## Project Report

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<td>50</td>
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<tr>
<td>6'-0&quot; x 6'-0&quot; Isolated Footing (F12)</td>
<td>52</td>
</tr>
<tr>
<td>8'-0&quot; x 8'-0&quot; Isolated Footing (F16)</td>
<td>58</td>
</tr>
<tr>
<td>8'-0&quot; x 8'-0&quot; Isolated Footing (F16)</td>
<td>64</td>
</tr>
<tr>
<td>10'-0&quot; x 10'-0&quot; Isolated Footing (F20)</td>
<td>70</td>
</tr>
<tr>
<td>17'-0&quot; x 8'-0&quot; Isolated Footing (F101)</td>
<td>76</td>
</tr>
</tbody>
</table>
Project Narrative

Architectural Vision

New York City’s skyline is decorated with a diverse set of skyscrapers and tall buildings. Due to an ever expanding population, the need for the development of residential buildings is prevalent now more than ever. However, due to the expensive cost of living in Manhattan, many residents have turned to the outer boroughs such as Queens and Brooklyn. There are many reasons to build in Brooklyn; there is more land territory to build than in Manhattan and the lower cost means that developers do not have to be billion dollar investors or corporations. In fact, New York’s largest residential construction boom exists in Brooklyn. In Downtown Brooklyn alone, more than $6.5 billion has been planned for new residential projects. Collaborative Design’s proposal is to conceptually design, structurally develop and provide construction plans to fill a void in luxury living in Brooklyn. Shuriken Tower will accomplish all the objectives required and deliver a structurally sound, aesthetically pleasing and economically feasible structure.

Project Objectives

The project is located at 8791 21st Ave. in Brooklyn NY. The owner requires that the structure have a minimum footprint of 12,000 square feet and a minimum of sixteen stories. The top stories must have a setback on all four sides. A total of two underground levels must be provided and the first underground level must be for parking. A green roof system must be investigated and proposed. An energy efficient exterior wall system must be implemented. The building should also include amenities such as a gymnasium, a swimming pool, etc. The owner has also requested submittals of architectural layout plans with means of egress in accordance with the International Building Code (IBC). A framing layout for the chosen structural system must be prepared. Drawings and specifications for the structural plans must be delivered. Drawings should include a foundation plan, a footing schedule, a beam schedule, sections, notes and typical details. Structural analysis and design of the gravity and
lateral loads and forces must be performed according to the relevant codes and standards. A sample manual of calculations to verify results of computer aided analysis and design must be completed. Lastly, an engineer’s cost estimate and construction schedule must be carried out.

**Shuriken Tower**

The owner has demanded a residential building where his profits and investments can be maximized. Shuriken Tower will contain of 19 stories with a three story setback for penthouses. The first floor will consist of a lobby and commercial space. The other 15 stories comprise of apartment dwellings, including studios, one and two bedrooms. The 17th floor penthouses will have private terraces. The first story underground will be an underground parking lot with 114 parking spaces. The sub-cellar will feature amenities such as a bowling alley, a swimming pool, a spa and personal storage to fulfill the owner’s requirements. A 2.5 in green roof system was selected and a curtain wall glass façade system was chosen (see drawings for details).

**Structural Plan**

In order to achieve this, a concrete flat plate construction optimal for residential construction was proposed. This is because the gravity system of a concrete flat plate consists of only an 8in to 12in concrete slab, as opposed to steel construction where the floor slab could reach up to two feet due to the depth of the beam, concrete slab, and metal deck and HVAC equipment. Hence, by saving up to a foot per floor, the owner will gain additional height and can add more stories to his building to maximize profit.

Concrete columns were laid out based on the Table 9.5 of the ACI318-11 provision on clear span. For an 8 inch thick slab, the maximum column to column clear span is 20ft. The majority of concrete columns were hidden between walls and closets to prevent obstruction and maximize space of residents. The columns were designed using spColumn. Since 20ft spans are insufficient for parking maneuvering, transfer beams were utilized to transfer loads from the ground floor to the cellar floor to increase span lengths. Four L shaped columns were placed in the four re-entrant corners of the structure since they are more effective than rectangular shaped columns. The slab reinforcement was
designed using SAFE. The lateral force resistant system includes four central core concrete shear walls that are orthogonal to each other which will run up from the sub-cellar all the way to the roof of the structure. Enclosed in these walls, are four passenger elevators and one freight elevator. In addition, four staircase perimeter C-shaped shear walls will provide torsional stability from the ground to the 16th floor. The shear walls, along with a rigid diaphragm was modeled and designed on ETABS.

In the penthouse levels, light gage metal stud framing was selected to decrease the load transfer of the slab below. Moreover, cold formed steel decreases the weight of the structure which effectively minimizes the effect of seismic loads of the structure. Most of the framing plans of the floors are similar (floors 2 to 16 are the same, floors 17 to 19 are the same) to allow for fast construction and economic savings. Formwork can be re-used and the method for pouring concrete and erection of steel can be repeated. Furthermore, the symmetrical layout of the building enables the mechanical, electrical and plumbing runs to be shared between adjacent runs, thereby maximizing the amount of leasable space.

The foundation system will entail isolated square footings for most columns. For locations where two columns are very close to another, a rectangular combined footing was utilized. The foundation wall and wall footing was analyzed and designed. All loads were determined in accordance to American Society of Civil Engineering (ASCE 7-10) Standard and the design of concrete structures was performed with reference to the American Concrete Institute (ACI318-11) Building Code. Light gage metal stud framing was designed with the assistance of MarinoWare manufacturer’s catalog.
Load Calculations

Dead Loads
The weights of materials for the calculation of dead load were determined with table C3-1 of ASCE7-10. The weight of normal weight concrete is assumed to 150pcf.

