STRUCTURAL CALCULATIONS
FOR
COMMUNITY CENTER AND STAGE
MISSION TWENTYFIVE 35
DOMINICAN REPUBLIC

MISSION
TWENTYFIVE 35

JUNE 17, 2017

PREPARED BY:
JOCelyn LU
MINDY TRIEU

JOURNEYMAN INTERNATIONAL
1330 MONTEREY STREET, SAN LUIS OBISPO, CA 93401
PROJECT DESCRIPTION:
Location - 19°17'19.07"N, 70°25'20.78"W, Dominican Republic

DESIGN CRITERIA:
Occupancy Classification - A (Assembly)  
[IBC Sect 302.1 & 304]
Fire Rating - 3 hours
[IBC Table 706.4]
Type of Construction - Type I-A  
[IBC Table 601]
Risk Category - II
[ASCE Table 1.5-1]
Aggregate Type - Carbonate aggregate concrete (limestone)
Minimum Finished Face-to-Face Wall Thickness - 4.4 inches
Minimum Slab Thickness - 5.7 inches
[IBC Table 721.1(2)]
Allowable Building Height Above Grade Plane - Unlimited
[IBC Table 721.1(3)]
Maximum Floor Area per level - Unlimited
[IBC Table 504.3]
Soil Type - Clayey sand
[IBC Table 506.2]
Allowable Bearing Pressure - 2000 psf
[IBC Table 1804.2]
Building Life Span - 50 years

MATERIAL SPECIFICATIONS (TYP UNO):
Concrete -
Normalweight Concrete 150 pcf
Pad Footings  
f'c = 4000 psi
Slab on Grade  
f'c = 4000 psi
Beams/Girders and Columns  
f'c = 4000 psi
Reinforcing Steel -
Reinforcing steel per ASTM A615 Grade 60
Welded Reinforcing per ASTM A706
Steel -
HSS Square Tubing  
fy = 60 ksi, E = 29,000 ksi
Masonry -
Materials and Workmanship per TMS 402-13/ACI 530-13/ASCE 5-13
TMS 602-13/ACI 530 1-13/ASCE 6-13
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<td>Office (Partitions)</td>
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2nd Floor Key Plan

- D: moment frame

[Diagram showing a floor plan with labeled sections and annotations]
Loads:

Worst case scenario beam was designed (along gridline A) to be conservative when placing this beam in order locations with smaller floor area.

- Load combo: 1.2D + 1.6L
- DL = 510 psf (3 m x 3.2 ft/m)
- 551.04 plf
  = 8.04 KN/m
- LL = 20 psf + 15 psf = 35 psf
  L = AT = (10m) (3m) = 30 m²
  R1 = 1.0
  R2 = 1.0 (beams ≠ no slope)
  LR = 1.0 K1 K2
  = (35 psf) (1.0) (1.0)
  = 35 psf
- LL = (35 psf) (3 m x 3.2 ft/m)
  = 344.4 plf
  = 5.03 KN/m
- Wt = 1.2D + 1.6L
  = 1.2 (8.04 KN/m) + 1.6 (5.03 KN/m)
  = 17.69 KN/m

See attached moments & shears found in RISA.
Member Labels:

Load Combo (1.2D+1.6L):

Shear Diagram:

Moment Diagram:

Member Forces:

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Mission Twenty-Five 35
Community Center-Dominican Republic
Mindy Trieu and Jocelyn Lu

**Calculation Topic:**
Kitchen Ist Floor Beam (B1)

\[ \text{Max. Mu} = 41.86 \text{ KN.m} \]

\[ w = \frac{1.07 \times 12}{60} = 0.28 \text{ ft} \]

\[ \frac{w}{f_y} = \frac{0.28}{50} = 0.0056 \]

\[ A = 5 \times 5 \times 0.5 = 12.5 \text{ in}^2 \]

\[ \Rightarrow A = 12.38 \text{ in}^3 \]

**Check #1**

\[ \frac{b}{t} \leq 0.55 \sqrt{\frac{f_y}{f_c}} \]

\[ 7.75 \leq 0.55 \sqrt{\frac{29000}{50}} \]

\[ 7.75 \leq 13.25 \checkmark \]

**Beam Self-Weight:**

\[ 28.48 \text{ plft} \]

\[ = 0.42 \text{ KN/m} \]

\[ 
\text{will not impact DL on beam much neg}
\]

**\( \Delta \text{allw} = \frac{ML}{EI} \)**

\[ = \frac{24 \times 5 \times 3.26 \times (12.5^2)}{300} \]

\[ = 0.84 \text{ in} \]

**\( \Delta \)**

\[ = \frac{6WL^4}{384EI} \]

\[ = \frac{5 (1.2 (551.04 \text{ plf} + 28.43 \text{ plf}) + 5.8 (344.04 \text{ plf})(12.5^2) (5 \times 3.26 \text{ ft/m} \times 12.5^4)}{384 \times (29,000,000 \text{ psi})(2.4 \text{ in}^4)} \]

\[ = \frac{5 (103.819 \text{ #/in})(19.0 \text{ in})^4}{384 (29,000,000 \text{ psi})(2.4 \text{ in}^4)} \]

\[ = 2.6 \text{ in} > 0.54 \text{ in} \checkmark \]

**Check #2**

\[ \frac{b}{t} \leq 0.55 \sqrt{\frac{f_y}{f_c}} \]

\[ 7.75 \leq 0.55 \sqrt{\frac{29000}{50}} \]

\[ 7.75 \leq 13.25 \checkmark \]

**\( \text{My} = 50 \times 5 \times 5.8 \) \text{ in}^2 \]

\[ = 44.7 \text{ in}^2 \]

**Check #1:**

\[ \frac{b}{t} \leq 0.55 \sqrt{\frac{f_y}{f_c}} \]

\[ 7.75 \leq 0.55 \sqrt{\frac{29000}{50}} \]

\[ 7.75 \leq 13.25 \checkmark \]

**Final Conclusion:**

The beam will impact moment, but \( z \) is significantly larger than previous and should suffice.
Kitchen/Restroom Beam (B1)

\[ \Delta = \frac{5 \times 10^3}{384} \]

\[ \Delta = 0.493'' \quad < \quad \Delta_{allow} = 0.596'' \quad \checkmark \]

Use: HSS 8 x 8 x 5/8 A992
Beam Design (B2)

Load Combination: 1.2D + L/4
PL = 2.5 psf + 0.4 psf (adjustment) = 34.7 psf
LL = 2.0 psf + 15 psf (partitions)

Ar = (4.225) = 15 m², Θ = AT = 1858 m²; R1 = 1

F = 15 ⇒ F ≤ 4; R2 = 1

Lr = 2.0 Rr R2 = (25 psf x 1.25) = 25 psf

w = 34.7 psf x 2.5 m = 3.28 m⁴ ft
⇒ (1.2 x 33.7 psf x 2.5 m² x 2.25) = 802 m³ psf

Mu = wL² / 8 = (802.4 psf x 3.28 m⁴ ft) / 8 = 3890 k-ft

Required 2x = 1.167 M₁ / F₁ = (11.7 x 38.9) / 60 = 19.99 in²

⇒ Try HSS 6 × 6 × 1/2, Zx = 19.8 in²

Check:

b/t ≤ 0.673 √F₁/3M₁ ≥ 7.70 ≤ 0.673 [27000 psi] = 13.25 √O.K.

Fy = 50 psf

Beam weight = 25.24 kip

w = 802.4 psf + 25.24 psf = 827.64 psf

Mu = (827.64 psf x 3.28 m⁴ ft) = 10,410.1 k-ft

Zx = (11.7 x 40.0 x 12.75) = 13.52 m² ≤ 3xHSS 6 × 6 × 1/2, Vx = 19.8 m² √O.K.

Ax = (11.7 x 40.0 x 12.75) / 2.0 = 0.656

A = 5.01

b = 34.0

A = (11.7 x 3.28) / 12.75 = 34.1

A = 5.01

USE HSS 6 × 6 × 1/2, A992 BEAM
Beam Sizing:

One-way continuous min. depth = \( \frac{d}{18.5} \)

\[
\frac{d}{18.5} = 13.12'' \times 0.21 > 12''/1 = 8.5'' = \frac{12''}{18.5} \text{ governing min. depth}
\]

Both end continuous min. depth = \( \frac{d/2}{21} \)

\[
\frac{d}{21} = 13.12'' \times 0.22 > 12''/1 = 7.5''
\]

ACI 318 Beam width: (1/2 beam depth)

\[
\left(\frac{1}{2} \times 12''\right) = \sqrt[3]{12''} \\
\text{Code minimum width} = 12''
\]

Estimated Beam Size = 12'' x 12''
Main Room, 2nd Flr. Beam - (B3)

Reference

DL: 41 psf \((12.5 \text{m} \times 0.25 \text{f/m²})\)
- 1.681 plf
- 1.681 klf

UL: 20 psf (40 LL reduction)
- At (44m) \((12.5 \text{m}^2)\) = 200 m²
- \(R_1 = 0.6\) b/c \(Ar = 55.74 \text{ m}^2\)
- \(R_2 = 1.0\) b/c no slope
- \(Lr = L_2 R_1 R_2\)
  - \((20 \text{ psf})(0.6)(1.0)\)
  - \(12 \text{ psf}\)
- UL = \((12 \text{ psf})(12.5 \text{ m} \times 3.28 \text{ f/m})\)
  - 492 plf
  - 0.492 klf

2D + Hel

\(W_D = (1.2) \times 1.681 \times 1.0 = 2.017 \text{ klf}\)
\(W_L = (1.6)(0.492 \text{ klf}) = 0.7872 \text{ klf}\)
\(W_T = 2.017 \times 0.7872 + 0.07872 \text{ klf} = 2.804 \text{ klf}\)

= 40.92 kN/m ← use in RISA analysis

Flexure:

\(A_{x, min}\):
\(A_{x, min} = \frac{200}{f_y} \cdot \text{bwd}\)
- cover the assume #8 bars
- \(200\)
- \((\frac{60,000 \text{ psi}}{12''}) (12'' - 1.5'' - 0.5'' - (\frac{3}{4})''(\frac{1}{2}))\)
- 0.38 in²

\(A_{x, min} = \frac{3 f_y}{f_y} \cdot \text{bwd}\)
- 3\(\sqrt{4000 \text{ psi}}\)
- \((\frac{60,000 \text{ psi}}{12''} (12'') \)
- 0.456 in² ← governs b/c greater
Member Labels:

Loading \((1.2D + 1.6L)\):
Shear Diagram:

Moment Diagram:
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### Calculation Topic

**Main Room 2nd Flr Beam - (B3)**

#### Reference

**Longitudinal Steel (over supports)**

- **Mu = PAsy (d - a/2)**
- Assume: \( d = h + 2.5" = 9.15" \)
- \( a = d/2 = 4.58" \)
- \( 0.082 \text{ kN} \cdot \text{m} = 45.051 \text{ kFt} \)

**FISA**

\[
(45.051 \text{ kFt})(12/1ft) = (1.99)(12)(0.9)(0.0 ksi)(9.5" - 4.58\text{'in'})
\]

**As req'd**

\[
\frac{(45.051 \text{ kFt})(12/1ft)}{(1.99)(9.0)(0.0 ksi)(7.915\text{'in'})} = 1.20 \text{ in}^2
\]

\( \Rightarrow \) Use 3 #6 bars

**Acf = 1.32 \text{ in}^2**

#### Check #1

\[ a = \frac{Asy}{0.85P_{Gb}} \]

\[ = \frac{(1.32 \text{ in}^2)(0.0 ksi)}{0.85(4 \text{ ksi})(12)} \]

\[ = 1.94 \text{ in} \]

\( \Phi_{Min} = \Phi_{Asy} (d - a/2) \)

\[ = (0.9)(1.94") (0.0 ksi)(9.625 - 1.94") \]

\[ = 90 \text{ kFt} \]

\[ = 75,500 \text{ kFt} + > 45,051 \text{ kFt} \]

#### Check #2

ACI 21.2.2

\[ C = 9.0 = 1.94"/0.85 = 2.28" \]

\( d = 12" - 1.5" - 0.5" - (9.0)/2 = 9.625" \)

\( Ec (d - c) = Ec \)

\[ 0.003 \left( 9.625" - 2.28" \right) \geq 0.004 \]

\[ 0.009 \leq 0.004 \]

\( \Rightarrow \) Tension controlled!

\[ \Rightarrow \text{Use 3 #6 bars over support} \]
Longitudinal Steel (between supports ~ 4m - 13.12' span)

\[ Mu = \frac{\text{Asy}}{fy (d - 9/2)} \]

- Assume \( d = h - 2.5" = 9.5" \)
- \( a = \frac{d}{2} = 3.17" \)
- \( 34.5 \text{ KN-m} = 25,158 \text{ kft} \)

\[ Mu = \frac{34.5 \text{ KN-m}}{25,158 \text{ kft}} = \frac{34.5}{25,158} \]

\[ = 0.714 \text{ in}^2 \]

\[ \Rightarrow \text{Try: 2 - #6 bars} \]
\[ A_s = 0.88 \text{ in}^2 \]

Check #1:
\[ \phi M_{n} = \phi A_{s} f_{y} (d - 9/2) \]
\[ = (0.85)(0.88 \text{ in}^2)(100 \text{ ksi})(9.5" - 4.5") \]
\[ = 0.72 \text{ kft} \]
\[ \phi M_{n} = \frac{426 \text{ kft}}{35.56 \text{ kft}} > 25.15 \text{ kft} \] \( \checkmark \)

Check #2:
\[ c = \frac{d}{2} = 1.29" / 0.65 = 1.92" \]
\[ c = 12" - 1.5" - 0.5" - \frac{1}{2} = 9.025" \]
\[ \epsilon_c (d - c) = 2 \epsilon _{e} \]
\[ 0.003 \left( \frac{9.025" - 1.52"}{1.52"} \right) \geq 0.004 \]
\[ 0.0010 \geq 0.004 \] \( \checkmark \)

17: tension controlled

\[ \Rightarrow \text{Use 2 - #6 bars between supports} \]
Hooked longitudinal bars @ ends:

\[ d = \sqrt{\frac{-2Mu}{f_{y}0.85cb}} + d^2 \]

where:
\[ d = 9.075" \text{ from bar over support (# columns)} \]
\[ 9.075" = \sqrt{\frac{-2(25.54 \text{ k-ft})(9.075")}{(0.9)(0.85)(4.61)(12")}} + (9.075")^2 \]
\[ = 9.075" - 8.714" \]
\[ = 0.361" \]
\[ A_{seb} = \frac{0.65(1.6b)}{f_{y}} = \frac{(0.65)(4.61)(12")(0.361")}{60 \text{ ksi}} = 0.027 \text{ in}^2 \]

\[ \rightarrow \text{use 2-#6 hooked bars} \]

\[ A_{seb} = 0.66 \text{ in}^2 \]

Check #1:
\[ a_{y} = \frac{0.65f_{y}}{0.65(4.61)(12")} = 1.29" \]

\[ f_{mn} = \phi f_{y} a_{y} (d - a/2) \]
\[ = (0.9)(0.65(4.61)(12"))(9.075" - 1.29") \]
\[ = 25.54 \text{ k-ft} > 25.54 \text{ k-ft} \checkmark \]

Check #2:
\[ c = \frac{d}{\beta} = \frac{9.075"}{1.52"} = 5.96 \text{ in} \]
\[ \frac{d}{c} = \frac{9.075"}{5.96} \geq 6.0 \]

\[ \epsilon_c \left( \frac{d-c}{c} \right) \geq 0.004 \]
\[ 0.003 \left( 9.075" - 5.96" \right) \geq 0.004 \]
\[ 0.014 \geq 0.004 \]
\[ \Rightarrow \text{tension controlled} \]

\[ \rightarrow \text{use 2-#6 hooked bars (closed by a cross tie)} \]

\[ \epsilon_{dn} = \frac{f_{y} f_{y} f_{y}}{f_{y}} \cdot \partial_{dn} = 0.01 \]
**Shear**

\[ V_c = 2 \sqrt{f_{c b}} \frac{b d}{2} \]

\[ = 2 \sqrt{4000 \text{ psi}} \frac{(12'')(9.025'')}{2} \]

\[ = 14.61 \text{ K} \]

\[ \phi V_n = \frac{\phi (V_c + V_s)}{\phi 10 \sqrt{f_{c b} d}} \]

\[ = \frac{(0.75)(10)(4000 \text{ psi}) (12'')(9.025'')}{51.78 \text{ K}} \]

\[ V_{n,\text{max}} = 88.5 \text{ KN} = 19.9 \text{ K} \]

\[ \Rightarrow V_c < V_{n,\text{max}} \quad \text{need shear reinf.} \]

\[ \phi V_n > V_n \quad \text{don't need to increase beam width} \]

**Max. Spacing:**

\[ V_{n,\text{max}} > \frac{6 \sqrt{f_{c b} d}}{\phi} \]

\[ \frac{19.9 \text{ K}}{0.75} > \frac{6 \sqrt{4000 \text{ psi}} (12'')(9.025'')}{31.78 \text{ K}} \]

\[ 26.053 \text{ K} > 43.83 \text{ K} \]

\[ \Rightarrow \text{use } S_{\text{max}} = d/2, \text{ not } d/4 \]

\[ S_{\text{max}} = \frac{d}{2} \]

\[ = \frac{9.025''}{2} \]

\[ = 4.51'' \]

**Min. Shear Reinforcement:**

\[ A_{v,\text{min}}/\delta = \text{greater of:} \]

\[ = 0.75 \sqrt{f_{c b} w_f} \]

\[ = (0.75) \sqrt{1000 \text{ psi}} (12'')(60,000 \text{ psi}) \]

\[ = 0.0095 \text{ in}^2/\text{in} \]

\[ = 50 \text{ bw/ft} \]

\[ = 50 (12'')(60,000 \text{ psi}) \]

\[ = 0.01 \text{ in}^2/\text{in} \]

\[ A_{v,\text{min}} = (0.01 \text{ in}^2/\text{in})(4.51 \text{ lin}) \]

\[ = 0.04561 \text{ in}^2 \]

\[ \Rightarrow \text{use } #3 \text{ stirrups} \]

\[ A' = (2)(0.11 \text{ lin}^2) \]

\[ = 0.22 \text{ lin}^2 \]
\[ \text{Shear Reinf Spacing:} \]

\[ \begin{align*}
A & \approx 75.2 \text{KN} \approx 16.9 \text{ k/lf} = 22.5 \text{ K} \\
B & \approx 88.5 \text{KN} \approx 19.9 \text{ k/lf} = 24.8 \text{ k (max)} \\
C & \approx 82.7 \text{KN} \approx 13.6 \text{ k/lf} = 24.8 \text{ K} \\
D & \approx 80.9 \text{KN} \approx 18.2 \text{ k/lf} = 24.3 \text{ K}
\end{align*} \]

\[ \text{4° = 0.75} \]

\[ \begin{align*}
\text{A-B slope} & = 22.5k + 24.8k \\
& \frac{(4m) \left(3.28 + \frac{12in}{12in}\right)}{4m} \\
& = 0.31 \text{ k/in}
\end{align*} \]

\[ \begin{align*}
\text{C-D slope} & = 24.8k + 24.3k \\
& \frac{(4m) \left(3.28 + \frac{12in}{12in}\right)}{4m} \\
& = 0.31 \text{ k/in}
\end{align*} \]

\[ \begin{align*}
\text{ACI 22.5.10} & \\
\text{specify} & = \frac{W}{L} \cdot V_e \\
& = \frac{(0.22 \text{ln}^2)(60 \text{ksi})(9.625^2)}{26.5 \text{ksi} - 14.81 \text{K}} \\
& = 10.69 \text{ in}
\end{align*} \]

\[ \text{Smax } \approx 4\text{"} \] since max shear requires a spacing larger than the code maximum, use max spacing throughout entire beam

\[ \text{use #3 stirrups @ 4\" o.c. throughout entire beam} \]
Final Beam Design (B3):

- 2-No. 6's hooked to column support (90° and hook)
- C = 2.28''
- Q = 1.94''
- T = C
- 2-No. 6's over the supports
- 2-No. 6's between supports
- No. 3 stirrups @ 4" o.c.
Main Room Quick Calc C1, C2

Concrete Slab:

Crack Width Continuous Min Depth = \( \frac{1}{8} \) 8.5

\[
\frac{10.7}{8.5} = 0.8 \times 12\:\text{"} = 9.6\:\text{"} = 12\:\text{"}
\]

Code governs.

Both end Continuous Min Depth = \( \frac{1}{2} \) 21

\[
\frac{21}{21} = 0.78 \times 12\:\text{"} = 9.4\:\text{"}
\]

Beam Width (1/2 Beam Depth)

\[
\left(\frac{1/2 \times 12}{12}\right) = \frac{1}{2}\:\text{"}
\]

Code Minimum Width = 12"
Member Labels:

Loading (1.2D + 1.6L):
Shear Diagram:

33.2 kft

Moment Diagram:

79.3 kft

42.5 kft
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<th>Shear[kN]</th>
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Main Room 2nd Floor Girder - (L1)

Girder Design (L1)

Load Combination: 1.2DL + 1.6LL
DL = 54 ksf
LL = 20 ksf

\[ F_y = 60,800 \quad F_e = 4,481 \quad R_x = 1 \]

\[ L_e = L \cdot R_x = (20 \times 0.6) = 12 \text{ ksf} \]

\[ \omega_{LF} = (1.2 \times 5.5 \times 12 \times 11 \times 3.28) = 242.4 \text{ ksf} = 25.376 \text{ kN/m} \]

\[ \omega_{LF} = (1.1 \times 12 \times 11 \times 3.28) = 149.2 \text{ ksf} = 16.12 \text{ kN/m} \]

\[ \omega_{LF} = (1.2 \times 5 \times 8 \times 8.23) = 171.3 \text{ ksf} = 18.77 \text{ kN/m} \]

\[ \omega_{LF} = (1.1 \times 12 \times 8 \times 8.23) = 50.2 \text{ ksf} = 5.55 \text{ kN/m} \]

\[ \omega = 107.65 \text{ kN/m} \]

Adjusted \( M_u \) (over the support):

\[ M_u = 79.2 \text{ kN/m} \]

Assume: \( a = \frac{1}{2} \)

\[ d = b + 2.5'' = 17.5'' \]

\[ A = \frac{M_u}{(0.9 \times 0.4 \times 17.5 - 8.83)} = 1.208 \text{ in}^2 \]

Try 3 - No. 5's, \( A_s = 1.22 \text{ m}^2 \) (over the supports)

Check:

\[ a = \frac{A_{stf}}{0.85 \times b} = \frac{(1.22 \times 12)}{(0.85 \times 12)} = 1.99'' \]

\[ c = \frac{a}{b} = \frac{2.64''}{0.85} = 2.5'' \]

\[ d'' = 1.5'' + \left( \frac{3}{2} \right) a_{stf} + \left( \frac{1}{2} \right) w_{stf} + \frac{1}{2} n_a\theta_{stf} = 2.27'' = 2.5'' \]

\[ E_S = 0.003 \quad (d'' - c) = 0.003 \quad (17.5 - 2.28) = 0.020 > 0.005 \]
Reference:

\[ A_{\text{min}} = 3 \frac{ft^2}{in^2} = \frac{3}{12} \times 12000 \times (12)(17.5) = 0.167 \text{ in}^2 < 1.32 \text{ in}^2 \]

\[ A_{\text{min}} = 2 \frac{ft^2}{in^2} = \frac{2}{12} \times 12000 \times (12)(17.5) = 0.7 \text{ in}^2 < 1.32 \text{ in}^2 \]

\[ \phi M_n = 0.7 \left(1.32 \times 0.6 \times \left(17.5 - \frac{17.2}{2}\right)^2\right) = 1178.26 \text{ kN-m} = 78.19 \text{ kft} \]

\[ \phi M_n = 98.19 \text{ kft} > M_n = 77.3 \text{ kft} \quad \checkmark \quad \text{O.K.} \]

\[ M_n = 57.7 \text{ kN/m} \Rightarrow 42.6 \text{ kft} \quad \text{(Between the spans)} \]

\[ A_{\text{req'd}} = \left(\frac{42.6 \times 12.7^2}{60 \times 17.5 - \frac{5.82}{2}}\right) = 0.049 \text{ m}^2 \]

\[ \Rightarrow \text{Ttry 2 - No. C's, } A_S = 0.88 \text{ m}^2 \text{ (b/w the supports)} \]

Check:

\[ q = \frac{A_{\text{fl}}}{0.85 \times b} = \frac{0.88 \times 12.7}{0.85 \times 12} = 1.27" \]

\[ \beta = 0.85 \]

\[ c = q / \beta = 1.27 / 0.85 = 1.527" \]

\[ \phi S = 0.003 \left(\frac{4 - c}{c}\right) = 0.003 \left(\frac{17.5 - 1.522}{1.522}\right) = 0.0315 > 0.005 \]

\[ A_{\text{min}} = 0.71 \text{ in}^2 < 0.88 \text{ in}^2 \quad \checkmark \]

\[ \phi M_n = 0.7 \left(0.88 \times 0.6 \times \left(17.5 - \frac{1.27}{2}\right)^2\right) = 800.75 \text{ kN-m} = 716.7 \text{ kft} \]

\[ \phi M_n = 616.7 \text{ kft} > 42.6 \text{ kft} \quad \checkmark \quad \text{O.K.} \]
Hooked long bars @ ends:

\[ M_u = 0.24 \text{ kN-m} = 460 \text{ k-lb-ft} \]

\[ a = \frac{-2M_u}{40.85\text{ ksi} \cdot b} + d^2 \]

\[ d = 17.5" \text{ (from prev.)} \]

\[ = 17.5" - \sqrt{-2 \left( \frac{4.2 \text{ k-ft}}{0.9 \text{ ksi}} \right) \left( \frac{12\text{ in}}{1\text{ ft}} \right) \frac{1.86\text{ in}}{(0.85\text{ ksi})(12\text{ in})}} + (17.5\text{ in})^2 \]

\[ = 0.881" \]

\[ A_{area} = 0.85\text{ ft}^2 \cdot ba = (0.85)(4\text{ ksi})(12\text{ in})(0.881\text{ in}) = 0.599\text{ in}^2 \]

\( \Rightarrow \) use 2 - #6 hooked bars, \( A = 0.88\text{ in}^2 \)

Check #1:

\[ a = \frac{M}{0.85\text{ ksi} \cdot (0.88\text{ in})} = 1.24" \]

\[ d^2 = \frac{4M (d - \frac{d}{2})}{(0.9\text{ ksi})(0.88\text{ in})(60\text{ ksi})(17.7\text{ in})(1.29\text{ in})} \]

\[ = 0.724\text{ k-ft} > 460\text{ k-ft} \checkmark \]

