100% DD
Structural Calculations

For
UO Housing Central Kitchen & Woodshop
1793 Columbia Street
Eugene, OR

Architect:
Robertson|Sherwood|Architects pc
132 East Broadway, Suite 540
Eugene, Oregon 97401

Project No: H-L 9368.1
August 8, 2014
# Index to Structural Calculations

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<th>Pg No.</th>
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# HOHBACH-LEWIN, INC.

**PROJECT NAME**: UO KITCHEN

## ROOF

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**SLOPE**: 1 in 12

**Roof Snow Load**: 20 psf

## WALLS

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PROJECT NAME: UO Kitchen

SNOW LOADS (per 2014 OSSC and ASCE 7-10)

1. Balanced Snow Load

Ground Snow Load: \[ P_g = 15 \text{ psf} \]
Exposure Category: Category: B
Terrain Category: Category: Partially Exposed
Exposure Factor: \[ C_e = 1.0 \]
Thermal Category: Category: Cold Ventilated Roofs
Thermal Factor: \[ C_t = 1.10 \]
Occupancy Category: Category: II
Importance Factor: \[ I = 1.0 \]

Flat Roof Snow Load: \[ P_f = 11.6 \text{ psf} \]

Roof Type: Type: Hip and Gable
Eave to Ridge Dist: \[ W = 64 \text{ ft} \]
Roof Slope: \[ s = 1 \text{ in 12} \]
Slope Angle: \[ \theta = 4.8 \text{ deg} \]
Roof Surface: Type: All Others
Slope Factor: \[ C_s = 1.00 \]

Sloped Roof Snow Load: \[ P_s = 11.6 \text{ psf} \]

2. Minimum Snow Load

Free Draining Roof: y/n: Yes
Oregon Exception for Rain Surcharg y/n: No

Minimum Roof Snow Load: \[ P_f \text{ min} = 20 \text{ psf} \]

Note: \( P_f \text{ min} \) applies only to balanced snow loads. It is not used in combination with partial, unbalanced, or drifted snow loads.
COMPONENTS & CLADDING WIND LOADING

PARAMETERS

RISK CATEGORY II
BASIC WIND SPEED 120 MPH
EX posURE CATEGORY B
MEAN ROOF HT: ~15 FT.
ROOF SLOPE 1 IN 12 5°

VELOCITY PRESSURE

\[ q_x = 0.10025 \times K_2 \times K_3 \times K_4 \times V^2 \]
\[ K_2 = 0.70 \quad (R \ h = 15') \]
\[ K_3 = 1.0 \]
\[ K_4 = 1.00 \]
\[ q_h = 21.9 \text{ PSF} \]

LOW RISE BUILDINGS (h ≤ 60')

\[ P = q_h \times (C_{Gp}) \times (G_{Cp}) \]
\[ C_{Gp} = \pm 0.18 \quad \text{(ENCLOSED BUILDING)} \]
## COMPONENTS & CLADDING WIND LOADING

### WALLS

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<th>P⁺</th>
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## COMPONENTS & CLADDING WIND LOADING

### ROOF

**Gable Roof w/ \( \theta \leq 7^\circ \)**

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### OVERHANG

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ROOF FRAMING

RJ-1 - 42' MAX SPAN JOISTS

DL = 20 P.S.F
SL = 20 P.S.F

SPACING = 2.67' O.C.
SPAN = 42'

W0 = 20 P.S.F (2.67') = 54 P.L.F
W2 = 20 P.S.F (2.64') = 54 P.L.F
W0+2 = 108 P.L.F

38" RED-L G.F. 141 P.L.F
26" RED-M G.F. 145 P.L.F

RJ-2 - 32' MAX SPAN JOISTS

W0+2 = 108 P.L.F
SPAN = 32'

24" RED-L G.F. 155 P.L.F

RJ-3 - 23' MAX SPAN JOISTS

W0+2 = 108 P.L.F
SPAN = 23'

11/4" RED-L G.F. 191 P.L.F
RED-L™ TRUSS ALLOWABLE UNIFORM LOAD TABLE (PLF) / PARALLEL CHORD

For economical truss design, see page 5.

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See pages 4 and 5 for available depths and profiles. For depths and profiles not shown, contact your RedBuilt™ technical representative for assistance.

Red numbers refer to 115% Total Load (TL).

General Notes
- Values shown are for demonstration of maximum allowable load capacities based on the following assumptions:
  - Simple span, uniformly loaded conditions, with provisions for positive drainage (1/4:12 slope minimum) in roof applications.
  - Span indicates distance from inside face to inside face of bearing.
  - Top chord no-notch bearing clips with 1/4" bearing. Higher values may be possible with other types of bearing clips.
  - Straight line interpolations may be made between depths and spans.
  - Values in areas may be increased for repetitive-member use as follows:
    - 7% for Red-L™ truss series and 4% for Red-L™ truss series.
    - Bolditalic values are controlled by minimum concentrated load analysis of 2,000 lb. Higher loads are possible where minimum concentrated load analysis is not required by code. Contact your RedBuilt™ technical representative for assistance.

General Notes continued on page 7

Trusses delivered to the jobsite are custom manufactured to resist only project specific application loads provided by the design professional. Actual trusses may not be able to resist the maximum loads shown in the tables above. For questions regarding actual truss capacity contact your RedBuilt™ technical representative.
ROOF FRAMING

RB-1

SPAN = 24'
ROOF TRIB = \(\frac{21'}{2} + \frac{42'}{2} \approx 32'\)

\(W_0 = 20\) psf (32') = 640 #/ft
\(W_{e} = 20\) psf (32') = 640 #/ft
\(W_{w} = 10\) psf (32') = 512 #/ft
\(W_{w} = -20\) psf (32') = -400 #/ft (AT > 100 ft², zone 112)

SEE FOLLOWING ENERCAU ANALYSIS

5/8 x 24 GLA
Wood Beam Design

Material Properties

Analysis Method: Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

Fb - Tension 2,400.0 psi
Fb - Compr 1,850.0 psi
Fc - Prll 1,650.0 psi
Fc - Perp 650.0 psi
Fv 265.0 psi
Ft 1,100.0 psi

E: Modulus of Elasticity
E:xx 1,800.0 ksi
E:yy 930.0 ksi
Em:xx 1,600.0 ksi
Em:yy 830.0 ksi
Density 32.210pcf

Wood Species: DF/DF
Wood Grade: 24F - V4
Beam Bracing: Beam is Fully Braced against lateral-torsion buckling

Calculations per IBC 2006, CBC 2007, 2005 NDS

Applied Loads
Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load: D = 0.640, S = 0.640, W = 0.5120 k/f, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending Stress Ratio = 0.904 1
Section used for this span = 5.125x24
fb : Actual = 2,296.12 psi
fb : Allowable = 2,541.01 psi
Load Combination = D+S+H
Location of maximum on span = 12.000 ft
Span # where maximum occurs = Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection 0.453 in Ratio = 635
Max Upward L+Lr+S Deflection 0.000 in Ratio = 0 <360
Max Downward Total Deflection 0.473 in Ratio = 609
Max Upward Total Deflection 0.000 in Ratio = 0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination | Segment Length | Span # | Max Stress Ratios | C_d | Summary of Moment Values | Summary of Shear Values |
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<td></td>
<td>1</td>
<td>0.332</td>
<td>0.194</td>
<td>1.000</td>
<td>48.06</td>
</tr>
</tbody>
</table>

Service loads entered. Load Factors will be applied for calculations.
## Wood Beam Design

### Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>Segment Length</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td></td>
<td>M_actual</td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.446</td>
<td>0.260</td>
<td>1.600</td>
<td>64.65</td>
</tr>
<tr>
<td>+D-0.750L+0.750S+0.450W+H</td>
<td>1</td>
<td>0.684</td>
<td>0.399</td>
<td>1.600</td>
<td>99.21</td>
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<td>0.332</td>
<td>0.194</td>
<td>1.600</td>
<td>48.06</td>
</tr>
<tr>
<td>+D-0.750L+0.750S+0.5250E+H</td>
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<td>82.62</td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.352</td>
<td>0.205</td>
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<td>50.95</td>
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<td>+D-0.600W+H</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.199</td>
<td>0.116</td>
<td>1.600</td>
<td>28.84</td>
</tr>
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</table>

### Overall Maximum Deflections - Unfactored Loads

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span</th>
<th>Max. * Defl</th>
<th>Location in Span</th>
<th>Load Combination</th>
<th>Max. * Defl</th>
<th>Location in Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only</td>
<td>1</td>
<td>0.4726</td>
<td>12.120</td>
<td></td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

### Vertical Reactions - Unfactored

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Support 1</th>
<th>Support 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall MAXimum</td>
<td>8.010</td>
<td>8.010</td>
</tr>
<tr>
<td>D Only</td>
<td>8.010</td>
<td>8.010</td>
</tr>
<tr>
<td>S Only</td>
<td>7.680</td>
<td>7.680</td>
</tr>
<tr>
<td>W Only</td>
<td>6.144</td>
<td>6.144</td>
</tr>
</tbody>
</table>
### Wood Beam Design

#### Material Properties

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Allowable Stress Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Combination</td>
<td>2006 IBC &amp; ASCE 7-05</td>
</tr>
</tbody>
</table>

- **Wood Species**: DF/DF
- **Wood Grade**: 24F - V4
- **Beam Bracing**: Completely Unbraced

### Calculations per IBC 2006, CBC 2007, 2005 NDS

- **Fb - Tension**: 2,400.0 psi
- **Fb - Compr**: 1,850.0 psi
- **Fc - Pfl**: 1,650.0 psi
- **Fc - Perp**: 650.0 psi
- **Fv**: 265.0 psi
- **Ft**: 1,100.0 psi

- **E**: Modulus of Elasticity
- **Ebend - xx**: 1,800.0 ksi
- **Eminb - xx**: 930.0 ksi
- **Ebend - yy**: 1,600.0 ksi
- **Eminb - yy**: 830.0 ksi

- **Density**: 32.210 pcf

### Applied Loads

- **Team self weight** calculated and added to loads
- **Load for Span Number 1**
  - **Uniform Load**: $D = 0.640$, $W = 0.60$ k/lf, Tributary Width = 1.0 ft

#### DESIGN SUMMARY

- **Maximum Bending Stress Ratio**: $0.589 : 1$
  - Section used for this span: 5.125x24
  - Load Combination: +D
  - Location of maximum on span: 12.000 ft
  - Span # where maximum occurs: #1

- **Maximum Shear Stress Ratio**: $0.344 : 1$
  - Section used for this span: 5.125x24
  - Load Combination: +D
  - Location of maximum on span: 0.000 ft
  - Span # where maximum occurs: #1

- **Maximum Deflection**:
  - Max Downward $L+Lr+S$ Deflection: 0.000 in
  - Max Upward $L+Lr+S$ Deflection: 0.000 in
  - Max Downward Total Deflection: 0.473 in
  - Max Upward Total Deflection: -0.566 in

### Maximum Forces & Stresses for Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>M</th>
<th>V</th>
<th>C_d</th>
<th>Actual M</th>
<th>fb-design</th>
<th>Fb-allow</th>
<th>Vactual</th>
<th>f-design</th>
<th>Fv-allow</th>
</tr>
</thead>
<tbody>
<tr>
<td>+D</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.589</td>
<td>0.344</td>
<td>0.900</td>
<td>48.06</td>
<td>1,172.22</td>
<td>1,988.62</td>
<td>6.73</td>
<td>82.06</td>
<td>238.50</td>
</tr>
<tr>
<td>+D+L+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.540</td>
<td>0.310</td>
<td>1.000</td>
<td>48.06</td>
<td>1,172.22</td>
<td>2,169.32</td>
<td>6.73</td>
<td>82.06</td>
<td>265.00</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.465</td>
<td>0.248</td>
<td>1.250</td>
<td>48.06</td>
<td>1,172.22</td>
<td>2,522.98</td>
<td>6.73</td>
<td>82.06</td>
<td>331.25</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.489</td>
<td>0.269</td>
<td>1.150</td>
<td>48.06</td>
<td>1,172.22</td>
<td>2,397.24</td>
<td>6.73</td>
<td>82.06</td>
<td>304.75</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.489</td>
<td>0.269</td>
<td>1.150</td>
<td>48.06</td>
<td>1,172.22</td>
<td>2,397.24</td>
<td>6.73</td>
<td>82.06</td>
<td>304.75</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.117</td>
<td>0.054</td>
<td>1.800</td>
<td>13.50</td>
<td>329.29</td>
<td>2,807.57</td>
<td>1.89</td>
<td>23.05</td>
<td>424.00</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 24.0 ft</td>
<td>1</td>
<td>0.418</td>
<td>0.194</td>
<td>1.500</td>
<td>48.06</td>
<td>1,172.22</td>
<td>2,807.57</td>
<td>6.73</td>
<td>82.06</td>
<td>424.00</td>
</tr>
</tbody>
</table>

**Design OK**
Wood Beam Design

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td></td>
<td>1</td>
<td>0.192</td>
<td>0.089</td>
<td>1.600</td>
</tr>
<tr>
<td>+D+0.750L+0.750S+0.450W+H</td>
<td></td>
<td>1</td>
<td>0.192</td>
<td>0.089</td>
<td>1.600</td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td></td>
<td>1</td>
<td>0.418</td>
<td>0.194</td>
<td>1.600</td>
</tr>
<tr>
<td>+D+0.750L+0.750S+0.5250E+H</td>
<td></td>
<td>1</td>
<td>0.418</td>
<td>0.194</td>
<td>1.600</td>
</tr>
<tr>
<td>Length = 24.0 ft</td>
<td></td>
<td>1</td>
<td>0.055</td>
<td>0.023</td>
<td>1.600</td>
</tr>
<tr>
<td>+0.60D+0.60W+H</td>
<td></td>
<td>1</td>
<td>0.251</td>
<td>0.116</td>
<td>1.600</td>
</tr>
</tbody>
</table>

Overall Maximum Deflections - Unfactored Loads

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span</th>
<th>Max. &quot;a&quot; Defl</th>
<th>Location in Span</th>
<th>Load Combination</th>
<th>Max. &quot;a&quot; Defl</th>
<th>Location in Span</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.0000</td>
<td>0.0000</td>
<td>W Only</td>
<td>-0.5664</td>
<td>12.120</td>
</tr>
</tbody>
</table>

Vertical Reactions - Unfactored

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Support 1</th>
<th>Support 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall MAXimum</td>
<td>-9.600</td>
<td>-9.600</td>
</tr>
<tr>
<td>D Only</td>
<td>8.010</td>
<td>8.010</td>
</tr>
<tr>
<td>W Only</td>
<td>-9.600</td>
<td>-9.600</td>
</tr>
</tbody>
</table>
ROOF FRAMING