Sample Calculation for Weight of Concrete Slab
The weight of a 12 inch concrete slab is:

\[
\frac{12in \cdot ft}{1 \cdot 12in \cdot ft^3} \cdot 150lb = 150lb/ft^2
\]

Sub-cellar Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0” Concrete Slab</td>
<td>70.5</td>
</tr>
<tr>
<td>Floor Finish (Ceramic Tile)</td>
<td>10</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
<tr>
<td>Total</td>
<td>86.5</td>
</tr>
</tbody>
</table>

Cellar Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0” Concrete Slab</td>
<td>150</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
<tr>
<td>Total</td>
<td>160</td>
</tr>
</tbody>
</table>

Ground (1st Floor - Inside) Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0” Concrete Slab</td>
<td>150</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Floor Finish (Ceramic Tile)</td>
<td>16</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
<tr>
<td>Total</td>
<td>176</td>
</tr>
</tbody>
</table>

Ground (1st Floor - Outside) Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0” Concrete Slab</td>
<td>150</td>
</tr>
<tr>
<td>12.0” Green Ground layer</td>
<td>45</td>
</tr>
<tr>
<td>Total</td>
<td>195</td>
</tr>
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</table>

2nd Floor Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.0” Concrete Slab</td>
<td>112.5</td>
</tr>
</tbody>
</table>
### Floor Finish (Ceramic Tile)
- **Weight (psf):** 10

### Miscellaneous
- **Weight (psf):** 6

### Ceiling
- **Weight (psf):** 5

### Total
- **Weight (psf):** 133.5

#### 3rd - 15th Floor Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.0” Concrete Slab</td>
<td>100</td>
</tr>
<tr>
<td>Floor Finish (Ceramic Tile)</td>
<td>10</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
</tbody>
</table>

| Total                           | 116          |

#### 16th Floor Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0” Concrete Slab</td>
<td>150</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Floor Finish (Ceramic Tile)</td>
<td>10</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
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</table>

| Total                           | 170          |

#### 17th - 18th Floor Dead Loads

<table>
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<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/16” Mtl. Deck w. 2 ½” Concrete Slab</td>
<td>26</td>
</tr>
<tr>
<td>Cold Form Framing</td>
<td>5</td>
</tr>
<tr>
<td>Floor Finish (Hardwood Floor)</td>
<td>4</td>
</tr>
<tr>
<td>Ceiling w. Finish</td>
<td>3</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
</tbody>
</table>

| Total                           | 44           |

#### Roof Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/16” Mtl. Deck w. 2 ½” Concrete Slab</td>
<td>26</td>
</tr>
<tr>
<td>Cold Form Framing</td>
<td>5</td>
</tr>
<tr>
<td>Roof Shingles</td>
<td>3</td>
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<tr>
<td>Rigid Insulation</td>
<td>1</td>
</tr>
<tr>
<td>Ceiling w. Finish</td>
<td>3</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>6</td>
</tr>
</tbody>
</table>

| Total                           | 44           |

---

1 ASCE 7-10 Table C3-1 p399
Live Loads

According to the International Building Code (IBC) Section 1607 Table 1607, which is equivalent to ASCE7-10 Table 4-1, the minimum live load for residential dwellings of multi-families is 40 psf. The exceptions of the 40psf live load are for the lobby floor. The lobby consists of retail spaces, a restaurant and a gymnasium, all of which with a minimum live load of 100 psf. The stairs and exit ways also contain a minimum live load of 100 psf. In addition, Table 4-1 specifies a minimum of 20 psf for roofs that may be “ordinary flat, pitched, and curved roofs that are not occupiable. However, since the penthouses on the setback floors have access to terraces, the live load will be designed for 60 psf.

Table 1 Live Loads Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-cellar Live Loads</td>
<td></td>
</tr>
<tr>
<td>Public Area</td>
<td>100</td>
</tr>
<tr>
<td>Storage Room</td>
<td>125</td>
</tr>
<tr>
<td>Bowling Alley</td>
<td>75</td>
</tr>
<tr>
<td>Swimming Pool</td>
<td>370</td>
</tr>
<tr>
<td>Locker Room</td>
<td>40</td>
</tr>
<tr>
<td>Hallway</td>
<td>60</td>
</tr>
<tr>
<td>Stair</td>
<td>100</td>
</tr>
<tr>
<td>Cellar Live Loads</td>
<td></td>
</tr>
<tr>
<td>Parking Area</td>
<td>40</td>
</tr>
<tr>
<td>Ground (1st Floor - Inside) Live Loads</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Weight (psf)</td>
</tr>
<tr>
<td>Public Area</td>
<td>100</td>
</tr>
<tr>
<td>Lobby</td>
<td>100</td>
</tr>
<tr>
<td>2nd Floor Live Loads</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Weight (psf)</td>
</tr>
<tr>
<td>Residential Area</td>
<td>40</td>
</tr>
<tr>
<td>Hallway</td>
<td>100</td>
</tr>
<tr>
<td>3rd - 15th Floor Live Loads</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Weight (psf)</td>
</tr>
<tr>
<td>Residential Area</td>
<td>40</td>
</tr>
<tr>
<td>Hallway</td>
<td>100</td>
</tr>
<tr>
<td>16th Floor Live Loads</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Weight (psf)</td>
</tr>
<tr>
<td>Residential Area</td>
<td>40</td>
</tr>
<tr>
<td>Hallway</td>
<td>100</td>
</tr>
</tbody>
</table>
### Terrace

<table>
<thead>
<tr>
<th>Location</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terrace</td>
<td>60</td>
</tr>
</tbody>
</table>

### 17th - 18th Floor Live Loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td>40</td>
</tr>
<tr>
<td>Hallway</td>
<td>100</td>
</tr>
</tbody>
</table>
Roof live Loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20</td>
</tr>
<tr>
<td>Green Roof Saturated Weight (2 ½” vegetated mat)</td>
<td>18</td>
</tr>
</tbody>
</table>

**Snow loads**

The analysis for snow loads is according to IBC 2012 Section 1608, which instructs that design snow loads are to be determined in accordance to Chapter 7 of ASCE7-10.