Check #2:

\[ \frac{d}{\sqrt{b}} = 1.24\text{ in} \cdot 0.85 = 1.52\text{ in} \]

\[ d = 20\text{ in} - 15\text{ in} - 0.5\text{ in} - \frac{1}{2} (\frac{7}{8}\text{ in}) = 17.625" \]

\[ ec \left( \frac{d+c}{2} \right) = 6" \]

\[ 0.003 \left( \frac{17.625" - 1.52"}{152"} \right) \geq 0.004 \]

\[ 0.03 \geq 0.004 \rightarrow \text{ tension controlled} \checkmark \]

\[ \text{Size 4.3} \]

\( \Rightarrow \) use 2 - #6 hooked bars (closed by n. cross tie)

\[ \text{Sec. 4.3.1} \]

\[ d_{in} = \text{greater of:} \]

\[ \frac{6d}{10} = 6 \text{ in} \cdot 10 \text{ in} = 60\text{ in} \cdot 60" \]

\[ = 14.23" \]

\[ \text{Net } = 9" \]
Shear Demand:

\[ V_{\text{max}} = 10 \cdot 1.2 \cdot f_{c} \cdot \text{b} \cdot \text{d} \cdot \left( 0.75 \cdot Y_{2.0} \cdot 4000 \cdot 12 \cdot 13.5 \right) = 70.8 \text{ kN} \]

\[ V_{\max} = 23.2 \text{ kN} \]

\[ V_{a} = 4.2 \cdot 1.2 \cdot f_{c} \cdot \text{b} \cdot \text{d} \cdot \left( 0.75 \cdot Y_{2.0} \cdot 4000 \cdot 12 \cdot 13.5 \right) = 20.5 \text{ kN} \]

\[ V_{\text{max}} = 23.2 \text{ kN} \neq V_{a} = 20.5 \text{ kN} \Rightarrow \text{Reinforcement required.} \]

\[ V_{\text{min}} = 14.8 \text{ kN} \leq V_{\text{max}} = 23.2 \text{ kN} \checkmark \text{Do not need to use beam width.} \]

Spacing requirements:

\[ V_{y} = \phi \cdot V_{\text{c, bw}} \]

\[ V_{y} = 23.2 \text{ kN} \cdot 1.2 \cdot f_{c} \cdot \text{b} \cdot \text{d} \cdot \left( 0.75 \cdot Y_{2.0} \cdot 4000 \cdot 12 \cdot 13.5 \right) = 77.7 \text{ kN} \]

\[ \frac{V_{y}}{\phi} = \frac{23.2 \text{ kN}}{0.75} = 41.3 \text{ kN} \Rightarrow \text{Sans = } \frac{d}{2} \text{ not } \frac{d}{4} \]

Maximum shear reinforcement - greater of \( A_{v, \text{min}} \) and \( A_{v, \text{mm}} \)

\[ A_{v, \text{mm}} = 0.75 \cdot f_{c} \cdot \text{bw} \cdot \left( 0.75 \times Y_{2.0} \times 12 \right) = 0.009 \text{ m}^2 \]

\[ A_{v, \text{min}} = \frac{50 \text{ kN}}{f_{y}} = \frac{50 \times 12}{60,000} = 0.01 \text{ m}^2 \]

\[ \Rightarrow \text{Use No. 2 stirrups, } \text{AV} = (2) (0.11) = 0.22 \text{ m}^2 \]

Shear Spacing:

\[ S_{\text{max}} = \frac{d}{2} = 175/2 = 8.75'' \]

\[ S_{\text{req'd}} = \frac{A_{v, \text{mm}}}{V_{y}} - V_{a} \]

\[ = \frac{23.2 \text{ kN}}{41.3 \text{ kN}} - 20.5 \text{ kN} \]

\[ = 7.48'' \]

\[ \Rightarrow \text{Smax} < \text{Sreq'd, use Smax for entire beam. Beam spacing is based on highest shear load.} \]

\[ \text{Use No. 3 stirrups @ 8'' O.C. throughout} \]
Final Girder Design (6.1):

- 2 - No. 6 Hooked bars 8" column supports (7/8" straighthook)
- C = 2.28"
- E_σ = 0.06% (Eo = 0.005)
- 3 - No. 4's over the support
- 2 - No. 4's between the supports
- No. 3 stirrups @ 6" O.C.
All trusses will be the same, but design will be based on worst case scenario. See column line between gridline 1 \& 2.

Truss shape: flow

Each bay = 1m in length

3.28' in length

Tri b width = 5m = 16.41'

Roof slope: 15:12

Load combo:

- 1.2 D + 1.6 L
- 1.4 D
- 1.2 D + 1.0 W + 0.5 L
- 0.7 D + 1.0 W

Live load reduction:

\[ A_t (10m)(5m) = 50m^2 \]

\[ F = 1.5 \]

\[ R_1 = 0.6 \quad \text{b/c} \quad A_t = 50.74m^2 \]

\[ R_2 = 1.0 \quad \text{b/c} \quad F = 4 \]

Load (gravity):

\[ A_r \text{ per node} = (5m \times 1m) \times \left( \frac{53.624ft^2}{5m^2} \right) = 26.91ft^2 \]

\[ P_{d,pl} = (11psf)(26.91 ft^2) = 296.00 \quad \text{#} \]

\[ = 13.17 \quad \text{KN} \]

\[ P_{u,pl} = (12psf)(26.91 ft^2) = 32.29 \quad \text{#} \]

Unfactored, will be factored on riser to account for load combinations.
Load (lateral):

used wind pressure b/c it directly hits each truss joint

\[ P = 57.28 \text{ psf} \]

\[ A_{1} = (0.125 \text{ m}) (5 \text{ m}) = 0.625 \text{ m}^2 = 6.727 \text{ ft}^2 \]

\[ P_{\text{total}} = (57.28 \text{ psf}) (6.727 \text{ ft}^2) = 384.9 \text{ ft-lb} = 1.71 \text{ kN} \]

~Truss analysis from KISA~

see next page
Load Combinations:

LC #1: 1.4D

LC #2: 1.2D + 1.6Lr

LC #3: 1.2D + 0.5Lr + 1.0W

LC #4: 0.9D + 1.0W
Member Labels:

Member Forces:

Top Chord:

<table>
<thead>
<tr>
<th>Member</th>
<th>S...</th>
<th>Axial[kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M59</td>
<td>1</td>
<td>max 638.291</td>
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<tr>
<td></td>
<td></td>
<td>min 196.602</td>
</tr>
</tbody>
</table>

Web:

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<tr>
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<th>S...</th>
<th>Axial[kN]</th>
</tr>
</thead>
<tbody>
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<td>M22</td>
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<td>max 305.077</td>
</tr>
<tr>
<td></td>
<td></td>
<td>min 88.823</td>
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</table>

Bottom Chord:

<table>
<thead>
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<th>S...</th>
<th>Axial[kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M49</td>
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<td>max 508.388</td>
</tr>
<tr>
<td></td>
<td></td>
<td>min 168.724</td>
</tr>
</tbody>
</table>
Governing Axial Loads:

Top Chord: \( 38.291 \text{ kN} = 143.5 \text{ k} \)

Web: \( 305.677 \text{ kN} = 68.6 \text{ k} \)

Bottom Chord: \( 508.888 \text{ kN} = 114.3 \text{ k} \)

Length of Top/Bottom Chord:

\[ L = \sqrt{(1 \text{ m})^2 + (0.125 \text{ m})^2} = 1.008 \text{ m} (3.31 \text{ ft}) \]

Length of Web:

\[ L = \sqrt{(1 \text{ m})^2 + (1.125 \text{ m})^2} = 1.505 \text{ m} (4.94 \text{ ft}) \]

Design:

Top Chord:

\[ KL = (1.0)(3.31 \text{ ft}) = 3.31 \text{ ft} \]

\[ Pu = 143.5 \text{ k} \]

Check:

\[ \frac{A}{t} \leq 0.55 \sqrt{\frac{E}{f_y}} \]

\[ 4^2 \leq 0.55 \sqrt{290000 \text{ ksi}} \]

\[ 0.385 \leq 0.55 \sqrt{400 \text{ ksi}} \]

\[ 12.8 \leq 13.81 \checkmark \]

Web:

\[ KL = (1)(4.94 \text{ ft}) = 4.94 \text{ ft} \]

\[ Pu = 68.6 \text{ k} \]

Check:

\[ \frac{A}{t} \leq 0.55 \sqrt{\frac{E}{f_y}} \]

\[ 3.025 \leq 13.81 \checkmark \]

Use:

- HSS 4 x 4 x 5/16
- Pn = 158 k

- HSS 3 x 3 x 1/4
- Pn = 93 k
**ROOF TRUSS DESIGN - (T1)**

**Bottom Chord**

KL = (1.0)(3.31') = 3.31' ft

Pu = 114.3 k

**Check:**

\[
\frac{0.6}{1.0} \leq 0.55 \sqrt{f_{y}}
\]

\[
\frac{0.375}{13.81} \leq 13.81
\]

\[
8 \leq 13.81 \checkmark
\]
See B3 Quick Cale for starter size: 12" x 12"

See B3 calcs for loads since mb mms are the same

\[ L \cdot P_L = 1.042 \text{ kN/m} \]
\[ L \cdot L_L = 0.722 \text{ kN/m} \]

Since loads and mb area is the same, B3 = B4

- 12" x 12" conc beam
- 3 #6 longitudinal bars over supports
- 2 #6 longitudinal bars between supports
- #3 stirrups @ 4" o.c. throughout beam
Final Beam Design (E4):

2 - No. L's hooked to column support (90° steel hooks)

3 - No. L's over the supports
2 - No. L's between supports

No. 3 stirrups @ 4" o.c.
Main Room Roof Girder - (6-2)

See GI quick calc for girder starting size: 12'' x 12''

Loads:
Load combo: 1.2D + 1.6L

DL = 5sf psf (8m x 3.28 ft/m)
  = 1469.4 plf
  = 1.47 klf
  = 21.4 KN/m

LL = 20 psf
  L = A_t \times \frac{(25m) (8m)}{200 m^2}
  R_1 = 0.6 w/c A_t \geq 55.7 ft^2
  R_2 = 1.0 w/c NO slope
  L_r = (0.6)(1.0)(20 psf)
  = 12 psf

LL = (12 psf) (8 m x 3.28 ft/m)
  = 394.88 plf
  = 4.6 KN/m

Wt = 1.2D + 1.6L
  = 2.27 klf
  = 33.04 KN/m

* Moment & shear found w/ Risa analysis (attached) *

12'' x 12''

Asmin

\[ \text{Asmin} = \frac{200}{f_y} \text{bw}d \]

\[ = \frac{200}{6000 \text{psi}} (12'')(9.5'') \]

\[ = 0.38 \text{ in}^2 \]

\[ \text{Asmin} = \frac{3 \sqrt{f_c}}{f_y} \text{bw}d \]

\[ = \frac{3 \sqrt{4000 \text{ psi}}}{60000 \text{ psi}} (12'')(9.5'') \]

\[ = 0.31 \text{ in}^2 \]
Member Labels:

Load Combo (1.2D+1.6L):

Shear Diagram:

Moment Diagram:

Member Forces:

<table>
<thead>
<tr>
<th>Member Section Forces</th>
<th>Member Label</th>
<th>Sec</th>
<th>Axial[kN]</th>
<th>Shear[kN]</th>
<th>Moment[kNm]</th>
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<td>2</td>
<td>43.82</td>
<td>15.092</td>
<td>-7.661</td>
</tr>
</tbody>
</table>
longitudinal steel (over supports):

\[ mu = \phi A_{fy} \left( d - \frac{d}{2} \right) \]

\[ L_{assume} = \frac{h - 2.5''}{3.17''} \]

\[ A_{c, revd} = \frac{M_{max}}{\phi_{fy} \left( d - \frac{d}{2} \right)} \]

\[ = \frac{52.63 \text{kft}}{(0.9)(60 \text{ksi})(9.5'' - 3.17'')} \]

\[ = 1.41 \text{in}^2 \]

\[ \Rightarrow my 3 - # 7 \text{ bars} \]

\[ A_s = 1.80 \text{in}^2 \quad > A_{s, min} \]

Check #1:

\[ a = \frac{A_s f_y}{0.85 f_{cb}} = \frac{(1.80 \text{in}^2)(60 \text{ksi})}{0.85(4 \text{ksi})(12'')} = 2.05'' \]

\[ \phi_{Mh} = \phi A_s f_y \left( d - \frac{d}{2} \right) \]

\[ = (0.9)(1.80 \text{in}^2)(60 \text{ksi})(9.5' - 3.12'') \]

\[ = 800 \text{ kipin} \quad > 520.3 \text{ kft} \quad \checkmark \]

Check #2:

\[ C = \frac{q/8}{0.85} = 3.12'' \]

\[ d = 12'' - 15'' \text{ cover} - 0.5'' - \frac{1}{2} (7/8'') = 9.5125'' \]

\[ f_{assumed} = 110 \text{ psi} \]

\[ E_o \left( \frac{d - c}{c} \right) \geq E_t \]

\[ 0.003 \left( \frac{9.5125'' - 3.12''}{3.12''} \right) \geq 0.004 \]

\[ 0.003 \geq 0.004 \quad \checkmark \]

\[ \Rightarrow \text{tension controlled} \]
Longitudinal Steel (between supports):

Worst case span: 5m = 16.4'

\[ Mu = \frac{4}{3} As f_y \left( d - \frac{q}{2} \right) \]

\[ L \geq Mu = 29.1 \text{ kN m} = 29.1 \text{ kft} \]
\[ d - h = 2.5'' = 9.5'' \]
\[ a = \frac{d}{h} = 3.17'' \]

\[ \text{Area of section:} \quad A = \frac{Mu}{fy (d - q/2)} = \frac{(29.1 \text{kft}) (12''/1ft)}{(0.9)(60 \text{ksi})(9.5''-317/2'')} = 0.021 \text{ in}^2 \]

\[ \Rightarrow 2 -\#6 \text{ bars } \quad A_s = 0.08 \text{ in}^2 > A_{min} \]

Check #1:
\[ a = \frac{0.08}{0.05} = \frac{(0.08 \text{ in}^2)(60 \text{ksi})}{(0.05)(14 \text{ksi})(12'')} = 1.29'' \]

\[ \# M_n = 4 As f_y (d - q/2) = (0.9)(0.08 \text{ in}^2)(60 \text{ksi})(9.625'' - 1.29''/2) \]
\[ = 261.7 \text{ kN} \]
\[ = 35.5 \text{ kft} > 29.1 \text{ kft} \]

Check #2:
\[ \varphi = \frac{9}{12} = 1.29''/0.85 = 1.52'' \]
\[ d - 12'' - 1.5'' \text{ cover} - 0.5'' - \frac{1}{2}(9.6''/2) = 9.025'' \]
\[ E (d - C) \geq 0.004 \]
\[ 0.003 \left( 9.025'' - 1.52'' \right) \geq 0.004 \]
\[ 0.016 \geq 0.004 \quad \checkmark \]
\[ \text{tension controlled} \]

\[ \Rightarrow \text{use 2-\#6 bars between supports.} \]
Hooked long bars @ ends:

\[ a = \sqrt{\frac{-2\mu}{f_c' + 0.85f_y}} + d^2 + d = 9.025" \text{ (u/b bar)} \]

\[ = 9.025" - \sqrt{\frac{-2(379 \text{ k-ft}) (12"/ft^2) (0.9)(0.85)(4 \text{ ksi})(12")}{(0.9)(0.85)(4 \text{ ksi})(12")} + (9.025")^2} \]

\[ = 1.29" \]

As req'd \[ \frac{0.85f_c'b\alpha}{f_y} \frac{0.85(12"/ft^2)(12")}{60 \text{ ksi}} = 0.995 \text{ in}^2 \]

\[ \Rightarrow \text{my 2-#7 hooked bars, } A_s = 1.20 \text{ in}^2 \]

Check #1:

\[ \alpha = \frac{Asy}{f_y} = \frac{(1.2 \text{ in}^2)(60 \text{ ksi})}{0.85(12"/ft^2)(12")} = 1.76" \]

\[ a = a = \frac{d}{d - 0.5"} = \frac{12" - 1.5" - 0.5" - \frac{1}{2}(7.5")}{9.51125} = 0.995 \text{ k-ft} \]

\[ \phi = 0.85(1.2 \text{ in}^2)(60 \text{ ksi})(1.51125" - 1.76") \]

\[ = 40.8 \text{ k-ft} > 37.9 \text{ k-ft} \checkmark \]

Check #2:

\[ a = \frac{9.025"}{0.85} = 2.1" \]

\[ d = 9.51125" \]

\[ Ec(d-c) \geq Ec \]

\[ 0.003 \frac{9.51125" - 2.1"}{2.1"} \geq 0.004 \]

\[ 0.01 \geq 0.004 \checkmark \Rightarrow \text{tension (allowed)} \]

\[ \text{s18.14.3} \Rightarrow \text{use 2-#7 hooked bars (closed by a cross-tie)} \]

\[ 2.54.3.1 \text{ load greater of: } \frac{50 \text{ lbs}}{12" \sqrt{f_{c'}}}, 8 \text{ db, 6"} \]

\[ = 11.4" \]

schedule

\[ b_{ext} = 10.5" \]
Shear Reinforcement:

\[ V_c = 2 \sqrt{f_c \cdot b w} \]
\[ = 2 \sqrt{4000 \text{ psi} \cdot 12''} \cdot (9.5/25'') \]
\[ = 145.14 \text{ kN} \]
\[ = 14.5 \text{ kips} \]

\[ \Phi V_n = \Phi (V_c + V_s) \]
\[ = 0.75 (0.75) (4000 \text{ psi} \cdot 12'' \cdot (9.5/25'')) \]
\[ = 54.4 \text{ kN} \]
\[ = 12.1 \text{ kips} \]

\[ V_{u \max} = 87.8 \text{ kN} = 19.74 \text{ kips} \]

\[ \Rightarrow V_c < V_{u \max} : \text{need shear reinforcement.} \]

\[ \text{Max spacing:} \]
\[ V_s > \frac{\sqrt{f_c \cdot b w}}{\Phi} \]
\[ = \frac{145.14}{0.75} \]
\[ = 193.52 \text{ kN} \]
\[ = 43.5 \text{ kips} \]

\[ 2(12'') \cdot 32 \text{ k} > 43.5 \text{ kips} \cdot X \]
\[ \Rightarrow \text{use } S_{\max} = \frac{d}{2}, \text{ not } d^1 \]

\[ S_{\max} = \frac{d}{2} \]
\[ = 9.5/25'' \div 2 \]
\[ = 4.78 \text{ in} \]

\[ \text{Min shear reinforcement:} \]

\[ A_{\min} / = \text{greater of} \]
\[ = 0.75 \sqrt{f_c \cdot b w} / \Phi \]
\[ = 0.75 \sqrt{4000 \text{ psi} \cdot 12'' / 60000 \text{ psi}} \]
\[ = 0.0095 \text{ in}^2 / \text{in} \]

\[ A_{\min} = (0.01 \text{ in}^2 / \text{in}) (4.78'') \]
\[ = 0.0478 \text{ in}^2 \]

\[ \Rightarrow \text{use #8 stirrups} \]
\[ A_V = 2(0.11 \text{ in}^2) \]
\[ = 0.22 \text{ in}^2 \]
Reference

Act 22.5.10
max V

\[
\text{S} = \frac{N_f + d}{V_f - V_c}
\]

\[
= \frac{(0.22 \text{ in}^3) (60 \text{ ksi})(9.5625 \text{ in})}{(260.32 \text{ k}) - 14.5 \text{ k}}
\]

\[
= 10.67
\]

Shear at 4" max shear location requires a spacing larger than code maximum, so use max spacing throughout.

\[
\rightarrow \text{ Use #3 stirrups @ 4" o.c. throughout beam}
\]
Final Girder Design (42):

- 2 No. 7's hooked and column support (90° stud hook)
- 3 = 3.12"
- 7 = 2.15"
- 2 No. 7's over the supports
- 2 No. 6's between supports
- 3 stirrups @ 4" o.c.
Stage Header (H1)

Schematic:

- Sill beam (H1)
- Roof girder (G2)
- 2F girder (G1)

Based upon allowable space above stage opening, beam preliminary size will be:

12" x 53"

Loads:

\[ DL = (2F \text{ girder loads} + \text{roof girder self wt.}) \]
\[ = 1.47 \text{ klf} + 0.15 \text{ klf} = 1.62 \text{ klf} = 23.6 \text{ KN/m} \]

Long span length = 18 m

Long span length = 10.4 m

Stage rated length = 34'

ASCE §4.8.2

- LR = 10.4 x (1.0) x (20 psf)
- = 208 psf

LR = 10.4 x (1.0) x (20 psf)
- = 208 psf

LL = (12 psf) x (5 x 3.28 ft/m)
- = 314.88 psi
- = 4.16 klf
- = 4.16 KN/m

Load combo = 1.2DL + 1.6LL
Member Labels:

Load Combo (1.2D+1.6L):

Shear Diagram:

Moment Diagram:

Member Forces:

<table>
<thead>
<tr>
<th>Member Label</th>
<th>X-axis Force</th>
<th>Shear Force</th>
<th>Moment Force</th>
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<tr>
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<td>-84</td>
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<td>M5</td>
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<tr>
<td>M20</td>
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<td>448,117</td>
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*Imperial units*
longitudinal steel (between supports):

\[ M_u = 294.6 \text{ k-ft} \]
\[ a = \frac{d}{\sqrt{\frac{2M_u}{1.05 fc' (\text{CIb}) + d^2}}} = \frac{53'' - 15'' - 0.5'' - 1.5''}{\sqrt{\frac{2(294.6 \text{ k-ft})}{1.05 (4000 (12''/12)) (1.5'')^2}} = 49.5''} \]
\[ = 49.5'' - \sqrt{\frac{-2(294.6 \text{ k-ft}) (12'/12) (0.9) (0.85) (4000) (1.5''}) = 49.5''^2} \]
\[ = 49.5'' - 17.52'' \]
\[ = 19.98'' \]

\[ A_{rein} = \frac{0.85(12'')(0.05)(4000)(12'')(1.5'')}{60 \text{ ksi}} = 1.346 \text{ in}^2 \]

\[ \text{Check 1:} \]
\[ \text{Check } \frac{A_{rein}}{0.85(12'')(0.05)(4000)(12'')} = \frac{(1.346 \text{ in}^2)}{60 \text{ ksi}} = 2.33'' \]

\[ \phi M_u = \phi A_{rein} f_y (d - a/2) = (0.9)(1.346 \text{ in}^2) (60 \text{ ksi}) (50.5'' - 2.33''/2) \]
\[ = 3506.8 \text{ k-ft} > 294.6 \text{ k-ft} \]

\[ \text{Check 2:} \]
\[ c = \frac{a/8}{0.85} = 2.74'' \]
\[ \varepsilon_c \left( \frac{d - c}{c} \right) \geq \varepsilon_c \]
\[ 0.002 \left( \frac{50.5'' - 2.74''}{2.74''} \right) \geq 0.004 \]
\[ 0.052 \geq 0.004 \]

\[ \text{min-reed} \rightarrow \text{Use 2-#8 long bars} \]

\[ A_{min} = \frac{200 \text{ ksi}}{60 \text{ ksi}} = \frac{200}{60000} (12'')(50.5'') = 2.02 \text{ in}^2 \]

\[ = \frac{3450}{6000} (12'')(50.5'') = 1.92 \text{ in}^2 \]
Hooked Bars (@ ends):

\[ d = \frac{-2m}{4 \times 0.05 \text{kips}} + \frac{d^2}{2} \]

\[ 50.5" = \sqrt{-2(454.5 \text{kft})(12'/100)\left(\frac{0.85}{\text{ksi}}\right)(12'\times 0.85')(12') - (12.5'')^2} \]

\[ 50.5" - 47.71" = 2.79" \]

Area = \( 0.85 \times 12 \times \left(\frac{0.85}{\text{ksi}}\right)(12')(0.03'') = 2.06 \text{ in}^2 \)

\( f_y = \frac{200 \text{ ksf}}{12' \times 0.85'} \)

\( \Rightarrow \text{Use 3-#8 Hooked Bars, } A_s = 2.37 \text{ in}^2 \)

\( d = 50.5" \text{ still assumes #8 bars} \)

Check #1:

\[ A_s = \frac{f_y}{200 \text{ ksi}} \times \left(\frac{0.85}{\text{ksi}}\right)(12')(0.05') \approx 3.49" \]

\( f = \frac{519.9 \text{ kft}}{454.5 \text{ kft}} > 1.11 \text{kips/ft} \)

Check #2:

\[ c = 9/8 = 3.49" / 0.85 = 4.11" \]

\[ \varepsilon = \frac{d-c}{c} \geq \varepsilon_t \]

\[ 0.003 \left(50.5" - 4.11"\right) \geq 0.004 \]

\[ 0.0034 > 0.004 \text{ tension controlled} \]

\[ f = 3.41 \text{ ft/kip} \times \frac{50.5'}{4.11'} = 2.0 \text{ ksi} \]

\[ \text{Length of 3-#8 Hooked Bars} = \text{greater of } \frac{2.79}{\text{ksi}} \times 866, 8' \]

\[ = 18.96" \]

\[ \text{Next: 12"} \]
Reference

Shear Reinforcement:

\[ V_c = \frac{f_{cc} b d}{2\sqrt{f_{cc} \psi}} \]
\[ = \frac{4000 \text{ psi} (12'') (50.5')}{71.65 \text{ k}} \]
\[ = 287.5 \text{ k} \]

\[ \psi_{V,h} = \psi (V_c + V_s) \]
\[ = \frac{10V_c b d}{(0.75)(10) \sqrt{4000 \text{ psi} (12'') (50.5')}} \]
\[ = 287.5 \text{ k} \]

K.4.6A
\[ V_{u,max} = 68.9 \text{ k} \]

\[ \Rightarrow V_u < V_c \quad \text{do not need shear reinforcement} \]
\[ \psi_{V,h} > V_u \quad \text{do not need to increase beam width} \]

\[ \Rightarrow \text{will add minimum shear reinforcment, need by code for constructability purposes} \]

Max Spacing
\[ \frac{d}{\psi} > 60 f_{cc} b d \]
\[ 68.9 \text{ k} > 60 \sqrt{4000 \text{ psi} (12'') (50.5')} \]
\[ 68.9 \text{ k} > 230 \text{ k} \quad \times \]

\[ \Rightarrow \text{use } S_{max} = d/2, \text{ not } d/4 \]

\[ S_{max} = \frac{d}{2} \]
\[ = 50.5' / 2 \]
\[ = 25.25' \]
\[ = 24' \]

Min. Shear Reinforcement:

\[ A_{v,min} = \text{greater of } \]
\[ = 0.75 f_{cc} b d / \psi \]
\[ = 0.0095 \text{ in}^2 / \text{ft} \]
\[ A_{v,min} = (0.01 \text{ in}^2 / \text{in}) (24'') \]
\[ = 0.21 \text{ in}^2 \]

\[ \Rightarrow \text{use } \#4 	ext{ stirrups @ 24'' o.c. throughout} \]
\[ A_v = 2(0.21 \text{ in}^2) \]
\[ = 0.42 \text{ in}^2 \]
Final Header Design (h=1):

3 - No. 8's hook end column support (90° sickle hook)

3 = 2.34"  
4 = 2.34"

No. 4 stirrups 2.5" o.c.