RB-2

SPAN = 33'
ROOF TRIB = \frac{23'}{2} + \frac{21'}{2} \approx 22'

W_0 = 20 \text{ PSF (22')} = 440 \text{ #/ft}
W_S = 20 \text{ PSF (22')} = 440 \text{ #/ft}
W_U = 16 \text{ PSF (22')} = 352 \text{ #/ft}
W_N = 25 \text{ PSF (22')} = 550 \text{ #/ft}

SEE FOLLOWING LATERAL ANALYSIS

5\frac{1}{8} \times 3\frac{1}{2} \text{ GUB}
Wood Beam Design

Material Properties

Analysis Method: Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05
Wood Species: DF/DF
Wood Grade: 24F - V4
Beam Bracing: Beam is Fully Braced against lateral-torsion buckling

Calculations per IBC 2006, CBC 2007, 2005 NDS

<table>
<thead>
<tr>
<th>Property</th>
<th>Fb - Tension</th>
<th>Fb - Compr</th>
<th>Fc - Prl</th>
<th>Fc - Perp</th>
<th>Fv</th>
<th>Ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>4,040 psi</td>
<td>1,850 psi</td>
<td>1,650 psi</td>
<td>650 psi</td>
<td>265 psi</td>
<td>1,100 psi</td>
</tr>
</tbody>
</table>

E: Modulus of Elasticity
- Ebend-xx = 1,800 ksi
- Ebend-yy = 930 ksi
- Ebend-y = 1,600 ksi
- Eminbend-xx = 830 ksi
- Density = 32.210 pcf

Applied Loads

Beam self weight calculated and added to loads
Load for Span Number 1:
- Uniform Load: D = 0.440, S = 0.440, W = 0.3520 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending Stress Ratio = 0.737 : 1
- Section used for this span: 5.125 x 31.5
- fb : Actual = 1,765.64 psi
- FB : Allowable = 2,395.34 psi

Maximum Shear Stress Ratio = 0.392 : 1
- Section used for this span
- fv : Actual = 119.38 psi
- Fv : Allowable = 304.75 psi

Load Combination
- +D+S+H

Location of maximum on span = 16.500 ft
Span # where maximum occurs = Span # 1

Maximum Deflection
- Max Downward L+L+R+S Deflection = 0.493 in
  Ratio = 804
- Max Upward L+L+R+S Deflection = 0.000 in
  Ratio = 0 < 360
- Max Downward Total Deflection = 0.533 in
  Ratio = 743
- Max Upward Total Deflection = 0.000 in
  Ratio = 0 < 240

Load Combination & Stresses for Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>M</th>
<th>V</th>
<th>C_d</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>+D</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.489</td>
<td>0.260</td>
<td>0.900</td>
<td>64.81 917.62 1,874.61</td>
<td>6.68 62.04 238.50</td>
</tr>
<tr>
<td>+D+L+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.441</td>
<td>0.234</td>
<td>1.000</td>
<td>64.81 917.62 2,082.90</td>
<td>6.68 62.04 265.00</td>
</tr>
<tr>
<td>+D+L+R+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.352</td>
<td>0.187</td>
<td>1.250</td>
<td>64.81 917.62 2,603.63</td>
<td>6.68 62.04 331.25</td>
</tr>
<tr>
<td>+D+L+R+S+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.737</td>
<td>0.392</td>
<td>1.150</td>
<td>124.71 1,765.64 2,395.34</td>
<td>12.85 119.38 304.75</td>
</tr>
<tr>
<td>+D+0.75L+0.75L+S+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.649</td>
<td>0.345</td>
<td>1.150</td>
<td>109.73 1,553.64 2,395.34</td>
<td>11.31 105.05 304.75</td>
</tr>
<tr>
<td>+D+0.50L+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.397</td>
<td>0.211</td>
<td>1.600</td>
<td>93.56 1,324.67 3,332.64</td>
<td>9.84 89.57 424.00</td>
</tr>
<tr>
<td>+D+0.70E+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.275</td>
<td>0.146</td>
<td>1.600</td>
<td>64.81 917.62 3,332.64</td>
<td>6.68 62.04 424.00</td>
</tr>
</tbody>
</table>
## Wood Beam Design

**Lic. #: KW-06003493**

**Description:** RB-2

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td></td>
<td>1</td>
<td>0.367</td>
<td>0.195</td>
<td>1.600</td>
</tr>
<tr>
<td>+D=0.750L+0.750S+0.450W+H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td></td>
<td>1</td>
<td>0.558</td>
<td>0.296</td>
<td>1.600</td>
</tr>
<tr>
<td>+D=0.750L+0.750L+0.5250E+H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td></td>
<td>1</td>
<td>0.275</td>
<td>0.146</td>
<td>1.600</td>
</tr>
<tr>
<td>+D=0.750L+0.750S+0.5250E+H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
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<td>1</td>
<td>0.456</td>
<td>0.248</td>
<td>1.600</td>
</tr>
<tr>
<td>+0.60D+0.60W+H</td>
<td></td>
<td>1</td>
<td>0.287</td>
<td>0.153</td>
<td>1.600</td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td></td>
<td>1</td>
<td>0.165</td>
<td>0.088</td>
<td>1.600</td>
</tr>
<tr>
<td>+0.60D+0.70E+H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td></td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Overall Maximum Deflections - Unfactored Loads

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span</th>
<th>Max. <em>,</em> Defl</th>
<th>Location in Span</th>
<th>Load Combination</th>
<th>Max. <em>,</em> Defl</th>
<th>Location in Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only</td>
<td>1</td>
<td>0.5330</td>
<td>16.665</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

### Vertical Reactions - Unfactored

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Support 1</th>
<th>Support 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall MAXIMUM</td>
<td>7.855</td>
<td>7.856</td>
</tr>
<tr>
<td>D Only</td>
<td>7.855</td>
<td>7.856</td>
</tr>
<tr>
<td>S Only</td>
<td>7.260</td>
<td>7.260</td>
</tr>
<tr>
<td>W Only</td>
<td>5.808</td>
<td>5.808</td>
</tr>
</tbody>
</table>
Wood Beam Design

Material Properties

Analysis Method: Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

Wood Species: DF/DF
Wood Grade: 24F - V4
Beam Bracing: Completely Unbraced

Calculations per IBC 2006, CBC 2007, 2005 NDS

- Fb - Tension: 2,400.0 psi
- Fb - Compr: 1,850.0 psi
- Fc - Plr: 1,650.0 psi
- Fc - Perp: 650.0 psi
- Fv: 265.0 psi
- Fl: 1,100.0 psi

E: Modulus of Elasticity
- Ebend-xx: 1,800.0 ksi
- Ebinbend - xx: 930.0 ksi
- Ebend-yy: 1,600.0 ksi
- Ebinbend - yy: 830.0 ksi

Density: 32.210 pcf

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load: D = 0.440, W = 0.550 klf, Tributary Width = 1.0 ft

Design Summary

- Maximum Bending Stress Ratio = 0.562
  - Section used for this span: 5.125x31.5
  - fb: Actual = 917.62 psi
  - FB: Allowable = 1,633.06 psi
  - Load Combination: +D
  - Location of maximum on span = 16.500 ft

- Maximum Shear Stress Ratio = 0.260
  - Section used for this span: 5.125x31.5
  - fv: Actual = 62.04 psi
  - Fv: Allowable = 238.50 psi

Maximum Deflection

- Max Downward L+L+R+ Deflection = 0.000 in
  - Ratio = 0 < 300
- Max Upward L+L+R+ Deflection = 0.000 in
  - Ratio = 0 < 300
- Max Downward Total Deflection = 0.533 in
  - Ratio = 7.43
- Max Upward Total Deflection = 0.616 in
  - Ratio = 0.43

Maximum Forces & Stresses for Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>C_d</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
<td>Mactual</td>
</tr>
<tr>
<td>+D</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.562</td>
<td>0.260</td>
<td>0.900</td>
<td>64.81</td>
</tr>
<tr>
<td>+D+L+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.544</td>
<td>0.234</td>
<td>1.000</td>
<td>64.81</td>
</tr>
<tr>
<td>+D+L+R+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.521</td>
<td>0.187</td>
<td>1.250</td>
<td>64.81</td>
</tr>
<tr>
<td>+D+S+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.528</td>
<td>0.204</td>
<td>1.150</td>
<td>64.81</td>
</tr>
<tr>
<td>+D+0.750L+0.750S+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.528</td>
<td>0.204</td>
<td>1.150</td>
<td>64.81</td>
</tr>
<tr>
<td>+D+0.600W+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.516</td>
<td>0.045</td>
<td>1.600</td>
<td>19.69</td>
</tr>
<tr>
<td>+D+0.750L+0.750L+0.450W+H</td>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.508</td>
<td>0.146</td>
<td>1.600</td>
<td>64.81</td>
</tr>
</tbody>
</table>
# Wood Beam Design

**License #: KW-06002492**

**Description:** RB-2 (L36ft)

<table>
<thead>
<tr>
<th>Load Combination Segment Length</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.244</td>
<td>0.070</td>
<td>1.600</td>
</tr>
<tr>
<td>+D=0.750L+L+0.750S+0.450W+H</td>
<td>1</td>
<td>0.244</td>
<td>0.070</td>
<td>1.600</td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.508</td>
<td>0.146</td>
<td>1.600</td>
</tr>
<tr>
<td>+D=0.750L+L+0.5250E+H</td>
<td>1</td>
<td>0.508</td>
<td>0.146</td>
<td>1.600</td>
</tr>
<tr>
<td>Length = 33.0 ft</td>
<td>1</td>
<td>0.061</td>
<td>0.014</td>
<td>1.600</td>
</tr>
<tr>
<td>+0.60D+0.60W+H</td>
<td>1</td>
<td>0.305</td>
<td>0.088</td>
<td>1.600</td>
</tr>
</tbody>
</table>

**Overall Maximum Deflections - Unfactored Loads**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span</th>
<th>Max. &quot;*&quot; Defl</th>
<th>Location in Span</th>
<th>Load Combination</th>
<th>Max. &quot;*&quot; Defl</th>
<th>Location in Span</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.0000</td>
<td>0.000</td>
<td>W Only</td>
<td>-0.6157</td>
<td>16.665</td>
</tr>
</tbody>
</table>

**Vertical Reactions - Unfactored**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Support 1</th>
<th>Support 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall MAXimum</td>
<td>-9.075</td>
<td>-9.075</td>
</tr>
<tr>
<td>D Only</td>
<td>7.856</td>
<td>7.856</td>
</tr>
<tr>
<td>W Only</td>
<td>-9.075</td>
<td>-9.075</td>
</tr>
</tbody>
</table>
ROOF FRAMING

RB-3

SPAN = 25'
ROOF TRIB = \frac{21' + 21'}{2} = 21'

W0 = 20 RSF (21') = 420 #/FT
Wf = 2.0 RSF (21') = 420 #/FT
Ww = 1.6 RSF (21') = 336 #/FT
Ww = -25 RSF (21') = -525 #/FT

SEE FOLLOWING ENGINEERING ANALYSIS

5\frac{1}{8} \times 21' GLA
Wood Beam Design

Material Properties

Analysis Method: Allowable Stress Design
Load Combination 2006 IBC & ASCE 7-05

Fb - Tension 2,400.0 psi
Fb - Compr 1,850.0 psi
Fb - Prl 1,650.0 psi
Fb - Perp 650.0 psi
Fv 265.0 psi
Fl 1,100.0 psi

E: Modulus of Elasticity
Ebd-xx 1,800.0 ksi
Eminbd-xx 930.0 ksi
Ebd-yy 1,600.0 ksi
Eminbd-yy 830.0 ksi
Density 32.210pcf

Wood Species: DF/DF
Wood Grade: 24F - V4
Beam Bracing: Beam is Fully Braced against lateral-torsion buckling

 applied loads

Design OK

DESIGN SUMMARY

Maximum Bending Stress Ratio = 0.839 : 1
Section used for this span = 5.125x21
fb : Actual = 2,150.51 psi
fb : Allowable = 2,564.68 psi
Load Combination = +D+S+H
Location of maximum on span = 12.500 ft
Span # where maximum occurs = Span # 1

Maximum Shear Stress Ratio = 0.430 : 1
Section used for this span = 5.125x21
fv : Actual = 130.97 psi
fv : Allowable = 304.75 psi
Load Combination = +D+S+H
Location of maximum on span = 23.375 ft
Span # where maximum occurs = Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection = 0.523 in Ratio = 574
Max Upward L+Lr+S Deflection = 0.000 in Ratio = 0 <360
Max Downward Total Deflection = 0.553 in Ratio = 542
Max Upward Total Deflection = 0.000 in Ratio = 0 <240

Maximum Forces & Stresses for Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>+D</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.551</td>
</tr>
<tr>
<td>+D+L+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.496</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.396</td>
</tr>
<tr>
<td>+D+S+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.839</td>
</tr>
<tr>
<td>+D+0.75OL+0.750S+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.737</td>
</tr>
<tr>
<td>+D+0.60W+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.450</td>
</tr>
<tr>
<td>+D+0.70E+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.310</td>
</tr>
<tr>
<td>Load Combination</td>
<td>Segment Length</td>
<td>Span #</td>
<td>Max Stress Ratios</td>
</tr>
<tr>
<td>------------------</td>
<td>----------------</td>
<td>--------</td>
<td>-------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>+D+0.750L+0.750S+0.450W+H</td>
<td>1</td>
<td>0.415</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>+D+0.750L+0.750S+0.450W+H</td>
<td>1</td>
<td>0.635</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>+D+0.750L+0.750S+0.450W+H</td>
<td>1</td>
<td>0.310</td>
</tr>
<tr>
<td>Overall Maximum Deflections - Unfactored Loads</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Combination</td>
<td>Span</td>
<td>Max, ** Defl</td>
<td>Location in Span</td>
</tr>
<tr>
<td>D Only</td>
<td>1</td>
<td>0.5526</td>
<td>12.625</td>
</tr>
<tr>
<td>Vertical Reactions - Unfactored</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Combination</td>
<td>Support 1</td>
<td>Support 2</td>
<td></td>
</tr>
<tr>
<td>Overall MAXimum</td>
<td>5.551</td>
<td>5.551</td>
<td></td>
</tr>
<tr>
<td>D Only</td>
<td>5.551</td>
<td>5.551</td>
<td></td>
</tr>
<tr>
<td>S Only</td>
<td>5.551</td>
<td>5.551</td>
<td></td>
</tr>
<tr>
<td>W Only</td>
<td>4.200</td>
<td>4.200</td>
<td></td>
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</tbody>
</table>
**Wood Beam Design**