**Snow Load Parameters**

<table>
<thead>
<tr>
<th>Snow Load Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category (ASCE7-10 Table 1.5-1)</td>
<td>II</td>
<td>-</td>
</tr>
<tr>
<td>Exposure Factor, $C_e$ (ASCE7-10 Table 7-2)</td>
<td>1.2</td>
<td>-</td>
</tr>
<tr>
<td>Thermal Factor, $C_t$ (ASCE7-10 Table 7-3)</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Importance Factor, $I_s$ (ASCE7-10 Table 1.5-2)</td>
<td>1.00</td>
<td>-</td>
</tr>
<tr>
<td>Roof Slope Factor, $C_s$ (ASCE7-10 Figure 7.2)</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Ground Snow Load, $p_g$ (ASCE Figure 7-1/IBC Figure 1608.2)</td>
<td>20</td>
<td>psf</td>
</tr>
<tr>
<td>Flat Roof Snow Loads, $p_f$ (ASCE7-10 Eq. 7.3-1)</td>
<td>16.8</td>
<td>psf</td>
</tr>
<tr>
<td>Minimum design snow load, $p_f$ (min)</td>
<td>20</td>
<td>psf</td>
</tr>
</tbody>
</table>

**Flat Roof Snow Load**

$$p_f = 0.7C_eC_sI_sC_ip_g = 0.7 \times 1.2 \times 1.0 \times 1.00 \times 20 \text{ psf} = 16.8 \text{ psf}$$

**Minimum Snow Load for Low Slope Roofs:**

$$p_m = I_sp_g = 1.00 \times 20 \text{ psf} = 20 \text{ psf}$$

**Balanced Snow Load:**

$$p_s = C_sp_f = 1.0 \times 16.8 \text{ psf} = 16.8 \text{ psf}$$

Since the roof is a flat roof, the roof slope is equal to 0 and the roof angle is 0°, therefore, the unbalanced snow load does not need to be considered.

**Density of Snow:**

$$\gamma = 0.13p_g + 14 \leq 30 \text{psf} = 0.13 \times 20 \text{ psf} + 14 = 16.6 \text{ lb/ft}^3$$
Height of Balanced Snow Load \( (h_b) \):

\[
h_b = \frac{p_s}{\gamma} = \frac{16.8 \text{ psf}}{16.6 \text{pcf}} = 1.01 \text{ ft}
\]

Snow Drift against Penthouse Wall

Height of wall (ceiling height of penthouse plus the bulkhead) \( (h_o) \): \( h_o = 46 \text{ ft} \)

Clear height from top of balanced snow load to top of upper roof \( (h_c) \):

\[
h_c = h_o - h_b = 46 \text{ ft} - 1.01 \text{ ft} = 45.0 \text{ ft} \, (\text{For Wall})
\]

\[
\text{Since} \quad \frac{h_c}{h_b} = \frac{45.0 \text{ ft}}{1.01 \text{ ft}} = 44.5 > 0.2 \rightarrow \text{drift loads need to be applied.}
\]

The snow drift will have 4 cases: drift of snow from the 16th floor (setback roof) to the penthouse wall in both the North to South and East to West directions. However since the building doubly symmetric, the drifts will be the same in both directions.

Windward Drift (Penthouse Wall)

<table>
<thead>
<tr>
<th>Snow Drift Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Roof Upwind of Drift, ( l_u )</td>
<td>45</td>
<td>ft</td>
</tr>
</tbody>
</table>

Height of Snow Drift

\[
h_d = \left(0.43 \sqrt[3]{l_u \frac{p_o + 10 - 1.5}{20 \text{ psf}}} \right) = \left(0.43 \sqrt[3]{45 \text{ ft}^3 \frac{20 \text{ psf} + 10 - 1.5}{20 \text{ psf}}} \right) = 2.1 \text{ ft}
\]

According to ASCE7-10 Section 7.7.1, the windward height drift should be taken as three-quarters of the drift height calculated.

\[
h_d = 0.75 \times 2.1 \text{ ft} = 1.58 \text{ ft}
\]

Drift Width

According to ASCE7-10 Section 7.7.1, if \( h_d < h_c \), the drift width is \( w = 4h_d \) but not greater than \( w = 8h_c \).

\[
w = 4(1.58 \text{ ft}) = 6.3 \text{ ft, but not greater than } w = 8 \times 45 \text{ ft} = 360 \text{ ft}
\]
$w = 6.3 \, ft$

Snow Drift against Penthouse Roof Parapet and Bulkhead Wall

Height of Railing ($h_o$): $h_o = 4 ft$

Height of wall (ceiling height of penthouse plus the bulkhead) ($h_o$): $h_o = 10 ft$

Clear height from top of balanced snow load to top of upper roof ($h_c$):

$$h_c = h_o - h_b = 4 ft - 1.01 ft = 2.99 \, ft \text{ (For Parapet)}$$

$$h_c = h_o - h_b = 10 ft - 1.01 ft = 8.99 \, ft \text{ (For Wall)}$$

Since $\frac{h_c}{h_b} = \frac{2.99 \, ft}{1.01 \, ft} = 3.0 > 0.2$ and $\frac{h_c}{h_b} = \frac{8.99 \, ft}{1.01 \, ft} = 8.9 > 0.2 \rightarrow$ drift loads need to be applied.

The snow drift will have four cases: drift of snow from the roof of the penthouse to the penthouse parapet in both the North to South and East to West directions and the drift of snow from the roof of the penthouse to the bulkhead walls, however since the building doubly symmetric, the drifts will be the same in both directions.

Windward Drift (Parapet)

<table>
<thead>
<tr>
<th>Snow Drift Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Roof Upwind of Drift, $l_u$</td>
<td>55</td>
<td>ft</td>
</tr>
</tbody>
</table>

Height of Snow Drift

$$h_d = \left(0.43 \sqrt[3]{l_u} \sqrt{\frac{20}{psf} + 10} - 1.5\right) = \left(0.43 \sqrt[3]{55} \sqrt{\frac{20}{psf} + 10} - 1.5\right) = 2.3 \, ft$$

According to ASCE7-10 Section 7.7.1, the windward height drift should be taken as three-quarters of the drift height calculated.

$$h_d = 0.75 \times 2.3 \, ft = 1.73 ft$$

Drift Width

According to ASCE7-10 Section 7.7.1, if $h_d < h_c$, the drift width is $w = 4h_d$ but not greater than $w = 8h_c$. 
\[ w = 4 \times 1.73 = 6.9 \text{ ft}, \text{ but not greater than } w = 8 \times 2.99 = 23.90 \text{ ft} \]

\[ w = 6.9 \text{ ft} \]

Windward Drift (Bulkhead Wall)