C = 2.94"  
E = 0.005

5"  
12"

$T$
Header Design - H2

Restrictions:
Height from 1st floor to top of window = 8'-2"
Depth of 1st floor girder on gridlines B and D (C2) = 20"
Max Depth = 30" - 20" = 10"
Max Width = width of 1st floor girder (C2) = 12"

Quick Calc: 12" x 18" header.

Load Combination: 1.2D + 1.0LL
DL = 69 psf (wall load) + (150 psf x 12/12) = 20 psf
LL = 20 psf

At = 1.5 m x (3.55 m) = 17.3 m² then: 17.3 m² / 2 = 8.65 m²

A = (6.9 psf x 0.5) + (150 psf x (20/12" x 12") x (12/12")) = 122.5 psf

w = 1.4 (20 psf x 1.0 x 1.0) = 373 psf

w = 1225.2 + 373 = 1598.5 psf = 1.6 klf

Mx = w x (1.6 klf x 1.4)² = 53.8 kft (max moment @ w span)

Flexure:
Mx = +4.0 x fy (h - h/2)²
Assume: h = 1'-2.5" = 15.5"
h/2 = 7.8" = 0.2 m²

A x = Mx = (53.8 x 12") / (15.5 x 60 ksf x 15.5 - 5.2) = 0.124 m²

Amin = greater of 200 lb/ft²
fy = 3.5 ksf

Diameter of reinforcing steel: 3/8" or 3.5 ksf

References:
A.\(A_{\text{min}} = \frac{2.00}{f_y} = \frac{2.00}{60.000} (12 \times 15.5) = 0.6 \text{ in}^2 = 0.926 \text{ m}^2\)

\(A_{\text{min}} = \frac{3.5 \sqrt{f_c}}{f_y} = \frac{3.5 \times 7000}{60.000} (12 \times 15.5) = 0.6 \text{ in}^2 = 0.926 \text{ m}^2\)

\(A_{\text{req}} = 0.926 \text{ in}^2 \Rightarrow \text{Ttry } 2 \text{ - No. L's, } A_f = 1.32 \text{ m}^2\)

Check:
\(a = \frac{f_{shf}}{0.85 f_c} = \frac{(1.32)(12)}{0.85(10.85)(9)(12)} = 1.94"\)
\(c = 9 - 1.94 = 2.28"\)
\(E = 0.85 = 2.86"\)

\(d = 18" - 1.5"\text{ elem. spacing} - \left(\frac{3}{8}\right)\text{ assured no.3 stumps}\)
\(\varepsilon = 0.002 = \frac{(d - c)}{c} = 0.007 = \frac{(15.5 - 2.28)}{2.28} = 0.113 > 0.005\) (tensile controlled) \(\Rightarrow f = 0.9\)

\(f_{\text{min}} = (0.9)(1.2)(10)(15.5 - 1.94/2) = 10.257 \text{ k} / \text{in} / 12" = 86.3 \text{ kip}\)

\(f_{\text{min}} = 86.3 \text{ kip} > M_y = 53.8 \text{ kip}\)

Note: Designed as simply supported - no moment at the ends. Add 2 - No. 6 hooks at ends. (7/8" std. hook)

Screws:
\(A_{\text{min}} \text{ is not required if: } 1) h \leq 24"\)
\(2) V_M < \phi 2 f_{\text{pc}} \text{ bond}\)
\(h = 18" \leq 24" \checkmark\)

\(V_M = \frac{W_L}{2} = \frac{1.6(15)(7)(15.5)}{2} = 13.12 \text{ kip}\)

\(\phi 2 f_{\text{pc}} \text{ bond} = (0.75)(2)4000 (12 \times 15.5) = 7.64 \text{ kip}\)
\[ v_c = 2 \sqrt{f_{c', \text{bw}d}} \]
\[ = 2 \sqrt{4000 \times (12 \times 15.5)} \]
\[ = 23.5 \text{ k} \]
\[ A V_n = 0.10 \sqrt{f_{c', \text{bw}d}} \]
\[ = 0.10 \times \sqrt{4000 \times (12 \times 15.5)} \]
\[ = 82.2 \text{ k} \]
\[ v_{u, \text{max}} = 13.12 \text{ k} \]

\[ \phi \frac{v_n}{v_u} \text{ do not need to increase, lam width} \]
\[ \text{max spacing} = \frac{v_u}{\phi} \times \frac{2}{0.6} \times f_{c', \text{bw}d} \]
\[ 13.12 \times \frac{1}{0.6} \times 4000 \times (12 \times 15.5) \]
\[ = 7.75 \]

\[ 17.5 \times 4 = 70 \text{ k} \Rightarrow \text{span} = \frac{d}{15.5} = \frac{2.015}{15.5} \approx 0.13 \]

\[ A V_{\text{min}} = 50 \text{ bw} \frac{f_y}{v_u} = \frac{(50 \times 12)}{60,000} = 0.01 \text{ in} \]

\[ A V_{\text{min}} = (0.01 \times 2) = 0.02 \text{ in}^2 \Rightarrow \text{No. 3 stirrups} \]

- use No. 3 stirrups @ 3' o.c.
- use 2 No. L's for flexural rein
- use 2 No. L hooks @ column support
Final Header design (4x2):

2 No 6's hooked 2 column support (90° steel hook)

C = 2.28"  
E = 0.005  
T = 1.94"

3 No 6's  
No 3 shank 8 @ 3" O.C.
Header Design - H2

Restrictions:
- Height from WL 2 to top of window = 3' - 2" = 36" - 24" = 12"
- Depth of 1" 2 beam (B3) on gridlines 12 = 12"
- Max. Depth = 36" - 12" = 24"
- Max. width = width of 1" 2 beam (B3) = 12"

Quick Calc.: 12" x 24" header

Load Combo: 1.2D + 1.6L
DL = 69 psf (wall load) + 1.5PSF(X12/12"
LL = 20 psf

AT = (4m^2)(2.55m) = 19.2m² => AT ≤ 18.58m²

F = 1.5 ≥ F = 1.4 => R1 = 1

C = (10.2R2 = (20 psf)(4.2) = 20 psf

w2 = 1.2 [(69 psf = 11) + (150 psf X (12/12" - 12/12") = 1105.2 plf

wl = 1.6 (20psf X 11.27") = 373 plf

wt = 1105.2 + 373 = 1478.2 plf = 1.5 klf

Mu = (1.5klf X 4m X 3.28 ft/m) / 8 = 2.25 kft (max moment b/w span)

Flexure:

Mu = φAfy (d - 2)

L = 7.82" = 0.64 ft

A_troy = (22.5 kft X 12") / (0.9 X 0.64 X 23.5 - 7.82) = 0.386 in²

As_min = greater of 200 fy / b/2d

3.3√φf_c / b w/ d
Window/Door Headers on Gridlines 1 & 3

\[ A_{\text{min}} = \frac{200}{40,000} \times (12 \times 22.5) = 0.74 \text{ in}^2 \]

Drives govern.

\[ A_{\text{min}} = \frac{3 \times \sqrt{4000}}{40,000} \times (12 \times 22.5) = 0.87 \text{ in}^2 \]

\[ A_{\text{req}} = 0.94 \text{ in}^2 \Rightarrow \text{Try } 2 - \text{ No. 1's} - A_{x} = 1.32 \text{ in}^2 \]

**Check:**

\[ a = \frac{A_{\text{req}}}{0.85 \times C} = \frac{(1.32 \times 12)}{0.85 	imes 4 \times 12} = 1.74'' \]

\[ C = \frac{a}{d} = \frac{1.74}{0.85} = 2.08'' \]

\[ d = 2.07 - 1.5 = 0.57'' \text{ clear spacing} \]

\[ E_{s} = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{2.07 - 1.7}{1.7} \right) = 0.003 \times 2.28 = 0.007 \frac{\text{kip}}{\text{in}} \text{ tension control} \]

\[ f_{\text{m}} = \frac{10.9 \times 1.5 \times 100 \times (2.4 - 1.7)}{2} = 1441.6 \text{ kip ft} = 134.8 \text{ kip ft} \]

\[ f_{\text{m}} = 134.8 \text{ kip ft} > M_{u} = 32.3 \text{ kip ft} \]

**Note:** Designed as simply supported - no moment at ends. Add 2 - No. 6 hooks at ends. (no stirrups)

**Steel:**

\[ A_{\text{min}} \text{ is greater of: } 1) \frac{0.75 \sqrt{f_{c}}}{f_y} \]

\[ 2) \frac{50 \text{ kip ft}}{f_y} \]

\[ A_{\text{min}} = \frac{0.75 \sqrt{4000}}{40,000} \times 12'' = 0.009 \text{ in}^2 \]

\[ A_{\text{min}} = \frac{50 \text{ kip ft}}{40,000} \times 12'' = 0.01 \text{ in}^2 \]
\[ V_c = 2 \sqrt{f_c \cdot l \cdot w \cdot d} \]
\[ f_{\text{pc}} = \frac{4000}{12 \cdot 24} = 13.4 \text{ k} \]

\[ \phi \cdot V_n = 0.75 \left( V_c + V_0 \right) = 0.75 \left( 13.4 + 7.84 \right) = 13.6 \text{ k} \]

\[ V_u = W_L = \frac{1.5 \times 4.0 \times 5.28}{2} = 7.84 \text{ k} \]

\[ V_c = 13.4 \text{ k} \geq V_u = 7.84 \text{ k} \Rightarrow \text{ shear reinforcement not required} \]

\[ \phi \cdot V_n = 13.6 \text{ k} \geq V_u \Rightarrow \text{ do not need toe mc. beam width} \]

*Note: Assume generous. \( V_{\text{reqd}} = 0.01 \text{ k}^2 \Rightarrow \text{ use No. 3 stirrups} \)

\[ V_c = 7.84 \text{ k} > \left( \frac{6 \times 4000}{12 \times 24} \right) = 7.84 \text{ k} \]

\[ d = 24 = 6'' \]

- Use No. 3 stirrups. 6'' o.c.
- Use 2 - No. 6's for flexural reinforcement.
- Use 2 - No. 6's @ ends w/ 90° standard hook.
Final Header Design (W2):

- 2 No. 8 hooks in column supports (10" stud hooks)
- E6 = 0.003
- C= 2.26"
- 3 No. 8's
- No. 3 stirrups 6" o.c.
worst case masonry wall designed for all cases:

\[ W_{\text{wall}} = 69 \text{ psf} \]
\[ D_L = 56 \text{ psf} \]
\[ L_L = 20 \text{ psf} \]

out of plane load (wind):
\[ P = 0.723 \text{ psf} \]

s. mortar partially grouted
\[ f_m = 2000 \text{ psf} \]
\[ M_b \text{ w} \text{ MDM} = 9^\circ \]
\[ 1.39 \text{ ft}^2 + 1.08 + 1 \text{ in. w} \]

\[ M_w = \frac{P \cdot L^2}{8} \left( 57.23 \text{ psf} \right) \left( 11^\circ \right)^2 = 869.6 \text{ ft}^3 \]

\[ P_u = (0.9 \text{ psf} \cdot 0.1 \text{ psf} \cdot 1.39 \text{ ft}) + 20 \text{ psf} \]
\[ = 174.25 \text{ psf} \]
\[ = 1.18 \text{ ft} \]

\[ 0.05 \text{ An } f_m = 0.05 \left( 12^\circ \times 12^\circ \right) \left( 2000 \text{ psi} \right) \]
\[ = 1400 \text{ psi} \]

\[ P_u \leq 0.05 \text{ An } f_m \]

\[ f_m = (1.3) \frac{144 \text{ psi}}{10} = 18.49 \text{ psi} \text{ (assume hollow blocs more conservative)} \]

\[ = (1.3) \left( 153.77 \text{ in}^3 \right) \left( 84 \text{ psi} \right) \]
\[ = 116 \text{ 991} \text{ psi} \]

\[ A_{\text{new}} = \mu_u + P_u \left( \frac{1}{12} \right) \]

\[ \phi \text{ ft}^2 (d - 9/2) \]

\[ = 80.65 \text{ ft}^2 \]

\[ = (12^\circ) (11.5^\circ) \]
\[ = \left( 200,000 \text{ psi} \right) (12.7^\circ - 11.5^\circ) \]
\[ = 0.024 \text{ in}^2 \]

\[ \Rightarrow M_{1} = 46 + @ A_S = 0.2 \text{ in}^2 \]

\[ a = \frac{A_{\text{new}} + d/3 = 12^\circ/3 = 44^\circ}{(11 \times 12^\circ)} - 1.5^\circ - 1.5^\circ - 0.5^\circ \]

\[ = (0.9) (60,000 \text{ psi}) (12.7^\circ - 11.5^\circ) \]
\[ = 0.024 \text{ in}^2 \]
**Reference**

\[ Mn = \frac{As f_y (d - \frac{d}{2})}{0.214 (60 \text{ksi}) (132^\circ - 0.926^\circ)} \]

\[ = 1573.4 \text{ kIN} \]

\[ = 131.58 \text{ kFT} \]

\[ = 121,537 \text{ #f} \]

\[ f_pMn = (0.9) Mn \]

\[ = 118,883.4 \text{ #f} \]

\[ Mn > 1.3 Mcr \ Check \]

**Lap Length**

\[ L_d = \frac{0.13 d_b f_y}{k \sqrt{f_m}} \]

\[ = \frac{0.13 (0.5 \text{ in})^2 (60,000 \text{ psi}) (1.05)}{(1.5 \text{ in}) \sqrt{2000 \text{ psi}}} \]

\[ = 29'' \]

\[ \Rightarrow \text{use 1-#4 longitudinal bar w/ ed: 29''} \]

**Shear**

\[ Vu_{max} = \frac{Wt}{2} = \frac{(116,17 \text{ lb}) (16.4'')}{2} = 952.6 \text{ ft} \]

\[ Mu = (0.65 \text{ conf}) + (1905 \#)(5.5) = 11.9 > 1.0 \]

\[ Vu = 952.6 \text{ ft} \]

\[ Vn_{max} = 4.06 \sqrt{f_m} g_y \]

\[ = 4 (7.025') \sqrt{2000 \text{ psi}} (0.75 \times 120.7'') \]

\[ = 1310 \text{ ft} \]

\[ f_vn = (0.8) (1310) \text{ ft} \]

\[ = 105328 \text{ ft} \]

\[ f_vn > Vu \Rightarrow \text{don't need shear rein. but add minimum} \]

\[ A_m = (0.007 b d v) = 0.007 (7.625') (25.7'') = 0.69 \text{ in}^2 \Rightarrow \text{try 1-#8} \text{ A} = 0.72 \in \]

\[ Vu = 0.5 (f_v) g_y d v \Rightarrow 952.6 \text{ ft} = 0.5 (0.72 \text{ in}^2) 60,000 \text{ psi} (120.7'') \Rightarrow s = \text{large, use 48'' O.C. to be conservative} \]

\[ \Rightarrow \text{use 1-#8 shear rein. @ 48'' O.C.} \]
Column Design - C-1

Lateral Loads: 1.2R + 1.0E
D.L. = 0.4 psf
L.L. = 20 psf

\[ A_T = (4\text{m} \times 2.5\text{m}) = 10\text{ m}^2 \Rightarrow A_T = 10.58 \text{ m}^2 \Rightarrow R_1 = 1 \]
\[ F_i = 1.5 \quad F_e = 4 \Rightarrow R_2 = 1 \]
\[ L_0 = L_0 \quad R_2 = 1 \Rightarrow (20)(10) = 200 \text{ psf} \]

Axial Design:
\[ P_{ax} = [(11.7)(4.1psf)] + [(10)(20psf)](4\text{m} \times 2.5\text{m} \times 3.28^2 \text{ft/k}}^2 = 11.32 \text{k} \]

Axial Diagram:

Flexural Design:
1. Face of other supports (B3):
\[ M_3 = \frac{w d^2}{12} \left( 201.7 \text{ kfp} \right) \left( 4\text{ m} \times 3.28 \text{ ft/m} \right)^2 = 31.51 \text{ kft} \]

2. Exterior face of the first interior support (B3):
\[ M_4 = \frac{w d^2}{10} \left( 201.7 \text{ kfp} \right) \left( 4\text{ m} \times 3.28 \text{ ft/m} \right)^2 = 39.31 \text{ kft} \]

3. Face of other supports (B4):
\[ M_5 = \frac{w d^2}{10} \left( 2.88 \text{ kfp} \right) \left( 4\text{ m} \times 3.28 \text{ ft/m} \right)^2 = 42.3 \text{ kft} \]
Example Exterior Force of the First Interior Support (E9)

\[ M_1 = \frac{W_a r^2}{12} = \frac{2.804 \times 4\times 2.8}{12} = 48.3 \text{ kft} \]

\[ M_{\text{proof}} = |M_1 - M_2| = |1.43.7 - 10.3| = 4.9 \text{ kft} \]

\[ M_{\text{min.}} = |M_2 - M_1| = |131.5 - 34.71| = 3.15 \text{ kft} \]

Mom. Diag:

\[ 4.9 \text{ kft} \]
\[ 3.15 \text{ kft} \]

\[ \text{Slenderness Check:} \]

12" CIP Column

Root > 2'-6" Fir

\[ K = 1.0 \]

1-8' 0", Ly = 7'-0" (clear height)

\[ r = 0.36; L = 0.3(12) = 3.6 \]

\[ \frac{K L}{r} < 3.4 + 12 \left( \frac{M_1}{M_2} \right) \]

\[ \frac{3.4}{3.6} \leq 3.4 + 12 \left( \frac{3.15}{4.4} \right) \]

\[ 2.3 < 2.5 \checkmark \text{Brace against side sway} \]

2'-6" Fir to Ground

\[ K = 1.0 \]

\[ l = 12'; h_4 = 11'-6" \] (clear height)

\[ r = 0.36; h = 3.6" \]

\[ \frac{K L}{r} < 3.4 + 12 \left( \frac{M_1}{M_2} \right) \]

\[ \frac{3.4}{3.6} \leq 3.4 + 12 \left( \frac{3.15}{4.4} \right) \]

\[ 38.33 < 42.6 \checkmark \text{Brace against side sway} \]
Final Information:

File Name: untitled.col
Object: Column
Code: ACI 318-14
Run Option: Investigation
Run Axis: X-axis

Material Properties:

Concrete: Standard
f'c = 4 ksi
Ec = 1605 ksi
Es = 3.4 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 50 ksi
Rs = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
Rectangular: Width = 12 in
Depth = 12 in

Gross section area, Ag = 144 in^2
Iy = 1728 in^4
rx = 3.4641 in
Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size | Diam (in) | Area (in^2) | Size | Diam (in) | Area (in^2) | Size | Diam (in) | Area (in^2) | Size | Diam (in) | Area (in^2)
--- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | ---
#3 | 0.39 | 0.11 | #4 | 0.50 | 0.20 | #5 | 0.63 | 0.31
#6 | 0.75 | 0.44 | #7 | 0.88 | 0.60 | #8 | 1.00 | 0.79
#9 | 1.13 | 1.00 | #10 | 1.27 | 1.27 | #11 | 1.41 | 1.56
14 | 1.69 | 2.25 | 16 | 2.25 | 3.08 | 4.00

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
\[ \alpha = 0.8, \quad \phi(b) = 0.9, \quad \phi(c) = 0.65 \]

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 0.88 in^2 at rho = 0.61% (Note: rho < 1.0%)
Minimum clear spacing = 3.56 in

#3 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

<table>
<thead>
<tr>
<th>No.</th>
<th>Pu</th>
<th>Max k-ft</th>
<th>PhiMax</th>
<th>PhiMin/Phi Na depth</th>
<th>Dl depth</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.32</td>
<td>4.39</td>
<td>25.02</td>
<td>5.699</td>
<td>1.57</td>
<td>9.94</td>
<td>0.01595</td>
</tr>
<tr>
<td>2</td>
<td>19.18</td>
<td>4.39</td>
<td>27.66</td>
<td>6.345</td>
<td>1.73</td>
<td>9.94</td>
<td>0.01426</td>
</tr>
</tbody>
</table>

*** End of output ***
Vertical Point Check:

\[ A_s = 8 - \text{No. 4} = 16 \text{ in}^2 \]

\[ A_{sm} = 0.01 \times 12^2 = 1.44 \text{ in}^2 < A_s, \text{ AS } \checkmark \]

\[ A_{sm} = 0.08 \times 12^2 = 11.52 \text{ in}^2 > A_s, \text{ AS } \checkmark \]

**UCE B - NO. 4 LONGITUDINAL BARS IN 12" 50 COLUMN**

**Column Transmission Requirement**

According to ACI 318

- \( f'_c \text{column} = 4.1 \text{ ksi} \)
- \( f'_c \text{floor} = 4.1 \text{ ksi} \)

Column transmission requirement doesn't apply because concrete strengths are equivalent between floors and columns.

Column size and spacing:

ACI 318 §25.7.2

Bar size is 1/2 of 1) No. 3 enclosing No. 10 bars
2) No. 4 enclosing No. 11 bars

**Use No. 3 TIES (GRAVITY)**

Spacing:

ACI 318 §18.7.5

\( d_{hoop} = 12" - 1.5" - 3/8" - 3/16" = 9.875" \)

\( \frac{h}{d_{hoop}} = \frac{1}{3} \times 12" \times 2" = 2.4" \)

\( h = 18" \)

Load is greater of:

1. \( d_{hoop} = 12" - 1.5" - 3/8" - 3/16" = 9.875" \)
2. \( \frac{h}{d_{hoop}} = \frac{1}{3} \times 12" \times 2" = 2.4" \)
3. \( h = 18" \)

\[ T \text{ governs} \]

**Gray is least of:**

1. \( \frac{1}{4} \text{ min. dim.} = \frac{1}{4} \times 12" = 3" \)
2. \( b \text{ (diam of smallest long)} \leq \frac{12"}{9/16} = 1" \)

\[ T \text{ governs} \]
Column Group #1 (C1)

5.1.1.1.2.1 (d) (long) = (6 * (1/8)) = 3" 
\[ \text{governs} \]

Clear spacing = \( \frac{4}{3} \text{d}_{\text{agg}} \) = 4/3 = 1.33" 

Max. & spacing is least of: 
1. 1/6" dia. = 16 (4/8) = 8" \[ \text{governs} \]
2. 1/8" dia. = 16 (3/8) = 10" 

Earthquake Requirements:

Assume elastic capacity of gravity framing is executed during an earthquake and "core non-linear behavior" is required.

Amount of Transverse Required:

\( P_u \leq 0.3 \gamma g f_e \cdot f'_e = 4 \text{ksi} \cdot 17.15K \)

19.18 \leq 0.2 \gamma g (0.672 \cdot 12 \cdot 2) \cdot 4 \) ksi

19.18 K \leq 172.8 K \checkmark 

Transverse resist is greater of:\n
a) \( 0.2 \left( \frac{f'_e}{f_y} \right) = 0.2 \left( \frac{32}{40} \right) = 0.015 \text{ in}^2 \) 
\[ \text{governs} \]

b) \( 0.09 \left( \frac{f'_e}{f_y} \right) = 0.09 \left( \frac{4}{40} \right) = 0.004 \text{ in}^2 \)

\( A_{ch} = 0.015 \text{ in}^2 \)

\( A_{ch} = 0.015 \times \frac{3}{2} (12" - 1.5" - 3/8" - 4/16") = 0.444375 \text{ in}^2 \)

\( A_{ch} = (3 \times 4/3) = 2 \left( 0.015 \text{ in}^2 \right) = 0.030 \text{ in}^2 \)

\( A_{ch} = 0.015 \cdot 2 \text{ in}^2 = 0.030 \text{ in}^2 \geq A_{ch} \)

\( \sqrt{A_{ch}} \text{ area} \)

USE NO. 4 TIES @ 2" O.C. (SEISMIC)
5.18.7.6 does not apply b/c not supporting resolutions from discontinued stiffeners, such as walls.

5.18.7.5.7 does not apply b/c clear spacing = 1.5' < 4'

Auto. Forces: run spec. column w/ φ = 1.0 \& fy = 1.35fy = 35 KSI.

\[ M_{\text{max}} = 81 \text{ kft} \text{ at e. critical point} \]

\[ V_c = \frac{2M_{\text{max}}}{h} \text{ e. full height} \]

\[ = \frac{2(81)}{20} \]

\[ = 8.1 \text{ k} \]

\[ V_c = \frac{2M_{\text{max}}}{h} \text{ e. mid height} \]

\[ = \frac{2(81)}{10} \]

\[ = 16.2 \text{ k} \]

\[ φV_n = φV_c + φV_e \]

\[ = 0 + φV_c \]

\[ = 0.75 \left( \frac{12\text{ in.}}{2\text{ in.}} \right) \]

\[ = 0.75 \left( 3 \times 2 \times 1.4 \times (12' - 1.5' - 4/8' \times (1/2)) \right) \]

\[ = 92.25 \text{ k} \]

\[ φV_n = 92.25 \text{ k} > V_c = 16.2 \text{ k} \text{ OK} \]
1. **Loads:**
   - **LTD:** DL (61 psf) at both levels w/o Level 2 in between.
   - **LL:** (20 psf) x (2m) x (2.5m) = 5 m². Worsen case scenario.
   - **R1:** 10 ft, w/o At = 18.5 m².
   - **R2:** 1.10
   - **LR:** (20 psf) x (1.0) x (1.0) = 20 psf.
   - **DL:** (61 psf) x (1.2) = 73.2 psf.
   - **PL:** (61 psf) x (1.2) = 73.2 psf.
   - **LL:** (20 psf) x (1.0) = 20 psf.
   - No LL on 2nd flr on column b/c no diaphragm.