**Material Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis Method</td>
<td>Allowable Stress Design</td>
</tr>
<tr>
<td>Load Combination</td>
<td>2006 IBC &amp; ASCE 7-05</td>
</tr>
<tr>
<td>Wood Species</td>
<td>DF/DF</td>
</tr>
<tr>
<td>Wood Grade</td>
<td>24F - V4</td>
</tr>
<tr>
<td>Beam Bracing</td>
<td>Completely Unbraced</td>
</tr>
<tr>
<td>Fb - Tension</td>
<td>2,400.0 psi</td>
</tr>
<tr>
<td>Fb - Compr</td>
<td>1,850.0 psi</td>
</tr>
<tr>
<td>Fc - Prll</td>
<td>1,650.0 psi</td>
</tr>
<tr>
<td>Fc - Perp</td>
<td>650.0 psi</td>
</tr>
<tr>
<td>Fv</td>
<td>265.0 psi</td>
</tr>
<tr>
<td>Ft</td>
<td>1,100.0 psi</td>
</tr>
<tr>
<td>Ebend- xx</td>
<td>1,800.0 ksi</td>
</tr>
<tr>
<td>Ebend- yy</td>
<td>1,600.0 ksi</td>
</tr>
<tr>
<td>Eminent bend- xx</td>
<td>930.0 ksi</td>
</tr>
<tr>
<td>Eminent bend- yy</td>
<td>830.0 ksi</td>
</tr>
<tr>
<td>Density</td>
<td>32.210 pcf</td>
</tr>
</tbody>
</table>

**Applied Loads**

Beam self weight calculated and added to loads.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Bending Stress Ratio</td>
<td>0.55 : 1</td>
</tr>
<tr>
<td>Section used for this span</td>
<td>5.125x21</td>
</tr>
<tr>
<td>fb : Actual</td>
<td>1,105.21 psi</td>
</tr>
<tr>
<td>FB : Allowable</td>
<td>2,007.14 psi</td>
</tr>
<tr>
<td>Load Combination</td>
<td>+D</td>
</tr>
<tr>
<td>Location of maximum on span</td>
<td>12.500 ft</td>
</tr>
<tr>
<td>Span # where maximum occurs</td>
<td>Span # 1</td>
</tr>
<tr>
<td>Maximum Deflection</td>
<td></td>
</tr>
<tr>
<td>Max Downward L+Lr+S Deflection</td>
<td>0.000 in</td>
</tr>
<tr>
<td>Max Upward L+Lr+S Deflection</td>
<td>0.000 in</td>
</tr>
<tr>
<td>Max Downward Total Deflection</td>
<td>0.553 in</td>
</tr>
<tr>
<td>Max Upward Total Deflection</td>
<td>-0.653 in</td>
</tr>
<tr>
<td>Ratio</td>
<td>0 &lt; 360</td>
</tr>
<tr>
<td>Ratio</td>
<td>0 &lt; 360</td>
</tr>
<tr>
<td>Ratio</td>
<td>542</td>
</tr>
<tr>
<td>Ratio</td>
<td>459</td>
</tr>
</tbody>
</table>

**Maximum Forces & Stresses for Load Combinations**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>+D</td>
<td></td>
<td>1</td>
<td>0.551</td>
<td>0.282</td>
<td>0.900</td>
</tr>
<tr>
<td>+D+L+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.499</td>
<td>0.254</td>
<td>1.000</td>
</tr>
<tr>
<td>+D+Lr+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.421</td>
<td>0.203</td>
<td>1.250</td>
</tr>
<tr>
<td>+D+S+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.447</td>
<td>0.221</td>
<td>1.150</td>
</tr>
<tr>
<td>+D+0.70L+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.447</td>
<td>0.221</td>
<td>1.150</td>
</tr>
<tr>
<td>+D+0.75L+0.750L+H</td>
<td>Length = 25.0 ft</td>
<td>1</td>
<td>0.368</td>
<td>0.159</td>
<td>1.600</td>
</tr>
</tbody>
</table>
## Wood Beam Design

**Description:** RB-3 (Upflight)

### Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Segment Length</th>
<th>Span #</th>
<th>Max Stress Ratios</th>
<th>Summary of Moment Values</th>
<th>Summary of Shear Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>M</td>
<td>V</td>
<td>C_d</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>1</td>
<td></td>
<td>0.172</td>
<td>0.074</td>
<td>1.60</td>
</tr>
<tr>
<td>+D+0.750L+0.7505+0.450W+H</td>
<td>1</td>
<td></td>
<td>0.172</td>
<td>0.074</td>
<td>1.60</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>1</td>
<td></td>
<td>0.368</td>
<td>0.159</td>
<td>1.60</td>
</tr>
<tr>
<td>+D+0.750L+0.7505+0.5250E+H</td>
<td>1</td>
<td></td>
<td>0.368</td>
<td>0.159</td>
<td>1.60</td>
</tr>
<tr>
<td>Length = 25.0 ft</td>
<td>1</td>
<td></td>
<td>0.052</td>
<td>0.017</td>
<td>1.60</td>
</tr>
<tr>
<td>+0.60D+H+0.60W+H</td>
<td>1</td>
<td></td>
<td>0.221</td>
<td>0.095</td>
<td>1.60</td>
</tr>
</tbody>
</table>

### Overall Maximum Deflections - Unfactored Loads

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Span</th>
<th>Max. *&quot; Dell</th>
<th>Location in Span</th>
<th>Load Combination</th>
<th>Max. *&quot; Dell</th>
<th>Location in Span</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.0000</td>
<td>0.0000</td>
<td>W Only</td>
<td>-0.6533</td>
<td>12.625</td>
</tr>
</tbody>
</table>

### Vertical Reactions - Unfactored

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Support 1</th>
<th>Support 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall MAXimum</td>
<td>-6.563</td>
<td>-6.563</td>
</tr>
<tr>
<td>D Only</td>
<td>5.551</td>
<td>5.551</td>
</tr>
<tr>
<td>W Only</td>
<td>-6.563</td>
<td>-6.563</td>
</tr>
</tbody>
</table>
HEADER DESIGN

EXTERIOR HEADERS

\[
\text{Roof Trib} = \frac{32}{2} = 16' \\
\text{Approx. 7' Wall Above}
\]

\[
\begin{align*}
W_0 &= 20 \text{ PSF (16')} \\
W_s &= 20 \text{ PSF (16')} \\
W_J &= 16 \text{ PSF (16')} \\
W_L &= -24 \text{ PSF (16')}
\end{align*}
\]

\[
\begin{align*}
L_w &= \frac{W_s}{E} & M &= \frac{W_s L^2}{8} \\
L_w &= \frac{W_L}{E} & \Delta &= \frac{S L^4}{384E I}
\end{align*}
\]

\[
L_{max} = 4'
\]

\[
\begin{align*}
R_0 &= 850 \# \\
R_s &= 640 \# \\
R_J &= 512 \# \\
R_L &= -837 \#
\end{align*}
\]

\[
\begin{align*}
W_0 &= 10,200 \text{ #} - \text{in} \\
W_s &= 7,680 \text{ #} - \text{in} \\
W_J &= 6,140 \text{ #} - \text{in} \\
W_L &= 9,980 \text{ #} - \text{in}
\end{align*}
\]

\[
\begin{align*}
A &= 16.5 \text{ in}^2 & F_v &= 126 \text{ PSI} < F_v = 115.1 \text{ PSI} = 207 \text{ PSI} \\
\beta &= 15.125 \text{ in}^3 & F_b &= 1,182 \text{ PSI} < F_b = 115 \text{ psi} = 1,100 \text{ PSI} = 1,495 \\
I &= 414.6 \text{ in}^4 & \Delta_s &= 0.03'' - \frac{L}{1600}
\end{align*}
\]

2-2x6 D.F. #1

2-2x6 D.F. #1
HEADER DESIGN

EXTERIOR HEADERS (CONT)

\[ L_{\text{max}} = 8' \]
\[ P_0 = 1,700 \text{ #} \]
\[ R_0 = 1,280 \text{ #} \]
\[ R_1 = 1,020 \text{ #} \]
\[ R_2 = 1,160 \text{ #} \]
\[ M_0 = 40,000 \text{ #} \]
\[ M_1 = 29,000 \text{ #} \]
\[ M_2 = 19,000 \text{ #} \]

\[ 3\frac{1}{2} \times 9\frac{1}{4} \text{ ISSE LSL} \]
\[ A = 32.38 \text{ IN}^2 \]
\[ f_v = 138 \text{ PSI} \]
\[ f_b = 49.9 \text{ IN}^2 \]
\[ f_b = 1,431 \text{ PSI} \]
\[ I = 230 \text{ IN}^4 \]
\[ \Delta_b = 0.09'' = \frac{L}{100} \]

USE \[ 3\frac{1}{2} \times 11\frac{1}{4} \] ISSE LSL

\[ L_{\text{max}} = 11'' \]

ONE 11'' HOR OCCURS ON LINE 2 (NOT SUPPORTING ROOF FRAMING)

\[ 2\frac{1}{2} \times 11\frac{1}{4} \text{ ISSE LSL FOR } \]

\[ \text{DIN: BY INSP} \]
HEADER DESIGN

INTERIOR HEADERS - LINE B

- INCLUDES DESIGN FOR LINE B & LINE D SOUTH

ROOF TIEB = 20' + 47' = 32'
APPROX. WALL ABOVE = 7'

WD = 20 PSF (32')
WS = 20 PSF (32')
WJ = 16 PSF (32')
WJ = -26 PSF (32')

1. Assume L = 4'

2. Assume 2-2x10 D. F. #1

A = 2175 in² → f_v = 140 psi < f'_v = 115.180 psi
S = 21.8 in³ → f_b = 765 psi < f'_b = 115.111 psi
E = 197,900 lb/in

ΔL = 0.02 → L = \frac{21,400}{2}
HEADER DESIGN

INTERIOR HEADERS - LINE D

MAXIMUM ROOF TRIB. OCCURS AT LINE D

\[ \text{ROOF TRIB} = \frac{37'}{2} + \frac{47'}{2} = 37' \]

APPROX. WALL ABOVE = 9.5'

\[ \begin{align*}
W_d &= 20 \text{ PSF}(37') + 12 \text{ PSF}(9.5') = 854 \text{ #/FT} \\
W_s &= 20 \text{ PSF}(37') = 740 \text{ #/FT} \\
W_r &= 16 \text{ PSF}(37') = 592 \text{ #/FT} \\
W_w &= -20 \text{ PSF}(37') = -962 \text{ #/FT}
\end{align*} \]

\[ R = \frac{WL}{L} \]

\[ M = \frac{W L^2}{2} \]

\[ \Delta = \frac{SWL^3}{3EI} \]

\[ L_{eff} = 6' - 4'' \]

\[ \begin{align*}
R_o &= 2.705 \# \\
R_s &= 21.315 \# \\
R_w &= 1.875 \# \\
R_i &= -3.050 \# \\
M_{d} &= 51,300 \text{ #-IN} \\
M_{s} &= 44,520 \text{ #-IN} \\
M_{w} &= 35,620 \text{ #-IN} \\
M_{w} &= -57,880 \text{ #-IN}
\end{align*} \]

3 1/2 x 9 1/4 LSSSE

\[ A = 31.58 \text{ IN}^2 \quad \rightarrow \quad F_v = 234 \text{ PSF} < F_v' = 1,115 \text{ PSF} \]

\[ b = 41.9 \text{ IN}^3 \quad \rightarrow \quad F_b = 1,922 \text{ PSF} < F_b' = 1,115 \text{ PSF} \]

\[ I = 250 \text{ IN}^4 \quad \rightarrow \quad \Delta_s = 0.08'' < 0.1'' \]

3 1/2 x 11 1/4 LSSSE LSL
HEADER BEARING STUDS

BEARING STUDS SUPPORT HEADER GRAVITY REACTIONS & ADD'L STUD TRIBUTARY

TYP. BEARING STUD # 1 = 8'

\[ F_c = 1,265 \text{ PSI} \]
\[ F_c' = 405 \text{ PSI} \]
\[ P_{allow} = 1,400 \text{ #} \]

2x8 D.F. # 2
\[ F_c = 1,052 \text{ PSI} \]
\[ F_c' = 405 \text{ PSI} \]
\[ P_{allow} = 2,340 \text{ #} \]

2x6 DF. # 2
\[ F_c = 546 \text{ PSI} \]
\[ F_c' = 405 \text{ PSI} \]
\[ P_{allow} = 2,130 \text{ #} \]

EXTERIOR HEADERS

\[ L_{max} = 4' \]

\[ \frac{\frac{4'}{2} \cdot 1.33'}{\frac{4'}{2}} = 1.33 \]

FROM HORIZ DSN:

\[ P_{d} = 850 \text{ #} \times 1.33 = 1,130 \text{ #} \]

\[ P_{s} = 640 \text{ #} \times 1.33 = 850 \text{ #} \]

\[ P_{d} = 512 \text{ #} \times 1.33 = 680 \text{ #} \]

\[ P_{h} = -352 \text{ #} \times 1.33 = -4710 \text{ #} \]

\[ P_{max} = 2,075 \text{ #} \]

1 - BEARING STUD

PERP. TO GRAIN BEARING FROM HORIZ TO STUDB

DF 4\# Fc = 615 PSI

2x2 BEARING ON 2x FLAT

2x FLAT BEARING ON 1x2 STUDB

\[ A_{BE} = 8.25\text{ in}^2 \]

\[ P_{allow} = F_c' \cdot A_{BE} = 5,160 \text{ #} \]

1 - BEARING STUD
STUD/COLUMN POSTS

HEADER BEARING STUDS (CONT.)

EXTERIOR HEADERS (CONT.)

L_{\text{max}} = 8'

\frac{(8' \times \frac{1.333}{2})}{2} = 11.67'

P_{\text{max}} = 3,144 #

[10 + 0.755' + 0.75'(0.6W)]

Z-L BEARING STUDS

PERP TO GRAIN BEARING FROM HOE TO STUDS

USE UDL F_{\text{FLAT}} = 800 PSI

DF FLAT F_{\text{FLAT}} = 625 PSI

3\frac{1}{2}'' UDL BEARING DF: FLAT \rightarrow OIL BY SMPS

2 x FLAT BEARING ON Z-2 x \text{ ABGC} = 5.15'' \times (2\times 1.5') = 15.5\text{in}^2

P_{\text{allow}} = F_{\text{FLAT}} \times \text{ ABGC} = 10,310 #

Z-L BEARING STUDS
STUDS/COLUMN/POSTS

HEADER BEARING STUDS (CONT.)