<table>
<thead>
<tr>
<th>Snow Drift Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Roof Upwind of Drift, ( l_u )</td>
<td>55</td>
<td>ft</td>
</tr>
</tbody>
</table>

Height of Snow Drift

\[ h_d = \left( 0.43 \sqrt[3]{l_u} \sqrt[p_g+10-1.5]{p_g} \right) = \left( 0.43 \sqrt[3]{55} \sqrt[4]{20} + 10 - 1.5 \right) = 2.3 \text{ ft} \]

According to ASCE7-10 Section 7.7.1, the windward height drift should be taken as three-quarters of the drift height calculated.

\[ h_d = 0.75 \times 2.3 \text{ ft} = 1.73 \text{ ft} \]

Drift Width

According to ASCE7-10 Section 7.7.1, if \( h_d < h_c \), the drift width is \( w = 4h_d \) but not greater than \( w = 8h_c \).

\[ w = 4(1.73 \text{ ft}) = 6.9 \text{ ft}, \text{ but not greater than } w = 8 \times 8.99 = 72 \text{ ft} \]

\[ w = 6.9 \text{ ft} \]

Leeward Drift (Bulkhead Wall)

<table>
<thead>
<tr>
<th>Snow Drift Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Roof Upwind of Drift, ( l_u )</td>
<td>50</td>
<td>ft</td>
</tr>
</tbody>
</table>

Height of Snow Drift

\[ h_d = \left( 0.43 \sqrt[3]{l_u} \sqrt[p_g+10-1.5]{p_g} \right) = \left( 0.43 \sqrt[3]{50} \sqrt[4]{20} + 10 - 1.5 \right) = 2.21 \text{ ft} \]

Drift Width

According to Section 7.7.1, if \( h_d < h_c \), the drift width is \( W = 4h_d \) but not greater than \( W = 8h_c \).

\[ W = 4(2.21 \text{ ft}) = 8.84 \text{ ft}, \text{ but not greater than } W = 8 \times 8.99 = 72 \text{ ft} \]
\[ w = 8.84 \text{ ft} \]

Rain on Surcharge Snow Load

According to ASCE 7-10 Section 7.10, a Rain on Snow Surcharge Load \( (\rho_{rs}) \) of 5 psf should be added when \( \rho_g \) is less than or equal to 20 psf.

\[ p_b = 20 \text{ psf} + 5 \text{psf} = 25 \text{psf} \]

Drift Surcharge Load \( (\rho_d) \)

\[ p_d = h_d \gamma = 1.58 \text{ ft} \times 16.6 \frac{\text{lb}}{\text{ft}^3} = 26.3 \text{ psf} \text{ (Windward drift against Penthouse Wall)} \]

\[ p_d = h_d \gamma = 1.73 \text{ ft} \times 16.6 \frac{\text{lb}}{\text{ft}^3} = 28.7 \text{ psf} \text{ (Windward drift against Parapet)} \]

\[ p_d = h_d \gamma = 1.73 \text{ ft} \times 16.6 \frac{\text{lb}}{\text{ft}^3} = 28.7 \text{ psf} \text{ (Windward drift against Bulkhead Wall)} \]

\[ p_d = h_d \gamma = 2.21 \text{ ft} \times 16.6 \frac{\text{lb}}{\text{ft}^3} = 36.7 \text{ psf} \text{ (Leeward drift against Bulkhead Wall)} \]

Total Snow Load

Total Snow Load = Balanced Snow Load + Drift Surcharge Snow Load

\[ p = 25 \text{psf} + 26.3 \text{psf} = 51.3 \text{psf} \text{ (Windward drift against Penthouse Wall)} \]

\[ p = 25 \text{psf} + 28.7 \text{psf} = 53.7 \text{psf} \text{ (Windward drift against Parapet)} \]

\[ p = 25 \text{psf} + 28.7 \text{psf} = 53.7 \text{psf} \text{ (Windward drift against Bulkhead Wall)} \]

\[ p = 25 \text{psf} + 36.7 \text{psf} = 61.7 \text{psf} \text{ (Leeward drift against Bulkhead Wall) (Controls)} \]

Summary Table

<table>
<thead>
<tr>
<th></th>
<th>Flat Snow Load (psf)</th>
<th>Snow Drift (psf)</th>
<th>Drift width (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulkhead Roof</td>
<td>25</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Bulkhead Wall</td>
<td>-----</td>
<td>36.7</td>
<td>8.84</td>
</tr>
<tr>
<td>Penthouse Roof</td>
<td>25</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Penthouse Parapet</td>
<td>-----</td>
<td>28.7</td>
<td>6.9</td>
</tr>
<tr>
<td>Penthouse Wall</td>
<td>-----</td>
<td>26.3</td>
<td>6.3</td>
</tr>
</tbody>
</table>
The calculation of earthquake load will be determined using the equivalent lateral force method as described in section 12.8 of ASCE 7-10.

Structure Information

- The seismic force resisting system is ordinary reinforced concrete shear walls
- Building is regular in plan and vertically (assumption needed to use the equivalent lateral force method)
- Site Class of Building is Site Class C
- The Occupancy Category of the Building is II
- The Importance Factor is 1.00 (ASCE 7-10 Table 1.5-2)

Table: Spectral Response Acceleration Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_s$ (mapped MCE, 5 percent damped, spectral response acceleration parameter for period of 1s)</td>
<td>0.269g*</td>
</tr>
<tr>
<td>$S_1$ (mapped MCE, 5 percent damped, spectral response acceleration parameter at a period of 1s)</td>
<td>0.070g*</td>
</tr>
<tr>
<td>$S_{ms}$ (the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for site class effects)</td>
<td>0.426g*</td>
</tr>
<tr>
<td>$S_{M1}$ (the MCE, 5 percent damped, spectral response acceleration for 1s period adjusted for site class effects)</td>
<td>0.168g*</td>
</tr>
<tr>
<td>$S_{DS}$ (design, 5 percent damped, spectral response acceleration at short periods)</td>
<td>0.284g*</td>
</tr>
<tr>
<td>$S_{D1}$ (design, 5 percent damped, spectral response acceleration parameter for 1s period)</td>
<td>0.112g*</td>
</tr>
</tbody>
</table>

*These values were determined with United States Geological Survey application. The links to the reports are presented in the references section below.
From table 11.6-1:

\[ 0.167g \leq S_{DS} = 0.215g < 0.33g \rightarrow \text{Seismic Design Category B} \]

From table 11.6-2:

\[ 0.067g \leq S_{D1} = 0.079g < 0.133g \rightarrow \text{Seismic Design Category B} \]

Therefore, the building will be designed based on seismic design category B.

