.9k \cdot 1.5k \]

\[ M = \sqrt{h \cdot \frac{5k(4')}{2}} = 11k \]
\[ V = 500 \text{ kN} = 4.9M \text{ (minimal)} \]

\[ f_y = 3.1k \text{ N/m}^2 + \frac{1}{3} \times 12 = 17.0 \pm 7.4 \text{ kN/m} \]

\[ f_y = \frac{64.18}{g} = 5.9 - 7.4 \text{ kN/m} \]

\[ A_s = (M) \left( \frac{f_y}{f_y} \right) = 0.02 \text{ in.} \]

\[ \text{USE } #4 \text{ as minimum} \]

\[ \text{NO NEED FOR EXACT ANALYSIS} \]

\[ M = 9(12.67) = 24.1\text{ kN-m} \]

\[ P = \left[ 28.92(12.67) - 4 \times 1.4\text{l} \right] / 30.24 \text{ sf} = 13.3 \text{ kN} \]

\[ f_y = \frac{P}{A} \pm \frac{M}{S} = \frac{13.3}{177} = 0.77 \text{ kN/m} \]

\[ 24.1 \times 12 = 11.3 \pm 4.3 \rightarrow 15.6 \text{ psi (avg)} \]

\[ g = 1900 \pm 1.6 \text{ psi minimal} \]

\[ o.02 \text{ in.} \]

\[ \text{ONLY PRESCRIPTIVE REINFORCMENT NEEDED - 2ND FLOOR} \]

\[ #4 \text{ AS JAMPS} \]
FIRST FLOOR

\[ f_b = \frac{P}{\frac{M}{68047}} = \frac{6.8K}{1171} = 5.8 \pm 6.4 \text{ psi} \]

Minimum: \( f_v = \frac{5500}{2812(40.7)} = 47 \text{ psi} \)

Increased tension: \( f_b \rightarrow \)

From (2) \# 4 bars => \( M = 3fsijd \)

\[ 0.20(24)(3)(4) = 122.8 \text{ in}^4 \rightarrow \text{Reg'd OK} \]

So far over capacity, no need for rebar check.
7  \( P = \frac{3P}{n} + M_{y_n} = \frac{9.8}{4} + \frac{55(9.35)}{211} = 2.45 \pm 0.99 \)  
\( P_a = 4.4 \)  
\( P_o = 0.5 \)  
\( 4.0 \rightarrow 2.45 \pm 0.99 \rightarrow P_B = 3.1 \)  
\( P_c = 1.6 \)

Check:  
\( 2P = 9.8 \sqrt{V} \)  
\( S = \frac{4.4}{4.16} \)  
\( M = \sqrt{\frac{4}{3}} = 1.4(4) = 2.8 \)  
\( V = 0.4(4) = 1.4 \)  
\( FB = 4.4 \pm 2.8 = 24.2 \pm 20.8 \) psi

\( F_b = 2.5 \sqrt{m} = 500 \) psi  
OK

\( f_v = \frac{1400}{3.39} = 8.6 \) psi  
Minimal
\[ V = 55(12.67) = 697.7 \text{ kN} \]

\[ f_b = \frac{13.3}{1177} \times 68097 = 11.3 \pm 12.3 \]

minimise on comp. & tension

use #4 from (3)

- In-plane shear
- #4 & 6 each jamb & edge (lap to foundation at 1st floor)
- #4 & 6 each jamb & edge (dowel to slab at 2nd floor)
From L9 - North Wall (First Floor)

1. Similar to 1 for 1st Floor P. L10

2. $V_{2b} = \frac{K_{2b}V}{2} = \frac{182(1.34)}{114} = 15.5$

$V_{2b} = \frac{V - V_{2c}}{2} = 22k$

$V_{2a} = 4.4k$

Distribute $P$ (use loads @ bottom)

$P = \frac{3P}{6} = \frac{13.8k}{3} = 4.6 + 2.8 = 7.4$

$P_{y} = 1.64$

$P_{y} = 1.64$

$P_{y} = 1.64$ edge (2b) 4.6 - 3.3 = 1.3

$P_{y} = 13.7$
**PROJECT NUMBER:** 9128  
**DATE:** 12/4/07  
**PAGE:** L18  
**PROJECT:** HYBRID-TYPE I  
**BY:** DB  
**SUBJECT:** SHEAR WALLS

---

**Diagram:**

- Shear wall with dimensions and forces indicated.
- Equations for shear force and moment calculations.