2. **Design Axial Forces:**
   - Pl = (DL + LL) x At
   - With At = 53.8 ft². Sm²
   - **Pr1:** (73.2 psf + 32 psf) (53.8 ft²)
   - = 5,600 k
   - **Pr2:** 9.0 k
   - **P3:** 8,600 k
   - (73.2 psf) (53.8 ft²)
   - = 9.0 k

3. **Design Bending Forces:**
   - i) Int. Face of Ext Support (Bir):
     - \( \overline{W} L^2 \)
     - \( \frac{110}{41} \) psf x (1.2) + 32 psf x (25 m x 3.28 ft/m) / 2 x (4 m x 3.28 ft/m)
     - = 35.82 k ft
   - ii) Int. Face of Ext Support (Girder):
     - \( \overline{W} L^2 \)
     - \( \frac{110}{41} \) psf x (1.2) + 32 psf x (6 m x 3.28 ft/m) x (5 m x 3.28 ft/m)²
     - = 105.2 k ft
Column Group #2 (C2)

3) Ext. face of ext. support (Brin):
   \[ \text{W} = \left( \frac{41 \text{ psf} \times 1.22}{6} \right) \left( 12.5 \text{ ft} \times 3.28 \text{ ft/m} \right) \left( 4 \text{ m} \times 3.28 \text{ ft/m} \right) \]
   \[ = 21.7 \text{ kFt} \]

4) Int. face of ext. support (Girder):
   \[ \text{W} = \left( \frac{50 \text{ psf} \times 1.22}{6} \right) \left( 6 \text{ m} \times 3.28 \text{ ft/m} \right) \left( 5 \text{ m} \times 3.28 \text{ ft/m} \right) \]
   \[ = 29.6 \text{ kFt} \]

(4) Check Stenderness:

- Match m/girder w/ Bu. 2.5 ft
- Roof \rightarrow 2nd Floor
  - K = 1
  - Mu = 7.0 ft (Cur HT)
  - R = 0.3 h
  - 0.3 (12') = 3.6 ft

Braced against side way:

\[ k_{uy} \leq 34 - 12 \left( \frac{\text{My/m}}{\text{My/m}} \right) \]
\[ = 34 - 12 \left( \frac{29.6 \text{ kFt}}{43.8 \text{ kFt}} \right) \]
\[ = 23.3 \leq 42.1 \checkmark \]

2nd Floor \rightarrow FND

\[ k_{uy} \leq 34 - 12 \left( \frac{\text{My/m}}{\text{My/m}} \right) \]
\[ = 34 - 12 \left( \frac{21.7 \text{ kFt}}{35.8 \text{ kFt}} \right) \]
\[ = 23.3 \leq 41.3 \checkmark \]

2.5 ft
12 x 12 in

Code: ACI 318-14
Units: English
Run axis: Biaxial
Run option: Investigation
Saidness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/10/17
Time: 17:45:53

File: u:senior project - jhc2.col
Project:
Column:
f_c = 4 ksi  fy = 60 ksi
E_c = 3605 ksi  Es = 29000 ksi
u = 0.003 in/in  e_yt = 0.00206897 in/in
Beta1 = 0.85
Confinement: Tied

Engineer:
Ag = 144 in^2  8 #6 bars
As = 3.52 in^2  rho = 2.44%
Xo = 0.00 in  lx = 1728 in^4
Yo = 0.00 in  ly = 1728 in^4
Min clear spacing = 3.00 in  Clear cover = 1.88 in
General Information:

File Name: u:senior project - jil\c2.col

Project:

Code: ACI 318-14

Run Option: Investigation
Run Axis: Biaxial

Material Properties:

Concrete: Standard
f'c = 4 ksi
Ec = 1605 ksi
fc = 3.4 ksi
Eps_u = 0.003 in/in
Beta1 = 0.89

Section:

Rectangular: Width = 12 in
Depth = 12 in

Gross section area, Ag = 144 in^2
Ix = 1728 in^4
rx = 3.4641 in
Xo = 0 in

Reinforcement:

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size</th>
<th>Dia (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.38</td>
<td>0.11</td>
</tr>
<tr>
<td>#6</td>
<td>0.75</td>
<td>0.44</td>
</tr>
<tr>
<td>#9</td>
<td>1.13</td>
<td>1.00</td>
</tr>
<tr>
<td>#14</td>
<td>1.69</td>
<td>2.25</td>
</tr>
</tbody>
</table>

Size | Dia (in) | Area (in^2) |
-----|----------|-------------|
#4   | 0.50     | 0.20        |
#7   | 0.88     | 0.60        |
#10  | 1.27     | 1.27        |
#18  | 2.26     | 4.00        |

Size | Dia (in) | Area (in^2) |
-----|----------|-------------|
#5   | 0.63     | 0.31        |
#8   | 1.00     | 0.79        |
#11  | 1.41     | 1.56        |

Confinement: Tied; #3 ties w/ #10 bars, #4 w/ larger bars.
in(a) = 0.9, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 3.52 in^2 at rho = 2.44%
Minimum clear spacing = 3.00 in

#6 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

<table>
<thead>
<tr>
<th>No.</th>
<th>Pu kip</th>
<th>Mux k-ft</th>
<th>Nuy k-ft</th>
<th>PhiMax k-ft</th>
<th>PhiMin/Mu NA Depth in</th>
<th>D_T Depth in</th>
<th>eps_u</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.60</td>
<td>29.60</td>
<td>21.70</td>
<td>46.77</td>
<td>34.25</td>
<td>1.580</td>
<td>6.29</td>
<td>13.57</td>
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<tr>
<td>2</td>
<td>5.65</td>
<td>43.80</td>
<td>35.82</td>
<td>44.65</td>
<td>36.52</td>
<td>1.019</td>
<td>6.30</td>
<td>13.69</td>
</tr>
</tbody>
</table>

*** End of output ***
Use: 6 #6 longitudinal bars in 12" sq column

Check vertical reinforcement:

\[ A_s = 6 \times (0.44 \text{ in}^2) = 3.52 \text{ in}^2 \]

\[ \min 0.01A_g = 0.01 \times (12\text{")\(12\text{")}) = 1.44 \text{ in}^2 \]

\[ A_s \geq 0.01A_g \checkmark \]

\[ \max 0.06A_g = 0.06 \times (12\text{")\(12\text{")}) = 8.64 \text{ in}^2 \]

\[ A_s \leq 0.06A_g \checkmark \]

7. Column transmission beam

\[ f = 0.3 \]

Will not apply b/c column + floor concrete strength is not same

8. Tie spacing/size:

Bar size:
- least of 1 #3 enclosing #10 bars or less
- #4 enclosing #11 bars or larger

\[ \Rightarrow \text{use } [3 \text{ ties} < \text{gravity} \]

Bar spacing:
- ENDO 2nd level
- \[ d_o = \text{greater of } \frac{d_{oi}}{12"} \]

\[ \text{least of } \frac{v}{4} = \frac{12\text{")}}{4} = 3\text{"} \text{ O.C. } \leq \text{govern} \]

\[ d_o = 4 + \left( \frac{14 - b_v}{8} \right) \leq \text{Min. } 4" \]

8.5.5

\[ S_1 = \text{least of } \left\{ \begin{array}{l} (d_{ii} \text{ long bar}) = 4.5" \text{ O.C. } \leq \text{govern} \text{ round to \(4"} \end{array} \right. \]
Mission Twenty-Five 35
Community Center-Dominican Republic
Mindy Trieu and Jocelyn Lu

Calculation Topic
Column group #2 (C2)

207 level root:

<table>
<thead>
<tr>
<th>Reference</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.7.5.1</td>
<td>4.5 - 8&quot; O.C.</td>
</tr>
<tr>
<td>18.5.3</td>
<td>4.5 - 8&quot; O.C.</td>
</tr>
</tbody>
</table>

5.60: Least of:

- 4.5 - 8" O.C. 
- (a) \( \frac{4}{3} \) (diameter) < 4.5" O.C. 
- 4" O.C. 

Spacing real:
CLR spacing = 4/3 dagg
+ 1/8"

Max spacing least of:

- 12"
- 18" (diag) < 18"

4. Truss reinforcement for DRM:
- Assume elastic capacity of gravity framing is exceeded during an earthquake; some nonlinear behavior is expected.

5. 18.7.5.4 0.3A_yf = (0.3)(12") (12") (4 ksi) = 172.8 k

Pu = 9.0 k < 0.3A_yf = 172.8 k
FC: 4 ksi < 16 ksi

Asn/sbo: greater of:

- \( 0.3 \left( \frac{A_y}{A_{so}} - 1 \right) \left( \frac{f_y}{f_{yf}} \right) \)
- 0.09 \( \left( \frac{f_y}{f_{yf}} \right) \)
- 0.09 \( \left( \frac{4}{60} \right) \)
- 0.006

Asn = 0.015556

Asn = (0.015556)(3") (12" - 15") - \( \frac{3}{4} \) " - \( \frac{1}{8} \) " = 0.138 in²

Arms: hup tie + 1 cross tie
3 (2 in²) = 0.83 in² < 0.96 in²
10. SMP checks:

§10.7.5.6 doesn't apply. Blc column isn't supporting discontinuous members, such as walls.

§10.7.5.7 doesn't apply. Blc cover = 1.5'' x 2.4''

11. Shear strength:

re-ran sp-column w/ $f_{lr} = 1.0$k. $f_{y} = 1.25k$. $f_{k} = 75k$.  

$M_{max} = balance at 107 kft$

$V_{e} = 2M_{e}r_{e}$ at full ht

$= \frac{2(107 kft)}{20'-0''} = 10.7 k$  

$V_{e} = 2M_{e}r_{e}$ at mid ht

$= \frac{2(107 kft)}{10'-0''} = 21.4 k$

$\phi V_{n} = \phi V_{c} + \phi V_{s}$

$\phi = 0.75$  

$= (0.75) \left( \frac{AV_{e+1}}{S} \right)$

$= 0.75 \left( \frac{3(0.2 in^2) (60 ksi) (12'' - 1.5'' - 0.8'' (1/2))}{3''} \right)$

$= 92.25 k$

$\phi V_{n} > V_{e}$
1. Loads:

- DL = 10.1 kips
- LL = 20 kips

\[ A_T = \frac{5 \text{ m}}{3 \text{ m}} = 1.667 \text{ m}^2 \]

- k = 1.2
- R1 = 1.0
- R2 = 1.0

\[ LR = 20 \text{ kips} (1.0)(1.0) = 20 \text{ kips} \]

\[ PLR = 0.1 \text{ kips} (1.2) = 1.2 \text{ kips} \]

\[ LLr = 20 \text{ kips} (1.0) = 20 \text{ kips} \]

2. Design Axial Forces:

- Ht at top:
  \[ = 9.5 + 1.2 \times 1.10 = 10.73 \text{ ft} \]

- \[ P_r = (73.2 \text{ kips} + 32 \text{ kips}) (1.667 \text{ m}^2) (0.25 \text{ ft/m}) \]

3. Design Bending Forces:

\[ I = 0.5 \text{ ft} \]

1) Ext. Face of first int. support (girder):

\[ \text{w} = \frac{(50 \text{ kips})(1.2) + 20 \text{ kips}(1.0)}{3 \text{ m} \times 3.28 \text{ ft/m}} \]

\[ = 26.25 \text{ kips} \]

2) Same as 1:

\[ = 26.25 \text{ kips} \]

\[ M_x = M_1 + M_2 = 52.5 \text{ kips ft} \]
File Name: untitled.col
Project: ACI 318-14
Run Option: Investigation
Run Axis: X-axis

Material Properties:

Concrete: Standard
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Eps_u = 0.003 in/in
Beta_l = 0.85

Steel: Standard
f_y = 60 ksi
E_s = 29000 ksi
Eps_yt = 0.002066997 in/in

Section:

Rectangular: Width = 12 in
Depth = 12 in
Gross section area, Ag = 144 in^2
I_y = 1728 in^4
I_y = 3.4641 in
x_o = 0 in

Reinforcement:

Bar Set: ASTM A615
Size Diam (in) Area (in^2)
# 3 0.38 1.11
# 6 0.75 0.44
# 9 1.33 1.00
# 14 1.69 2.25

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
\( \phi_a = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 2.48 in^2 at \( \rho_o = 1.72\% \)
Minimum clear spacing = 3.19 in

8 #5 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

<table>
<thead>
<tr>
<th>No.</th>
<th>P</th>
<th>Mu</th>
<th>PhiMax</th>
<th>PhiMax/Mu</th>
<th>Mu DA</th>
<th>depth</th>
<th>depth</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.98</td>
<td>52.50</td>
<td>55.21</td>
<td>1.052</td>
<td>2.80</td>
<td>9.81</td>
<td>0.00750</td>
<td>0.900</td>
<td></td>
</tr>
</tbody>
</table>

*** End of output ***
4. Check slenderness:

12" x 12" SQ COL

\[ \frac{12}{3.60} = 3.33 > 3 \]

6.25

\[ \text{column braced against sideway} \]

\[ \begin{align*}
K_m & = 34 + 12 \\
K & = 1 \\
\lambda & = 10.73 \\
A & = 0.31 \\
0.3 & = 3.60 \\
3.60 & = 40 \\
\end{align*} \]

5. Column:

\[ \text{check vertical reenforcement:} \]

\[ \begin{align*}
\text{use} & \quad 6 \# 5 \\
\text{long. bars in 12" sq. column} \\
\end{align*} \]

18.74

\[ \begin{align*}
A_s & = 8 (0.31 \text{in}^2) \\
& = 2.48 \text{in}^2 \\

\min & = 0.01 A_g (0.01) (12") (12") \\
& = 1.44 \text{in}^2 \\
A_s & > 0.01 A_g \\

\max & = 0.010 A_g (0.01) (12") (12") \\
& = 0.64 \text{in}^2 \\
A_s & < 0.010 A_g \\
\end{align*} \]

7. Column trans mission reac: 

\[ \text{floor/roof system: steel} \quad w / f_y = 60 \text{ksi} \]

\[ \text{column system: concrete} \quad w / f_c = 4 \text{ksi} \]

\[ f_c \neq 1.4 (f_y) \]

Therefore, section will not apply
B. Tie spacing (size)

Bar size:
- least of { #3 enclosing #10 bars
  #4 for #11 bars or larger

  ⇒ use #3 ties (gravity)

Bar spacing:

- Roof ⇒ FND
  \[ l_{o} = \text{greater of} \left\{ \frac{d_{oc}}{2} = 12'' \right\} \]
  \[ \frac{d_{in}}{2} = 21.16'' \leq \text{governs, round to 22''} \]

- \[ l_{so} = \text{least of} \left\{ \begin{array}{c}
  14b' := 3'' \text{ o.c.} \\
  6(d_{in}) = 3.75''
\end{array} \right\} \left\{ \begin{array}{c}
  l_{o} + 14h_{x} = \text{min 4''}
\end{array} \right\} \]

- \[ s = \text{least of} \left\{ \begin{array}{c}
  6(d_{in}) = 3.75'' \leq \text{govern, round to 3''}
\end{array} \right\} \]

25.7.2

- Spacing Rebar

- CK spacing: 4/3 dag
  4/3''

- max spacing: least of
  \[ \begin{array}{c}
  6(d_{in}) = 10'' \\
  18(d_{in}) = 18''
\end{array} \leq \text{govern} \]

- Traverse reinforcement for SMF
  - Assume elastic capacity of gravity framing is exceeded during an earthquake:ismic nonlinear behavior is new

- \[ 0.3f'c = 0.3(12'')(12'')(4ksi) = 172.8K \]

- \[ Pu = 0.9f'c < 0.3f'c = 172.8K \]

- \[ f'c = 4ksi < 10ksi \]

- \[ \frac{A_{s}}{A_{n}} \text{ greater of } \left\{ \begin{array}{c}
  a) \frac{0.3(A_{s} - 1)}{f'c} \\
  b) 0.09 \left( \frac{f'c}{f_{y}} \right)
\end{array} \right\} \]

- \[ 0.2 \left( \frac{(12'')^2 - 1}{100} \right) \left( \frac{4}{100} \right) = 0.09 \left( \frac{4}{100} \right) \]
COLUMN group #3 (C3)

Try #4

\[ V_b = 0.015556 \]
\[ A_{bh} = (0.015556)(3)(12" - 1.5" - 3/8" - 5/8") \]
\[ = 0.443 \text{ in}^2 \]

Anes check: \( \sum (F_\text{int}) = 0 \)

\[ F_\text{int} = F_\text{L} + F_\text{C} \]
\[ = 2 \text{ (2 in.)} \]
\[ = 4 \text{ in.} \]

\[ \Rightarrow \text{use #4 nec @ 3" O.C. (Seismic)} \]

10. SMIF checks

\[ f_{18.7.5.6} \text{ doesn't apply w/ column isn't supporting discontinuous member such as walls} \]
\[ f_{18.7.5.7} \text{ doesn't apply w/ column cover is } 15" < 2" \]

11. Shear strength:

\[ 18.7.6 \]

re-ran w/ sp column \( V = 1.0 \cdot f_y \cdot 1.25 \cdot f_y = 79 \text{ ksi} \)

\[ M_{\text{max}} \cdot \text{B.E. pr. = 98 kft} \]

\[ V_e = 2M_{\text{pr}} \cdot \text{ @ full h} \]
\[ V_e = 2M_{\text{pr}} \cdot \text{ @ mid h} \]
\[ M_{\text{pr}} = \frac{2(98k \text{ ft})}{5.37} \]
\[ = 34.7 \text{ k} \]

\[ \phi V_n = \phi V_e + \phi V_s \]
\[ = 0 + \phi V_s \]
\[ = 0.75 \left( \frac{A_{bh} f_y}{f_y} \right) \]
\[ = 0.75 \left( \frac{3(0.2 \text{ in}^2)(60 \text{ ksi})(12" - 1.5" - 3/8" - 5/8")}{8"} \right) \]
\[ = 92.65 \text{ k} \]

\[ \phi V_n > V_e \]
12 x 12 in

Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Strengtheness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/10/17
Time: 20:23:41

STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-A-2356E-23197

File: U:\Senior Project - J\C3.col
Project:
Column:
f'c = 4 ksi
Ec = 3605 ksi
\nu = 0.003 in/in
beta = 0.85

fy = 75 ksi
Es = 29000 ksi
e_{yt} = 0.00258621 in/in

Ag = 144 in^2
As = 2.48 in^2
Xo = 0.00 in
Yo = 0.00 in
Min clear spacing = 3.19 in

Engineer:
8 #5 bars
r = 1.72%
lx = 1728 in^4
ly = 1728 in^4
Clear cover = 1.88 in
Column Design - C.4

Load Combo: 1.2D + 1.0L_L
DL = 64 kips
LL = 20 kips

\[ A = (5.5 \times 1.6) = 8.8 \text{ m}^2 \quad \Rightarrow \quad Area = 18.58 \text{ m}^2 \quad \beta_1 = 1 \]

\[ F = 1.5 \Rightarrow F = 4 \quad \beta_2 = 1 \]

\[ L_r = L_0 \beta_2 = (2 \times 4) = 8 \text{ kips} \]

Axial Design:

\[ M_p = \left[ (1.2 \times L_1 \text{kip}) + (1.0 \times 2.5 \text{kip}) \right] \left( 5.5 \times 1.5 \times 3.28 \text{ ft}^3 \right) = 7.34 \text{ k} \]

\[ M_{Ed} = 9.34 \text{ k} + \left( 1.2 \times (1.5 \times 5.5 \times 3.28 \text{ ft}^3) \right) = 15.83 \text{ k} \]

Axial Diagram:

\[ 9.34 \text{ k} \]

\[ 15.83 \text{ k} \]

\[ 12^\prime \]

Flexural Design:

1) Interior face of exterior support

\[ M_1 = \frac{wLdL^2}{12} = \frac{[(1.2 \times 41 \text{ kips}) + (1.0 \times 20 \text{ kips})] \times (3.0 \times 3.28 \text{ ft}^3)}{12} \times (9.84 \text{ ft})^2 = 4.84 \text{ kft} \]

2) Interior face of exterior support

\[ M_2 = \frac{wLdL^2}{12} = \frac{[(1.2 \times 51 \text{ kips}) + (1.0 \times 20 \text{ kips})] \times (5.0 \times 3.28 \text{ ft}^3)}{12} \times (10.4 \text{ ft})^2 = 27.2 \text{ kft} \]
**Column Group # 4 - C4**

**M_3**) End Span

\[ M_3 = \frac{w \cdot a \cdot L^2}{14} \]

\[ = \left[ 1.2 \times 11 \text{ pcf} \right] \left[ 2 \times 3.2 \text{ ft} \right] \left[ 6.5 \text{ ft} \right]^2 \] \frac{1}{14} \]

\[ = 0.911 \text{ kft} \]

**M_4**) Interior free of exterior support

\[ M_4 = \frac{w \cdot a \cdot L^2}{16} \]

\[ = \frac{11.2 \times 11 \text{ pcf} \left( 2 \times 3.2 \text{ ft} \right)}{16} \left( 1.8 \text{ ft} \right)^2 \]

\[ = 2.92 \text{ kft} \]

**M_5**) Interior free of exterior support

\[ M_5 = \frac{w \cdot a \cdot L^2}{16} \]

\[ = \frac{11.2 \times 11 \text{ pcf} \left( 5.5 \times 3.2 \text{ ft} \right)}{16} \left( 5.5 \times 2.25 \right)^2 \]

\[ = 18.5 \text{ kft} \]

**M_{free} = |M_2 - M_2 - M_3| = \left| 10.911 - 4.97 - 27.2 \right| = 31.3 \text{ kft} \]

**M_{free} = |M_4 - M_5| = \left| 2.92 - 18.5 \right| = 15.57 \text{ kft} \]

---

**ACI 318 Slenderness check:**

\[ \frac{L_0}{r} \]

\[ L_0 = 10 \text{ ft} \]

\[ r = 0.3h = 0.3(12) = 3.6 \text{ in} \]

\[ k = \frac{2.4 + 12}{3.6} \]

\[ k \]
\[
\frac{1.24 \times 12.2\%}{3.6} = 3.4 + 12 \left( \frac{15.57}{31.3} \right)
\]

23.3 \leq 27.97 \quad \text{Bolted against side sway}

2^{nd} \text{ FIC to ground}

k = 1.0

\Delta h = 11' - 6" \quad \text{(clear height)}

e = 3.6"

\[
\frac{1.24 \times 12.2\%}{3.6} = 3.4 + 12 \left( \frac{15.57}{21.2} \right)
\]

10.4 \leq 27.17 \quad \text{Bolted against side sway}

COLUMN IS NOT SLENDER
Column Group \# 4 - C4

Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/10/17
Time: 21:09:14

12 x 12 in

---

File: untitled.col
Project:
Column:
\[ f_c = 4 \text{ ksi} \]
\[ E_c = 3605 \text{ ksi} \]
\[ f_c = 3.4 \text{ ksi} \]
\[ a_u = 0.003 \text{ in/in} \]
\[ \beta_1 = 0.85 \]
Confinement: Tied

---

Engineer:
\[ a_y = 60 \text{ ksi} \]
\[ E_s = 29000 \text{ ksi} \]
\[ e_{yt} = 0.00206897 \text{ in/in} \]
\[ X_o = 0.00 \text{ in} \]
\[ Y_o = 0.00 \text{ in} \]
Min clear spacing = 3.38 in
Clear cover = 1.88 in

---

8 #4 bars
\[ \rho = 1.11\% \]
\[ I_x = 1728 \text{ in}^4 \]
\[ I_y = 1728 \text{ in}^4 \]
**Central Information:**
- **File Name:** untitled.col
- **Object:**
  - **Code:** ACI 318-14
  - **Run Option:** Investigation
  - **Run Axis:** X-axis

**Material Properties:**
- **Concrete:** Standard
  - $f'_c = 4$ ksi
  - $f_c = 3.4$ ksi
  - $E_{ps,u} = 0.003$ in/in
  - $E_{stal} = 0.85$

**Section:**
- **Rectangular:** Width = 12 in, Depth = 12 in
  - $I_x = 1728$ in$^4$
  - $I_y = 3464$ in$^4$
  - $x_o = 0$ in

**Reinforcement:**
- **Bar Set:** ASTM A615
  - Size | Dia (in) | Area (in$^2$) | Size | Dia (in) | Area (in$^2$) | Size | Dia (in) | Area (in$^2$)
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.30</td>
<td>0.11</td>
<td>4</td>
<td>0.50</td>
<td>0.20</td>
<td>5</td>
<td>0.63</td>
<td>0.31</td>
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<tr>
<td>6</td>
<td>0.75</td>
<td>0.44</td>
<td>7</td>
<td>1.00</td>
<td>0.60</td>
<td>8</td>
<td>1.00</td>
<td>0.79</td>
</tr>
<tr>
<td>9</td>
<td>1.13</td>
<td>1.00</td>
<td>10</td>
<td>1.27</td>
<td>1.27</td>
<td>11</td>
<td>1.41</td>
<td>1.56</td>
</tr>
<tr>
<td>14</td>
<td>1.69</td>
<td>2.25</td>
<td>16</td>
<td>2.26</td>
<td>4.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Confinement:** Tied; #3 ties with #10 bars, #4 with larger bar.
- $d(a) = 0.8$, $\phi_i(b) = 0.9$, $\phi_i(c) = 0.65$

**Layout:** Rectangular
**Pattern:** All Sides Equal (Cover to transverse reinforcement)
**Total steel area:** $A_s = 1.60$ in$^2$ at $\rho = 1.11$
**Minimum clear spacing:** $S = 3.38$ in

**#4 Cover = 1.5 in**

**Factored Loads and Moments with Corresponding Capacities:**

<table>
<thead>
<tr>
<th>No.</th>
<th>$P_u$ (kips)</th>
<th>$M_u$ (k-ft)</th>
<th>$\Phi/M_u$</th>
<th>$\Phi_{MN/u}$</th>
<th>$\mu_{L}$</th>
<th>$\delta$ (in)</th>
<th>$\delta_{L}$ (in)</th>
<th>$e_{ps,t}$</th>
<th>$\Phi_{ps,t}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.34</td>
<td>31.30</td>
<td>37.62</td>
<td>1.202</td>
<td>2.07</td>
<td>9.88</td>
<td>0.01231</td>
<td>0.900</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>15.83</td>
<td>15.97</td>
<td>39.83</td>
<td>2.558</td>
<td>2.19</td>
<td>9.88</td>
<td>0.01051</td>
<td>0.900</td>
<td></td>
</tr>
</tbody>
</table>

***End of output***
Mission Twenty-Five 35
Community Center-Dominican Republic

Mindy Trieu and Jocelyn Lu

Reference: 6J 218

Vertical Point Check:

\[ A_{c1} = 6.01 \text{ in}^2 \]

\[ A_{c2} = (0.01 \times 12^2) = 1.44 \text{ in}^2 = A_c \]

\[ A_{c3} = 0.08 \times (0.08 \times 12^2) = 11.52 \text{ in}^2 > A_c \]

Use 8-NO. 4 longitudinal bars in 12" sq column.

Column Transmission Req't:

\[ f_{sec} = 4 \text{ ksi} \]

\[ f_{tac} = 4 \text{ ksi} \]

Column transmission req't does not apply because the beam strengths are equivalent to floors and columns.