From table 12.2-1, for an ordinary reinforced concrete shear wall building, the response modification factor is 5. There are no height limitations for this building based on seismic design category B. Based on Table 12.6-1, the equivalent lateral force analysis is permitted.

Natural Period:

\[ T_n = \frac{1}{n_i} = \frac{1}{0.38Hz} = 2.63 \text{s} \]

Long Transition Period:

\[ T_L = 6\text{s from Figure 22-12} \]

Coefficient for Upper Limit on Calculated Period:

\[ C_u = 1.70 \text{ for } S_{D1} = 0.079 \text{ (interpolated from Table 12.8-1)} \]

Approximate Period Parameters:

\[ C_t = 0.02 \text{ for all other structural systems (Table 12.8-2)} \]

\[ x = 0.75 \text{ for all other structural systems (Table 12.8-2)} \]

Approximate Fundamental Period

\[ T_a = C_t h_n^x = 0.02(199.5ft)^{0.75} = 1.06 \text{s} \]

Upper Limit of Period:

\[ T_{upper\ limit} = C_u T_a = C_u C_t h_n^x = 1.70 \times 1.06 \text{s} = 1.80 \text{s} \]

\[ T = T_a \leq T_{upper\ limit} \therefore T = 1.06 \text{s} \]

Seismic Response Coefficient

\[ C_S = \frac{S_{DS}}{R/I} = \frac{0.215}{5/1.0} = 0.0430 \]
$C_S = \frac{S_{D_1}}{T(R/I)}$ for $T \leq T_L = \frac{0.079}{1.065 \times 5/1.0} = 0.0149$

$C_S$ should not exceed...

$C_S = 0.044S_{D_1}I \geq 0.01 = 0.044 \times 0.215 \times 1.0 = 0.0010 \leq 0.01$

$\therefore C_S = 0.0149$

Sample Calculation of Weight of Floor (3rd Floor)

Base Shear, $V$ (kip)

$V = C_S W = 0.0149 \times 44,798k = 670k$

Sample Calculation of equivalent lateral force: (3rd Floor)

$C_{vx} = \frac{w_x h_x^k}{\sum_i (w_i h_i^k)} = \frac{(2769k)(24.5ft)^{1.28}}{(14,199,208k)} = 0.012$

$F_x = C_{vx}V = 0.0149 \times 670 = 7.85k$
### WIND LOAD

The determination of wind loads is according to the directional procedure in ASCE 7-10 Chapter 27. For a flexible structure, the gust factor $G_f$, needs to be calculated in accordance to ASCE 7-10 Chapter 26, Section 26.9.5. The basic wind speed based on our location (40.5993 and -73.9999) on a Risk Category II is 116mph.

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>$W_i$ (k)</th>
<th>Floor Height(ft)</th>
<th>Floor height from base level $h_i$ (ft)</th>
<th>$w_x h_x^k$</th>
<th>$C_{vX}$</th>
<th>$F_x$ (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>3,227</td>
<td>15.0</td>
<td>15.0</td>
<td>103,555</td>
<td>0.007</td>
<td>4.88</td>
</tr>
<tr>
<td>3</td>
<td>2,769</td>
<td>9.5</td>
<td>24.5</td>
<td>166,591</td>
<td>0.012</td>
<td>7.85</td>
</tr>
<tr>
<td>4</td>
<td>2,769</td>
<td>9.5</td>
<td>34.0</td>
<td>253,473</td>
<td>0.018</td>
<td>11.95</td>
</tr>
<tr>
<td>5</td>
<td>2,769</td>
<td>9.5</td>
<td>43.5</td>
<td>347,531</td>
<td>0.024</td>
<td>16.39</td>
</tr>
<tr>
<td>6</td>
<td>2,769</td>
<td>9.5</td>
<td>53.0</td>
<td>447,581</td>
<td>0.032</td>
<td>21.10</td>
</tr>
<tr>
<td>7</td>
<td>2,769</td>
<td>9.5</td>
<td>62.5</td>
<td>552,821</td>
<td>0.039</td>
<td>26.07</td>
</tr>
<tr>
<td>8</td>
<td>2,769</td>
<td>9.5</td>
<td>72.0</td>
<td>662,666</td>
<td>0.047</td>
<td>31.25</td>
</tr>
<tr>
<td>9</td>
<td>2,769</td>
<td>9.5</td>
<td>81.5</td>
<td>776,669</td>
<td>0.055</td>
<td>36.62</td>
</tr>
<tr>
<td>10</td>
<td>2,769</td>
<td>9.5</td>
<td>91.0</td>
<td>894,472</td>
<td>0.063</td>
<td>42.18</td>
</tr>
<tr>
<td>11</td>
<td>2,769</td>
<td>9.5</td>
<td>100.5</td>
<td>1,015,786</td>
<td>0.072</td>
<td>47.90</td>
</tr>
<tr>
<td>12</td>
<td>2,769</td>
<td>9.5</td>
<td>110.0</td>
<td>1,140,368</td>
<td>0.080</td>
<td>53.77</td>
</tr>
<tr>
<td>13</td>
<td>2,777</td>
<td>9.5</td>
<td>119.5</td>
<td>1,271,675</td>
<td>0.090</td>
<td>59.96</td>
</tr>
<tr>
<td>14</td>
<td>2,785</td>
<td>10.0</td>
<td>129.5</td>
<td>1,413,607</td>
<td>0.100</td>
<td>66.65</td>
</tr>
<tr>
<td>15</td>
<td>2,785</td>
<td>10.0</td>
<td>139.5</td>
<td>1,554,910</td>
<td>0.110</td>
<td>73.32</td>
</tr>
<tr>
<td>PH1</td>
<td>3,619</td>
<td>10.0</td>
<td>149.5</td>
<td>2,207,944</td>
<td>0.155</td>
<td>104.11</td>
</tr>
<tr>
<td>PH2</td>
<td>591</td>
<td>10.0</td>
<td>159.5</td>
<td>391,369</td>
<td>0.028</td>
<td>18.45</td>
</tr>
<tr>
<td>PH3</td>
<td>591</td>
<td>10.0</td>
<td>169.5</td>
<td>423,070</td>
<td>0.030</td>
<td>19.95</td>
</tr>
<tr>
<td>Roof</td>
<td>585</td>
<td>10.0</td>
<td>179.5</td>
<td>451,336</td>
<td>0.032</td>
<td>21.28</td>
</tr>
<tr>
<td>Bulkhead</td>
<td>73</td>
<td>10.0</td>
<td>189.5</td>
<td>60,412</td>
<td>0.004</td>
<td>2.85</td>
</tr>
<tr>
<td>Bulkhead Roof</td>
<td>72</td>
<td>10.0</td>
<td>199.5</td>
<td>63,373</td>
<td>0.004</td>
<td>2.99</td>
</tr>
<tr>
<td>Sum:</td>
<td>44,798</td>
<td></td>
<td></td>
<td>14,199,208</td>
<td>1.000</td>
<td>669.51</td>
</tr>
</tbody>
</table>
In order to use the directional procedure, the following conditions must be satisfied (Section 27.1.2).