**Equation:**

- \( M = 1.34 \times 7.33 = 9.7 \times 10^4 \)
- \( f_b = 18.2 \pm 20.1 \)  
  - Compression only

- \( f_b = 4.4 \times 12 = 54.0 \)  
  - Tension

**Axial Stress:**

- \( A_s = \frac{M}{f_b} = \frac{4.4 \times 12}{180} = 0.40\)  

**Shear Force:**

- \( F_V = 4.4 \times 10^2 \pm 5.4 \text{ psi} \)  
  - Minimal
NORTH WALL

#4 TYPE EA. JAMPS
SOUTH WALL

BETWEEN N. WALL
RESULTS MIN. REINF
REQ'D

#4
WEST WALL (EAST WALL SIMILAR)

SECOND FLOOR

OVERTURNING

\[ M_{ov} = 6.3 \times 12.92^2 = 814 \text{ kip-ft} \]
\[ P = 4 \times \frac{1}{2} \left( 171 \times 12.92 \right) = 4(167) = 894 \text{ kip} \]
\[ \frac{M_{ov}}{P \times \frac{L}{2}} = \frac{814}{894 \times \frac{12}{2}} = 1.07 \text{ tension, expected} \]
PROJECT: HYBRID-TYPE E  BY: DIB
SUBJECT: SHEAR WALLS

\[ P = 4.1 \left(5.59(111)\right) = 1.2k \]

\[ M = 5.59(63) = 35.2'k \]

\[ \sigma_t = \frac{2.5 \pm 17.8}{17.1 \times 40.7} = 9.1 \text{ psi} \text{ minimal} \]

\[ f_u = 6.3 \pm 0.1 \text{ psi} \]

\[ A_s = \frac{35.2 \times 12}{24(9)(197)} = 0.10 \text{ in}^2 \]

\[ f_{s, y} = 4.0 \text{ k} \]

\[ P_x = 2P + M = 4.1 \pm 7.5 \text{ k} \text{ uplift} \]

\[ P_{2a} = 11.6 \text{ k} \]

\[ P_{2b} = 8.4 \text{ k} \text{ (tension)} \]

For (6D+n), \( \Rightarrow \text{TENSION INCREASES} \)

\[ P_{2b} = 6(41) - 7.5 = -5.0 \text{ k} \]

\[ f_b = 3.35(625) \frac{44}{9} = 10.7 \text{ k} \]

\[ f_b = 42 \pm 23.8 \text{ comp. only} \]

\[ f_b = 25.2 \pm 23.8 \text{ no known} \]
TRY \( A_T = \frac{V_h}{24} \)

\[ A_{\text{flexure}} = \frac{M}{f_s \cdot d} = \frac{6.3 \times 12}{24.9(64)} \]

\[ = 0.050\% \]

\[ V_h = \frac{3.15 \cdot 0.4}{2} = 6.3 \text{ in} \]

\[ \frac{C_m}{K_d} \]

\[ \Rightarrow f_s = f_m \left( \frac{d-K_d}{K_d} \right) \]

\[ C_m = \frac{1}{2} f_m \text{ bef } K_d \]

\[ \Rightarrow \frac{C_m}{K_d} \]

\[ \Rightarrow \frac{f_m}{K_d} \]

\[ \Rightarrow f_s = \frac{f_m}{(d-K_d) \cdot K_d} \]

\[ \text{if } K_d = 12'' \]

\[ 20.34 f_m + 5000 = 20.33 f_m \Rightarrow f_m = 585 \text{ psi} \]

\[ f_s = 545 \text{ psi} \]

\[ \text{if } K_d = 8'' \]

\[ 13.56 f_m + 5000 = 16.66 f_m \Rightarrow f_m = 151 \text{ psi} \]

\[ f_s = 273 \text{ psi} \]

check \( M \):

\[ M = C_m \times \left[ \frac{22 - K_d}{3} \right] + T \left( \frac{6.4 - 36}{28} \right) \]

\[ = 200 \text{ in.}^2 \text{ in.} / 16.7 > 6.3 \text{ in.}^2 / \text{ft} \]

\[ = \text{OK} \]
\[ f_v = 3.15^{1/6} \times 0.72(40) = 12.9 \text{ psi} \ (\text{min.}) \]

\[ \text{check } f_v = 6.3 = 9.0 \text{ psi} \ (\text{min.}) \]

\[ f_v = 42.238 \text{ psi}; \quad \therefore 2a = 65.3 \text{ psi} \leq \frac{f_v}{2} = 500 \text{ psi}; \quad \text{OK} \]