INTERIOR HEADERS

\[
L_{\text{MAX}} = 4' \quad \text{(LINE B)}
\]

From HOR DSN:
\[
P_o = 1,448 \times 1.33 = 1,950 \quad \#
\]
\[
P_2 = 1,280 \times 1.33 = 1,710 \quad \#
\]
\[
P_u = 1,644 \times 1.33 = 2,170 \quad \#
\]

\[
P_{\text{MAX}} = 3,830\# \quad \text{[D + 0.75\# + 0.75(0.6W)]}
\]

Z-BEARING STUDS

DEEP TO GRAIN BEARING - HOR TO STUOS

DF HOR = F_C = 625 PSI

2-2x HOR BEARING ON Zx FLAT \rightarrow O.K. BY INSPECTION

2x FLAT BEARING ON 2-2x STUDS

\[
A_{\text{BEG}} = 5.15'' \times (2x15'') = 6.25\text{ in}^2
\]

\[
P_{\text{ALLOW}} = F_c \times A_{\text{BEG}} = 10,510 \quad \# \rightarrow \text{Z-BEARING STUDS}
\]

\[
L_{\text{MAX}} = 6'4'' \quad \text{(LINE D)}
\]

From HOR DSN:
\[
P_o = 2,705 \# \times 1.21 = 3,275 \#\]
\[
P_2 = 2,445 \# \times 1.21 = 2,940 \#\]
\[
P_u = 3,050 \# \times 1.21 = 3,690 \#\]

\[
P_{\text{MAX}} = 6,125 \# \quad \text{[D + 0.75\# + 0.75(0.6W)]}
\]

Z-BEARING STUDS

DEEP TO GRAIN BEARING - HOR TO STUOS

LSL HOR TO Zx FLAT \rightarrow O.K. BY INSPECTION

2x FLAT BEARING ON 2-2x STUDS

\[
A_{\text{BEG}} = 5.5'' \times (2x15'')
\]
STUDS/COLUMNS/POSTS

INTERIOR BEARING WALL STUDS

LINE D

ROOF TRIB = $\frac{42'1}{2} + \frac{32'1}{2} = 37'1$

W_o = 20 PSF (37') = 740 #/1FT
W_s = 20 PSF (37') = 740 #/1FT
W_t = 16 PSF (37') = 592 #/1FT

WALL HT. = 17.5'
W_o,w = 12 PSF (17.5') = 210 #/1FT

STUDS AT 16" O.C. → CHECK STUD DIRECTLY BELOW

ROOF DIST = 2.67" ROOF TRIB

P_0 = 2.67 [W_o,w] + 1.33 [W_o,n] = 2,255 #
F_s = 2.67 [W_t] = 1,975 #
F_o = 2.67 [W_s] = 1,580 #

LOAD COMBINATIONS

1. D + S
   P = 4,030 # → DESIGN FOR LC 2

2. D + 0.75 S + 0.15 (0.6 W)
   P = 4,450 #

2 x 8 DF #2 AT 16" O.C.

A = 10.875 in² → f_c = 409 PSI ≤ f_c = \frac{1.16 \times 1105}{(4)} \times 0.125 \times 1.358 \text{ PSI} = 533 \text{ PSI}

f_c = 409 PSI ≤ f_{c1} = \frac{1.25 \times 405 \text{ PSI}}{(4)} = 506 \text{ PSI}

2 x 8 DF #2 STUDS

AT 16" O.C.

OK! BY INSPECTION

AT LINE B
STUDS/COLUMNS/POSTS

INTERIOR BEARING WALL STUDS

LINE D @ LOCKER ROOM/HALLWAY

ROOF TRIBUTARY = 22' + \frac{21'}{2} = 22' 1"

W = 20, R = 20  PSF (22') = 440 #/ft
W = 10, R = 20  PSF (22') = 350 #/ft

WALL HT = 15', 5"  
WALL W = 12  PSF (15.5') = 1800 #/ft

STUDS AT 16" O.C.  \rightarrow  CHECK STUD DIRECTLY BELOW  
ROOF GDL 5'-2", 1/2" ROOF TRIB.

P_o = 9.67 [W/w x 1 + 1.33' x 0.05'] = 1425 #  
P_s = 2.67 [W/w x 2] = 1775 #  
P_w = 2.67 [W/w x 1] = 1940 #

LOAD COMBINATIONS

1. D+S  
P = 2600 #

2. D+0.75S - 0.75(D+W)  
P = 2175 #  \rightarrow  DESIGN FOR LOAD 2

2x6 D.F. #2 AT 16" O.C.

A = 0.25 in²  \rightarrow  F_c = 331 PSF < F_c = \frac{160 \times 1.1}{(0.6) (1.1) \times 0.169 \times 1.350 PSI} = 401 PSI

F_c = 331 PSF < F_c = \frac{125 \times 405 PSI = 506 PSI}{(0.6) (1.1) \times 0.169 \times 1.350 PSI} \rightarrow \text{HEM F.S.}
STUDS/COLUMNS/POSTS

COLUMN C1

ROOF TRIBUTARY: \[
\left( \frac{23}{2} + \frac{21}{2} \right) \times \left( \frac{60}{2} \right) = 660 \text{ FT}^2
\]

\[
P_0 = 21 \text{ PSF} \times \left( \frac{660}{600} \right) = 13.9 \text{ K}
\]

\[
P_N = 2.0 \text{ PSF} \times \left( \frac{660}{600} \right) = 13.2 \text{ k}
\]

\[
P_W = 1.0 \text{ PSF} \times \left( \frac{660}{600} \right) = 10.6 \text{ K}
\]

\[
P_W = 2.5 \text{ PSF} \times \left( \frac{660}{600} \right) = 16.5 \text{ K}
\]

LOAD COMBINATIONS

1. 1.20 + 1.65' + 0.5W \quad P_u = 43.1 \text{ K}

HSS 5x5x1/4

\[
K_L = 115 \text{ MAX} \quad (CONSEC. AT THIS LOC'N)
\]

\[
K_N = 80.3 \text{ K} \quad > \quad P_u = 43.1 \text{ K}
\]

HSS 5x5x1/4
USGS—Provided Output

\[ S_s = 0.762 \text{ g} \quad S_{HS} = 0.911 \text{ g} \quad S_{HH} = 0.607 \text{ g} \]
\[ S_t = 0.400 \text{ g} \quad S_{HT} = 0.640 \text{ g} \quad S_{HT} = 0.427 \text{ g} \]

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 refer to the application and select the "2009 NEHRP" building code reference document.

SEISMIC BASE SHEAR COEFFICIENT PER 2012 IBC/2014 OSSC

PROJECT NAME: UO Kitchen

### Mapped Spectral Accelerations (%g) at Site Class B

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Design Site Class</th>
<th>Site Coefficients (%g)</th>
<th>$F_a$</th>
<th>$F_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>D</td>
<td>TABLE 1613.3.3(1)</td>
<td>1.20</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TABLE 1613.3.3(2)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Adjusted Maximum Considered EQ (MCE) Spectral Response Acceleration Parameters (2012 IBC §1613.3.3):

$$S_{MS} = F_a^*S_3 = 0.91 \text{ g}$$  \hspace{1cm} (EQN 10-37)

$$S_{M1} = F_v^*S_1 = 0.64 \text{ g}$$  \hspace{1cm} (EQN 10-38)

### Design Spectral Response Acceleration Parameters (2012 IBC §1613.3.4)

$$S_{DS} = 2/3*S_{MS} = 0.61 \text{ g}$$  \hspace{1cm} (EQN 16-39)

$$S_{D1} = 2/3*S_{M1} = 0.43 \text{ g}$$  \hspace{1cm} (EQN 16-40)

### Design Coefficients and Factors for Seismic Force-Resisting System (ASCE7-10 §112.2)

Seismic Force-Resisting System per ASCE 7-10 Table 12.2-1:

**A. BEARING WALL SYSTEMS**

16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets

- Response Modification Coefficient, $R$: 6.5  \hspace{1cm} (Table 12.2-1)
- Overstrength Factor, $\Omega_3$: 3  \hspace{1cm} (Table 12.2-1)
- Deflection Amplification Factor, $C_d$: 4  \hspace{1cm} (Table 12.2-1)
- Structural System Limitations: 65 ft  \hspace{1cm} (Table 12.2-1)

**Importance Factor (ASCE 7-10 §11.5)**

Nature of Use or Occupancy per IBC 2012 Table 1604.5:

- Risk Category: II  \hspace{1cm} (Table 1604.5)
- $I = 1.00$  \hspace{1cm} (Table 1.5-2)

**Buildings and other structures except those listed in Risk Categories I, III, and IV**

Note: See IBC 2012 Table 1604.5 and ASCE 7-10 Table 1.5-1 for additional information.

**Seismic Design Category (2012 IBC §1613.3.5)**

- Design Category based on $S_3$: N/A  \hspace{1cm} (IBC 2012 §1613.3.5)
- Design Category based on $S_{DS}$: D  \hspace{1cm} (Table 1613.3.5(1))
- Design Category based on $S_{D1}$: D  \hspace{1cm} (Table 1613.3.5(2))

**Design Seismic Design Category:** D

**Period Determination (ASCE 7-10 §12.8.2)**

- Structure Type: All other structural systems  \hspace{1cm} (Table 12.8-2)
- $C_i = 0.02$  \hspace{1cm} (Table 12.8-2)
- $x = 0.75$  \hspace{1cm} (Table 12.8-2)
- $h_n = 25.00 \text{ ft}$  \hspace{1cm} (Table 12.8-7)
- $T = T_s = C_i h_n^{1/2} = 0.224 \text{ s}$  \hspace{1cm} (EQN 12.8-7)
- $T_i = 16 \text{ s}$  \hspace{1cm} (ASCE 7-10 §11.4.5 & Fig. 22-12 to 22-16)

**Seismic Response Coefficient (ASCE 7-10 §12.8.1)**

- $\rho = 1.00$  \hspace{1cm} (ASCE 7-10 §12.3.4)
- $C_i = S_{DF}(R/I) = 0.093 \text{ W}$  \hspace{1cm} (EQN 12.8-2)
- For $T = T_i$: $C_{min} = S_{DF}(T^{*}/R/I) = 0.294 \text{ W}$  \hspace{1cm} (EQN 12.8-3)
- For $T > T_i$: $C_{max} = S_{DF}(T^{*}/T_i^{*}/R/I) = N/A$  \hspace{1cm} (EQN 12.8-4)
- $C_{min} = 0.044S_{DF}[\rho >0.01] = 0.027 \text{ W}$  \hspace{1cm} (EQN 12.8-5)
- For $S_p = 0.6$: $C_{max} = 0.5S_p/R/I = N/A$  \hspace{1cm} (EQN 12.8-6)
- $V = \rho^*C_i^{*}W = 0.093 \text{ W}$  \hspace{1cm} (STRENGTH LEVEL)
- $V_{ASD} = 0.7V = 0.066 \text{ W}$  \hspace{1cm} (ASD)
# LFRS Seismic Loading

## Seismic Weight

See Seismic Area Key Plan

<table>
<thead>
<tr>
<th>Label</th>
<th>Area</th>
<th>DL</th>
<th>W</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>5,700 ft(^2)</td>
<td>26 psf</td>
<td>148 k</td>
</tr>
<tr>
<td>A2</td>
<td>2,710 ft(^2)</td>
<td>26 psf</td>
<td>72 k</td>
</tr>
<tr>
<td>A3</td>
<td>9,160 ft(^2)</td>
<td>26 psf</td>
<td>250 k</td>
</tr>
<tr>
<td>A4</td>
<td>2,300 ft(^2)</td>
<td>12 psf</td>
<td>88 k</td>
</tr>
<tr>
<td>Ext. Walls</td>
<td>15(\frac{1}{2}) x (12') + 2 x (18')</td>
<td>15 psf</td>
<td>71 k</td>
</tr>
</tbody>
</table>

\[
\text{EXT. WALLS} = 15\frac{1}{2} \times 12' + 2 \times 18' = 4,740 \text{ ft}^2
\]

\[\sum W = 569 \text{ k}\]

## Base Shear

\[ V_{ASD} = 0.065 W = 36,198.5 \text{ #} \]
**LFRS SEISMIC LOADING**

**TRANSVERSE SEISMIC LOADING**

\[
\text{Vertical Load} \text{ on \ LFRS} = 361,985 \text{ #}
\]

**Building Seismic Shear**

\[
\text{Building Seismic Shear} = \frac{V_{lsd}}{\text{Area}} = \frac{35,165 \text{ #}}{18,070 \text{ FT}^2} = 1.95 \text{ #/FT}
\]

\[
\sum n_k, k_2, k_3
\]

**Seismic Loading Based on Area of Roof**

\[
V_{lsd} = \frac{V_{lsdG}}{\text{Area}} = \frac{35,165 \text{ #}}{18,070 \text{ FT}^2} = 1.95 \text{ #/FT}
\]

\[
V_{l1} = \frac{V_{lsd}}{\text{Area}} = \frac{35,165 \text{ #}}{18,070 \text{ FT}^2} = 197 \text{ #/FT}
\]

**Vertical Load on LFRS**

\[
V_{l1} = V_{lsd} \times B_{l1} = 197 \text{ #/FT}
\]

**Vertical Load on Roof**

\[
V_{l2} = V_{lsd} \times B_{l2} = 248 \text{ #/FT}
\]

**Vertical Load on Roof**

\[
V_{l3} = V_{lsd} \times B_{l3} = 125 \text{ #/FT}
\]

**Vertical Load on Roof**

\[
V_{l4} = \frac{V_{lsd} \times W_u}{T_{lrd}} = 1,820 \text{ #}
\]

**Verify Base Shear**

\[
N_{tot} = V_{l1} (L_{l1}) + V_{l2} (L_{l2}) + V_{l3} (L_{l3}) + V_{l4} (L_{l4})
\]

\[
= 37,060 \text{ #} \quad \text{or} \quad V_{lsd} = 361,985 \text{ #}
\]
LONGITUDINAL SEISMIC LOADING

\[ V_{L,0} = 36,985 \text{ #} \]

BUILDING SEISMIC SHEAR = \( V_{L,0} \times \sum \bar{W}_1, \bar{W}_2, \bar{W}_3 \text{, West Walls} \)
\[ = 35,165 \text{ #} \]