1. The building is a regular shaped building as defined in Section 26.2.
2. The building does not have response characteristics making it subject to across wind loading, vortex shedding, etc.

Search Results

Latitude: 40.5993
Longitude: -73.9999

ASCE 7-10 Wind Speeds
(3-sec peak gust MPH):

Risk Category I: 106
Risk Category II: 116
Risk Category III-IV: 124
MRI** 10 Year: 76
MRI** 25 Year: 85
MRI** 50 Year: 90
MRI** 100 Year: 96

ASCE 7-05: 107
ASCE 7-93: 81

Wind Speed Map
Table 2 Wind Load Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category (Table 1.5-1)</td>
<td>II</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>B</td>
</tr>
<tr>
<td>Basic Wind Speed, V (Wind Speed Map)</td>
<td>116 mph</td>
</tr>
<tr>
<td>Wind Directionality Factor, Kd (Table 26.6-1)</td>
<td>0.85</td>
</tr>
<tr>
<td>Velocity Pressure Exposure Coefficient, K\text{\textsubscript{z}} (Table 27.3-1)</td>
<td>1.20</td>
</tr>
<tr>
<td>Topographic factor, K\text{\textsubscript{zt}} (Section 26.8.2)</td>
<td>1.0</td>
</tr>
<tr>
<td>Wind Gust Factor Coefficient, G\text{\textsubscript{f}} (Section 26.9.5), calculation below</td>
<td>0.877</td>
</tr>
<tr>
<td>Internal Pressure Coefficient, GCp\text{\textsubscript{i}} (Figure 6-5) for enclosed buildings</td>
<td>± 0.18</td>
</tr>
<tr>
<td>Damping Ratio, β</td>
<td>2%</td>
</tr>
<tr>
<td>Natural Frequency, n\text{\textsubscript{1}} (calculation below)</td>
<td>0.38Hz</td>
</tr>
</tbody>
</table>

Table 3 Building Dimension Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Width, B</td>
<td>250.0 ft</td>
</tr>
<tr>
<td>Building Length, L</td>
<td>250.0 ft</td>
</tr>
<tr>
<td>Mean Roof Height, h</td>
<td>199.5 ft</td>
</tr>
</tbody>
</table>

The approximate natural frequency for structural steel and concrete buildings with other lateral force resisting systems (not moment frame) can be determined based on equation 26.9-4 in Section 26.9.3.

\[
n_1 = \frac{75}{h} = \frac{75}{199.5 \text{ ft}} = 0.38 \text{ Hz}
\]

From table 26.9-1, the following terrain exposure constants can be defined for an Exposure Category B.

Table 4 Terrain Exposure Constants

| Mean Hourly Wind Speed Power Law Exponent, \(\hat{\alpha}\) | 0.250 |

From table 26.9-1, the following terrain exposure constants can be defined for an Exposure Category B.
<table>
<thead>
<tr>
<th>Mean Hourly Wind Speed Factor, $\bar{b}$</th>
<th>0.450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbulence Intensity Factor, $c$</td>
<td>0.300</td>
</tr>
<tr>
<td>Integral Length Scale Factor, $\ell$</td>
<td>320 ft</td>
</tr>
<tr>
<td>Integral Length Scale Power Law Exponent, $\epsilon$</td>
<td>0.333</td>
</tr>
<tr>
<td>Exposure Constant, $z_{min}$</td>
<td>30.0 ft</td>
</tr>
</tbody>
</table>

Equivalent Height of Structure (Section 26.9.4)

$$z = 0.6h \geq z_{min} = 0.6(201.0\text{ft}) \geq 30.0\text{ft}$$

$$z = 121 \text{ft}$$

Mean Hourly Speed

$$\bar{V}_Z = \bar{b} \left( \frac{Z}{33} \right)^{\alpha} V \left( \frac{88}{60} \right) = (0.45) \left( \frac{121 \text{ft}}{33} \right)^{0.25} (116 \text{mph}) \left( \frac{88}{60} \right) = 106 \text{ft/s}$$

$$\eta_b = \frac{4.6n_B \bar{V}}{\bar{V}_Z} = \frac{4.6 \times 0.38 \text{Hz} \times 250 \text{ft}}{105 \text{ ft/s}} = 4.09$$

$$\eta_h = \frac{4.6n_h \bar{V}}{\bar{V}_Z} = \frac{4.6 \times 0.38 \text{Hz} \times 199.5 \text{ft}}{105 \text{ ft/s}} = 3.27$$

$$\eta_L = \frac{15.4n_L \bar{V}}{\bar{V}_Z} = \frac{15.4 \times 0.38 \text{Hz} \times 250 \text{ft}}{105 \text{ ft/s}} = 13.70$$

$$R_B = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2} (1 - e^{-2\eta_b}) = \frac{1}{4.09} - \frac{1}{2(4.09)^2} (1 - e^{-2(4.09)}) = 0.215$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2} (1 - e^{-2\eta_h}) = \frac{1}{3.27} - \frac{1}{2(3.27)^2} (1 - e^{-2(3.27)}) = 0.259$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2} (1 - e^{-2\eta_L}) = \frac{1}{13.70} - \frac{1}{2(13.70)^2} (1 - e^{-2(13.70)}) = 0.070$$