**First Floor**  
**Expect Tension**

\[ M_0 = 31.5(3650) = 11761 \text{ in} \cdot \text{lb} \]

\[ T_0 = 31.5 \approx 45.3 \text{ psi} \]

\[ V_0 = 31.5 \approx 45.3 \text{ psi} \]

\[ M = \frac{1761 \times 12}{31.5(197)} \approx 34 \Rightarrow F_v = \frac{1}{3} \left[ 4 - M \frac{f_m}{V_d} \right] = 47.3 \text{ psi} \ (\text{D+V}) \]

\[ M = \frac{1761 \times 12}{31.5(197)} \approx 34 \Rightarrow F_v = \frac{1}{3} \left[ 4 - M \frac{f_m}{V_d} \right] = 47.3 \text{ psi} \ (\text{D+V}) \]

\[ F_v > f_v \Rightarrow \text{no shear reinforcement req'd} \]

**Flexure:** Try \( 2 - \#6 \) \( A_s = 0.88 \text{ in}^2 > 0.50 \)

\[ n_p = \frac{F_m}{f_{ck}} = \frac{21 \times 10^6}{900 \times 1500} = 2.15 \]

\[ n_p = 21.5 \left( \frac{2.28}{3.39(190)} \right) = 0.029 \]

\[ K = \frac{n_p^2 + n_p - n_p^2}{2} = 0.136 \]

\[ j = 1 - \frac{K}{3} = 0.929 \]
\[ R_d = 214 \times (190) = 40,711 \text{ kN} < p_{2a} = 6.25' \text{ kN} \]

\[ f_m = \frac{2M}{K_j b_d^2} = \frac{2 \times 176.1 \times 12800}{0.29 \times (214 \times 3.39 \times 190)^2} = 174 \text{ psi} \ll f_m = 500 \text{ psi} \text{ OK} \]

\[ \therefore \text{ use } 2\# 6 \text{ @ ea end} \]

\[ \text{Use } M \text{ @ bottom of wall } = 31.5 \times (192) = 4070 \text{ kN-m} \]

\[ P_x = \frac{3P + M}{2} = 4.1 \pm 4.07 \]

\[ P_{2a} = 4.1 \pm 37.5 \text{ kN} \]

\[ P_{2b} = -33.4 \text{ kN} \text{ TENSION (P_T)} \]

\[ (D+W) \quad P_{2a} = 4.1 \pm 37.5 \text{ kN} \]

\[ (D+W) \quad P_{4a} = 4.1(6) - 37.5 = -35.0 \text{ kN} \]

\[ A_5 = P_t \div f_s = 95.0 \div 2415 = 0.046 \text{ in}^2 \]

\[ M = Vh \div 2 = 31.5 \times 4 = 126 \text{ kNm} \]

\[ A_s = \frac{31.5 \times 12}{24(3.9 \times 68''}) = 0.268'' \]

\[ EAS = 1.71 \text{ kN/} \text{in} \]

\[ C + P_T = T_5 \]

\[ A_{b/2} \ll \text{ create } T_5 \]
PROJECT NUMBER: 9128  DATE: 12/20/07  PAGE: 59
PROJECT: HYBRID - TYPE I  BY: D7B
SUBJECT: SHEAR WALLS

1. \[ C_m + P_T = T_s \]

2. \[ V_2 f_m \times K_d \times b_{eff} + 35,000 = 0.44 \times (f_{s1} + f_{s2}) \]

3. \[ E_m = \frac{f_m}{K_d} = \frac{f_{s1}}{E_{s1}} = \frac{f_{s2}}{E_{s2}} = \frac{60Kd}{68-Kd} \]

4. \[ f_{s1} = f_{s2} \left( \frac{60Kd}{68-Kd} \right) \]

5. \[ f_{s2} = \frac{E_s f_m}{K_d} (60-Kd) = n f_m (68-Kd) \]

6. \[ \frac{1}{2} f_m \times K_d \times b_{eff} + 35,000 = 0.44 \left( f_{s2} \left( \frac{60-Kd}{68-Kd} \right) + 1 \right) \]

7. \[ M_{eff} = C_m \times (37.5 - \frac{Kd}{2}) + T_s (60.37) + T_s (68.37) \]
TRY $K_d = 12'' \Rightarrow \begin{align*}
&\frac{1}{2} f_{cm}(12)(3.39) + 35000 = 0.44 f_s \left( \frac{48}{56} + 1 \right) \\
&\frac{n f_m}{f_m} \left( \frac{56}{12} \right) \\
&20.34 f_m + 35000 = 81.9 f_m \Rightarrow f_m = 568 \text{ psi} > F_m_{ng} \\
&61.65 f_m = 35000 \\
&f_s = 5696 \text{ psi} > F_s_{ng}
\end{align*}$