Tie size and spacing:

Use NO. 3 ties (gravity)

Bar size is least of:

1) NO. 3 enclosing NO. 10 bars
2) NO. 4 enclosing NO. 11 bars

Spacing:

Do is greater of:

1) \[ d = 12" - 1.5" = 9.5" \]
2) \[ d = 4(0.65) = 2.6" \]
3) 18"

\[ \text{governs} \]

S is least of:

1) \[ 4b \text{ (min col dim)} = 4(12") = 3" \]
2) \[ d \text{ (dim of smallest long)} = 12(1") = 12" \]

\[ \text{governs} \]

S is least of:

1) \[ 6" \text{ (long)} = 6(1") = 6" \]
2) \[ 6" \text{ (long)} = 6(1") = 6" \]
clear spacing = \( \left( \frac{1}{2} \times \text{dagg} \right) = \frac{4}{3} = 15'' \)

max & spacing is least of:
1. 11 d:bg = 11 (\( \frac{2}{3}'' \)) = 18''
2. 48 d:la = 48 (3'') = 58''

Earthquake Requirements:

Assume plastic capacity of gravity framing is executed during an earthquake and some non-linear behavior is required.

Amount of Transverse Ref. 41.

Pt 4 0.2 Ag fcc = 9Ksi - 41.6 = 15.83 k

15.83 k ≤ (0.3) (12×2.4)

15.83 k ≤ 172.6 k  \( \Rightarrow \) Trans is greater

a) 0.2 \( \left( \frac{Ag}{f_{cc}} \right) \left( \frac{f_{c}}{f_{y}} \right) = 0.2 \left( \frac{12}{9} - 1 \right) \left( \frac{12}{6} \right) = 0.015 \text{ in}^2 \)

b) 0.09 \( \left( \frac{f_{c}}{f_{y}} \right) = 0.09 \left( \frac{12}{6} \right) = 0.006 \text{ in}^2 \)

Arch = 0.015 \text{ in}^2

SBC

Arch = 0.015 \( \left( 3' \times 3' \times 3'' - 1.5 - 3/8'' - \frac{1}{2}'' \right) = 0.444 \text{ in}^2 \)

\( M_{sc} = 2.15 \times 4' = (3 \times 0.2) = 0.60 \text{ in}^2 \)  \( \Rightarrow \) Arch √

USE NO. 4 TIES @ 2'' O.C. (SEISMIC)
More Checks:

8.18.5.2.6 does not apply b/c not supporting reactions from discontinuous stiff masses such as walls.

8.18.7.5.7 does not apply b/c clear spacing = 1.5" x 4".

Shear forces - run up column to f = 1.0 k / 125 kg = 75 k.

\[ V_{max} = 141 k \] ft E Balance point

\[ V_e = \frac{2V_{max}}{d_e} \] E full height

\[ = \frac{(2)(141)}{20'} \]

\[ = 14.1 k \]

\[ V_e = \frac{2V_{max}}{d_e} \] E mid height

\[ = \frac{(2)(141)}{10'} \]

\[ = 28.2 k \]

\[ \phi V_n = \phi V_c + \phi V_s \]

\[ = 0 + \phi V_s \]

\[ = 0.15 \left( \frac{2\pi D - 1.5 - 1.5(0.5)}{3'} \right) \]

\[ = 92.25 k \]

\[ \phi V_n = 92.25 k \]

\[ V_e = 28.2 k \] O.K.
LTO

1. Loads

DL = 101 psf
LL = 20 psf

1st Level

\( R_1 = 1.2 \times 0.011 \times A_t \)
\( = 1.2 \times 0.011 \times (40 \text{ m}^2) \)
\( = 0.07 \)

\( R_2 = 1.0 \times (15.44) \)
\( = 15.44 \)

\( L_1 = (20 \text{ psf}) \times (0.70) \times (1.0) \)
\( = 15.2 \text{ psf} \)

\( L_2 = (12 \text{ psf}) \times (1.0) \)
\( = 12 \text{ psf} \)

2nd Level

\( A_t = (5 \text{ m}) \times (11 \text{ m}) \)
\( = 55 \text{ m}^2 \)

\( R_1 = 1.2 \times 0.011 \times A_t \)
\( = 1.2 \times 0.011 \times (55 \text{ m}^2) \)
\( = 0.07 \)

\( R_2 = 1.0 \)

\( L_2 = (20 \text{ psf}) \times (0.60) \times (1.0) \)
\( = 12 \text{ psf} \)

2. Design Axial Forces

\( P_1 = (73.2 \text{ psf} + 24.32 \text{ psf}) \times (40 \text{ m}^2) \times (3.28 \text{ ft/m})^2 \)
\( = 92 \text{ k} \)

\( P_2 = (73.2 \text{ psf} + 19.2 \text{ psf}) \times (55 \text{ m}^2) \times (3.28 \text{ ft/m})^2 \)
\( = 106.7 \text{ k} + 42 \text{ k} \)
\( = 148.7 \text{ k} \)

Roof Cir. HT = 7' - 0''

2nd Level Cir. HT = 11' - 0''
3. Design Bending Forces

1) Ext. face of first int. support (deg.) (girder)

$$\frac{Wc}{L} = \frac{(560 \text{ psf})(1.2)}{10} = 67.2 \text{ Kft}$$

2) Face of all other supports (girder)

$$\frac{Wc}{L} = \frac{(560 \text{ psf})(1.2)}{10} = 67.2 \text{ Kft}$$

3) Ext. face of first int. support (girder)

$$\frac{Wc}{L} = \frac{(560 \text{ psf})(1.2)}{10} = 67.2 \text{ Kft}$$

4) Face of all other supports (girder)

$$\frac{Wc}{L} = \frac{(560 \text{ psf})(1.2)}{10} = 67.2 \text{ Kft}$$

5) Ext. face of first int. support (cm)

$$\frac{Wc}{L} = \frac{(560 \text{ psf})(1.2)}{10} = 67.2 \text{ Kft}$$

$$M_{x1} = 170 \text{ Kft} - 67.2 \text{ Kft} = 102.8 \text{ Kft}$$

$$M_{y1} = 0$$

$$M_{x2} = 102.8 \text{ Kft} - 70 \text{ Kft} = 32.8 \text{ Kft}$$

$$M_{y2} = 18 \text{ Kft}$$
4. Check slenderness:

12" x 12" sq column

**ROOF ⇒ 2nd FLOOR**

$F = 1$

$W = 7' - 0"$ (circ ht)

$R = 0.3h$

$3.0'$

**Braced against sidesway:**

$x$-axis:

$\frac{12 + (12/12)}{3.0'} \leq 24 + 12 \left( \frac{6}{10.1} \right)$

$23.3 \leq 24 + 12 \left( \frac{6}{10.1} \right)$

$23.3 \leq 34$

$\checkmark$

$y$-axis:

$\frac{12 + (12/12)}{3.0'} \leq 24 + 12 \left( \frac{6}{10.1} \right)$

$38.3 \leq 24 + 12 \left( \frac{6}{10.1} \right)$

$38.3 \leq 40$

$\checkmark$
Column Group #5 - 55

12 x 12 in

Code: ACI 318-14
Units: English
Run axis: Biaxial
Run option: Investigation
Genderness: Not considered
Column type: Architectural
Bars: ASTM A615
Date: 06/16/17
Time: 02:10:09

StructuresPoint - spColumn v5.11 (TM). Licensed to: Cal Poly University. License ID: 64929-1050704-4-2356E-24F6B

File: untitled.col
Project:
Column:
\( f_c = 4 \) ksi
\( E_c = 3605 \) ksi
\( f_y = 60 \) ksi
\( E_s = 29000 \) ksi
\( \epsilon_{yt} = 0.00206897 \) in/in
\( \mu = 3.4 \) ksi
\( \beta = 0.003 \) in/in
\( \beta_{ta} = 0.85 \)
Confinement: Tied

Engineer:
\( \gamma = 144 \) in^2
\( A_s = 1.60 \) in^2
\( X_o = 0.00 \) in
\( Y_o = 0.00 \) in
Min clear spacing = 3.38 in
Clear cover = 1.88 in

\( \rho = 1.11% \)
\( I_x = 1728 \) in^4
\( I_y = 1728 \) in^4
Column Group #5 - C5

12 x 12 in

Code: ACI 318-14
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Architectural
Bars: ASTM A615
Date: 06/16/17
Time: 02:10:31

STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: Cal Poly University. License ID: 64929-1050703-4-2356E-24F6B

File: untitled.col
Project:
Column:
\( f_c = 4 \text{ ksi} \)
\( E_c = 3605 \text{ ksi} \)
\( f_y = 60 \text{ ksi} \)
\( E_s = 29000 \text{ ksi} \)
\( x_{yf} = 0.00206897 \text{ in/in} \)
\( \beta_1 = 0.85 \)
\( b_u = 0.003 \text{ in/in} \)

Engineer:
\( A_g = 144 \text{ in}^2 \)
\( A_s = 1.60 \text{ in}^2 \)
\( x_o = 0.00 \text{ in} \)
\( y_o = 0.00 \text{ in} \)
Min clear spacing = 3.38 in
Clear cover = 1.88 in

8 #4 bars
\( \rho = 1.11\% \)
\( l_x = 1728 \text{ in}^4 \)
\( l_y = 1728 \text{ in}^4 \)
General Information:
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File Name: untitled.col
Project:
Column:
Code: ACI 318-14
Run Option: Investigation
Run Axis: Biaxial

Material Properties:
====================
Concrete: Standard
f'c = 4 ksi
Ec = 3605 ksi
fc = 3 ksi
Eps_u = 0.003 in/in
Beta_t = 0.85

Steel: Standard
fy = 60 ksi
Es = 25000 ksi
Eps_yt = 0.00200097 in/in

Section:
========
Rectangular: Width = 12 in
Depth = 12 in
Gross section area, Ag = 144 in^2
Iy = 1728 in^4
ry = 3.4641 in
Xo = 0 in

Reinforcement:
---------------
Bar Set: ASTM A615
Size Diam (in) Area (in^2) Size Diam (in) Area (in^2) Size Diam (in) Area (in^2)
# 3 0.31 0.11 # 4 0.50 0.20 # 5 0.63 0.31
# 6 0.75 0.44 # 7 0.88 0.60 # 6 1.00 0.79
# 9 1.13 1.20 # 10 1.27 1.27 # 11 1.41 1.56
# 14 1.69 2.25 # 14 2.26 4.00

Confinement: Tied; #3 tied with #16 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 1.60 in^2 at rho = 1.11%
Minimum clear spacing = 3.38 in

8 #4 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:
--------------------------------------------------------
<table>
<thead>
<tr>
<th>No.</th>
<th>Pu kip</th>
<th>Mxx k-ft</th>
<th>Myy k-ft</th>
<th>PhiMax k-ft</th>
<th>PhiMxx k-ft</th>
<th>PhiMyy k-ft</th>
<th>PhiMmx/MuNk in</th>
<th>Nk in</th>
<th>depth in</th>
<th>Dk in</th>
<th>depth in</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>42.00</td>
<td>6.30</td>
<td>0.00</td>
<td>48.29</td>
<td>0.09</td>
<td>7.665</td>
<td>2.78</td>
<td>9.98</td>
<td>0.00766</td>
<td>0.900</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>96.70</td>
<td>7.80</td>
<td>48.10</td>
<td>8.35</td>
<td>51.49</td>
<td>1.071</td>
<td>5.76</td>
<td>11.62</td>
<td>0.00306</td>
<td>0.735</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*** End of output ***
Column Group #5 - #7

Vertical Reinforcing:
\[ A_s = 8 - \frac{W_o}{h} = 1.6 \text{ in}^2 \]

As min = 0.01 A0 (0.01)(12^2) = 1.44 in^2 ≤ As √

As max = 0.08 A0 (0.08)(12^2) = 11.52 in^2 > As √

**USE 8 - NO. 4 LONGITUDINAL BARS**

**IN 12" SQ COLUMN**

Column Transmission Req't:

ACI 318
§5.15.2

f'c = 4 ksi, f'p = 4 ksi

Column transmission req't does not apply because the zone strengths are equivalent b/w floors and columns.

Bar Size and Spacing:

ACI 318
§25.7.2

Bar size is least of:
1) No. 3 enclosing No. 10 bars
2) No. 4 enclosing No. 11 bars

**USE NO. 3 TIES (GRAVITY)**

Spacing:

ACI 318
§18.7.5

d0 is greater of:
1) d0 = 12" - 1.5" - 3/8" - 1/4" = 9.875"
2) \( \frac{V_e}{ld} = (\frac{V_e}{12}) = 21 = 0.416 \) = governs
3) 16"

s0 is least of:
1) \( \frac{d0}{4} \) (min col diam) = \( \frac{(4)(12)}{4} = 3" \) = governs
2) l0 (diam + smallest long.) = (12 * 4\%) = 6"

s0 is least of:
1) 6" (diam) = \( (6)(4\%) = 3" \) = governs
2) 6"
### Column Group #5 - C5

<table>
<thead>
<tr>
<th>Reference</th>
<th>ACI 218</th>
<th>25.7.2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>clear spacing = ((\frac{1}{2}) \times d_{agg}) = (\frac{3}{4}'') = 15''</td>
<td></td>
</tr>
<tr>
<td></td>
<td>max. spacing is least of 1) (11'') d = 11'' ((\frac{1}{8})) = 9'' governs</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2) (4) d_{N} = (4'') ((\frac{3}{4})) = 3'' governs</td>
<td></td>
</tr>
</tbody>
</table>

### Earthquake Requirements:

Assume elastic capacity of gravity framing is exceeded during an earthquake, and some non-linear behavior is rapid.

### Amount of Transverse Per Eq. 41

\(P_t = 0.3A_g f_c\); \(f_c = 4\) ksi; \(P_t = 15.82\) k

\(96.7\) k \(> (0.3 \times 12 \times 12 (3.4))\)

\(96.7\) k \(> 172.8\) k \(\Rightarrow\) Trans is greater

9) \(0.2 (\frac{A_t}{A_{net}}) (\frac{f_c}{f_y}) = 0.3 \left(\frac{12}{9^2} \right) (\frac{4}{6}) = 0.015\) in

### ACI 218

5) \(187.5\) in

6) \(0.09 (\frac{f_y}{f_c}) = 0.09 (\frac{4}{3}) = 0.009\) in

### ACI 318

5) \(A_{sh} = 0.015\) in

5) \(A_{sh} = (0.015 \times 3 \times 12'' - 1.5 - 3/8'' - 9/8'') = 0.444\) in

### ACI 318

5) \(A_{sh} = 3 - N \times 4' = (3 \times 0.2) = 0.6\) in

### USE NO. 4 TIES @ 3'' O.C. (SEISMIC)
More Checks:

818.5.7.6 does not apply. We are not supporting reactions from discontinued stiff members such as walls.

818.7.5.7 does not apply. We are supporting 1.5' x 4'.

Stair forces - run span column of A = 1.6; f_y = 125; f_g = 75 kpsi.

\[ M_{\text{min}} = 53.6 \text{kft} \] at balance point.

\[ V_c = \frac{2 \text{ Mpr}}{w_4} \]

\[ = \frac{(2 \times 53)}{201} \]

\[ = 5.3 \text{k} \]

\[ V_c = \frac{2 \text{ Mpr}}{w_4} \]

\[ = \frac{(2 \times 53)}{101} \]

\[ = 10.6 \text{k} \]

\[ \phi V_n = \phi V_c + \phi V_s \]

\[ = 0 + \phi V_s \]

\[ = 0.75 \left( \frac{V_s (f_y)}{6} \right) \]

\[ = 0.75 \left( \frac{125 \times 0.2 \times 0.12 (12 - 1.5 - \left[ \frac{V_s (f_y)}{6} \right])}{8} \right) \]

\[ = 12.25 \text{k} \]

\[ \phi V_n = 92.25 \text{k} \leq V_c = 10.6 \text{k} \checkmark \text{D.K.} \]
Column Design - C6

Load Combo: 1.2D + 1.0L
DL = 61 psf
LL = 20 psf

At = 991 sf / 2.28 = 0.44 m²  ⇒  At = 55.45  ⇒  P1 = 0.6

F1 = 1  ⇒  F = 4  ⇒  R2 = 1

Lr = Lc(R1 + R2) = (1.20 m × 0.44 m²) = 12 psf

Axial Design:

Pcr = [(1.2 X 61 psf) + (1.5 X 12 psf)](9.77) = 91.6 k

Mcr = 91.6 k × (1.2 X 12 psf X 9.77) = 14.4 k ft

Detailed Diagram:

M3 = 0.5 × w × l² = (2.01 kft)(11²) = 2.51 k ft

M4 = 0.5 × w × l² = (2.01 kft)(11²) = 46.94 k ft
1.) Exterior force of the first interior support

\[ M_1 = \frac{w \cdot a^2}{12} = \frac{(2.604 \times 11.2^2)}{12} = 71.8 \text{ kft} \]

2.) Force of other supports

\[ M_2 = \frac{w \cdot a \cdot b}{12} = \frac{(2.604 \times 11.2 \cdot 11)}{12} = 65.3 \text{ kft} \]

\[ M_{max} = \left| M_1 - M_2 \right| = \left| 71.8 - 65.3 \right| = 6.5 \text{ kft} \]

\[ M_{min} = \left| M_2 - M_1 \right| = \left| 65.3 - 46.7 \right| = 18.6 \text{ kft} \]

Moment Diagram

Shear Force Check:

ACI 318

12" 50 Column

Roof = 2nd Floor

\[ k = 1.0 \]

\[ h_u = 7' - 0" = 7 \times 12" = 84" \]

\[ c = 0.2 \times \frac{h}{12} = 0.2 \times \frac{84}{12} = 4.2" \]

\[ \frac{k \cdot h}{r} \leq 24 + 12 \left( \frac{M_1}{M_2} \right) \]

\[ \frac{2.604 \times 12^2}{4.2} \leq 24 + 12 \left( \frac{71.8}{65.3} \right) \]

\[ 23.3 \leq 42.7 \checkmark \text{ Braced against sidesway} \]
Reference

2nd Floor Ground

K = 1.0

\( a_0 = 11.5'' \) (clear height)
\( r = 3.6'' \)

\( 13.68 \times 12'' = 3.9 + 12(4.67) \)
\[ \frac{\text{2.1}}{12} \]

\[ 10.48 \leq 42.7 \quad \text{Breed against side sway} \]

COLUMN IS NOT SLENDER

Vertical Bending Check:

Assuming \( 0.01\, \text{ft}^2 = 0.01\times12^2 = 1.44\, \text{ft}^2 \)

Assuming \( 0.06\, \text{ft}^2 = 0.06\times12^2 = 11.52\, \text{ft}^2 \)

Use 8 - No. 4 longitudinal bars in 16' column

Note: Refer to Reference Column design C1 or C4 for tie size and spacing calculations.

Use No. 4 ties @ 3" O.C. (Gravity)
Column Group 6 - C6

P (kip)

600

fs=0

(Pmax)

(Pmax)

fs=0.5fy

fs=0

fs=0.5fy

fs=0

fs=0.5fy

(Pmin)

-100

(Pmin)

Mx (k-ft)

-90

90

12 x 12 in

Code: ACI 318-14

Units: English

In axis: About X-axis

Pmin option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/11/17

Time: 00:11:38

STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: Cal Poly University. License ID: 64920-1050703-4-2356E-200AE

File: untitled.col

Project:

Column:

f_c = 4 ksi

E_c = 3605 ksi

f_c = 3.4 ksi

u = 0.003 in/in

Beta1 = 0.85

Confinement: Tied

fy = 60 ksi

Es = 29000 ksi

t_e = 0.00206897 in/in

Engineer:

Ag = 144 in^2

As = 1.60 in^2

X_o = 0.00 in

Y_o = 0.00 in

Min clear spacing = 3.38 in

8 #4 bars

rho = 1.11%

lx = 1728 in^4

ly = 1728 in^4

Clear cover = 1.88 in
Cal Information:

File Name: untitled.col
Project:
Code: ACI 318-14
Run Option: Investigation
Run Axis: X-axis

Material Properties:

Concrete: Standard
f′c  =  4 ksi
Ec  =  3605 ksi
fc  =  3.4 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy  =  60 ksi
Es  =  29000 ksi
Eps_yt = 0.00206997 in/in

Section:

Rectangular: Width = 12 in
Gross section area, Ag = 144 in^2
Ix = 1728 in^4
rx = 3.4641 in
Xo = 0 in

Depth = 12 in
Iy = 1728 in^4
ry = 3.4641 in
Xo = 0 in

Reinforcement:

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 3</td>
<td>0.13</td>
<td># 4</td>
<td>0.60</td>
<td># 5</td>
<td>0.31</td>
</tr>
<tr>
<td># 6</td>
<td>0.13</td>
<td># 7</td>
<td>0.60</td>
<td># 8</td>
<td>0.31</td>
</tr>
<tr>
<td># 9</td>
<td>1.00</td>
<td># 10</td>
<td>1.27</td>
<td># 11</td>
<td>1.41</td>
</tr>
<tr>
<td># 14</td>
<td>1.69</td>
<td># 10</td>
<td>1.27</td>
<td># 11</td>
<td>1.41</td>
</tr>
</tbody>
</table>

Confined: Tied; #10 ties with #10 bars, #4 with larger bars.
\( \mu(a) = 0.8, \ phi(b) = 0.9, \ phi(c) = 0.65 \)

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 1.60 in^2 at rho = 1.11%
Minimum clear spacing = 3.38 in

# 4 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

<table>
<thead>
<tr>
<th>No.</th>
<th>Pu (kip)</th>
<th>Max k-ft</th>
<th>PhiMax</th>
<th>PhiMin/Ma</th>
<th>NA depth</th>
<th>Dt depth</th>
<th>depth</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>91.60</td>
<td>6.50</td>
<td>8.69</td>
<td>9.88</td>
<td>0.00440</td>
<td>0.00133</td>
<td>0.650</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>164.14</td>
<td>4.70</td>
<td>11.04</td>
<td>9.88</td>
<td>0.00440</td>
<td>0.00133</td>
<td>0.650</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*** End of output ***
Column Group #7 (C7)

Loads:
- DL: 11 psf
- LL: 20 psf

Buoyant:
- bA = (0.5m)(3m) + (0.5m)(3m) = 27 m²
- R1 = 1.2 - 0.014A
  = 1.2 - 0.014(27 m²)
  = 0.903
- R2 = 1.0, R1 > R2
- L0 = (20 psf)(0.903)(1.0)
  = 18.06 psf

Pu = ((1.2)(11 psf) + (1.0)(18.06 psf))(27 m²)(3.28 ft/m)
  = 12.3 k

Design:
- x = 10.5' + 11'
  = 21.5'
- Try HSS 3 x 3 x 1/8

Checks:
- Buckling:
  - λr = b/k = 1.40 \sqrt{E/\rho}
  = 1.40 \sqrt{210000000/400}
  = 35.15 max
  - λr = 22.9 < 35.15

- Slenderness:
  - \frac{L}{t} \leq 200
  - (1.0)(10.5')(11'/14') = 200
  - 117"< 107.8 \leq 200

USE HSS 3 x 3 x 1/8
### Stage Steel Columns

<table>
<thead>
<tr>
<th>Reference</th>
<th>Stage Steel Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$A_t = (17') (17') = 289 \text{ ft}^2$</td>
</tr>
<tr>
<td></td>
<td>$DL = 11 \text{ PSF}$</td>
</tr>
<tr>
<td></td>
<td>$LL = 20 \text{ PSF}$</td>
</tr>
<tr>
<td></td>
<td>$\frac{A_t}{L} = 289 \text{ ft}^2$</td>
</tr>
<tr>
<td></td>
<td>$R_1 = 1.2 - 0.001 A_t$</td>
</tr>
<tr>
<td></td>
<td>$R_2 = 0.911$</td>
</tr>
<tr>
<td></td>
<td>$R = 1.0$</td>
</tr>
<tr>
<td></td>
<td>$L_r = (20 \text{ PSF}) (0.911)(1.0)$</td>
</tr>
<tr>
<td></td>
<td>$18.22 \text{ PSF}$</td>
</tr>
</tbody>
</table>

$$Pu = \left(1.2(11 \text{ PSF}) + 1.6 (18.22 \text{ PSF}) \right) (289 \text{ ft}^2)$$

$$Pu = 12.24 \text{ K}$$

### Design

**Slanted Column**

$L = 16' - 0''$

$K_L = (1.0)(16.2'')$

$K_{B1} = 12.7$

**Use**

HSS 3" x 3" x 3/16"

$A_{sys} = 15.1 \text{ K}$

$F_y = 40 \text{ ksi}$

### Checks

**Buckling**

$$\lambda = \frac{L}{r} = 1.40 \sqrt{E / F_y}$$

$35.15$

$$\lambda = 14.12 < \lambda_r$$

**Sloppiness**

$$k_2 \leq 2$$

$$(1.0)(1.0)(12'/1') = 200$$

$$1.14'' \leq 200$$

$108.4 \leq 200$
story height < 60'
  - low rise building, partially enclosed

1) Risk category: 2

2) Wind loads based on Florida because it has closest wind conditions to Dominican Republic

\[ V = 150 \text{ mph}, 60 \text{ m/s} \]

3) Wind load parameters:
   - \( K_d = 0.85 \)
   - Exposure category: B
   - \( K_z = 1.0 \)
   - Partially enclosed (60% of stage)

\[ G_{pi} = 0.55 \]

4) \( h = 22' \) (mean roof)
   - \( K_h = 0.47 \)

\[ h_{diag} = 2 = 25' \approx 7.6 \text{ m} \]
   - \( K_2 = 0.16 \)

5) \[ L_2 = 0.00256 \left( \frac{K_2 K_z K_d V^2}{0.00256 (0.600)(1.0)(0.85)(180)^2} \right) \]
   - 41.53 psf

\[ L_n = 0.00256(0.64)(1.0)(0.85)(180)^2 \]
   - 75.12 psf

N/S:
   - Windward \( C_p = 0.8 \)
   - \( u_b = 0.7102 \)
   - Leeward \( C_p = 0.5 \)

E/W:
   - Windward \( C_p = 0.8 \)
   - \( u_b = 1.41 \)
   - Leeward \( C_p = 0.3 \)

7) Flexible diaphragm, rigid building

- Windward:
  \[ P = G C_{pi} - q_2 (G C_{pi}) \]
  \[ = (41.53 \text{ psf})(0.85)(0.8) - (41.53 \text{ psf})(0.55) \]
  \[ = 57.23 \text{ psf} \]

- Worst case wind load due to windward side and full building height

\[ P = (77.23 \text{ psf})(21.42') \]
USGS Design Maps Summary Report

User-Specified Input

Building Code Reference Document: ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates: 38.52466°N, 118.62459°W

Site Soil Classification: Site Class D - "Stiff Soil"

Risk Category: I/II/III

USGS-Provided Output

\[ S_s = 1.455 \text{ g} \quad S_{ms} = 1.455 \text{ g} \quad S_{zs} = 0.970 \text{ g} \quad S_{ps} = 0.496 \text{ g} \]

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

For PGA, T1, C0, and Cm values, please view the detailed report.