SEISMIC LOADING BASED ON AREA OF ROOF
\[ V_{L,0} = \frac{V_{L,0}}{A_{R,0}} = \frac{35,165 \text{ #}}{18,070 \text{ ft}^2} = 19.5 \text{ #/ft} \]

\[ \bar{W}_1 \]
\[ B_{L,1} = 144' \quad \rightarrow \quad V_{L,1} = V_{L,0} \times B_{L,1} = 2,811 \text{ #/ft} \]

\[ \bar{W}_2 \]
\[ B_{L,2} = 189' \quad \rightarrow \quad V_{L,2} = V_{L,0} \times B_{L,2} = 3,697 \text{ #/ft} \]

\[ \bar{W}_3 \]
\[ B_{L,3} = 89' \quad \rightarrow \quad V_{L,3} = V_{L,0} \times B_{L,3} = 1,749 \text{ #/ft} \]

\[ V_4 \]
Pt. Load from Low Roof
\[ V_4 = \frac{V_{L,0}}{W_T} = 1,820 \text{ #} \]

VERIFY BASE SHEAR
\[ V_{TOT} = V_{L,1} (L_{L,1}) + V_{L,2} (L_{L,2}) + V_{L,3} (L_{L,3}) + V_4 \]
\[ = 37,090 \text{ #} \quad < \quad V_{L,0} = 36,985 \text{ #} \]
LFRS SEISMIC LOADING

SEISMIC SHEAR DISTRIBUTION

TRANSVERSE DIRECTION

LINE 2
\[ V_2 = V_{T1} \times \left( \frac{13}{2} \right) + V_{4} = 11.2 \, K \]

LINE 3
\[ V_3 = V_{T1} \times \left( \frac{13}{2} \right) + V_{T2} \times \left( \frac{32}{4} \right) = 13.3 \, K \]

LINE 3.7
\[ V_{3.7} = V_{T2} \times \left( \frac{32}{2} + 12 \right) + V_{T3} \times \left( \frac{45 - 12}{2} \right) = 9.0 \, K \]

LINE 4
RENDONANT ELEMENT (LOAD ACCOUNTED FOR @ LINE 3.7)
\[ V_{4} = V_{T2} \times \left( \frac{12}{2} \right) = 1.5 \, K \]

LINE 5
\[ V_5 = V_{T3} \times \left( \frac{45}{2} + \frac{12}{2} \right) = 3.6 \, K \]

VERIFY BASE SHEAR:
\[ V_{TOT} = V_2 + V_3 + V_{3.7} + V_5 \]
\[ 37.1 \, K \approx V_{ASO} = 37.0 \, K \]

+ RENDONANT SW @ LINE 4 \[ V_{4} = 1.5 \, K \]
LEFS SEISMIC LOADING

SEISMIC SHEAR DISTRIBUTION

LONGITUDINAL DIRECTION

LINE A

\[ V_A = V_{L1} \times \left( \frac{63}{2} \right) = 8.9 \text{ K} \]

LINE D

\[ V_D = V_{L1} \times \left( \frac{63}{2} \right) + V_{L2} \times \left( \frac{33}{2} \right) = 10.0 \text{ K} \]

LINE E

\[ V_E = V_{L2} \times \left( \frac{33}{2} \right) + V_{L3} \times \left( \frac{31}{2} \right) + V_4 = 10.6 \text{ K} \]

LINE F

\[ V_F = V_{L3} \times \left( \frac{31}{2} \right) = 2.7 \text{ K} \]

VERIFY BASE SHEARS

\[ V_{\text{total}} = V_A + V_D + V_E + V_F \]
\[ = 37.2 \text{ K} \]
\[ \approx V_{H=0} = 37.0 \text{ K} \]
1. FRS WIND LOADING

PARAMETERS

- Risk Category: II
- Basic Wind Speed: 120 MPH
- Exposure Category: B
- Mean Roof HT: ~15 FT (LEAVE HT: @ θ<10°)
- Roof Slope: 1 IN 12 ≤ 5°

2. VELOCITY PRESSURE

\[ B_e = 0.00256 \times K_d \times K_t \times K_h \times V^2 \]
\[ K_d = 0.85 \]
\[ K_t = 1.0 \]
\[ K_h = 1 \]
\[ B_{20} = 19.4 \text{ PSF} \]
\[ B_{11} = 9.15 \text{ PSF} \]

3. DESIGN WIND PRESSURE - LOW RISE BUILDING (H<60')

\[ P = g \left[ (G_{Cp}) - (G_{CPV}) \right] \]
\[ G_{Cp} = 0.18 \] (ENCLOSED BUILDING)
\[ G_{CPV} = 0.85 \]
LEFS WIND LOADING

WALL LOADING

WINNOWARD WALLS

\[ C_p = 0.8 \]

\[ +G_1C_p \]

\[ P_{w0} = 9.7 \text{ PSF} \]
\[ P_{w1} = 9.0 \text{ PSF} \]

\[ -G_1C_p \]

\[ P_{w0} = 16.7 \text{ PSF} \]
\[ P_{w1} = 15.4 \text{ PSF} \]

LEEWARD WALLS

TRANSVERSE DIRECTION

\[ \frac{L/H = 64'}{190} = 0.34 \]

\[ C_p = -0.5 \]

\[ +G_1C_p \]

\[ P_{n0} = -10.8 \text{ PSF} \]

\[ -G_1C_p \]

\[ P_{n0} = -4.4 \text{ PSF} \]

LONGITUDINAL DIRECTION

\[ \frac{L/H = 190'}{124} = 1.50 \]

\[ C_p = -0.4 \]

\[ +G_1C_p \]

\[ P_{n0} = 9.3 \text{ PSF} \]

\[ -G_1C_p \]

\[ P_{n0} = -2.9 \text{ PSF} \]
LEAK WIND LOADING

ROOF LOADING

NORMAL TO RIDGE (TRANSVERSE)

\[ \frac{h}{L} = \frac{15'}{64'} = 0.24 \]

\[ C_p = -0.19, -0.18 \]

\[ + \text{CPi} \]

\[ P = -16.7 \text{ PSF}, -6.0 \text{ PSF} \]

\[ \theta < 10^\circ \]

\[ -\text{CPi} \]

\[ P = -10.5 \text{ PSF}, 0.5 \text{ PSF} \]

PARALLEL TO RIDGE (LONITUDINAL)

SAME AS TRANS. DIR. - SEE ABOVE
LEFS WIND LOADING

SECTION 1

\[
W_{ROOF} = W_L (16'2'') - W_L (13.5'2'') + W_{R,W} (2.75'2') - W_{R,L} (5.25'2')
\]

\[
W_{L} = 16.7 \text{ PSF}
W_{L} = -9.4 \text{ PSF}
W_{R,W} = -7.9 \text{ PSF}
W_{R,L} = -10.5 \text{ PSF}
\]

\[
W_{ROOF} = 223 \# / \text{FT} \quad \text{STR LEVEL}
\]

\[
W_{ROOF} A/D = 134 \# / \text{FT} \quad \text{CONTROLS}
\]

\[
W_{ROOF} A/D = 132 \# / \text{FT}
\]

\[
\frac{W}{A} = 134 \# / \text{FT}
\]
UPRS WIND LOADING

SECTION 2

W_{\text{roof}} = W_{\text{U}} (13.67' \times \frac{1}{2}) + W_{\text{L}} (11.5' \times \frac{1}{2}) + W_{\text{Rin}} (5.0' \times \frac{1}{2}) - W_{\text{Rout}} (5.25')

W_{U} = 9.7 \text{ psf}
W_{L} = 10.8 \text{ psf}
W_{\text{Rin}} = -10.0 \text{ psf}
W_{\text{Rout}} = -16.7 \text{ psf}

W_{\text{roof}} = 187 \# / \text{ft}
\text{sta. level}

W_{\text{roof, ASD}} = 173 \# / \text{ft}

\text{controls: } T/2 = 11.6 \# / \text{ft}
WIND LOADING

SECTION 3

\[ W_{\text{roof}} = W_W(9'11\frac{1}{2}) - W_L(11'5\frac{1}{2}) = W_{WL}(7'5') \]

\[ W_W = 9.7 \text{ PSF} \]
\[ W_L = -10.8 \text{ PSF} \]
\[ W_{WL} = -16.9 \text{ PSF} \]

\[ W_{\text{roof}} = 238 \# / \text{ft} \]

\[ W_{\text{roof}, \text{asg}} = W_{\text{roof}} \times 0.16 \]
\[ = 143 \# / \text{ft} \]

\[ \text{CONTROLS} \]
\[ 1/3 = 143 \# / \text{ft} \text{ (asg)} \]
TRANSVERSE WIND LOADING

L18 9368.1
LFRS WIND LOADING

WIND SHEAR DISTRIBUTION

TRANSVERSE DIRECTION

LINE 2

\[ V_2 = V_{t1} \times \left( \frac{L1}{2} \right) = 0.7 \, K \]

LINE 3

\[ V_3 = V_{t1} \times \left( \frac{L1}{2} \right) + V_{t2} \times \left( \frac{3L1}{2} \right) = 8.6 \, K \]

LINE 3.7

\[ V_{3.7} = V_{t2} \left( \frac{3L1}{2} + 2' \right) + V_{t3} \left( \frac{4.5'}{2} - 12' \right) = 5.6 \, K \]

LINE 4

REDUNDANT ELEMENT (LOAD ACCOUNTED FOR @ LINE 3.7)

\[ V_4 = V_{t2} \times \left( \frac{12'}{2} \right) = 0.7 \, K \]

LINE 5

\[ V_5 = V_{t2} \times \left( \frac{4.5'}{2} + 12' \right) = 4.1 \, K \]

BY INSPECTION - SEISMIC LOADING GOVERNS SheAR WALLS DESIGN IN THE LONGITUDINAL DIRECTION
SHEAR WALL DESIGN

METHODOLOGY/EQUATIONS

- SHEAR WALLS DESIGNED FOR ASO LEVEL SEISMIC/WIND FORCES

EQUATIONS

VELOCITIES

\[ V = \frac{V}{L} \]

OVERTURNING

\[ \begin{align*}
T & = \frac{(0.6 - 0.0854)d}{L_{seg}} \\
T & = \frac{mot - (0.6 - 0.0854)mo}{L_{seg}}
\end{align*} \]

SHEAR WALL CAPACITY (2008 SOPS TABLE 4.5.4)

- 15/32" 2000 STRUC PANEL SHEARING
- 8d NAILS

<table>
<thead>
<tr>
<th>NAIL SIZE</th>
<th>ALLOWABLE SHEAR CAP</th>
<th>FIXATING</th>
</tr>
</thead>
<tbody>
<tr>
<td>6&quot;</td>
<td>260 #/ft</td>
<td>2x</td>
</tr>
<tr>
<td>4&quot;</td>
<td>300 #/ft</td>
<td>2x</td>
</tr>
<tr>
<td>3&quot;</td>
<td>350 #/ft</td>
<td>3x</td>
</tr>
<tr>
<td>2&quot;</td>
<td>410 #/ft</td>
<td>3x</td>
</tr>
</tbody>
</table>

* LIMIT TO 350 #/FT TO MEET 2008 SOPS

<table>
<thead>
<tr>
<th>HD CAPACITY</th>
<th>LABEL</th>
<th>HD</th>
<th>ALLOWABLE TENSION</th>
</tr>
</thead>
<tbody>
<tr>
<td>HD-1</td>
<td>HDV2</td>
<td>3,075 #</td>
<td></td>
</tr>
<tr>
<td>HD-2</td>
<td>HDVB</td>
<td>7,870 #</td>
<td></td>
</tr>
</tbody>
</table>
SHEAR WALL DESIGN

LINE 1

\[ V_{EQ} = 11.2 \text{ k} \]
\[ V_{w} = 4.7 \text{ k} \]

SEGMENT I: 6', SEGMENT II: 5', SEGMENT III: 0.25', SEGMENT IV: 5.5', SEGMENT V: 7.75', \( L_{TOT} = 34.5' \)

\( W_{D} = 15 \text{ PSF (15')} = 225 \text{ #/ft} \)

WEIGHT OF WALL

EDGE NAILING

\[ N = \frac{V}{L_{TOT}} = 325 \text{ #/ft} \]

ASPECT RATIO PENALTY = \( \frac{2 \times 5.5'}{10'} = 0.16 \)

8d at 2" o.c. G.F. 640 #/ft x 0.16 = 390 #/ft

2d at 2" o.c.

OVERTURNING - SEGMENT IV

\[ M_{OD} = 30,390 \text{ #}-\text{ft} \]
\[ M_{OD} = 3100 \text{ #}-\text{ft} \]
\[ T = 31210 \text{ #} \]

H0-2
SHEAR WALL DESIGN

LINE 3

\[ V_{eq} = 13.3 \text{ k} \]
\[ V = 8.6 \text{ k} \]

SEGMENT I = 38.3 k
SEGMENT II = 37.3 k
SEGMENT III = 31.7 k

\[ L_{tot} = 102.1' \]

\[ W_o = 15 \text{ psf (15')} = 22.5 \text{ #/ft} \]

\[ \text{UT OF WALL} \]

EDGE NAILING

\[ V = \frac{V}{L_{tot}} = 131 \text{ #/ft} \]

8d AT 6" O.C. G/F, 260 #/ft

8d AT 6" O.C.

OVERTURNING SEASON III

\[ \text{Eq.} \]
\[ M_{ot} = 69,780 \text{ #-ft} \]
\[ M_D = 110,450 \text{ #-ft} \]
\[ T = 415 \text{ #} \]
\[ \text{HD - 1} \]
SHEAR WALL DESIGN

LINE 3.7

\[ V_{Ed} = 9.0 \text{k} \]
\[ V_{w} = 5.46 \text{k} \]
\[ @ 14' \]
\[ \text{SEGMENT} \ i = 63.25' \]
\[ w_0 = \frac{15 \text{ PSF (14')}}{\text{WT. OF. WALL}} = 210 \text{ #/FT} \]

EDGE NAILING

\[ N = \frac{V}{L} = 143 \text{ #/FT} \]

Bd AT 6" O.C. 6xF, 260 #/FT

Bd AT 6" O.C.

OVERTURNING

\[ \text{Eq} \]
\[ M_{ot} = 126,000 \text{ #-FT} \]
\[ M_{o} = 420,000 \text{ #-FT} \]
\[ T = -1,430 \text{ #} \]

NO HOILS
SHEAR WALL DESIGN

LINE 4

\[ \frac{V_{eq}}{V} = \frac{1.5 \text{ k}}{0.75 \text{ k}} \quad \frac{8'}{16'} \quad \text{at} \quad 16' \text{s} \quad \text{left} \quad \text{cont.} \]

SEGMENT 1 = 8'

\[ W_D = 15 \text{ PBF} (16.5') = 248 \text{ #/ft} \]

\[ \frac{\text{Wt. of Wall}}{\text{ft. of Wall}} \]

EDGE NAILING

\[ V = \frac{V}{\text{Hut}} = 188 \text{ #/ft} \]

8d @ 6" o.c. @ G.F. 260 #/ft.