TRY ADDED REINFORCEMENT

\[ f_{s2} = n f_m \left( \frac{68 - K_d}{K_d} \right) \]
\[ f_{s1} = f_{s2} \left( \frac{60 - K_d}{68 - K_d} \right) \]
\[ f_{s3} = f_{s2} \left( \frac{52 - K_d}{68 - K_d} \right) \]
\[ f_{s4} = f_{s2} \left( \frac{44 - K_d}{68 - K_d} \right) \]

\[ \frac{1}{2} f_m (K_d)(3.39) + 35000 = 0.44 f_{s2} \left[ 1 + \frac{60 - K_d}{68 - K_d} + \frac{52 - K_d}{68 - K_d} + \frac{44 - K_d}{68 - K_d} \right] \]
if $K_d = 3''$

1. $5.09 \text{ ft} + 35000 = 0.44 (\text{ft})(68-K_d) \left[ 1 + \frac{57 + 49 + 41 + 33}{65} \right] \left[ \frac{65}{65} \right] = 3.262$

3109 ft + 35000 = 6681.5 ft

66341 ft + 35000 = $f_m = 52.5$ psi < $F_m$ OK
$f_m = 24376$ > $F_s$ NG

ADD ANOTHER BAR ($^4'6'$) WITH $K_d = 3''$

$f_s = \frac{36-K_d}{68-K_d}$

5.09 ft + 35000 = 0.44 \left[ 21.5 \text{ ft} \right] \left[ \frac{65}{3} \right] \left[ 1 + \frac{57 + 49 + 41 + 33}{65} \right] = 272.57 \text{ ft}

$f_m = 35000 / 767.48 = 45.6$ psi < $F_m$

$f_s = 2124.4$ psi < $F_s$

check $\frac{M}{A} = \sum \frac{T_s (\sin \theta)}{6} + T_s (\cos \theta) + T_s (\theta) + T_s (\phi) + T_s (-\lambda) = 61763.8$ 11'-

$8356 \quad 9348 \quad 8197 \quad 6179 \quad 5896 \quad 2794 \quad 2794 \quad 2934 \quad 315'$

$M = 315$ 11'

$\therefore 5'-6"$ AT $\frac{1}{2}$ PIER
\( V = 31.5 \sqrt{h} \)

\[ M = Vh = 31.5 \sqrt{h} \]

\[ f_{06} = \frac{416}{6.25(40.7)} \pm \frac{31.5 \times 12}{339(6.25 \times 12)^2} = 164 \pm 119 \]

\[ p = 416 \text{ kN} \]

\[ f_{06} = \frac{416}{6.25(40.7)} \pm \frac{31.5 \times 12}{339(6.25 \times 12)^2} = 164 \pm 119 \]

Totally in compression: \( f_{06} = 283 \text{ psi} \)

\[ f_m = \frac{1}{3} f_{06} = 560 \text{ psi} \]

No reinf. needed.

Check: \( f_v = \frac{31.5 \sqrt{h}}{6.25(40.7)} \approx 62 \text{ psi} \)

\[ M = \frac{31.5 \sqrt{h}}{339(6.0)} \leq 0.4 < 1.0 \]

\[ f_v = \frac{1}{3} \left[ \frac{4 - M}{Vd} \right] f_{06} = 46.5 \text{ psi} \]

Requires shear reinforcement.

**With Reinforcement:**

\[ f_v = \frac{1}{2} \left[ \frac{4 - M}{Vd} \right] f_{06} = 70.9 \text{ psi} < f_v \]

\[ f_{v(max)} = 120 - 45 \left( \frac{M}{Vd} \right) = 120 - 45(4) = 102 \text{ psi} \]

\[ A_v = \frac{V_s}{f_{sd}} = \frac{(31.5 \sqrt{h})(16)^{3/2}}{(24)(60)} = 0.183 \text{ in}^2 \]

\( \#3 \text{ rebar @ 16 in} \)

Spacing: 2.3, 5, 3, 2.
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Project: HYBRID-TYPE I  By: 

Subject: SHEET WALLS

- Movement Joint
- 1 5/8 ea. JAMB 4 EDGE
- Roof
- 2nd
- 1st

In-Plane

West Wall (East Wall: Sim.)