Intensity of Turbulence (Eq. 26.9-7)

$$I_z = c(33/z)^{1.6} = 0.300(33/121\text{ft})^{1.6} = 0.242$$

Integral Length Scale of Turbulence (Eq. 26.9-9)

$$L_Z = l \left( \frac{Z}{33} \right)^{\epsilon} = 320 \text{ft} \left( \frac{121 \text{ft}}{33} \right)^{0.333} = 492 \text{ft}$$
Reduced Frequency (Eq. 26.9-14)

\[
N_1 = \frac{n_1 L_z}{V_z} = \frac{(0.38Hz)(492ft)}{106ft/s} = 1.75Hz
\]

\[
R_n = \frac{7.47N_1}{(1 + 10.3N_1)^{5/3}} = \frac{7.47(1.75Hz)}{(1 + 10.3(1.75Hz))^{5/3}} = 0.096
\]

Resonant Response Factor (Eq. 26.9-12)

\[
R = \frac{1}{\sqrt{\beta R_n R_h R_B(0.53 + 0.47R_v)}} = \frac{1}{\sqrt{0.02}}(0.096)(0.259)(0.215)(0.53 + 0.47(0.070)) = 0.389
\]

Peak Factor for Resonant Response (Eq. 26.9-11)

\[
g_R = \frac{\sqrt{2\ln(3600n_1)} + 0.577}{\sqrt{2\ln(3600n_1)}} = \sqrt{2\ln(3600)(0.38)} + \frac{0.577}{\sqrt{2\ln(3600)(0.38)}} = 3.95
\]

Peak Factor for Wind Response, and Resonant Response (Section 26.9.5)

\[
g_Q = g_R = 3.40
\]

Background Response Factor (Eq. 26.9-8)

\[
Q = \frac{1}{\sqrt{1 + 0.63\left(\frac{B + h}{L_z}\right)^{0.63}}} = \frac{1}{\sqrt{1 + 0.63\left(\frac{250ft + 199.5ft}{492ft}\right)^{0.63}}} = 0.792
\]

\[
G_f = 0.925\left(\frac{1 + 1.7L_z\sqrt{g_Q^2Q^2 + g_R R^2}}{1 + 1.7g_vL_z}\right)
\]

\[
G_f = 0.925\left(\frac{1 + 1.7(0.242)\sqrt{(3.40)^2(0.792)^2 + (3.95)(0.389)^2}}{1 + 1.7(3.40)(0.242)}\right) = 0.877
\]

Velocity Pressure

\[
q_z = 0.00256K_xK_{zt}K_dV^2
\]

\[
q_z = 0.00256 \times 1.20 \times 1.0 \times 0.85 \times (116mph)^2 = 35.25 \frac{lb}{ft^2}
\]

Table 5 Wall Pressure Coefficients

| Windward Wall Width, B | 250 ft |
Sample Calculation for Height $z = 82\text{ ft}$

$$K_z = 2.01 \left( \frac{z}{Z_0} \right)^{2/\alpha} = 2.01 \left( \frac{82\text{ ft}}{1200\text{ ft}} \right)^{2/7.0} = 0.93$$

Velocity Pressure

$$q_z = 0.00256 K_z K_w K_d V^2$$

$$q_z = 0.00256 \times 0.93 \times 1.0 \times 0.85 \times (116\text{ mph})^2 = 27.29 \frac{\text{lb}}{\text{ft}^2}$$

Windward Pressure:

$$p_W = q_z G_f C_{pw} = 27.29 \text{ psf} \times 0.877 \times 0.80 = 19.15 \text{ psf}$$

Leeward Pressure:

$$p_L = q_z G_f C_{pl} = 35.25 \text{ psf} \times 0.877 \times -0.50 = -15.46 \text{ psf}$$

Side Wall Pressure:

$$p_s = q_z G_f C_{ps} = 35.25 \text{ psf} \times 0.877 \times -0.70 = -21.64 \text{ psf}$$
<table>
<thead>
<tr>
<th>Height, z</th>
<th>$K_z$</th>
<th>$q_z$</th>
<th>Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>WW</td>
</tr>
<tr>
<td>0 ft</td>
<td>0.57</td>
<td>16.83 psf</td>
<td>11.81 psf</td>
</tr>
<tr>
<td>15 ft</td>
<td>0.57</td>
<td>16.83 psf</td>
<td>11.81 psf</td>
</tr>
<tr>
<td>25 ft</td>
<td>0.66</td>
<td>19.36 psf</td>
<td>13.59 psf</td>
</tr>
<tr>
<td>34 ft</td>
<td>0.73</td>
<td>21.26 psf</td>
<td>14.92 psf</td>
</tr>
<tr>
<td>44 ft</td>
<td>0.78</td>
<td>22.81 psf</td>
<td>16.01 psf</td>
</tr>
<tr>
<td>53 ft</td>
<td>0.82</td>
<td>24.14 psf</td>
<td>16.94 psf</td>
</tr>
<tr>
<td>63 ft</td>
<td>0.86</td>
<td>25.30 psf</td>
<td>17.75 psf</td>
</tr>
<tr>
<td>72 ft</td>
<td>0.90</td>
<td>26.34 psf</td>
<td>18.49 psf</td>
</tr>
<tr>
<td>82 ft</td>
<td>0.93</td>
<td>27.29 psf</td>
<td>19.15 psf</td>
</tr>
<tr>
<td>91 ft</td>
<td>0.96</td>
<td>28.17 psf</td>
<td>19.77 psf</td>
</tr>
<tr>
<td>101 ft</td>
<td>0.99</td>
<td>28.98 psf</td>
<td>20.34 psf</td>
</tr>
<tr>
<td>110 ft</td>
<td>1.02</td>
<td>29.73 psf</td>
<td>20.87 psf</td>
</tr>
<tr>
<td>120 ft</td>
<td>1.04</td>
<td>30.45 psf</td>
<td>21.37 psf</td>
</tr>
<tr>
<td>129 ft</td>
<td>1.06</td>
<td>31.12 psf</td>
<td>21.84 psf</td>
</tr>
<tr>
<td>139 ft</td>
<td>1.08</td>
<td>31.76 psf</td>
<td>22.29 psf</td>
</tr>
<tr>
<td>149 ft</td>
<td>1.11</td>
<td>32.40 psf</td>
<td>22.74 psf</td>
</tr>
<tr>
<td>159 ft</td>
<td>1.13</td>
<td>33.01 psf</td>
<td>23.16 psf</td>
</tr>
<tr>
<td>169 ft</td>
<td>1.15</td>
<td>33.59 psf</td>
<td>23.57 psf</td>
</tr>
<tr>
<td>179 ft</td>
<td>1.17</td>
<td>34.15 psf</td>
<td>23.96 psf</td>
</tr>
<tr>
<td>189 ft</td>
<td>1.18</td>
<td>34.68 psf</td>
<td>24.34 psf</td>
</tr>
<tr>
<td>200 ft</td>
<td>1.20</td>
<td>35.25 psf</td>
<td>24.74 psf</td>
</tr>
</tbody>
</table>
Horizontal Irregularities