8d @ 6" o.c.

OVERTURNING

\[ EQ. \]

\[ M_{tot} = 24,760 \text{ #-ft} \]

\[ M_{0} = 7,194 \text{ #-ft} \]

\[ T = 2,588 \text{ #} \]

\[ \#0 - 1 \]
SHEAR WALL DESIGN

LINE 5

\[ N_{eq} = 3.02 \times 4.11 \times 14' = 144' \times 14' \leftarrow \text{CONT} \]

SEGMENT I = 21'
SEGMENT II = 24'
\[ \text{TOT} = 45' \]

\[ W_D = 15 \text{ PSF} (14') = 210 \text{ #/ft} \]

EDGE NAILING

\[ V = \frac{V}{L_{tot}} = 92' \text{ #/ft} \]

Bd. At 6" O.C. G.F. 200 #/ft

Bd. At 6" O.C.,

OVERTURNING - SEGMENT I

Eq.

\[ \text{MOT} = 27,050 \]
\[ \text{MD} = 416,1300 \]
\[ T = 150 \text{ #} \]

H0-1
SHEAR WALL DESIGN

LINE A

$V_{eq} = 8.9 \text{ k}$ @ 13.8'

SEGMENT I: $14.35^\circ$
SEGMENT II: $16^\circ$
SEGMENT III: $20^\circ$
SEGMENT IV: $10.33^\circ$
SEGMENT V: $10.75^\circ$

$L_{tot} = 79.08'$

$W_D = \frac{15 \text{ PSF (13')} + 20 \text{ PSF (21'2')}}{\text{WT. OF WALL}} = 405 \text{ #/FT}$

EDGE NAILING

$N = \frac{V}{L_{tot}} = 11.3 \text{ #/ft}$

BD AT 6" O.C. G.F.: 260 #/FT
BD AT 6" O.C.

OVERTURNING - SEGMENT

$M_0 = 11,800 \text{ #.-FT}$

$M_0 = 12,160 \text{ #.-FT}$

$T = 730 \text{ #}$

$H_0 = 1$
SHEAR WALL DESIGN

LINE D

\[ V_{eq} = 15.0 \text{ k} \quad @ \quad 18.75' \]

SEGMENT I = 58.75''
SEGMENT II = 17.75''
SEGMENT III = 50''

\[ L_{TOT} = 126.5' \]

\[ N_D = 15 \text{ psf (1/2'') \quad \frac{18.75'}{\text{VALL}} + 20 \text{ psf (4/2'') \quad = 690 \#/ft} \]

EDGE MACHING

\[ V = \frac{V}{L_{TOT}} = 119 \quad \#/ft \]

8d at 6'' o/c; G/F 260 \#/ft

8d at 6'' o/c.

OVERTURNING - SEGMENT II

EQ.

\[ M_{TOT} = 39,600 \quad \#-ft \]
\[ M_{O} = 108,700 \quad \#-ft \]
\[ T = -920 \# \]

NO H.D.'S REQ.
SHEAR WALL DESIGN

LINE E

\[ V_{eq} = 10.6 \text{k} \quad \text{at} \quad 10' \]

SEGMENT I = 42.5'
SEGMENT II = 22'
\[ L_{tot} = 64.5' \]

\[ W_0 = 15 \text{PSF} \left( \frac{15'}{\text{wall}} \right) + 20 \text{PSF} \left( \frac{32'}{\text{roof}} \right) = 545 \text{#/ft} \]

EDGE NAILING

\[ N = \frac{V}{L_{tot}} = 164 \text{ #/ft} \]

\[ \theta d \quad \text{at} \quad 6'' \quad 0.5'' \quad G.I.F. \quad 260 \text{ #/ft} \]

\[ \theta d \quad \text{at} \quad 6'' \quad 0.5'' \]

OVERTURNING - SEGMENT II

EQ:

\[ W_{tot} = 571,730 \]
\[ W_0 = 131,890 \]
\[ T = -4,600 \text{ #} \]

NO HO'S REQ.
SHEAR WALL DESIGN

LINE F

\[ V_{eq} = 2.7K \quad @ \quad 11.5' \]

SEGMENT I = 26.5'\quad 3 \quad L_{TOT} = 43.6' \]

\[ W_0 = \frac{15 \text{ PSF} (11')}{\text{WALL}} + \frac{20 \text{ PSF} (21'/2')}{\text{Roof}} = 375 \text{ #/FT} \]

EDGE NAILING

\[ V = \frac{V}{L_{TOT}} = 62 \text{ #/FT} \]

8d AT 6" OIC; G.N.F. 260 #/FT

8d AT 6" OIC

OVERTURNING - SEGMENT II

\[ Eq. \]

\[ W_{TOT} = 12,840 \text{ #-FT} \]

\[ W_0 = 60,750 \text{ #-FT} \]

\[ T = -1,025 \text{ #} \]

HD-1
**DIAPHRAGM/CHORD/COLLECTORS**

**DESIGN FORCES (ASCE 7-10 12.10.1.1)**

\[ F_{px} = \frac{F_x \cdot W_x}{2W_x} \rightarrow 1-\text{STORY} \]

\[ F_{px} = F_x \]

\[ F_{px} = 0.085 \text{W} \]

\[ F_{px,\text{MAX}} = 0.12 \cdot 50 \cdot 1.0 \cdot 0.17 = 0.085 \text{W} \]

\[ F_{px,\text{MAX}} = 0.14 \cdot 50 \cdot 1.0 \cdot 0.17 = 0.171 \text{W} \]

\[ F_{px} = 0.085 \text{W} \]

\[ W = 5.69 \text{K} \]

\[ F_{px} = 48.4 \text{K} \]

**STRUCTURAL IRREGULARITIES (ASCE 7-10 12.3.2)**

**HOLE STRUC. IRREG. #2 - REENTRANT CORNER IRREG.**

\[ B_{tot} = 12.7' \]

\[ L_{tot} = 18.4' \]

\[ B_1 = 6.3' \]

\[ L_1 = 4.8' \]

\[ \frac{B_1}{B_{tot}} = 0.50 > 0.15 \]

\[ \frac{L_1}{L_{tot}} = 0.24 > 0.15 \]

PER ASCE 7-10 12.3.4

- DESIGN CONN. OF DIAPHR. TO LFES FOR 125% F_{px}
- DESIGN COLLECTORS/CONNECTIONS FOR 125% F_{px}
## Diaphragm/Chords/Collectors

### Diaphragm Design - Transverse Direction

<table>
<thead>
<tr>
<th>LINE</th>
<th>Fx (k)</th>
<th>Fpx (k)</th>
<th>1/2SFPx(k)</th>
<th>LD</th>
<th>FpxxLd</th>
<th>12S</th>
<th>Fpx</th>
<th>LD</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>11.2</td>
<td>14.6</td>
<td>18.1</td>
<td>96'</td>
<td></td>
<td>191</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>13.5</td>
<td>17.4</td>
<td>21.7</td>
<td>127'</td>
<td></td>
<td>171</td>
<td></td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>9</td>
<td>11.8</td>
<td>14.7</td>
<td>127'</td>
<td></td>
<td>116</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>2.0</td>
<td>2.5</td>
<td>58'</td>
<td></td>
<td>66</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>31.6</td>
<td>4.7</td>
<td>6.9</td>
<td>64'</td>
<td></td>
<td>92</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Unblowned
1/2" Sheathing w/ 10d Nails at 6" o.c. Edges
12" o.c. Field to 3x Members
G.F. 240 #/ft (min.)

### Diaphragm Design - Longi Direction

<table>
<thead>
<tr>
<th>LINE</th>
<th>Fx (k)</th>
<th>Fpx (k)</th>
<th>1/2SFPx(k)</th>
<th>LD</th>
<th>FpxxLd</th>
<th>12S</th>
<th>Fpx</th>
<th>LD</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>89</td>
<td>11.6</td>
<td>14.5</td>
<td>144'</td>
<td></td>
<td>101</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>15</td>
<td>19.6</td>
<td>24.5</td>
<td>189'</td>
<td></td>
<td>130</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>10.6</td>
<td>13.9</td>
<td>17.3</td>
<td>189'</td>
<td></td>
<td>92</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>2.7</td>
<td>3.15</td>
<td>4.4</td>
<td>89'</td>
<td></td>
<td>49</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Diaph. G.F. 240 #/ft (min.)
DIAPHRAGM/CHORDS/COLLECTORS

CHORDS

MAXIMUM CHORD FORCE OCCURS IN THE TRANSVERSE DIR. BETWEEN LINES 2 & 3

\[ V = \frac{N_a}{L} \times 0.085 = 245 \text{ #/ft} \]

SEE FIG. 6b

\[ M_{\text{Dia.}} = \frac{V \cdot L^2}{8} = 306.3 \text{ K-ft} \]

\[ T_{\text{Chord}} = \frac{P_{\text{Load}} - M_{\text{Dia.}}}{E} = 3110 \text{ #} \]
OUT-OF-PLANE WALL DESIGN

EAVE WALLS

CHECK STUDS AT LINE E

GRANITY LOADING

ROOF TRIB = \( \frac{32}{2} = 16' \)

\( \frac{W_{O,20} \text{ PSF} (16')}{2} = 320 \frac{\#}{\text{FT}} \)

\( \frac{W_{S,20} \text{ PSF} (16')}{2} = 320 \frac{\#}{\text{FT}} \)

\( \frac{W_{W,R} \text{ PSF} (16')}{2} = 250 \frac{\#}{\text{FT}} \)

WALL HT. = 15'

\( \frac{W_{O,20} \text{ PSF} (15/2)}{2} = 112.5 \frac{\#}{\text{FT}} \)

(all at midpt of wall, located at max moment)

STUDS AT 16" O.C. --- CHECK STUD JOIST DIRECTLY BELOW 2.47' ROOF TRIB

\( P_d = 2.67 \left[ \frac{W_{O,20}}{2} + 1.33 \left( W_{O,20} \right) \right] = 1,095 \# \)

\( P_s = 2.67 \left[ \frac{W_{S,20}}{2} + 1.33 \left( W_{W,R} \right) \right] = 685 \# \)

OUT-OF-PLANE LOADING

STUD H.D. = 14' - 8"

At \( \frac{1}{16} \) = 70 FT²

@ \( \frac{1}{16} \) = 50 FT²

\( P_w = 25 \text{ PSF} \) (tone 4)

\( \text{Woop} = P_w (133') = 33 \frac{\#}{\text{FT}} \)

OUT-OF-PLANE DEFLECTION

\( \Delta = \frac{SWL^4}{384EI} \)

Check at service level --- 0.16

Components & cladding def. factor = 0.7

\( \Delta \text{SER} = 0.17 \times 0.7 \times \frac{SW \text{ Woop} L^4}{384EI} = 14,431,000 \times \text{IN}^3 \)

\( 2 \times 6 \) DEF. #2

\( EI = 33,275,000 \frac{\#}{\text{IN}^2} \)

\( \Delta \text{SER} = 0.14" \)

--- 4" 0.1"
OUT-OF-PLANE WALL DESIGN

EAVE WAALS (CONT.)

LOAD COMBINATIONS
1. D + 0.16W
   P = 1,420 #
   M = 6,1390 #-IN

2. D + 0.75S + 0.6W
   P = 1,955 #
   M = 4,796 #-IN

FLEXURE

\[ F_b = \frac{M}{S} \]
\[ S = 7.56 \text{ in}^2 \]
\[ F_{b1} = 845 \text{ psi} \]
\[ F_{b2} = 635 \text{ psi} \]
\[ F_b = \frac{1}{10} \cdot \frac{1.3}{(C_D)} \cdot \frac{1.35}{(C_F)} \cdot 900 \text{ psi} = 2,527 \text{ psi} \]

ALLOWABLE COMPRESSION

\[ \frac{\sqrt{F_c^2}}{F_c} + \frac{F_b}{F_b(1-F_c/F_{c.e})} \leq 1.0 \]
\[ F_c = \frac{P}{A} \]

\[ F_c = \frac{1.6}{(C_D)} \cdot \frac{1.1}{(C_F)} \cdot 0.187 \cdot 1.350 \text{ psi} = 444 \text{ psi} \]

L.C., #1 \[ F_b = 845 \text{ psi} \]
\[ F_c = 444 \text{ psi} \]
\[ F_c \text{ allow} = 243 \text{ psi} \]
\[ F_c = 173 \text{ psi} \]

L.C., #2 \[ F_b = 687 \text{ psi} \]
\[ F_c = 444 \text{ psi} \]
\[ F_c \text{ allow} = 275 \text{ psi} \]
\[ F_c = 237 \text{ psi} \]

2x6 DiFl #2 STUDS AT 16" OC
OUT-OF-PLANE WALL DESIGN

RAKE WALLS

CHECK STUDS AT RIDGE
AXIAL LOADING IS NEGLECTIBLE CHECK FLEXURE/DEFLECTION

OUT-OF-PLANE LOADING

STUD AT 17'  
\[ A_{t} = \frac{L_{t}}{S} = 96 \text{ FT}^2 \]

@ 100 FT^2  
\[ P_{w} = 24 \text{ PSF} \quad \text{(ZONE 4)} \]

\[ W_{oop} = P_{w} (1.52') = 32 \text{ #/FT} \]

DEFLECTION

\[ \Delta_{S3} = 0.17 \times 0.16 \times \frac{W_{oop} \cdot L_{c}^4}{8 \cdot E \cdot I} = \frac{25,257,000 \text{ #.IN}^3}{E \cdot I} \]

2x6 D.F. #2  
\[ E \cdot I = 33,275,000 \text{ #.IN}^2 \]

\[ \Delta_{S3} = 0.176'' \quad \text{AT 268} \text{ #/IN} \]

FLEXURE

\[ M = 0.26 \times W_{oop} \cdot L_{c}^2 \quad = 8,325 \text{ #.-IN} \]

\[ F_{b} = 1101 \text{ PSI} \quad < \quad F_{b} = 1.6 \times 1.3 \times 1.35 \times 900 \text{ PSI} = 2,527 \text{ PSI} \]