- 10 #6
- #3 @ 16" dowel to tension
- 10 #6
- #3 @ 16"
LATERAL LOAD - 20 psf used

NORTH WALLS - BOTH FLOORS

\[
W = \frac{8' \times 20}{4} = 40 \text{ psf on the pier}
\]

\[
M = \frac{W(12.6')^2}{2} = \frac{6.4' \times 12' - 4}{2} = 1.6' \times \text{in/ft}
\]

\[
A_s = \frac{M x 12}{E s j d} = 0.24 \text{ in/ft} \times 4' = 9.6 \text{ in.} \text{ try } 3 - 8\# 6 (A_s = 1.32 \text{ in.}) (0.33/\text{ft})
\]

\[
h_p = \frac{21.5 (0.33)}{12 (3.8)} = 0.156 \quad K = 0.424 \quad j = 0.859 \quad K_d > 0.4 \text{ grout solid}
\]

\[
M_5 = A_s F_j j d = 0.33 (24' \times 0.859' \times 3.8') \approx 2.2 ' \times \text{in/ft} \text{ or}
\]

\[
M_m = F_m k_j b d^2 = \frac{500}{2} (0.424 \times 0.859' \times 3.8')^2 \approx 13 \text{ in} < 1.6 b_5
\]

Reqd F_m = \frac{1}{2} (500) = 615

Reqd F_m = 1846

Type S mortar

Requires CMU strength 2800 psi
**NORTH WALL**

\[ M = 20 \left( \frac{12.67}{B} \right)^2 = 4.81 \, \text{ft} \]

\[ A_S = 0.06 \, \text{in}^2 / \text{ft} \]

\[ t_y = 4 \times 40' = 0.6 \]

\[ J = 9.90 \]

\[ K_d = 0.80'' < t \text{force shell ok} \]

\[ M_n = \frac{F_n}{2} k' b d^2 = 0.71 \, \text{in} \text{ft} \text{ ok} \]

\[ W = 9.0 \, \text{psf} \text{ ft} \]

\[ W_e = \left[ 1.5 + \frac{9.0}{6} \right] = 33.2 \, \text{psf ft} \]

\[ M_{\text{max}} = 39.2 \, \text{psf ft} \left( \frac{12.67}{B} \right)^2 = 0.52 \, \text{in} \text{ ft} \]

\[ A_S = 0.07 \, \text{in}^2 / \text{ft} \]

\[ t_y = 4 \times 40' = 0.89'' \text{ ok} \]

\[ \text{Kd} = 0.80'' < t_y \text{ ok} \]

\[ M_S = \frac{A_S}{12} = 0.53 \, \text{in} \text{ ft} \text{ ok} \]

\[ M_m = \frac{F_n}{2} k' b d^2 = 0.78 \, \text{in} \text{ ft} \text{ ok} \]
WEST WALL

\[ W_e = \left( \frac{6.25 + 4.67}{2} \right) \frac{20}{6.25} = 27.5 \text{ psi} \]

\[ M_3 = 27.5 \left( \frac{12.67}{8} \right)^2 = 5.6 \text{ in.-ft} \]

\[ A_s \approx 0.06 \text{ in.}^2 \quad # 6 @ 24\text{"} (A_s = 0.22) \]

\[ h_p = \frac{21.5 (122)}{12 (3.8)} = 1.10 \quad k = 1.38 > t_{face \ shell} \]

\[ M_3 + A_s F_s \frac{1}{12} = 1.5 \text{ in.} \quad \text{OK} \]

\[ M_m = \frac{F_m}{2} k \frac{b d^2}{12} = 1.2 \text{ in.} / \text{ft} \quad \text{OK} \]

\[ K_d = 1.38 > t_{face \ shell} \quad f'_{m} = 1.50 \text{ psi} \]

\[ M_m = \frac{F_m}{2} k \frac{b d^2}{12} = 1.1 \text{ in.} > 0.56 \text{ in.} \]
SUMMARY INCLUDES RESULTS OF SHEAR WALL & OUT-OF-PLANE DESIGNS

NORTH WALL

SOUTH WALL

SEE NEXT PAGE FOR REINFORCEMENT
3 - #6 PER PIER; GROUT PIER SOLID WALL $f_m = 1850$ psi
Type S mortar; $CMU = 2500$ psi by Unit Strength

3 - #4
$fm = 1500$ psi
Type S mortar; $CMU = 1900$ psi by Unit Strength

SOLID WALL = #4 @ 40" OC
$fm = 1500$ psi
2nd

1st

3-#4

4-#4

+ 4c 40"

f'm = 1500 psi
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PROJECT: HYBRID - TYPE I    BY: DB
SUBJECT: SUMMARY

- 4 - #6 EA PIER
- 10 - #6 EA PIER
- 4 - #350 16" EACH PIER
- f' m = 1500 psi