Seismic design category is D (ASCE 7-10 TL 11.01).
Seismic Force Analysis

Building Properties

<table>
<thead>
<tr>
<th>Level</th>
<th>Floor Area (ft^2)</th>
<th>Floor Area (m^2)</th>
<th>Wall Perimeter (ft)</th>
<th>Wall Perimeter (m)</th>
<th>Tributary Height (ft)</th>
<th>Tributary Height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5548.80</td>
<td>515.50</td>
<td>337.93</td>
<td>103.00</td>
<td>11.00</td>
<td>3.35</td>
</tr>
<tr>
<td>Roof</td>
<td>4305.56</td>
<td>400.00</td>
<td>269.03</td>
<td>82.00</td>
<td>6.00</td>
<td>1.52</td>
</tr>
</tbody>
</table>

Loads Per Level

<table>
<thead>
<tr>
<th>Level</th>
<th>Roof Level</th>
<th>Floor Level</th>
<th>Exterior Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>262,639.16</td>
<td>-</td>
<td>92,815.01</td>
</tr>
<tr>
<td>Floor Level</td>
<td>338,476.54</td>
<td>#</td>
<td>256,486.59</td>
</tr>
</tbody>
</table>

Total Load

<table>
<thead>
<tr>
<th>Weight</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Weight at Roof Level</td>
<td>355,454.17 #</td>
</tr>
<tr>
<td>Total Weight at Floor Level</td>
<td>594,963.14 #</td>
</tr>
<tr>
<td>Total Building Weight</td>
<td>950,417.30 #</td>
</tr>
</tbody>
</table>

Mapped Spectral Acceleration:

Values gathered from USGS Design Maps Summary Report. City of Hawthorne's geographical location was chosen because it closely mirrored seismic location of Dominican Republic.

Seismic Response Coefficient

Response Modification Factor - ASCE Table 12.2-1
Seismic Importance Factor - ASCE Table 1.5-2
Seismic Design Category - IBC 302.1
Risk Category - ASCE Table 1.5-1

Special Reinforced Concrete Moment Frame

<table>
<thead>
<tr>
<th>R</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>le</td>
<td>1.0</td>
</tr>
<tr>
<td>SDC</td>
<td>V D</td>
</tr>
<tr>
<td>Risk Category</td>
<td>2</td>
</tr>
</tbody>
</table>

Fundamental Period

| Ct   | 0.02 |
| h_n | 23.28 ft |
| x   | 0.75 |
| T   | 0.21197 sec |

Cs = 0.12125
Cs, max = 0.29250
Cs, min = 0.0427
Lateral loads due to wind forces are less than lateral loads due to seismic forces. Therefore, seismic forces will govern lateral design.

### Vertical Force Distribution

<table>
<thead>
<tr>
<th>Level</th>
<th>Floor Wt. (lb)</th>
<th>Height (ft)</th>
<th>W*(h^k) (lb-ft)</th>
<th>CvX</th>
<th>Fx, Story Force/Shear (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>355,454</td>
<td>23.28</td>
<td>8,274,973</td>
<td>0.537</td>
<td>61,863</td>
</tr>
<tr>
<td>2</td>
<td>594,963</td>
<td>12.00</td>
<td>7,139,558</td>
<td>0.463</td>
<td>53,375</td>
</tr>
<tr>
<td>Σ</td>
<td></td>
<td></td>
<td>15,414,531</td>
<td></td>
<td>115,238.10</td>
</tr>
</tbody>
</table>

### Diaphragm Design Forces

<table>
<thead>
<tr>
<th>Level</th>
<th>Floor Wt.</th>
<th>Fx (lb)</th>
<th>Fp (lb)</th>
<th>Fp-max (lb)</th>
<th>Fp-min (lb)</th>
<th>Fp-design (lb)</th>
<th>Ax (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>355,454</td>
<td>61,863</td>
<td>2,657</td>
<td>137,916</td>
<td>68,958</td>
<td>68,958</td>
<td>0.1940</td>
</tr>
<tr>
<td>2</td>
<td>594,963</td>
<td>53,375</td>
<td>4,448</td>
<td>230,846</td>
<td>115,423</td>
<td>115,423</td>
<td>0.1940</td>
</tr>
</tbody>
</table>

**ASCE 12.8 and 12.10:**

- \( CvX = \frac{(Wx*(hx^k))}{(Wt*(ht^k))} \)
- \( Fx = CvX \times V \)
- \( Fp = \frac{\text{Sum}(Fx)}{\text{Sum}(Wt)} \times Wt. \)
- \( Fp-max = 0.4 \times Sds \times Wt. \)
- \( Fp-min = 0.2 \times Sds \times Wt. \)
- \( Ax = \frac{Fp-design}{Wt.} \)
Inequality checks

Structures w/ SDC D can't have vertical irregularities → must check

12.3.1.2 Reduce θ to 1.0 b/c it is SDC D w/ each story resisting more than 25% of the base shear in each direction to comply w/ T12.3-3

- Lateral force-resisting element = moment frame

- N/κ = 5 total bays

- E/W = 6 total bays

⇒ removal of one bay = 16.7% - 20% less strength

So θ = 1.0

Horizontal checks:

- Type 1A) \[ \frac{d_{\text{avg}}}{d_{\text{max}}} < 1.2 \]

\[ 0.000972 = 1.27 > 1.2 \]

- Type 1B) Same as Type 1A ✓

- Type 2) No reentrant corners ✓

- Type 3) No diaphragm discontinuity ✓

- Type 4) No out-of-plane offsets b/c concrete moment frame will extend to both stories ✓

- Type 5) No nonparallel system irregularity b/c building is aligned only major orthogonal axes ✓

Vertical check:

- Type 1A) No stiffness soft story irregularity b/c lateral system is the same between stories ✓

- Type 1B) Same as 1A ✓

- Type 2) No weight mass irregularity b/c only technically have roof loads ✓

- Type 3) No vertical geometric irregularity b/c horizontal dimensions of seismic system is the same between stories ✓

- Type 4) Doesn't apply b/c no in-plane offset of vertical srs. system ✓

- Type 5A) No discontinuity of lateral strength-weak story ✓

- Type 5B) Same as 5A ✓

Diaphragm ratio check:

Roof:

\[ \frac{25}{10} = 1.50 \times 25 < 3 \]

Max: 1.75 < 3 ✓
**Load Combo:**

\[
\begin{align*}
1.0 & \cdot (1.2 + 0.25ps) D + P & + L \\
1.394 & \cdot (1.0) Q & + L \\
\end{align*}
\]

**Etabs Inputs:**

**ROOF:**

\[
\text{BM DL: } (41 \text{ psf})(41') = 1,681 \text{ kip} \\
\text{BM LL: } (20 \text{ psf})(41') = 0.820 \text{ kip}
\]

**2ND LVL:**

\[
\text{Girder DL: } (50 \text{ psf})(20.24') = 1.409 \text{ kip} \\
\text{Girder LL: } (20 \text{ psf})(20.24') = 0.524 \text{ kip}
\]

- Only girder will take gravity load due to the location of the diaphragm.

**ROOF: Center of Mass:**

\[(41', 26.24') \quad \text{-- apply } F_x = 621.8 \text{ kip}
\]

**2ND LVL: Center of Where Building will Rotate About:**

\[(33.03', 42.32') \quad \text{-- apply } F_x = (0.62)(53.275') = 35.228 \text{ kip}
\]

\[(41.18', 33.49') \quad \text{-- apply } F_x = (0.34)(53.275') = 18.148 \text{ kip}
\]

\[D_2 = 18.233 \text{ ft}^2 \approx 66\% \text{ of } D_{tot} \\
D_3 = 67.8 \text{ ft}^2 \approx 34\% \text{ of } D_{tot} \\
D_{tot} = 2001.1 \text{ ft}^2\]
See hand calculations for determination of loads applied.
Inputs/Modeling Parameters:

MATERIAL PROPERTIES:

- Material Property Data
  - General Data
    - Material Name: HSC035
  - Material Type: Concrete
  - Directional Symmetry Type: Neutral
  - Material Density Color
  - Material Notes
  - Material Weight and Mass
    - Specific Weight Density
    - Weight per Unit Volume
    - Mass per Unit Volume

Mechanical Property Data
- Modulus of Elasticity, E
- Poisson's Ratio, v
- Coefficient of Thermal Expansion, \( \alpha \)
- Shear Modulus, G

Design Property Data
- Design Material Data
- Reference Material Data
- Material Density Property Data

LOAD ADJUSTMENTS:

- Load Combination Data
  - Load Combination Name: Combo 1
  - Construction Type
  - Notes
  - Auto Combination

Before Combination of Load Case/Combo Results

<table>
<thead>
<tr>
<th>Load Name</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td>1.324</td>
</tr>
<tr>
<td>Live</td>
<td>1</td>
</tr>
<tr>
<td>Seismic</td>
<td></td>
</tr>
</tbody>
</table>

Self-weight multiplier set equal to zero because the loads applied in the ETABS model already accounts for appropriate gravity and lateral loads.
Inputs/Modeling Parameters:

BEAM/GIRDER PROPERTIES:

COLUMN PROPERTIES:

Column and beam sizes based on quick calculations and readjusting as necessary for constructability purposes.

(diaphragm modelled as semi-rigid.)
N/S Loading:

STORY FORCES:

DEFLECTION:

MF-1

MF-2

MF-3

MF-4

See hand calculations for determination of loads applied.
See hand calculations for determination of loads applied.
See hand calculations for determination of loads applied.
See hand calculations for determination of loads applied.
E/W Loading:

STORY FORCES:

DEFLECTION:

See hand calculations for determination of loads applied.
See hand calculations for determination of loads applied.
See hand calculations for determination of loads applied.
E/W Loading:

See hand calculations for determination of loads applied.

MOMENT:

MF-1

1  2  2.5  3  4  4.5  5  6  7

MF-2

1  2  2.5  3  4  4.5  5  6  7

MF-3

1  1  1  1  1  1  1  1  1

MF-4

5  5  5  5  5  5  5  5  5
Deflected Building (N-S Loading):

Deflected Building (E-W Loading):

Joint Deflection:

<table>
<thead>
<tr>
<th>Joint Deflection (ETABS)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>N-S Force</strong></td>
</tr>
<tr>
<td>Level</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>Roof</td>
</tr>
<tr>
<td>2nd</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>E-W Force</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>Roof</td>
</tr>
<tr>
<td>2nd</td>
</tr>
</tbody>
</table>
Maximum Drift: \( a = 0.020h \)

**N-S Loading:**

2nd Floor: \( hx = 12' - 0" \)
\( a = 0.020 (12') (12''/ft) = 2.88" \)

Design: \( dx = \frac{cdfx}{Ie} \)
\( w/cd = 5.5 \)
\( Ie = 1.0 \)

\( dx = (8.5) (0.107049") \)
\( = 0.59" \) \( \checkmark \)

E-W Loading:

2nd Floor: \( hx = 12' - 0" \)
\( a = 2.88" \)

\( dx = (8.5) (0.170547") \)
\( = 0.94" \) \( \checkmark \)

Use story drift as grout gap between concrete framing system and masonry walls to prevent dual axial lateral system.
Design is based on worst case moment frame - MF 4. One moment frame will be design for entire building.

**Moment Frame, Beam Design (B5)**

**ETABS Analysis**

**Demand:**
- \( M_y = 39.2 \text{ kft} \) (over the supports)
- \( M_y = 29.75 \text{ kft} \) (between the supports)
- \( V_y = 16.4 \text{ k} \)

**Longitudinal Steel (over supports):**

- \( f_y = 60 \text{ ksi} \)
- \( f_y = 0.9f_y = 54 \text{ ksi} \)
- \( l_s \text{ (assumed)} = 4 \text{ in} \) (12.25 - 8.25 = 4 in)
- \( f_y = 0.9f_y = 54 \text{ ksi} \)

- \( A_{min} = \frac{200 \text{ ksw}}{f_y} = \frac{200 \times 12 \times 9.5}{54} = 69.6 \text{ in}^2 \)

- \( A_{min} = 2.3 \frac{f_y \times l_s}{f_y} = 2.3 \times 54 \times 12 \times 9.5 = 0.31 \text{ in}^2 < A_s \checkmark \)

**Check:**

- \( a = \frac{M_y}{f_y (l_s - d)} = \frac{39.2 \times 12}{54 \times (4 - 1.76)} = 1.76 \text{ in} \)
- \( b = 0.85 \)
- \( c = 1 \)
- \( d = 12 - 1.76 \text{ in} = 10.24 \text{ in} \)
- \( f_y = 0.9f_y = 54 \text{ ksi} \)

- \( d = 12 - 1.76 \text{ in} \) (cover - 0.5" assumed - (7/8"/1") = 9.5")

- \( \varepsilon_s = 0.002 \left( \frac{d - c}{c} \right) = 0.002 \left( \frac{12 - 1.76}{1.76} \right) = 0 \) \[0.007 > 0.002 \]

- \( \phi_M = \phi = 0.9 \)

- \( \phi_M N = \phi f_y (l_s - d) = (0.9) \times 54 \times 12 \times 9.5 = 5438 \text{ k} \)

- \( \phi_M N = 0.9 f_y l_s = 0.9 \times 54 \times 12 \times 9.5 = 5438 \text{ k} \)
\[ \phi M_n = 46.5 \text{ kft} \quad > \quad M_y = 39.2 \text{ kft} \quad \checkmark \quad O.K. \]

**USE 2-NO. 7 LONG BARS OVER SUPPORTS**

Longitudinal Steel (between the supports):

\[ M_y = \frac{12}{12}(d-\frac{d}{2}) \]

\[ = (2.995)(12.5) \]

\[ = (0.9)(12)(7.15) \]

\[ A_s = 0.09 \text{ in}^2 \quad \text{Try: 2 - NO. 6's, } A_s = 0.88 \text{ in}^2 \]

\[ A_s = 0.88 \text{ in}^2 \quad < \quad A_s \]

Check:

\[ a = \frac{A_s y}{0.85\sqrt{f_y}} = \frac{(0.88)(12)}{(0.85)(6)} = 1.29'' \]

\[ c = \frac{9}{0.85} = 10.7'' \]

\[ d = 12 - 1.5 - 0.5 - (12/2) = 7.25'' = 7.5'' \]

\[ E_t = 0.002 \left( \frac{d-c}{c} \right) = 0.002 \left( \frac{7.5-1.5}{1.52} \right) = 0.01575 \times 0.004 \quad \checkmark \quad \text{tens controlled, } d = 0.9 \]

\[ \phi M_n = \frac{\phi M}{d-\frac{d}{2}} \]

\[ = (0.9)(0.88)(6)(9.5 - 1.27) = 92.079 \text{ kft} \quad \Rightarrow \quad 85.1 \text{ kft} \]

\[ \phi M_n = 85.1 \text{ kft} \quad > \quad M_y = 29.75 \text{ kft} \quad \checkmark \quad O.K. \]

**USE 2-NO. 6 LONG BARS B/W SUPPORTS**
Hooked Longitudinal Bars C. ends:

\[ M_y = 37.2 \text{ kft} \]

\[ a = \frac{-2M_y}{f_y c_f c_b} + d^2 \quad \therefore d = 9.5'' \]

\[ = 9.5 - \frac{(1.95)(1.92)(12)}{\sqrt{(0.95)(0.85)(4)(12)}} + 9.5^2 \]

\[ = 9.5 - \frac{29.72}{\sqrt{0.95(0.85)(4)(12)}} + 9.5^2 \]

\[ = 9.5 - 2.97 + 9.5^2 \]

\[ = 9.5 - 2.97 + 9.09 \]

\[ = 2.07'' \]

\[ A_{reqd} = 0.85 f_y c_f c_b \quad \therefore (0.85)(0.85)(4)(12) = 0.993 \text{ m}^2 \]

\[ \Rightarrow \text{Try} 2 - \text{No.} 7's, A_{S} = 1.21 \text{ m}^2 \]

Check:

\[ a = \frac{f_y c_f c_b}{0.85 f_y c_f c_b} \quad \therefore a = 1.36'' \]

\[ c = 2.07'' \]

\[ A_{S} = 0.682 \left( \frac{9.5 - 2.07}{2.07} \right) = 0.617 < 0.88 \quad \text{ten's controlled, } 4 = 0.9 \]

\[ 4M_n = (0.9)(1.2)(4.6)(9.5 - 1.2) = 46.5 \text{ kft} \]

\[ 4M_n = 46.5 \text{ kft} \quad \therefore M_n = 39.2 \text{ kft} \quad \checkmark \text{D.K.} \]

Use 2 - No. 7 Hooked Bars
(Closed by a Crosstie)

Greaser of:

\[ \ell_{th} = \frac{f_y f_y f_y}{50} \quad \therefore \ell_{th} = 16.6'' \]
Shear:

\[ V_c = 2 \sqrt{f_c} \times \text{lw} \]

\[ = (2 \sqrt{4000} \times 12 \times 7.5) \]

\[ = 14.9 \text{k} \]

\[ \phi V_n = 0.75 \sqrt{f_c} \times \text{lw} \]

\[ = 0.75 \times \sqrt{4000} \times (12 \times 7.5) \]

\[ = 54.1 \text{k} \]

\[ V_{max} = 11.4 \text{k} \]

\[ V_c < V_n : \text{need shear reinforcement} \]

\[ V_n > V_c : \text{don't need to increase beam width} \]

max spacing = \[ \frac{V_{max}}{\phi} \]

\[ \frac{11.4}{0.75} \approx 15 \text{k} \times (12 \times 7.5) \]

\[ = 43.2 \text{k} \Rightarrow \text{use } \frac{d}{2} \text{, not } d \]

\[ S_{max} = \frac{d}{2} = 9.75 = 4.75'' \]

\[ f_{om} \text{ is greater of: 1) } 0.75 \sqrt{f_c} \times \text{lw} \text{ 2) } 50 \text{ lw} \]

\[ 1) 0.75 \sqrt{4000} \times (12 \times 40,000) = 0.0095 \text{ in}^2/y \text{in} \]

\[ 2) 50 \times (12 \times 40,000) = 0.61 \text{ in}^2/y \text{in} \] (goes over)

\[ A_{min} = (0.01 \text{ in}^2/y)(9.75'') = 0.0476 \text{ in}^2 \Rightarrow \text{No. 3 stirrups} \]

\[ s_{o} = d = \frac{A_{min}}{\phi V_n} = \frac{(0.72 \text{ in}^2)(40)(9.5)}{(0.75)} = 16.71'' \Rightarrow \text{Use } S_{max} = 4.75'' \times 4'' \]

USE NO. 3 STIRRUPS @ 4'' O.C. THROUGHOUT BM.
Final Beam Design (B5):

2 - No. 7's hooked to column support (90° steel bracket)

2 - No. 7's over the supports
2 - No. 6's between supports

No. 3 stirrups @ 4" o.c.
Moment Frame - 2nd Flr Beam (BL)

Design is based on worst case moment frame = MFy ≤ 0.4 ks, 1.5 k ≤ V ≤ 2 k.

Moment Frame Beam Design (BL):

**ETABS**
- **Demands:** Mu = 2.9 kft (over the supports)
  - Mu = 1.8 kft (between the supports)
  - Vr = 12 kft

**Quick Calc.**

- **Longitudinal Steel** (over the supports):
  - Mu = Mf y (d/2)
  - Assumed d = h/2 = 7.5
  - φy = 0.9 %
  - As = 0.814 m²

**Check:**

- **As, min.** = 2.00 b(x) = \( \frac{2.00 \times 12 \times 7.5}{60,000} = 0.381 \text{ in}^2 \)
  - φy = 0.9%
  - As, min. = 0.36 m²

- **Check:**
  - \( a = \frac{xt} {0.85 f_y} = \frac{0.36 \times 12} {0.85 \times 40} = 1.29 \text{ in} \)
  - \( c = \frac{a}{0.85} = 1.52 \text{ in} \)
  - \( d = 12 - 1.5 - 0.5 - \left( \frac{12}{2} \right) = 9.25 \approx 9.5 \text{ in} \)

- \( e_i = \left( 0.003 \left( \frac{d - c}{e} \right) \right) = 0.003 \left( \frac{9.5 - 1.5}{1.52} \right) = 0.01575 \approx 0.004 \)
\[ M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \]
\[ = (0.7)(0.88)(60)(9.5 - \frac{1.27}{2}) = 920.7 \text{ kN}\cdot\text{m} \]
\[ M_n = 35.1 \text{ kft} \times d_n = 2.9 \text{ kft} \checkmark \text{O.K.} \]

**USE 2-NO.6 LONG. BARS OVER SUPPORTS**

**Longitudinal steel (b/w supports):**

\[ M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \]

Let some assumptions as previously stated

\[ A_s = \frac{18(k) + 12\%}{60(0.9)(0.6)(7.5 - \frac{3.44}{2})} = 0.565 \text{ m}^2 \]

\[ \Rightarrow \text{Try } 2 - \text{No.5's, } A_s = 0.67 \text{m}^2 \]

\[ A_{min} = 0.28 \text{m}^2 < A_s \checkmark \]

**Check**

\[ \sigma = \frac{f_y}{0.85f_{ck}} = \frac{0.52(410)}{0.85(410)} = 0.911'' \]

\[ d = 1.07'' \]

\[ d = 9.5'' \]

\[ \varepsilon_s = 0.003 \left( \frac{7.5 - 1.07}{1.07} \right) = 0.022 > 0.004 \checkmark \text{ tens controlled, } \phi = 0.9 \]

\[ M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) = (0.9)(0.62)(60)(9.5 - \frac{0.911}{2}) = 302 \text{ kN}\cdot\text{m} \]

\[ M_n = 25.23 \text{ kft} \times d_n = 18 \text{ kft} \checkmark \text{O.K.} \]

**USE 2-NO.5 LONG. BARS B/W SUPPORTS**
Hooked longitudinal bars & supports:

\[ q = \frac{d = \frac{-2 \mu y}{\phi 0.85 fcL} + d^2}{2} \]

\[ = \frac{9.5}{\sqrt{(0.7 \times 0.85)(4.12) + 9.5^2}} \]

\[ q = 1.06\text{"} \]

\[ A_{re} = \frac{0.85 fc L a}{f_y} = \frac{(0.85)(4.12)(1.06)}{60} = 0.0718\text{ in}^2 \]

Try 2 - No. 6's, \( A_{re} = 0.078\text{ in}^2 \)

Check:

\[ q = \frac{A_{as} f_y}{0.85 fc L} = \frac{(0.98)(L_0)}{(0.85)(4.12)(L_0)} = 1.29\text{"} \]

\[ c = 1.522\text{"} \]

\[ d = 9.5\text{"} \]

\[ s = (0.062)(9.5 - 1.522) = 0.0152 < 0.004 \sqrt{\phi} \text{ tension controlled, } k = 0.9 \]

\[ \phi Mn = \phi f_{as} y (d - \frac{d}{2}) \]

\[ = (0.91)(0.88)(60)(9.5 - 1.522) = 520.6\text{ k"} \]

\[ \phi Mn = 520.6\text{ k"} > \mu y = 29\text{ k"} \]

\[ \text{USE 2 - NO. 6 HOOKED BARS (CLOSED BY A CROSSTIE)} \]

\[ L_0 = 14.23\text{"} \]
\[ V_C = 2 \sqrt{f_y LWd} \]
\[ = (2 \times 1000 \times 12 \times 9.5) \]
\[ = 11.4 \text{k} \]

\[ 4V_C = 11.4 \times 4 \text{k} \]
\[ = 57.6 \text{k} \]

\[ N_{max} = 12 \text{k} ft \]

\[ V_C + V_U > V_U \quad \text{do not need stirrups, reinf} \]
\[ 4V_C > V_U \quad \text{do not need to inc. lum width} \]

\[ 16 \text{k} + 48.2 \text{k} \Rightarrow use \frac{d}{2}, \text{not} \frac{d}{4} \]

\[ V_{max} = \frac{d}{2} = \frac{9.5}{2} = 4.75'' \approx 4'' \]

\[ A_{nom} \text{ is greater of: } 1) 0.0075 f_y LW \quad \text{or} \quad 2) \frac{fy}{f_y} \]
\[ 1) 0.0075 \times 4000 \times 12 \times 9.5 = 0.855'' \text{ in} \]
\[ 2) 5d \times 0.0075 = 0.075'' \text{ in} \]

\[ A_{nom} = (0.02 \times 9.5) = 0.1975'' \text{ in}^2 \Rightarrow \text{No. 3 stirrups, } AV = 0.22'' \text{ in}^2 \]

\[ f_{cd} = \frac{f_{yd}}{f_y} = \frac{10.22 \times 1000 \times 7.5}{12 \times 0.75} = 70.22'' \text{ in}^2\]

\[ f_{cd} - V_C = \left( \frac{12}{0.75} \right) - 11.4 = 14.4 \text{ use } S_{max} = 4'' \]

\[ \text{USE NO. 3 STIRRUPS C4'' O.C. THROUGHOUT BM.} \]
Final Beam Design (B6):

- 2-No. 6's hooked @ column support (90° stud hook)
- 2-No. 6's over the supports
- 2-No. 5's between supports
- 3 stirrups @ 4" o.c.
Design is based on worst case moment frame. MF4 one moment frame will be designed for entire building.