2x6 D.F. #2 AT 16'' O.C.
OUT-OF-PLANE WALL DESIGN

HEADERS - OUT-OF-PLANE

BUILT-UP HEADERS HAVE 2x FUAT T&D FOR OOP

Z-2x6 FUAT

\[ L_{\text{max}} = 8' \]
\[ A_t = 8' \times \left( \frac{12'}{2} \right) = 70 \text{ ft}^2 \]
\[ P_{\text{u}} = 2.5 \text{ PSF (STU)} \]
\[ W = 2.5 \text{ PSF} \left( \frac{12'}{2} \right) = 213 \text{ #/ft} \text{ (STU)} \]

\[ R = \frac{W L}{E} \]
\[ M = \frac{W L^2}{8} \]
\[ A = \frac{W L^2}{24} \text{ (EI)} \]
\[ S = 151 \text{ in}^3 \quad f_y = 47 \text{ PSI} \]
\[ S = 151 \text{ in}^3 \quad f_t = 813 \text{ PSI} \]
\[ I = 4160 \text{ in}^4 \quad \Delta w = 0.113'' \rightarrow \frac{L}{40} \]

Z-2x6 FUAT
OUT-OF-PLANE WALL DESIGN

HEADER KING STUDS

\[ L_{\text{max}} = 4' - 0'' \]

COOP TRIBUTARY \( \leq \frac{4'}{2} + \frac{1.53'}{2} = 2.61' \)

2 X TRIB TYP. STUD

2 - 2X6 STUDS OK! BY INSPECTION

LOWEST \( L_{\text{max}} = 8' - 0'' \)

COOP TRIBUTARY \( \leq \frac{8'}{2} + \frac{1.33'}{2} = 4.67' \)

2 X TRIB TYP. STUD TAKE

3 - 2X6 STUDS OK! BY INSPECTION \( \Rightarrow \) DOES NOT OCCUR AT 17' STUDS
WALL STILL ANCHORAGE

IN-PLANE LOADING

AT 6" E.N. \[ V_{\text{max}} = 188 \text{ #/ft} \] @ LINE 4
AT 2" E.N. \[ V_{\text{max}} = 325 \text{ #/ft} \] @ LINE 2

EXTERIOR WALLS

5/8" L-BOLTS CONFORMING TO 2014 OESC
EXCEPTION 1905.1.9

6" E.N.

\[ \frac{2\pi}{12} = 0.17 \\ \frac{12}{16} = 0.75 \\
\]

\[ V = 188 \text{ #/ft} \]

\[ A_{\text{req}} = \frac{V}{f_{\text{u}}} = 7.3' \rightarrow 4' SPC \]

7" E.N.

\[ \frac{2\pi}{12} = 0.26 \\ \frac{12}{16} = 0.75 \\
\]

\[ V = 325 \text{ #/ft} \]

\[ A_{\text{req}} = 5.2' \rightarrow 4' SPC \]

INTERIOR WALLS

HALL INT WALLS ARE 6" E.N.

* 5/8" SCH 40 TITEN HO 1/4" EMB.

DESIGN PER ACI 318-11 D13.3.5.3 (W)

6" E.N.

\[ V_{\text{ACO}} = 4' \times 188 \text{ #/ft} = 752 \text{#} \]

\[ V_{u} = 1,075 \text{#} \]

\[ S_{L}V = 3' \times 1,075 \text{#} = 3,225 \text{#} \]

SEE FOLLOWING SIMPSON ANALYSIS

5/8" TITEN HO AT 4' OK.
1. Project Information
Customer company:
Customer contact name:
Customer e-mail:
Comment:

2. Input Data & Anchor Parameters
General
Design method: ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.625
Nominal Embedment depth (inch): 4.000
Effective Embedment depth, \( h_{e} \) (inch): 2.970
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
\( h_{bw} \) (inch): 6.00
\( C_{rr} \) (inch): 4.50
\( C_{mr} \) (inch): 1.75
\( S_{mr} \) (inch): 3.00

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: not satisfied
Ductility section for shear: D.3.3.5.3 (c) is satisfied
\( \Omega_{c} \) factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Base Material
Concrete: Normal-weight
Concrete thickness, \( h \) (inch): 24.00
State: Cracked
Compressive strength, \( f_{c} \) (psi): 3000
\( \Psi_{s,v} \): 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Do not evaluate concrete breakout in tension: No
Do not evaluate concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate

<Figure 1>
Recommended Anchor
Anchor Name: Titen HD® - 5/8"Ø Titen HD (THDB model), hnom: 4" (102mm)
Code Report Listing: ICC-ES ESR-2713
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, N_{ax} (lb)</th>
<th>Shear load x, V_{ax} (lb)</th>
<th>Shear load y, V_{ay} (lb)</th>
<th>Shear load combined, \sqrt{(V_{ax})^2+(V_{ay})^2 (lb)}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>3225.0</td>
<td>0.0</td>
<td>3225.0</td>
</tr>
<tr>
<td>Sum</td>
<td>0.0</td>
<td>3225.0</td>
<td>0.0</td>
<td>3225.0</td>
</tr>
</tbody>
</table>

Maximum concrete compression strain (\varepsilon_{c})= 0.00
Maximum concrete compression stress (psi)= 0
Resultant tension force (lb)= 0
Resultant compression force (lb)= 0
Eccentricity of resultant tension forces in x-axis, \varepsilon_{ax} (inch)= 0.00
Eccentricity of resultant tension forces in y-axis, \varepsilon_{ay} (inch)= 0.00
Eccentricity of resultant shear forces in x-axis, \varepsilon_{ax} (inch)= 0.00
Eccentricity of resultant shear forces in y-axis, \varepsilon_{ay} (inch)= 0.00

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

<table>
<thead>
<tr>
<th>V_{ax} (lb)</th>
<th>\phi_{out}</th>
<th>\phi</th>
<th>\phi_{out}\phi V_{ax} (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8000</td>
<td>1.0</td>
<td>0.60</td>
<td>4800</td>
</tr>
</tbody>
</table>

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in x-direction:

\[ V_{bx} = \min(\frac{\sqrt{3}}{d_s}, d_s, d_s) \cdot \frac{f_{c}_{con}}{\tan^{1.5}} + 9 \lambda x^{1.5} \cdot \tan^{1.5} (\text{Eq. D-33 & Eq. D-34}) \]

<table>
<thead>
<tr>
<th>l_s (in)</th>
<th>d_s (in)</th>
<th>\lambda_s</th>
<th>\Gamma_s (psi)</th>
<th>c_{con} (in)</th>
<th>V_{bx} (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.97</td>
<td>0.63</td>
<td>1.0</td>
<td>16.00</td>
<td>3000</td>
<td>26494</td>
</tr>
</tbody>
</table>
\[ \phi V_{bx} = \phi (A_{con} / A_{tot}) \psi_{d,c} V_{d,c} \psi_{c} V_{c} \psi_{b} V_{b} \text{ (Sec. D.4.1 & Eq. D-30)} \]
\[ A_{bc} (in^2) \quad A_{tot} (in^2) \quad \psi_{d,c} V_{d,c} \quad \psi_{c} V_{c} \quad \psi_{b} V_{b} \quad V_{bx} (lb) \quad \phi \quad \phi V_{bx} (lb) \]
\[ 576.00 \quad 1152.00 \quad 0.850 \quad 1.000 \quad 1.000 \quad 26494 \quad 0.70 \quad 7882 \]

Shear parallel to edge in x-direction:

\[ V_{by} = \min(\frac{\sqrt{3}}{d_s}, d_s, d_s) \cdot \frac{f_{c}_{con}}{\tan^{1.5}} + 9 \lambda x^{1.5} \cdot \tan^{1.5} (\text{Eq. D-33 & Eq. D-34}) \]

<table>
<thead>
<tr>
<th>l_s (in)</th>
<th>d_s (in)</th>
<th>\lambda_s</th>
<th>\Gamma_s (psi)</th>
<th>c_{con} (in)</th>
<th>V_{by} (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.97</td>
<td>0.63</td>
<td>1.0</td>
<td>12.00</td>
<td>3000</td>
<td>17209</td>
</tr>
</tbody>
</table>
\[ \phi V_{by} = \phi (A_{con} / A_{tot}) \psi_{d,c} V_{d,c} \psi_{c} V_{c} \psi_{b} V_{b} \text{ (Sec. D.4.1 & Eq. D-30)} \]
\[ A_{bc} (in^2) \quad A_{tot} (in^2) \quad \psi_{d,c} V_{d,c} \quad \psi_{c} V_{c} \quad \psi_{b} V_{b} \quad V_{by} (lb) \quad \phi \quad \phi V_{by} (lb) \]
\[ 648.00 \quad 648.00 \quad 1.000 \quad 1.000 \quad 1.000 \quad 17209 \quad 0.70 \quad 24092 \]

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

\[ \phi V_{yp} = \phi V_{con} = \phi V_{con} \cdot A_{con} \cdot V_{con} \cdot N_{bc} (\text{Eq. D-40}) \]

<table>
<thead>
<tr>
<th>\lambda_p</th>
<th>A_{tot} (in^2)</th>
<th>A_{con} (in^2)</th>
<th>\psi_{d,c} V_{d,c}</th>
<th>\psi_{c} V_{c}</th>
<th>\psi_{b} V_{b}</th>
<th>N_{bc} (lb)</th>
<th>\phi</th>
<th>\phi V_{yp} (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>79.39</td>
<td>79.39</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>4766</td>
<td>0.70</td>
<td>6672</td>
</tr>
</tbody>
</table>

11. Interaction of Tensile and Shear Forces (Sec. D.7)

<table>
<thead>
<tr>
<th>Shear</th>
<th>Factored Load, V_{ax} (lb)</th>
<th>Design Strength, \sigma_{V_{ax}} (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>3225</td>
<td>4800</td>
<td>0.67</td>
<td>Pass (Governs)</td>
</tr>
<tr>
<td>T Concrete breakout x+</td>
<td>3225</td>
<td>7882</td>
<td>0.41</td>
<td>Pass</td>
</tr>
</tbody>
</table>

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
5/8"Ø Titen HD (THDB model), hnom:4" (102mm) meets the selected design criteria.

12. Warnings
- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections D.8.1 and D.8.2 for torqued cast-in-place anchor is waived per designer option.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer’s product literature for hole cleaning and installation instructions.
WALL STUD ANCHORAGE

OUT-OF-PLANE LOADING

MAX. STUD HT. ≈ 17'
AT ≈ 100 FT² → PW = 24 PSF (STE)

STUD ANCHORS AT 4'-0" O.C.

\[ PW = 1.10 \cdot PW \cdot \left( \frac{17'}{2} \right) \cdot 4' = 816 \text{ #} \]

\[ PSO = 0.16 \cdot PW = 490 \text{ #} \]

1/2" Hi-Fi STUD PLATE

\[ 2' = 2.0 \cdot 2' = 1.6 \cdot 420# = 672# \]

\[ PSO = 490 \text{ #} \]

5/8" Ø ANCH. AT 4'-0" O.C.

ANCH. TO CONCRETE

\[ PV = 816 \text{ #} \]

SEE FOLLOWING SIMPSON ANALYSIS

5/8" Ø ANCHORS AT 4'-0" O.C.
1. Project Information
Customer company: 
Customer contact name: 
Customer e-mail: 
Comment: 

2. Input Data & Anchor Parameters
General
Design method: ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, \( h_0 \) (inch): 7.000
Anchor category: -
Anchor ductility: Yes
\( h_{sw} \) (inch): 8.38
\( C_{min} \) (inch): 0.81
\( S_{min} \) (inch): 2.50

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Base Material
Concrete: Normal-weight
Concrete thickness, \( h \) (inch): 24.00
State: Uncracked
Compressive strength, \( f'_c \) (psi): 3000
\( \psi_{LV} \): 1.4
Reinforcement condition: B tension, B shear
Supplemental reinforcement: No
Do not evaluate concrete breakout in tension: No
Do not evaluate concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Recommended Anchor
Anchor Name: J- or L-Bolt - 5/8"Ø J- or L-Bolt, F1554 Gr. 36

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.447.3671 www.strongtie.com
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_{ax}$ (lb)</th>
<th>Shear load x, $V_{ax}$ (lb)</th>
<th>Shear load y, $V_{ay}$ (lb)</th>
<th>Shear load combined, $\sqrt{V_{ax}^2 + V_{ay}^2}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>816.0</td>
<td>816.0</td>
</tr>
<tr>
<td>Sum</td>
<td>0.0</td>
<td>0.0</td>
<td>816.0</td>
<td>816.0</td>
</tr>
</tbody>
</table>

Maximum concrete compression strain (%e): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, $\epsilon_{e_x}$ (inch): 0.00
Eccentricity of resultant tension forces in y-axis, $\epsilon_{e_y}$ (inch): 0.00
Eccentricity of resultant shear forces in x-axis, $\epsilon_{e_x}$ (inch): 0.00
Eccentricity of resultant shear forces in y-axis, $\epsilon_{e_y}$ (inch): 0.00

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

<table>
<thead>
<tr>
<th>$V_{as}$ (lb)</th>
<th>$\varphi_{mst}$</th>
<th>$\varphi$</th>
<th>$\varphi_{min}V_{as}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7865</td>
<td>1.0</td>
<td>0.65</td>
<td>5112</td>
</tr>
</tbody>
</table>

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min[(7/6)(d_s)^2 / d_e \sqrt{f_{c}c_{ay}}^{1.5}, 9 \lambda_{s} / \sqrt{f_{c}c_{ay}}^{1.5}]$ (Eq. D-33 & Eq. D-34)

<table>
<thead>
<tr>
<th>$d_e$ (in)</th>
<th>$d_s$ (in)</th>
<th>$\lambda_s$</th>
<th>$f_c$ (psi)</th>
<th>$c_{ay}$ (in)</th>
<th>$V_{by}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00</td>
<td>0.63</td>
<td>1.00</td>
<td>3000</td>
<td>3.00</td>
<td>2387</td>
</tr>
</tbody>
</table>

$\varphi V_{by} = \varphi (A_{f}_c / A_{wet}) \varphi_{st} V_{c} \varphi_{bl} V_{by}$ (Sec. D.4.1 & Eq. D-30)

<table>
<thead>
<tr>
<th>$A_{wet}$ (in$^2$)</th>
<th>$A_{f}_c$ (in$^2$)</th>
<th>$\varphi_{st}$</th>
<th>$V_{c}$</th>
<th>$\varphi_{bl}$</th>
<th>$V_{by}$ (lb)</th>
<th>$\varphi$</th>
<th>$\varphi V_{by}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40.50</td>
<td>40.50</td>
<td>1.00</td>
<td>1.400</td>
<td>1.00</td>
<td>2387</td>
<td>0.70</td>
<td>2340</td>
</tr>
</tbody>
</table>

Shear parallel to edge in y-direction:

$V_{by} = \min[(7/6)(d_s)^2 / d_e \sqrt{f_{c}c_{ay}}^{1.5}, 9 \lambda_{s} / \sqrt{f_{c}c_{ay}}^{1.5}]$ (Eq. D-33 & Eq. D-34)

<table>
<thead>
<tr>
<th>$d_e$ (in)</th>
<th>$d_s$ (in)</th>
<th>$\lambda_s$</th>
<th>$f_c$ (psi)</th>
<th>$c_{ay}$ (in)</th>
<th>$V_{by}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00</td>
<td>0.63</td>
<td>1.00</td>
<td>3000</td>
<td>16.00</td>
<td>29403</td>
</tr>
</tbody>
</table>

$\varphi V_{by} = \varphi (2)(A_{f}_c / A_{wet}) \varphi_{st} V_{c} \varphi_{dl} V_{by}$ (Sec. D.4.1 & Eq. D-30)

<table>
<thead>
<tr>
<th>$A_{wet}$ (in$^2$)</th>
<th>$A_{f}_c$ (in$^2$)</th>
<th>$\varphi_{st}$</th>
<th>$V_{c}$</th>
<th>$\varphi_{dl}$</th>
<th>$V_{by}$ (lb)</th>
<th>$\varphi$</th>
<th>$\varphi V_{by}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>144.00</td>
<td>1152.00</td>
<td>1.00</td>
<td>1.400</td>
<td>1.00</td>
<td>29403</td>
<td>0.70</td>
<td>7204</td>
</tr>
</tbody>
</table>

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\varphi V_{py} = \varphi L_{py} N_{py} = \varphi A_{wet} \varphi_{st} V_{c} \varphi_{py} V_{by} N_{py}$ (Eq. D-40)

<table>
<thead>
<tr>
<th>$A_{wet}$ (in$^2$)</th>
<th>$V_{c}$</th>
<th>$\varphi_{py}$</th>
<th>$N_{py}$ (lb)</th>
<th>$\varphi$</th>
<th>$\varphi V_{py}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>126.00</td>
<td>441.00</td>
<td>0.786</td>
<td>0.73</td>
<td>5564</td>
</tr>
</tbody>
</table>

11. Interaction of Tensile and Shear Forces (Sec. D.7)

<table>
<thead>
<tr>
<th>Shear</th>
<th>Design Strength, $\sigma V_{as}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>816</td>
<td>0.16</td>
<td>Pass</td>
</tr>
<tr>
<td>T Concrete breakout y+</td>
<td>816</td>
<td>0.35</td>
<td>Pass (Governs)</td>
</tr>
</tbody>
</table>
5/8"Ø J- or L-Bolt, F1554 Gr. 36 with hef = 7.000 inch meets the selected design criteria.

12. Warnings
- Minimum spacing and edge distance requirement of 6ds per ACI 318 Sections D.8.1 and D.8.2 for torqued cast-in-place anchor is waived per designer option.
- Designer must exercise own judgement to determine if this design is suitable.
FOUNDATION DESIGN

BEARING WALL FOOTINGS - EXTERIOR BEARING WALLS

LINE E - EXTERIOR BEARING WALL

ROOF CONTRIBUTORY = $\frac{32'}{6'} = 16'$

$W_{O,K} = 2.0$ PSF (16') = 320 $\#/ft$
$W_{S,K} = 2.0$ PSF (16') = 320 $\#/ft$
$W_{W,K} = 1.0$ PSF (16') = 256 $\#/ft$
$W_{W,K} = -2.5$ PSF (16') = -400 $\#/ft$

WALL HT = 15'
$W_{O,W} = 15$ PSF (15') = 225 $\#/ft$

2' WIDE x 1'-0" DEEP (BELOW 1'-0" CONC.)

$W_{FTG} = 600$ $\#/ft$

D+5

$W = 1,485$ $\#/ft$
$g = \frac{w}{h} = 733$ PSF
$g$ allow = 2,000 PSF

$0.40 + 0.16W$
$W = 447$ $\#/ft$

2'-0" WIDE x 1'-0" DEEP FTG

O.K.
FOUNDATION DESIGN

BEARING WALL FOOTINGS - INTERIOR

LINE D - INTERIOR BEARING WALL

\[
\text{ROOF TRIBUTARY} = \frac{42' \times 32'}{2} = 37'
\]

\[
\begin{align*}
W_{DF} &= 20 \text{ kips/ft} \\
W_{DF} &= 20 \text{ kips/ft} \\
W_{DF} &= 16 \text{ kips/ft} \\
W_{DF} &= 22.5 \text{ kips/ft}
\end{align*}
\]

\[
\text{WALL HT} = 17.5'
\]

\[
W_{o.w} = 1 \text{ kips/ft} \times 2 \text{ kips/ft} = 210 \text{ kips/ft}
\]

\[
2' \text{ WIDE} \times 1' \text{ DEEP} \text{ BELOW CONC,}
\]

\[
W_{fig} = 600 \text{ kips/ft}
\]

\[
\begin{align*}
D+S \\
W &= 2.290 \text{ kips/ft} \\
B &= 1,145 \text{ kips/ft} \\
(a) &= 2,000 \text{ kips/ft} \text{ O.K.}
\end{align*}
\]

\[
W_{o.w} = 0.16W
\]

\[
W = 375 \text{ kips/ft} > 0 \text{ O.K.}
\]

\[
2' \text{-O" WIDE} \times 1' \text{-O" DEEP FIG}
\]
FOOTING DESIGN

SPREAD FOOTINGS

MAX LOADING OCCURS AT COLUMN C1 (SEE PG. B29)

\[ P_0 = 13.9 \, k \]
\[ P_0 = 13.8 \, k \]
\[ P_0 = 10.16 \, k \]
\[ P_0 = 716.5 \, k \]

\[ 5' \times 5' \times 15' \, FTG \]

\[ W_{FTG} = 150 \, PCF (5' \times 5' \times (15' + 1')) \]

\[ 9.4 \, k \]

\[ D + S \]

\[ P_{total} = 3615.5 \, k \]

\[ g = \frac{P}{A} = 1460 \, PSF \]

\[ g_{allow} = 2000 \, PSF \]

\[ 0.160 - 0.160 \]

\[ P_{total} = 41.1 \, k \]

\[ 0 \]

\[ 0.160 \]

\[ 5' \times 5' \times 1'-6'' \, FTG \]
FOUNDATION DESIGN

BEARING WALL FOOTINGS

COLUMN AT CONT. WALL FOOTING

MAXIMUM LOADING OCCURS AT COLUMN SUPPORTING KB-2

\[ P_0 = 7.9\text{ k} \]
\[ P_1 = 2.8\text{ k} \]
\[ P_2 = 2.8\text{ k} \]
\[ P_4 = -9.1\text{ k} \]

CONTINUOUS 2'-WIDE X 11'- DEEP FOR KB6 (BELOW CONCRETE)

\[ W_{FTG} = 600\text{ #/ft} \]
\[ W_{WALL} = 15\text{ PSF}(16\text{"}) = 240\text{ #/ft} \]

D+S

\[ P = 152\text{ k} \]

\[ L_{REQ} = \frac{P}{(B_{LOW}\times B \times (W_{FTG} + W_{WALL}))} = 4.81\text{', O.K.} \]

\[ 0(60 + 0.6W) \]

CHECK WITH 4.81'-LENGTH

\[ P_0 = 11.94\text{ k} \]
\[ P_2 = -9.1\text{ k} \]
\[ 2(0.5P_0 + 0.5P_2) + 11700 \text{ #} \geq 700 \text{ O.K.} \]
FOUNTAIN DESIGN

BEARING WALL FOOTINGS

COL. AT CONT. WALL FOOTING (CONT.)

BASE DESIGN

BOTTOM BARS: 12.0 + 1.68 + 0.50 (W) \[ Pu = \frac{28.1K}{3,000} \text{ PSF} \]

\[ M_u = 3u \times (4.81') \left( \frac{W}{2} \right) \]
\[ = 71.2 \text{ k-ft} \]

3 - #5 BOTTOM BARS

SEE PG. 48 \[ \phi M_u = 33.6 \text{ k-ft} \]

TOP BARS

\[ P_0 = \frac{7.9K}{9.1} \text{ K} \]

\[ 0.9D + 1.0W : \quad Pu = -2.0K \]

\[ L_{req} = \frac{Pu}{(0.7)(W + 1.0W)} = 2.65' \rightarrow 1.33' \text{ CANT.} \]

\[ Mu_{ftg} = 0.9(W + 1.0W) \times 133'^2 \]
\[ = 0.7 \text{ k-ft}^2 \]

1 - #5 BAR AT TOP OF STEM WALL O.K. BY INSP.
FOUNDATION DESIGN

SHEAR WALL FOOTINGS - AT EXT. BEARING WALL

LINE D - 11'-0" SEGMENT

LATERAL LOADING
\[ V_0 = 15,010 \text{ (kips)} \]
\[ L_{HFL} = 110.25' \]

SEISMIC LOAD TO 11' WALL SEGMENT
\[ V = V_0 \times \frac{L_{HFL}}{110.25'} = 2,180 \text{ kips} \]

GRAVITY LOADING
\[ P_{LW} = 15 \text{ PSF} (H_W) (L) + 20 \text{ PSF} (L_{ROOF}) (L) = 7,440 \text{ kips} \]
\[ P_{HTG} = 150 \text{ kips/ft} (B_{HTG}) (H_{HTG}) (L + 2 \times L_T) \]

2'-0" WIDE x 1'-0" DEEP FTG (BELOW 1'-0" CONC.)

\[ L_T = 2' \]
\[ B_{HTG} = 2' \]
\[ H_T = 2' \]
\[ P_{HTG} = 12,000 \text{ kips} \]
\[ P_0 = 19,440 \text{ kips} \]
\[ M_{HTG} = \frac{V (H_W + H_T)}{4} = 11,430 \text{ kips-ft} (A SD) \]
\[ D = 0.71: \]
\[ P = 19,440 \text{ kips} \]
\[ M = 11,430 \text{ kips-ft} \]
\[ C = \frac{M}{P} = 2.13' \]
\[ \gamma_{MAX} = \frac{(P)}{(A)} (1 + \frac{L_E}{L_{TOT}}) = 797 \text{ PSF} < \gamma_{ALLOW} = 3,000 \text{ PSF} \]

2'-0" WIDE x 1'-0" DEEP FTG
FOUNDATION DESIGN

SHALLOW WALL FOOTING

LINE 2 - S' WALL SEGMENT

LATERAL LOADING

\[ V_s = 112 \text{ k} \]

\[ L_{wall} = 34'15" \]

SEISMIC LOAD TO S' WALL SEGMENT

\[ V = V_s \times \frac{L_{wall}}{1625} = 1625 \text{ # (A50)} \]

GRAVITY LOADING

\[ P_w = 15 \text{ PSF}(H_w)(L) \]

\[ P_{ftg} = 160 \text{ PCF}(2')(1.5'(1')/(L+2\times L_T) \]

2' WIDE x 1' DEEP FTG (BELOW 1' CONK.)

TRY 4' TOE

\[ P_{w} = 1.27 \text{ k} \]

\[ P_{ftg} = 1.8 \text{ k} \]

\[ M_{tot} = V(H_w + H_f) \]

\[ = 30.1 \text{ k} \text{ ft} \]

\[ e = \frac{M_{tot}}{P_{tot}} = 5.6' \]

\[ g_{max} = \frac{2 \times 0.6 \text{ k}}{8(L_{tot} + 2 - e)} = 248.6 \text{ PSF} < g_{mu} = 3,000 \text{ PSF} \]

2' WIDE x 1' DEEP FTG
FOUNDATION DESIGN

SHEAR WALL FOOTING

LINE 2 - 5' WALL SEGMENT (CONT.)

REINFORCING

\[ V_u = 2,320 \, \text{#} \]
\[ M_u = V_u \cdot (H_u + H_F) = 441 \, \text{K-ft} \]
\[ P_d = 9107 \, \text{K} \]

0:778D + 1:0E

\[ R = 7.06 \, \text{K} \quad M_u = 441 \, \text{K-ft} \quad e = 0.625 \, \text{ft} \]

\[ V_u = 7106 \, \text{K} \]
\[ M_u = T = 2615 \, \text{K-ft} \]

ENTIRE RESULTANT @ THE

FLEXURAL REINF. - BOTTOM BARS

3-#5 \quad A_s = 0.92 \, \text{in}^2 \quad a = 0.91 \, \text{in}

\[ \phi M_n = f_y A_s \quad (d - a/2) \]
\[ \phi = 0.90 \]
\[ f_y = 60 \, \text{ksi} \]
\[ d = 8\text{in} \]

\[ \phi M_n = 33.6 \, \text{K-ft} \quad > M_u = 2615 \, \text{K-ft} \]

SHEAR REINF.

\[ \phi V_c = \frac{f_c}{f_y} \times \phi \times \phi \]
\[ \phi = 0.75 \]
\[ f_c = 3000 \, \text{ksi} \]
\[ \phi = 0.85 \]
\[ a = 2\text{in} \]

\[ \phi V_c = 10.7 \, \text{K} \quad > \quad V_u = 7106 \, \text{K} \]

NO SHEAR REINF. REQ.
FOUNDATION DESIGN

EXTERIOR WALL FOOTINGS - OOP DESIGN

 DESIGN AT RAKE WALL NEAR RIDGE

WALL HT < 18'

GRAVITY LOADING

\[ W_0 = 15 \text{ pcf (18')} = 270 \# / \text{ft} \]

OUT-OF-PLANE LOADING

SEE PG 135 \( P_W = 24 \text{ pcf (STK)} \)

\[ V = P_W (1'HALL/2) = 216 \# / \text{ft} (STK) \]

ETC/SLAB DL

\[ W_0 = 150 \text{ pcf} [0.5' \times 0.5' + 2' \times 2.7'] = 640 \# / \text{ft} \]

\[ 0.6W + 0.16W \]

\[ W = 545 \# / \text{ft} \]

\[ V = 130 \# \]

\[ M = V (1'H) = 325 \# / \text{ft} \]

\[ P = W (1') = 545 \# \]

\[ e = M / P = 0.60 \]

\[ g_{max} = \frac{2 \cdot P}{3(B/2 - C)(1')^2} = 908 \text{ psf} < \gamma_{HALL} = 3,000 \text{ psf} \text{ OK} \]

SLIDING

\[ M = 0.14D \text{ (PER GEO - ULTIMATE VALVE - CONSERVE FOR ASD)} \]

\[ M_P = 218 \# > V = 130 \# \text{ OK} \]