**Moment Frame Column Design (C1):**

**ETABS Analysis**

**Demand**

- P_{HR} = 31.74 k
- P_{LR} = 51.2 k
- M_{LR} = 12.14 k ft
- M_{LR} = 2.05 k ft

**Slenderness Check**

- \( \frac{k_{d}}{c} = \frac{3.4 + 12\left(\frac{M_{LR}}{M_{t}}\right)}{3.4} \)
- Quick check
  - 12" 12" 3.4
  - 23.33 < 36.02 \( \checkmark \) Braced against sidesway

**2nd Flr -> Ground**

- \( k = 1.0 \)
- \( d_u = 11" - 0" \)
- \( c = 3.6" \)
- \( K_d = 3.4 + 12\left(\frac{2.05}{12.14}\right) \)
- \( \frac{1 \times 11.5(12)}{3.6} \leq 36.02 \)
- 28.32 < 36.02 \( \times \) Not braced against sidesway

*Must use moment magnification*
**Moment Frame Column - (C1)**

**Reference**

ACI 318

**Equation**

\[ P_c = \frac{E I_{eff}}{(E I)^2} \]

\[ = \frac{(0.2E I_{eff})}{1 + 0.1E I_{eff}} \]

\[ = \frac{1.13 \times 10^6}{1 + 0.1 \times 0.17 \times 10^6} = 1.13 \times 10^6 \]

\[ b = \frac{C_m}{1 - \frac{P_{a}}{0.75P_c}} \]

\[ = \frac{1.0}{1 - \frac{54.2}{0.75 \times 1.13 \times 10^6}} = 1.13 > 1.0 \]

\[ M_c = 0.8 M_2 = 1.13 \times 2.05 = 2.33 \text{ kft} \]

Check if: \[ M_c < 1.4 M_1 \]

\[ 12.14 \leq (1.4)2.33 \]

\[ 12.14 < 3.25 \]

Design for \( M_{floor} = 2.33 \text{ kft} \)
Moment Frame Column - C1

12 x 12 in

Code: ACI 318-14
Units: English
Run axis: About X-axis
"n" option: Investigation
Tenderness: Not considered
Column type: Architectural
Bars: ASTM A615
Date: 06/16/17
Time: 01:32:18

File: untitled.col
Project:
Column:
\( f_c = 4 \text{ ksi} \)
\( E_c = 3605 \text{ ksi} \)
\( \varepsilon = 3.4 \text{ ksi} \)
\( u = 0.003 \text{ in/in} \)
\( \beta_1 = 0.85 \)
Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Engineer:
\( A_g = 144 \text{ in}^2 \)
\( A_s = 1.60 \text{ in}^2 \)
\( x_0 = 0.00 \text{ in} \)
\( y_0 = 0.00 \text{ in} \)
Min clear spacing = 3.38 in
Clear cover = 1.88 in

8 #4 bars
\( \rho_o = 1.11\% \)
\( l_x = 1728 \text{ in}^4 \)
\( l_y = 1728 \text{ in}^4 \)
General Information:

File Name: untitled.col

Project:
Column:
Code: ACI 318-14
Run Option: Investigation
Run Axis: X-axis

Material Properties:

Concrete: Standard
f'c' = 4 ksi
Ec = 3600 ksi
f'c = 3.4 ksi
Eps_u = 0.003 in/in
Beta1 = 0.05

Steel: Standard
FY = 60 ksi
Ks = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:

Rectangular: Width = 12 in
Depth = 12 in

Gross section area, Ag = 144 in^2
Iy = 1728 in^4
ry = 3.4641 in
Yo = 0 in

Reinforcement:

Bar Set: ASTM A615
Size Diam (in) Area (in^2) Size Diam (in) Area (in^2) Size Diam (in) Area (in^2)
# 3 0.38 0.11 # 4 0.50 0.20 # 5 0.63 0.31
# 6 0.75 0.44 # 7 0.88 0.60 # 8 1.00 0.79
# 9 1.33 1.00 # 10 1.27 1.27 # 11 1.41 1.56
# 14 1.69 2.25 # 18 2.26 4.00

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 1.60 in^2 at rho = 1.11%
Minimum clear spacing = 3.38 in

8 #4 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

<table>
<thead>
<tr>
<th>No.</th>
<th>Pu (kips)</th>
<th>Mx (k-ft)</th>
<th>PhiMax</th>
<th>PhiMn/Nu</th>
<th>NA</th>
<th>depth (in)</th>
<th>depth (in)</th>
<th>eps_t (in)</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.24</td>
<td>12.14</td>
<td>44.91</td>
<td>3.699</td>
<td>9.88</td>
<td>0.00865</td>
<td>0.900</td>
<td>0.00865</td>
<td>0.900</td>
</tr>
<tr>
<td>2</td>
<td>34.26</td>
<td>2.33</td>
<td>51.97</td>
<td>22.303</td>
<td>3.06</td>
<td>9.88</td>
<td>0.00677</td>
<td>0.900</td>
<td>0.900</td>
</tr>
</tbody>
</table>

*** End of output ***
From spec column, the moment frame column will require 8 - No. 4's.

See column group #1 for vertical rein check calculations.

See column group #1 for tie size spacing calculations and shear force checks \( (M_p = 59kft \leq M_p = 81kft) \)

Earthquake Requirements:

Assume elastic capacity of gravity framing is exceeded during an earthquake and some non-linear behavior is required.

Amount of Transverse Req'd:

\[ P_u \leq 0.35 \frac{Ag}{f_y} \]

\[ 5.4.2 \leq (0.3)(12'' \times 12'' \times fyv) \]

\[ 5.4.2 \leq 172.8k \Rightarrow \text{Transverse is greater of a) or b) } \]

a) \[ 0.3 \left( \frac{Ag}{A_{ch}} - \frac{f_y}{f_y} \right) = 0.3 \left( \frac{12^2}{72} - 1 \right) \left( \frac{f_y}{f_y} \right) = 0.015 \text{ in}^2 \]

b) \[ 0.09 \left( \frac{f_y}{f_y} \right) = 0.09 \left( \frac{f_y}{f_y} \right) = 0.006 \text{ in}^2 \]

\[ A_{ch} = 0.015 \]

2'' from column group #1 checks

\[ A_{ch} = (0.015)(2'' \times 12'' - 1.5 - 3/8 - 4/16) = 0.444375 \text{ in}^2 \]

\[ A_{tie} = (3)(No. 4) = (3)(0.20) = 0.60 \text{ in}^2 > A_{ch} \]

\[ USE NO. 4 TIES @ 3'' O.C. (SEISMIC) \]
Beam Check

Dimensional limits:

(a) clear span $d_n$ shall be at least $4d$

$$d_n = 4d = 12.12$$

$$4d = 4 \times (12.5') = 57' = 6.5'$$

Largest $d$ of all beams

$$d_n = 12.12' > 4d = 6.5' \checkmark \text{O.K.}$$

(b) width $bw$ shall be at least the lesser of $0.2h$ and 10"

bw of all beams = 12"

$b$ (largest beam) = 20"

$$0.2h = 0.2 \times 20" = 4" < 10" \text{ governs}$$

$$12" \times 10" \checkmark \text{O.K.}$$

(c) projection of beam width beyond the width of the supporting column on each side shall not exceed the lesser of $c_2$ and $0.75c$.

beam width ($s$) = 12"

$$c_2 = 12"$$

$$0.75c = (0.75 \times 12") = 9"$$

projection width = 0"

$$0" < 9" \checkmark \text{O.K.}$$

Longitudinal Reinforcement:

Beams shall have at least 2 continuous bars @ both top and bottom files. The amount of reinforcement shall be at least that required by § 9.6.1.2 and the reinforcement ratio $f$ shall not exceed 0.025.

* Reference calculations B1-B4, G1-G2 for § 9.6.1.2 reg't.

All beams have at least 2 cont. bars @ top and bottom \checkmark \text{O.K.}
Lap splices should not be used in locations:

1. Within the joint.
2. Within a distance of twice the beam depth from the face of the joint.
3. Within a distance of twice the beam depth from critical sections where flexural yielding is likely to occur as a result of internal displacements beyond the elastic range of behavior.

ld for straight bars:

- No. 3 - ld = 3"
- No. 4 - ld = 2"
- No. 5 - ld = 3"
- No. 6 - ld = 9"
- No. 7 - ld = 10.5"
- No. 8 - ld = 12"
- No. 9 - ld = 3"
- No. 10 - ld = 2"
- No. 11 - ld = 3.75"
- No. 14 - ld = 4.5"
- No. 18 - ld = 5.25"
Transverse Reinforcement:

Hoops shall be provided in the following regions:

(a) over a length equal to twice the beam depth measured from the face of the supporting column toward midspan at both ends of the beam.

(b) over lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur as a result of lateral displacement beyond the elastic range behavior.

Diagram:

- Hoop spacing shall not exceed:
  
  (a) \( \frac{d}{4} \)

  \[ d \left( \frac{B_2, b_2, b_2}{4} \right) = 9.5/4 = 2.375'' = 2'' \]

  \[ d \left( \text{left} \right) = 17.5''/4 = 4.375'' = 4'' \]

  (governs)

(b) \( \frac{d}{12} \)

- Smallest primary flexural rein. reqd. 101

  \[ \left( \frac{d}{2} \right) \text{nom.} = \left( \frac{d}{2} \right) \text{nom.} = 4.5'' \]

(c) 6''
Column Check

All columns were calculated in accordance to § 18.17 - columns of special moment frames.

Joints of special moment frames

8.6.2.4

Depth $h$ of the joint shall not be less than $\frac{1}{2}$ beam framing into the joint system.

Beams: B3, B4, and Girder G2 - 12" x 12"

Girder C1 - 12" x 20"

$h = \left(\frac{1}{2}(12)\right) = 6" \leq 12" \checkmark \text{O.K.}$

$h = \left(\frac{1}{2}(20)\right) = 10" \leq 12" \checkmark \text{O.K.}$

18.8.4.3

Effective joint width shall not exceed the lesser of:

1. Beam width plus joint depth

   Beam width = 12"
   Joint Depth = 12"

   Effective joint width = 12" + 2.4" = 14.4" \checkmark \text{O.K.}

2. 2x's the smaller perpendicular dist. from the longitudinal axis of beam to column side = 12"
Development length of hooks:

\[ L_d \] shall be at least the greater of:

(a) \( 8d \)

\[ b(No.8) = 8 \left( \frac{\sqrt{8}}{\pi} \right) = 6'' \]

(b) \( 6'' \)

(c) \[ L_d = \frac{f_y dB}{6s f_t e} \]

\[ = \frac{(60,000)(1.5)}{(6)(3,900)} \]

\[ = 10.94'' \approx 11'' \]

Note: The hook shall be located within the confined core of a column, with the hook bent into the joint.
Load Combo: $3940 + Q = L$

Values are obtained from ETABS model.

Column design is based on worst case loading.

Footprint 1 - F1

Based on column group #1 - C1 (12" x 12")

$A_t = 9719\text{sf}$

Pulprf = 116.47k

Pullf = 116.91k

Pullf + Pulprf = 33.55k

Footprint 2 - F2

Based on column group #2 - C2 (12" x 12")

Pull = 24.8k

Pulprf = 23.2k

Pulprf + Pull = 98k

Footprint 3 - F3

Based on column group #3 - C3 (12" x 12")

ETABS values < Gravity design values. Gravity values govern.

Pull = $0.1 psf(114.4 sf) = 13.1k$

Pulprf = $[(1.2)(1.0) + (1.0)(2.0)](141.4) = 16.9k$

Footprint 4 - F4

Based on column group #4 - C4 (1.55 2x3 x 2/16)

$A_t = 290\text{sf}$

Pull = $(81 psf)(290 \text{sf}) = 23.9k$

Pulprf = $[(1.2)(1.0) + (1.0)(2.0)](290 \text{sf}) = 30.5k$
To simplify column schedule, columns with same reinforcements will be grouped:

\[ C_1, C_4, C_5, C_6 \Rightarrow \text{MF COL} = F_1 \]
\[ C_2 = F_2 \]
\[ C_3 = F_3 \]
\[ C_7, C_8 = F_4 \]

After further analysis, all moment frame columns will also be grouped under \( C \), denoted above.
### COLUMN PROPERTIES

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>61 psf</td>
</tr>
<tr>
<td>LL</td>
<td>20 psf</td>
</tr>
<tr>
<td>$f_c$</td>
<td>4 ksi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>60 ksi</td>
</tr>
<tr>
<td>$q_u$</td>
<td>2 ksi</td>
</tr>
<tr>
<td>M (k-ft)</td>
<td>0 (concentric)</td>
</tr>
<tr>
<td>$P_u$</td>
<td>33.5 kips</td>
</tr>
<tr>
<td>$d_{col}$</td>
<td>12 inches</td>
</tr>
</tbody>
</table>

### TWO-WAY (PUNCHING) SHEAR

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>1 (square column)</td>
</tr>
<tr>
<td>$V_u$</td>
<td>27.68 kips</td>
</tr>
<tr>
<td>$b_0$</td>
<td>80 inches</td>
</tr>
<tr>
<td>$\phi V_c$ 1</td>
<td>182.15 kips</td>
</tr>
<tr>
<td>$\alpha_s$</td>
<td>30 (exterior column)</td>
</tr>
<tr>
<td>$\phi V_c$ 2</td>
<td>151.79 kips</td>
</tr>
<tr>
<td>$\phi V_c$ 3</td>
<td>121.43 kips</td>
</tr>
</tbody>
</table>

**SMALLEST $V_c$ GOVERNS**

**CHECK:** $V_u < \phi V_c$

### FLEXURAL DESIGN

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_u$</td>
<td>2.36 k-ft/ft</td>
</tr>
<tr>
<td>$I_f$</td>
<td>7.2 inches</td>
</tr>
<tr>
<td>Assume $\phi$</td>
<td>0.9</td>
</tr>
<tr>
<td>$A_s_{min}$</td>
<td>0.14 in$^2$/ft</td>
</tr>
<tr>
<td>$A_s$</td>
<td>0.07 in$^2$/ft</td>
</tr>
</tbody>
</table>

**REQUIRED REINFORCEMENT**

$A_s = 0.14 in^2/ft$

### FOOTPRINT OF FTG

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{FTG}$</td>
<td>16.75 ft$^2$</td>
</tr>
<tr>
<td>$L = W$</td>
<td>4.09 ft</td>
</tr>
</tbody>
</table>

**TRY 4'-0" SQ FTG**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L = W$</td>
<td>4 ft</td>
</tr>
<tr>
<td>$q_u{\text{Actual}}$</td>
<td>2.09 ksf</td>
</tr>
</tbody>
</table>

### ONE-WAY BEAM SHEAR

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{min}$</td>
<td>2.39 inches</td>
</tr>
<tr>
<td>$V_u$</td>
<td>2.72 kips</td>
</tr>
<tr>
<td>$\phi$</td>
<td>0.75</td>
</tr>
<tr>
<td>$\phi V_c$</td>
<td>2.72 kips</td>
</tr>
<tr>
<td>$h$</td>
<td>6.39 &gt; 6&quot; minimum</td>
</tr>
</tbody>
</table>

**TRY $H = 12"$**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover</td>
<td>3 inches</td>
</tr>
<tr>
<td>$d = h - (3&quot; \text{ cover} + 1&quot;)$</td>
<td>8 inches</td>
</tr>
<tr>
<td>$\phi V_c$</td>
<td>9.11 kips</td>
</tr>
<tr>
<td>$V_u$</td>
<td>1.74 kips</td>
</tr>
</tbody>
</table>

**CHECK:** $V_u < \phi V_c$

### 4' SQ FTG W/ #4 @ 12" C.C. EACH WAY

$H=18"$ min
**COLUMN PROPERTIES**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>61 psf</td>
</tr>
<tr>
<td>LL</td>
<td>20 psf</td>
</tr>
<tr>
<td>$f_c$</td>
<td>4 ksi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>60 ksi</td>
</tr>
<tr>
<td>$q_u$</td>
<td>2 ksf</td>
</tr>
<tr>
<td>$M$ (k-ft)</td>
<td>0 (concentric)</td>
</tr>
<tr>
<td>$P_u$</td>
<td>48 kips</td>
</tr>
<tr>
<td>$d_{col}$</td>
<td>12 inches</td>
</tr>
</tbody>
</table>

**ECCENTRICITY**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e = M/P$</td>
<td>0 ft</td>
</tr>
<tr>
<td>$L/6$</td>
<td>0.8164966 ft</td>
</tr>
<tr>
<td>$q_{min}$</td>
<td>1.92 ksf</td>
</tr>
<tr>
<td>$q_{max}$</td>
<td>1.92 ksf</td>
</tr>
</tbody>
</table>

**FOOTPRINT OF FTG**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{FTG}$</td>
<td>24 ft$^2$</td>
</tr>
<tr>
<td>$L = W$</td>
<td>4.90 ft</td>
</tr>
</tbody>
</table>

**ONE-WAY BEAM SHEAR**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{min}$</td>
<td>2.96 inches</td>
</tr>
<tr>
<td>$V_u$</td>
<td>3.37 kips</td>
</tr>
<tr>
<td>$\phi$</td>
<td>0.75</td>
</tr>
<tr>
<td>$\phi_{Vc}$</td>
<td>3.37 kips</td>
</tr>
<tr>
<td>$h$</td>
<td>6.96 &gt; 6&quot; minimum</td>
</tr>
</tbody>
</table>

**TWO-WAY (PUNCHING) SHEAR**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>1 (square column)</td>
</tr>
<tr>
<td>$V_u$</td>
<td>42.67 kips</td>
</tr>
<tr>
<td>$d_e$</td>
<td>80 inches</td>
</tr>
<tr>
<td>$\phi_{Vc}$</td>
<td>182.15 kips</td>
</tr>
<tr>
<td>$a_e$</td>
<td>30 (exterior column)</td>
</tr>
<tr>
<td>$\phi_{Vc}$</td>
<td>151.79 kips</td>
</tr>
<tr>
<td>$h$</td>
<td>121.43 kips</td>
</tr>
</tbody>
</table>

**SMALLEST $V_c$ GOVERNS**

**FLEXURAL DESIGN**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_u$</td>
<td>3.84 k-ft/ft</td>
</tr>
<tr>
<td>If $(d-a)/2 = 0.9d$</td>
<td>7.2 inches</td>
</tr>
<tr>
<td>Assume $\phi = 0.9$</td>
<td>0.9</td>
</tr>
<tr>
<td>$A_{sm}$</td>
<td>0.15 in$^2$/ft</td>
</tr>
<tr>
<td>$A_s$</td>
<td>0.12 in$^2$/ft</td>
</tr>
</tbody>
</table>

**REQUIRED REINFORCEMENT**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$</td>
<td>0.2 in$^2$/ft</td>
</tr>
</tbody>
</table>

**REINFORCEMENT**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Checks:</td>
<td></td>
</tr>
<tr>
<td>1) $a &lt; a_{assumed}$</td>
<td>1.6 inches</td>
</tr>
<tr>
<td>$a_{assumed}$</td>
<td>0.0588235 inches</td>
</tr>
<tr>
<td>$a$</td>
<td>As is O.K.</td>
</tr>
<tr>
<td>2) $\epsilon_s$</td>
<td>0.021</td>
</tr>
<tr>
<td>$c = a$</td>
<td>0.0588235 inches</td>
</tr>
<tr>
<td>$\epsilon_s &gt; 0.005$, TENSION CONTROLLED</td>
<td>$\phi = 0.9$</td>
</tr>
</tbody>
</table>

**5' SQ FTG W/ #4 @ 12" C.C. EACH WAY**

**CHECK: $V_u < \phi_{Vc}$**

$H = 18"$ min
**COLUMN PROPERTIES**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>61 psf</td>
</tr>
<tr>
<td>LL</td>
<td>20 psf</td>
</tr>
<tr>
<td>f'c</td>
<td>4 ksi</td>
</tr>
<tr>
<td>fy</td>
<td>60 ksi</td>
</tr>
<tr>
<td>qu</td>
<td>2 ksf</td>
</tr>
<tr>
<td>M (k-ft)</td>
<td>0 (concentric)</td>
</tr>
<tr>
<td>P (ASD)</td>
<td>13.1 kips</td>
</tr>
<tr>
<td>P (LRFD)</td>
<td>16.9 kips</td>
</tr>
<tr>
<td>d&lt;sub&gt;col&lt;/sub&gt;</td>
<td>12 inches</td>
</tr>
</tbody>
</table>

**TWO-WAY (PUNCHING) SHEAR**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>square column</td>
</tr>
<tr>
<td>Vu</td>
<td>11.68 kips</td>
</tr>
<tr>
<td>( \delta_0 )</td>
<td>80 inches</td>
</tr>
<tr>
<td>( \phi_{Vc} ) 1</td>
<td>182.15 kips</td>
</tr>
<tr>
<td>( \phi_{Vc} ) 2</td>
<td>30 (exterior column)</td>
</tr>
<tr>
<td>( \phi_{Vc} ) 3</td>
<td>151.79 kips</td>
</tr>
<tr>
<td>( \phi_{Vc} ) 4</td>
<td>121.43 kips</td>
</tr>
</tbody>
</table>

**SMALLEST \( \phi_{Vc} \) GOVERNS**

**CHECK:** \( Vu < \phi_{Vc} \)

**ECCENTRICITY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>e = M/P</td>
<td>0 ft</td>
</tr>
<tr>
<td>L/6</td>
<td>0.4265495 ft</td>
</tr>
</tbody>
</table>

**FOOTPRINT OF FTG**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A FTG</td>
<td>6.55 ft&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>L = W</td>
<td>2.56 ft</td>
</tr>
<tr>
<td>qu&lt;sub&gt;min&lt;/sub&gt;</td>
<td>1.4555556 ksf</td>
</tr>
<tr>
<td>qu&lt;sub&gt;max&lt;/sub&gt;</td>
<td>1.4555556 ksf</td>
</tr>
</tbody>
</table>

**TRY 3' 0" SQ FTG**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L = W</td>
<td>3 ft</td>
</tr>
<tr>
<td>d&lt;sub&gt;quActual&lt;/sub&gt;</td>
<td>1.88 ksf</td>
</tr>
</tbody>
</table>

**ONE-WAY BEAM SHEAR**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d&lt;sub&gt;min&lt;/sub&gt;</td>
<td>1.45 inches</td>
</tr>
<tr>
<td>Vu</td>
<td>1.65 kips</td>
</tr>
<tr>
<td>( \phi )</td>
<td>0.75</td>
</tr>
<tr>
<td>( \phi_{Vc} )</td>
<td>1.66 kips</td>
</tr>
<tr>
<td>( h )</td>
<td>5.46 &gt; 6&quot; minimum</td>
</tr>
<tr>
<td>TRY H = 12&quot;</td>
<td>12 (increments of 3&quot;)</td>
</tr>
</tbody>
</table>

**CHECK: Vu < \phi_{Vc}**

**REINFORCEMENT**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>As</td>
<td>0.12 in&lt;sup&gt;2&lt;/sup&gt;/ft</td>
</tr>
</tbody>
</table>

**REQUIRED REINFORCEMENT**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>As&lt;sub&gt;min&lt;/sub&gt;</td>
<td>0.12 in&lt;sup&gt;2&lt;/sup&gt;/ft</td>
</tr>
<tr>
<td>As</td>
<td>0.03 in&lt;sup&gt;2&lt;/sup&gt;/ft</td>
</tr>
</tbody>
</table>

**REINFORCEMENT**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 @ 12&quot; C.C. EACH WAY</td>
<td></td>
</tr>
<tr>
<td>Checks:</td>
<td></td>
</tr>
<tr>
<td>1) ( a &lt; a_{\text{assumed}} )</td>
<td></td>
</tr>
<tr>
<td>a&lt;sub&gt;assumed&lt;/sub&gt;</td>
<td>1.6 inches</td>
</tr>
<tr>
<td>a</td>
<td>0.0980392 inches</td>
</tr>
<tr>
<td>As is O.K.</td>
<td></td>
</tr>
<tr>
<td>2) ( \varepsilon_s )</td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_s )</td>
<td>0.02</td>
</tr>
<tr>
<td>c</td>
<td>0.0980392 inches</td>
</tr>
<tr>
<td>( \varepsilon_s &gt; 0.005 ), TENSION CONTROLLED</td>
<td></td>
</tr>
</tbody>
</table>

**3' SQ FTG W/ #4 @ 12" C.C. EACH WAY**

\( h = 18" \text{ min} \)
### COLUMN PROPERTIES

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>61 psf</td>
</tr>
<tr>
<td>LL</td>
<td>20 psf</td>
</tr>
<tr>
<td>f_c</td>
<td>4 ksi</td>
</tr>
<tr>
<td>f_y</td>
<td>60 ksi</td>
</tr>
<tr>
<td>q_u</td>
<td>2 ksi</td>
</tr>
<tr>
<td>M (k-ft)</td>
<td>0 (concentric)</td>
</tr>
<tr>
<td>P (ASD)</td>
<td>23.5 kips</td>
</tr>
<tr>
<td>P (LRFD)</td>
<td>30.5 kips</td>
</tr>
<tr>
<td>d_col</td>
<td>12 inches</td>
</tr>
</tbody>
</table>

### TWO-WAY (PUNCHING) SHEAR

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>β</td>
<td>1 (square column)</td>
</tr>
<tr>
<td>V_u</td>
<td>25.20 kips</td>
</tr>
<tr>
<td>d_o</td>
<td>80 inches</td>
</tr>
<tr>
<td>d_s</td>
<td>30 (exterior column)</td>
</tr>
<tr>
<td>Vc_1</td>
<td>182.15 kips</td>
</tr>
<tr>
<td>Vc_2</td>
<td>151.79 kips</td>
</tr>
<tr>
<td>Vc_3</td>
<td>121.43 kips</td>
</tr>
</tbody>
</table>

**SMALLEST Vc GOVERS**

CHECK: V_u < Vc

### FLEXURAL DESIGN

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu</td>
<td>2.14 k-ft/ft</td>
</tr>
<tr>
<td>d</td>
<td>7.2 inches</td>
</tr>
<tr>
<td>φ</td>
<td>0.9</td>
</tr>
<tr>
<td>A_s_min</td>
<td>0.13 in²/ft</td>
</tr>
<tr>
<td>A_s</td>
<td>0.07 in²/ft</td>
</tr>
</tbody>
</table>

**REQUIRED REINFORCEMENT**

A_s = 0.13 in²/ft

### REINFORCEMENT

A_s = 0.2 in²/ft

#4 @ 12" C.C. EACH WAY

Checks:

1. a < a assumed
   - a assumed = 1.6 inches
   - a = 0.0735294 inches
   - A_s is O.K.

2. ε_s < 0.005, TENSION CONTROLLED
   - ε_s = 0.021
   - c = a
   - c = 0.0735294 inches

### FOOTPRINT OF FTG

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_FTG</td>
<td>11.75 ft²</td>
</tr>
<tr>
<td>L x W</td>
<td>3.43 ft</td>
</tr>
</tbody>
</table>

TRY 4'-0" SQ FTG

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L = W</td>
<td>4 ft</td>
</tr>
<tr>
<td>q_uActual</td>
<td>1.91 ksf</td>
</tr>
</tbody>
</table>

### ONE-WAY BEAM SHEAR

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d_min</td>
<td>2.20 inches</td>
</tr>
<tr>
<td>V_u</td>
<td>2.51 kips</td>
</tr>
<tr>
<td>φ</td>
<td>0.75</td>
</tr>
<tr>
<td>Vc</td>
<td>2.51 kips</td>
</tr>
<tr>
<td>h</td>
<td>12 (increments of 3&quot;)</td>
</tr>
</tbody>
</table>

TRY H = 12"

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>cover</td>
<td>3 inches</td>
</tr>
<tr>
<td>d</td>
<td>8 inches</td>
</tr>
<tr>
<td>φ Vc</td>
<td>9.11 kips</td>
</tr>
<tr>
<td>V_u</td>
<td>1.59 kips</td>
</tr>
</tbody>
</table>

**CHECK: V_u < φ Vc**

4' SQ FTG W/ #4 @ 12" C.C. EACH WAY

H = 18" min
Steel Deck to Steel Truss:

3/4" Bolt, 24'0.06

3" x 18 Gauge Galvanized Convoluted Steel Decking (N Beck - Type PLN3)

Top Chord of Truss - HEC 4.4 x 5/16

Lateral loading

\[ V = 61.863 \text{ ft} = 61.9 \text{ k} \]

\[ L = 2.5 \text{ m} \times 3.23 \text{ ft/m} = 82 \text{ ft} \]

\[ V = 61.9 \text{ k} \times \frac{82 \text{ ft}}{82 \text{ ft}} = 0.754 \text{ kL} \]

Truss width = 5 m x 3.23 ft/m = 16.2'

Shear per bolt = \((0.154 \text{ kL})(16.2') = 12.36 \text{ k} \)

\( \phi V_n = 17.9 \text{ k} \) ️  \( V_n = 12.36 \text{ k} \) ️ D.K.

USE 3/4" DIAM. BOLTS TO CONNECT DECKING TO TRUSS.
Steel decking to HSS beam:

Reference "Steel Decking to Steel Truss" for configuration.

\[ V = 35,228 \text{#} = 35.2 \text{k} \]
\[ L = 17.24 \text{ft} \]
\[ u' = 35.2 \text{k} \]
\[ u = 17.24 \text{ft} \]

Tub width = 8.23'

Steel per bolt = \((0.523)(8.23) = 4.36 \text{k}\)

Try \(\frac{5}{8}''\) diam bolts, @ 24° 10'

Available shear = 12.4 K

\[ fV_a = 12.4 \text{ K} \]
\[ Vu = 4.36 \text{ K} \]

USE 5/8'' DIA M BOLTS TO CONNECT STEEL DECKING HSS BEAMS.
**Steel Truss to Concrete Beam:**

- **HSS 3x3 x ½” Truss Web**
- **E10XX Weld**
- **HSS 3x2 x ¾” Bottom Chord**
- **3/4” φ 6 bolts**
- **12” x 12” Concrete Grider**
- **Steel Plate**

**Calculation:**

- $V = 61.56 \times 3.4 = 61.9\text{k}$
- $L = 82\text{ft}$
- $V = 61.9\text{k} \div 82\text{ft} = 0.751\text{ kFt}$
- **Tensile capacity = 14.4**
- **Shear per bolt = (0.75 \times 14.4) = 12.3\text{ kFt}**
- **Try 3/4” diam bolts**
- **Available shear = 17.9\text{k}**

$\delta V = 17.9\text{k} \div \sqrt{V} = 12.3\text{k} \checkmark 0.1$

**USE 3/4” DIAM. BOLTS TO CONNECT STEEL TRUSS TO CONCRETE BM.**
Steel Beam to Masonry Wall / Concrete Column:

HSS Joint

Steel plate

Fillet weld (CJP)

5/8" A325

Concrete column

Masonry wall

ETABS Analysis

\[ v = 35.22 \text{ kips} = 35.2 \text{ k} \]

\[ L = 6.724 \text{ ft} \]

\[ v = 35.2 \text{ k} = 0.523 \text{ kip/ft} \]

\[ \frac{6.724}{35.6} \text{ ft} \]

- Tab width = 8.28

- Shear per bolt = (0.523 x 8.33) = 4.34 k

- Try 5/8" dia bolts

- Available shear = 12.4 k

- \[ 4w = 12.4 \text{ k} > \sqrt{u} = 4.34 \text{ k} \]

- O.K.

**USE 5/8" DIAM. BOLTS TO CONNECT ANGLES TO MASONRY WALL**
Steel Decking / HSS Beams to Concrete Beam:

Concrete Beam

6 Bolts @ 24" O.C.

Steel Plate

Steel Decking

HSS Beam

Masonry Wall

Reference: "Steel Decking to HSS Beam" for connection information.

ETABS Analysis

(Floor View)

V = 25.2 k

L = 67.24 ft

V' = 25.2 k / 67.24 ft

Tub width = 8.33'

Shear per bolt = (8.33' x 0.523 k/ft) = 4.34 k

Try 5/8" diam bolts.

Available shear = 12.4 k

$V_n = 12.4 k \geq V_n = 4.34 k \checkmark$

USE 5/8" DIAM BOLTS TO CONNECT HSS BEAM / STEEL DECKING TO FACE OF CONCRETE BEAM.
CONNECTION: Concrete frame to masonry wall

Concrete beam/girder

Concrete beam/girder

Horizontal expansion joint

Gap = Story drift

0.247" min

Masonry wall

Section

Plan

As 3700
Max. Joint Movement = 10mm
Max. Joint spacing = 2m

ACI T.25-4.2.2

$Le = \left( \frac{fy le}{25} \right) \frac{1}{c}$

$= \left( \frac{160 ksi \times 12 (1/16)}{25 \times 1.0 \sqrt{1000 \, ksi}} \right)$

$Le = 18.8"$

⇒ #4 hooks in horizontal expansion joint

S = 8" O.C.

Le = 18.8"

Concrete column to masonry wall

Masonry wall

Column

Gap

Gap is filled solely w/ sealant as a connection between the two elements so the masonry wall won't affect the moment frame (lateral system)

Gap: Story drift

$g = 0.247" \text{ min}$
GENERAL NOTES

1. ALL NEW CONSTRUCTION SHALL COMPLY WITH THE CONTRACT DOCUMENTS AND THE CURRENT EDITION OF THE 2015 IBC.

2. ALL DRAWINGS AND SPECIFICATIONS SHOW THE LATEST EDITION OF THE CODES, RULES, REGULATIONS, STANDARDS, MANUFACTURER’S INSTRUCTIONS OR REQUIREMENTS OF REGULATORY AGENCIES IN EFFECT AT THE DATE OF SUBMISSION OF BID UNLESS THE DOCUMENT DATE IS SHOWN.

3. REFERENCE TO CODES, RULES, REGULATIONS, STANDARDS, MANUFACTURER’S INSTRUCTIONS OR REQUIREMENTS OF REGULATORY AGENCIES IS TO THE LATEST PRINTED EDITION OF EACH IN EFFECT AT THE DATE OF SUBMISSION OF BID UNLESS THE DOCUMENT DATE IS SHOWN.

4. TYPICAL DETAILS AND GENERAL NOTES APPLY TO ALL PARTS OF THE WORK EXCEPT WHERE SPECIFICALLY DETAILED OR UNLESS NOTED OTHERWISE.

5. REFER TO ARCHITECTURAL DRAWINGS FOR FLOOR DEPRESSIONS, EDGE OF SLAB, OPENINGS, SLOPES, DRAINS, CURBS, PADS, EMBEDDED ITEMS, NON-WELDING OF REINFORCING STEEL SHALL BE PERFORMED BY WELDERS SPECIFICALLY CERTIFIED FOR REINFORCING STEEL. USE E90XX ELECTRODES.

6. REFER TO MECHANICAL AND ELECTRICAL DRAWINGS FOR SLEEVES, OPENINGS, AND HANGERS FOR PIPES, DUCTS AND BEARING PARTITIONS, ETC.

7. KEY AND DOWEL POUR JOINTS AS SHOWN ON THE PLANS. ANY DEVIATION FROM POUR JOINTS SHOWN ON THE PLANS MUST BE APPROVED BY THE OWNER’S REPRESENTATIVE.

8. DEFECTIVE CONCRETE (VOIDS, ROCK POCKETS, HONEYCOMBS, CRACKING, ETC.) SHALL BE REMOVED AND REPLACED AS DIRECTED BY THE OWNER’S REPRESENTATIVE.

9. CONTRACTOR SHALL CAREFULLY REVIEW THE DRAWINGS TO IDENTIFY THE SCOPE OF WORK REQUIRED, VISIT THE SITE TO RELATE THE SCOPE OF WORK TO THE SITE, AND DRAW CONCLUSIONS AS TO WHETHER THE WORK IS AS PERMITTED AND IN ACCORDANCE WITH THE CONTRACT DOCUMENTS, BEFORE PROCEEDING WITH THE WORK.

10. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES, AND GUIDES REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE, SIDEWALKS, AND OTHERS MAY BE SUBJECTED TO DURING AND/OR AFTER CONSTRUCTION.

11. THE CONTRACTOR SHALL RESOLVE ANY CONFLICTS ON THE DRAWINGS OR IN THE SPECIFICATIONS WITH THE OWNER’S REPRESENTATIVE BEFORE PROCEEDING WITH THE WORK.

12. REVIEW OF THE SHOP DRAWINGS SHALL NOT BE CONSTRUED AS AN AUTHORIZATION TO DEVIATE FROM CONTRACT DOCUMENTS.

13. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES, AND GUIDES REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE, SIDEWALKS, AND OTHERS MAY BE SUBJECTED TO DURING AND/OR AFTER CONSTRUCTION.

14. STRUCTURAL OBSERVATIONS PERFORMED BY ENGINEER DURING CONSTRUCTION ARE NOT THE CONTINUOUS AND SPECIAL INSPECTION SERVICES PERMITTED BY THE CONTRACT DOCUMENTS, NOR IS THE ENGINEER RESPONSIBLE FOR THE CORRECT COMPLETION OF THE WORK AS PERMITTED BY THE CONTRACT DOCUMENTS.

15. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES, AND GUIDES REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE, SIDEWALKS, AND OTHERS MAY BE SUBJECTED TO DURING AND/OR AFTER CONSTRUCTION.

16. STRUCTURAL OBSERVATIONS PERFORMED BY ENGINEER DURING CONSTRUCTION ARE NOT THE CONTINUOUS AND SPECIAL INSPECTION SERVICES PERMITTED BY THE CONTRACT DOCUMENTS, NOR IS THE ENGINEER RESPONSIBLE FOR THE CORRECT COMPLETION OF THE WORK AS PERMITTED BY THE CONTRACT DOCUMENTS.

17. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES, AND GUIDES REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE, SIDEWALKS, AND OTHERS MAY BE SUBJECTED TO DURING AND/OR AFTER CONSTRUCTION.

18. REVIEW OF THE SHOP DRAWINGS SHALL NOT BE CONSTRUED AS AN AUTHORIZATION TO DEVIATE FROM CONTRACT DOCUMENTS.

19. SHOP DRAWINGS WILL NOT BE PROCESSED DUE TO INCOMPLETENESS, LACK OF COORDINATION WITH RESPECT TO CERTAIN ITEMS OR TO ADDRESS SPECIFIC ITEMS AS REQUIRED PER THE CONTRACT DOCUMENTS.

20. ALLOW FOURTEEN WORKING DAYS FOR PROCESSING SHOP DRAWINGS AFTER RECEIPT.
CORNER REINFORCING AT CONCRETE WALLS AND FOOTINGS

CORNER BARS MAY BE USED IN LIEU OF SINGLE BEND

LAP

SINGLE LAYER BARS

LAP TYP. AT SPLICE

2 ADDITIONAL TYP. VERTICAL

DOUBLE LAYER BARS

STANDARD HOOK DETAILS FOR PRINCIPAL REINFORCEMENT

STANDARD HOOK DETAILS FOR STIRRUPS & TIES

TYPICAL REINFORCEMENT DETAILS

CONCRETE MEMBER PER SCHEDULE

NOTE: 1. LE (TOP) NOT SHOWN FOR CLARITY
2. LE (FRONT)

TYPICAL SPLICE AND SCHEDULE

STANDARD HOOK DETAILS FOR TENSION BARS

Development Length for Tension Bars

Hook Type

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Min. Bend Dia. (in)</th>
<th></th>
<th>Hook Type</th>
<th>Bar Size</th>
<th>Min. Bend Dia. (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90°</td>
<td>3</td>
<td>0.75</td>
<td>3</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>4</td>
<td>0.90</td>
<td>4</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>5</td>
<td>1.25</td>
<td>5</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>6</td>
<td>1.60</td>
<td>6</td>
<td>1.60</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>7</td>
<td>2.00</td>
<td>7</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>8</td>
<td>2.40</td>
<td>8</td>
<td>2.40</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>9</td>
<td>2.80</td>
<td>9</td>
<td>2.80</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>10</td>
<td>3.20</td>
<td>10</td>
<td>3.20</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>11</td>
<td>3.60</td>
<td>11</td>
<td>3.60</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>12</td>
<td>4.00</td>
<td>12</td>
<td>4.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>13</td>
<td>4.50</td>
<td>13</td>
<td>4.50</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>14</td>
<td>5.00</td>
<td>14</td>
<td>5.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>15</td>
<td>5.50</td>
<td>15</td>
<td>5.50</td>
<td></td>
</tr>
</tbody>
</table>

STANDARD HOOK DETAILS FOR TENSION BARS

Development Length for Tension Bars

Hook Type

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Min. Bend Dia. (in)</th>
<th></th>
<th>Hook Type</th>
<th>Bar Size</th>
<th>Min. Bend Dia. (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90°</td>
<td>3</td>
<td>0.75</td>
<td>3</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>4</td>
<td>0.90</td>
<td>4</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>5</td>
<td>1.25</td>
<td>5</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>6</td>
<td>1.60</td>
<td>6</td>
<td>1.60</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>7</td>
<td>2.00</td>
<td>7</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>8</td>
<td>2.40</td>
<td>8</td>
<td>2.40</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>9</td>
<td>2.80</td>
<td>9</td>
<td>2.80</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>10</td>
<td>3.20</td>
<td>10</td>
<td>3.20</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>11</td>
<td>3.60</td>
<td>11</td>
<td>3.60</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>12</td>
<td>4.00</td>
<td>12</td>
<td>4.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>13</td>
<td>4.50</td>
<td>13</td>
<td>4.50</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>14</td>
<td>5.00</td>
<td>14</td>
<td>5.00</td>
<td></td>
</tr>
<tr>
<td>90°</td>
<td>15</td>
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STANDARD HOOK DETAILS FOR TENSION BARS

Development Length for Tension Bars

Hook Type

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Min. Bend Dia. (in)</th>
<th></th>
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</table>
FOR SUBBASE UNDER SLAB SEE GEOTECHNICAL REPORT AND SPECS. TYP. 1/4" TYP. FOR SLAB THICKNESS SEE PLAN.

REBAR OR WWF SEE PLAN TYP. 1 1/2"

NOTE: CLR. SLAB REINFORCING SEE PLAN TYP.

COLD JOINT STOP REINF AT JOINT.

#4 DIA x 2' - 0" LONG PLAIN DOWEL @ 18" o.c.

GREASE ONE SIDE 12" 12" 12" 12" 12" "T"/2 "T" "T"/2

CONTROL JOINTS TO LOCATED AT COLUMN CENTER LINES AND AT 20' - 0" O.C. MAX. AND EVERY 400 SQUARE FEET.

IF SAW-CUT CONTROL JOINT TO BE USED, SAW-CUT WITHIN 24 HOURS OF POUR.

CUT 1/2 REINF. ACROSS CONTROL JOINT AS REQUIRED TO CLEAR STRIP OR SAW CUT DEPRESS REINFORCEMENT UNDER JOINT AS REQ. TO CLEAR STRIP OR SAW CUT

SEE PLAN FOR "T".

NOTES:
SEE PLAN FOR SLAB REINF. (TYP., U.N.O.)

1/8" WIDE x T/4 DEEP PRE FORMED STRIP OR SAW CUT THE SAME DAY OF THE POUR IF SAW-CUT CONTROL JOINT TO BE USED, SAW-CUT WITHIN 24 HOURS OF POUR.

SLAB "T" & REINF. SEE PLAN

FINISH FLOOR #4 CONT. EXTEND ALL AROUND

CUT 1/2 REINF. ACROSS CONTROL JOINT AS REQUIRED TO CLEAR STRIP OR SAW CUT DEPRESS REINFORCEMENT UNDER JOINT AS REQ. TO CLEAR STRIP OR SAW CUT

CONCRETE COLUMN #3 @ 12" O.C.

SIZE TO MATCH SLAB REINF.

PAINT JOINT w/ BOND BREAKER FOR FOOTING SIZE AND REINFORCING SEE PLAN OR SCHEDULE.

CONSTRUCTION JOINT

NOTE:
1. CONTROL JOINTS TO LOCATED AT COLUMN CENTER LINES AND AT 20' - 0" O.C. MAX. AND EVERY 400 SQUARE FEET.
2. IF SAW-CUT CONTROL JOINT TO BE USED, SAW-CUT WITHIN 24 HOURS OF POUR.

FINISH GRADE 1' - 6" MIN.

REBAR OR WWF SEE PLAN @ 24" O.C.

NEW FLOOR #4 CONT. CREATE A 6" DEPRESSION END OF JOINTS

SLAB JOINT AT CONCRETE COLUMN #4 TOP & BOTTOM.

SLAB JOINT TYP.

TYPICAL SLAB ON GRADE DETAILS

TYPICAL SLAB ON GRADE DEPRESSION DETAIL

TYPICAL SLAB ON GRADE JOINTING AT CONCRETE COLUMN

TYPICAL SLAB ON GRADE EDGE

TYPICAL SLAB ON GRADE CONTROL JOINT

1" = 1'-0" 1 TYPICAL SLAB ON GRADE

1" = 1'-0" 2 TYPICAL SLAB ON GRADE

1" = 1'-0" 3 TYPICAL SLAB ON GRADE

1" = 1'-0" 4 TYPICAL SLAB ON GRADE

1" = 1'-0" 5 TYPICAL SLAB ON GRADE

1" = 1'-0" 6 TYPICAL SLAB ON GRADE

Must be checked by licensed engineer before construction.
1. Foundation to be prepared in accordance to geotechnical report.
2. C1 indicates concrete column, see for size and reinforcement.
3. F1 indicates foundation, see for size and reinforcement.
4. T.O.F. indicates top of foundation reference finished floor (0'-0').

Must be checked by licensed engineer before construction.
Notes:
1. B1 indicates concrete beam; see your size and reinforcement.
2. G1 indicates concrete girder; see your size and reinforcement.
3. Opening location and sizes to be verified with architectural drawings.
Notes:
1. Opening location and sizes to be verified with architectural drawings.
CONCRETE COLUMN.
SEE PLAN.

FTG. BEYOND
SEE PLAN

HSS 8x8x5/8

MASONRY WALL

CONCRETE COLUMN.
SEE PLAN.

3" x 18 GA STEEL DECKING

FTG. BEYOND
SEE PLAN

HSS 8x8x5/8

MASONRY WALL

CONCRETE COLUMN.
SEE PLAN.

MASONRY WALL

CONCRETE COLUMN.
SEE PLAN.

0'-0"

12'-0"

6" SLAB ON GRADE.
SEE PLAN

4" SAND
5" GRAVEL

MASONRY WALL

CONCRETE COLUMN.
SEE PLAN.

STAGE HEADER.
SEE PLAN

CONCRETE BEAM.
SEE PLAN.

STEEL TRUSS.
SEE PLAN

MASONRY WALL

CONCRETE BEAM.
SEE PLAN.

4" SAND
5" GRAVEL

MASONRY WALL

CONCRETE COLUMN.
SEE PLAN.

FTG. BEYOND
SEE PLAN

HSS 6x6x1/2

SEE DETAILS FOR
BOLT SIZE

CONCRETE COLUMN.
SEE PLAN.

S4.6

S4.5

S4.6

S4.5

3" x 18 GA STEEL DECKING

FTG. BEYOND
SEE PLAN

1' - 6 1/8"

5' - 0"

1' - 6"

4' - 0"

1' - 6"

3' - 0"

1' - 6"

4' - 0"

1' - 6"

5' - 0"

1' - 6"

4' - 0"

1' - 6"

5' - 0"

1' - 6"

4' - 0"

1' - 6"
3" x 18 GA STEEL DECKING

HSS 8x8x5/8

HSS 3x3x3/8 STEEL COLUMN

MASONRY WALL

5" SLAB ON GRADE. SEE PLAN.

MASONRY WALL

4" SAND

5" GRAVEL

12'-0"

SEE DETAILS FOR BOLT SIZE

CONCRETE COLUMN. SEE PLAN.

CONCRETE COLUMN. SEE PLAN.

1'-0"

SEE PLAN.
PLAN SECTION

ELEVATION SECTION

FOOTING SCHEDULE

<table>
<thead>
<tr>
<th>MARK</th>
<th>SIZE</th>
<th>DEPTH</th>
<th>REINFORCING DETAILS</th>
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<tr>
<td>F1</td>
<td>4'-0&quot; SQ.</td>
<td>1'-6&quot;</td>
<td>No. 4 @ 12&quot; O.C. EACH WAY</td>
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<tr>
<td>F2</td>
<td>5'-0&quot; SQ.</td>
<td>1'-6&quot;</td>
<td>No. 4 @ 12&quot; O.C. EACH WAY</td>
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<tr>
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<td>No. 4 @ 12&quot; O.C. EACH WAY</td>
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CONCRETE COLUMN
SEE PLAN OR SCHEDULE

DOWELS TO MATCH VERT. COLUMN REINF.

OF FOOTING AND COLUMN

DEPTH

REINFORCING DETAILS

MARK

"W" x "L"

F1 F2 F3 F4

F1 4'-0" SQ. 1'-6"
F2 5'-0" SQ. 1'-6"
F3 3'-0" SQ. 1'-6"
F4 4'-0" SQ. 1'-6"

No. 4 @ 12" O.C. EACH WAY
No. 4 @ 12" O.C. EACH WAY
No. 4 @ 12" O.C. EACH WAY
No. 4 @ 12" O.C. EACH WAY

Must be checked by licensed engineer before construction
FOR FOOTING DETAIL SEE ZONE A

WHERE BM DO NOT OCCUR

NOTE:

= 1'-6" MIN. OR H/6 OR MAX. COL. DIM. WHICHEVER IS GREATER

WHERE NO BEAMS OCCUR TO COLUMN OF FOOTING AND COLUMN

CLASS B SPLICE FOR COLUMN SIZE AND REINF. SEE COLUMN SCHEDULE

BEAM WHERE OCCURS DOWELS TO MATCH VERTICAL (ELIMINATE DOWELS IF VERTICAL REINF. IS HOOKED 90°)

CLASS B LAP @ T.O. COL.

COLUMN SCHEDULE

<table>
<thead>
<tr>
<th>COLUMN TYPE</th>
<th>C1/4/5/6</th>
<th>C2</th>
<th>C3</th>
<th>C7</th>
<th>C8</th>
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<tr>
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<td>12&quot; x 12&quot;</td>
<td>HES 6/8</td>
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<td>6/8</td>
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<tr>
<td>6/8</td>
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ALTERNATE 135 DEGREES HOOK TO OPPOSITE END FOR EACH CONSECUTIVE VERTICAL TIE TYP.

COLUMN DETAIL AND SCHEDULE

COLUMN DETAIL AND SCHEDULE

COLUMN TYPE A

COLUMN TYPE B

COLUMN TYPE C

COLUMN DETAIL AND SCHEDULE

COLUMN DETAIL AND SCHEDULE
CLEAR SPAN - Ln1

PROVIDE 25% MAX. TOP BARS, 2 BARS MIN. OR PER SCHEDULE BOTTOM BARS PER SCHEDULE

CENTER LINE OF SUPPORT

STIRRUPS SEE SCHEDULE FOR 25% OF BOTTOM BARS (2 BARS MIN.), PROVIDE STD., HOOK @ WALL AND COLUMNS. EXTEND 6" INTO SUPPORT FOR REMAINDER OF BARS.

STD. HOOKS TYP. 3" MAX.

SPLICE TYP. CLASS "A" OR PER SCHEDULE, WHICHEVER IS GREATER (ld min)

TOP BARS TYP. 2" MAX.

SPLICE TYP. CLASS "A" OR PER SCHEDULE, WHICHEVER IS GREATER (ld min)

NOTES:
1. FOR TOP BARS AT INTERIOR SUPPORT, PROVIDE THE LARGEST STEEL AREA REQUIRED FOR ADJACENT SPANS.
2. TOP LEFT BARS ARE SOUTH AND WEST END, TOP RIGHT ARE NORTH AND EAST END.
3. SEE LAP SPLICE SCHEDULE FOR "ld".

U.N.O. ON PLANS OR SCHEDULE.

FOR 25% OF BOTTOM BARS (2 BARS MIN.), PROVIDE SCHEDULE FOR WIDTH.

CONCRETE BEAM SECTION
# Beam Schedule

<table>
<thead>
<tr>
<th>MARK</th>
<th>WIDTH</th>
<th>DEPTH</th>
<th>SUPPORT BEAM ON FACE OF WALL</th>
<th>TRANSVERSE REINFORCEMENT</th>
<th>REMARKS</th>
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<tbody>
<tr>
<td>B1</td>
<td>-</td>
<td>-</td>
<td>12&quot; HSS 8x8x5/8</td>
<td>4% Ø6’s @ 2' o.c.</td>
<td></td>
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<tr>
<td>B2</td>
<td>-</td>
<td>-</td>
<td>12&quot; HSS 6x6x1/2</td>
<td>4% Ø6’s @ 2' o.c.</td>
<td></td>
</tr>
<tr>
<td>B3 &amp; B4</td>
<td>12”</td>
<td>12&quot;</td>
<td>7.14” 8,12” 12&quot;</td>
<td>4% Ø6” @ 2’ o.c.</td>
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<tr>
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<tr>
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<tr>
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<td>H3</td>
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<td>4% Ø6” @ 2’ o.c.</td>
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</tbody>
</table>

**REMARKS**

- Must be checked by licensed engineer before construction.

**Client**
MissionTwenty
-Five35

**Engineers**
Jocelyn Lu
Mindy Trieu

**Scale**
1” = 1'-0"
CONCRETE COLUMN PER PLAN
CONCRETE BEAM PER PLAN
MIN. 29" Ld PER CALCS
#4 LONGITUDINAL BAR
FOOTING PER PLAN AND SCHEDULE
GROUT WIDTH PER DETAIL
#8 SHEAR REINF. BAR @ 48" O.C. IN GROUTED CELLS
HORIZONTAL EXPANSION JOINT PER DETAIL
#4 HOOKED BAR @ 8' O.C.
CONCRETE BEAM PER PLAN
MIN. 19" Ld PER CALCS
HORIZONTAL EXPANSION JOINT
MIN. 1/4" GROUT
MASONRY WALL PER DETAIL
MASONRY WALL PER DETAIL
MIN. 1/4" GROUT
Must be checked by licensed
engineer before construction
HSS 4x4x5/16 TOP CHORD
HSS 4x4x5/16 BOTTOM CHORD
HSS 4x4x5/16 WEB
WELD PER MANUFACTURER
CONCRETE BEAM PER PLAN
3/4" DIA. BOLTS AT @ 24" O.C.
3 x 18 GA GALVANIZED CORRUGATED STEEL DECKING
(N - DECK, TYPE PLN3)
TRUSS PER DETAIL
MASONRY WALL PER PLAN AND DETAIL
MIN. (2) 3/4" DIA. BOLTS

TRUSS DETAILS
1/2" = 1'-0" 1 ROOF TRUSS
1" = 1'-0" 2 TRUSS TO CONCRETE BEAM AND ROOF DECKING

Must be checked by licensed engineer before construction