Table 12.3-1 (Horizontal Structure Irregularities)

<table>
<thead>
<tr>
<th>Type</th>
<th>Irregularity</th>
<th>Description</th>
<th>Applicable</th>
<th>Reason</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Torsional Irregularity (Refer to Section 16.2.2 for Seismic Design Category B)</td>
<td>Exists if maximum story drift (computed with accidental torsion factor = 1.0) at one end of structure transverse to an axis is &gt; 1.2 times the average of the story drifts at the two ends of the structure; applicable only to semi-rigid and rigid diaphragms</td>
<td>NO</td>
<td>Extreme torsional irregularity applies</td>
</tr>
<tr>
<td>1b</td>
<td>Extreme Torsional Irregularity (Section 16.2.2 for SDS B)</td>
<td>Exists if maximum story drift (computed with accidental torsion factor = 1.0) at one end of structure transverse to an axis is &gt; 1.4 times the average of the story drifts at the two ends of the structure; applicable only to semi-rigid and rigid diaphragms</td>
<td>YES</td>
<td>$\Delta_2 &gt; 1.4(\frac{\Delta_1 + \Delta_2}{2})$</td>
</tr>
<tr>
<td>2</td>
<td>Reentrant Corner Irregularity (SDS D,E,F)</td>
<td>Exists if both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>3</td>
<td>Diaphragm Discontinuity Irregularity (SDS D)</td>
<td>Exists if there are a abrupt discontinuity or variation in stiffness, including one having a cutout or open greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>4</td>
<td>Out of Plane Offset Irregularity (16.2.2 for SDS B)</td>
<td>Exists if there is a discontinuity in the LRFS such as out of plane offset of at least one vertical dimension</td>
<td>NO</td>
<td>The Lateral Force Resistant System (concrete shear walls) is continuous</td>
</tr>
<tr>
<td>5</td>
<td>Nonparallel System Irregularity (16.2.2 for SDS B)</td>
<td>Exists if vertical LRFS are not parallel to major orthogonal axes of the seismic force resisting system</td>
<td>NO</td>
<td>The Lateral Force Resistant System (concrete shear walls) is orthogonal to each other</td>
</tr>
</tbody>
</table>

The extreme torsional irregularity is applicable because the story drift at one end of the structure transverse to an axis is greater than 1.4 times the average of the story drifts at two ends of the story (ASCE7-10 Table 12.3-1 Type 1b). In the building, the maximum deflection at the north staircase shear wall on floor 16 was 5.023in and the deflection at the south wall staircase shear wall on floor 16 was 1.808in due to the earthquake force applied at 5% eccentricity.
When the extreme torsional irregularity applies, the 5% accidental torsion as required per ASCE7-10 Section 12.8.4.2 (apply a 5% eccentricity to the earthquake force in both orthogonal directions) is amplified by the accidental torsional amplification factor ($A_s$) in accordance to ASCE7-10 Section 12.8.4.3. The torsional amplification factor is calculated as:

$$1.0 \leq A_s = \left( \frac{\delta_{max}}{1.2 \delta_{avg}} \right)^2 \leq 3.0 = \left( \frac{5.023in}{1.2 \cdot 1.808in} \right)^2 = 1.50$$

Therefore, the 5% eccentricity must be amplified by 1.50 times to 7.5% and the design of the lateral force resisting system shear walls will be designed in ETABS with such a load case.

In addition to magnifying the accidental irregularity, ASCE7-10 Section 12.7.3 requires that the structural model for the structure should be constructed for the purpose of determining member forces, structural displacements resulting from applied loads and any imposed displacements or P-delta effects. The model should also include the stiffness and strength of elements that pertain to the distribution of the forces and deformations in the structure. The model should also include the stiffness properties of the concrete cracked sections. The model should also be a 3D model with the appropriate diaphragm stiffness. These requirements were enforced into the ETABS Model.
Vertical Irregularities

Table 12.3-2 (Vertical Structure Irregularities)

<table>
<thead>
<tr>
<th>Type</th>
<th>Irregularity</th>
<th>Description</th>
<th>Applicable</th>
<th>Reason</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Stiffness-Soft Story Irregularity (SDS D,E,F)</td>
<td>Exists if a story in which the LRFS is less than 70% of the story above or less than 80% of the average of the three stories above</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>1b</td>
<td>Stiffness-Extreme Soft Story Irregularity (SDS D,E,F)</td>
<td>Exists if a story in which the LRFS is less than 60% of the story above or less than 70% of the average of the three stories above</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>2</td>
<td>Weight (Mass) Irregularity (SDS D,E,F)</td>
<td>Exists if effective mass of any story is more than 150% of the effective mass of an adjacent story</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>3</td>
<td>Vertical Geometric Irregularity (SDS D,E,F)</td>
<td>Exists if horizontal dimension of the seismic force resisting system is more than 130% of that in an adjacent story</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>4</td>
<td>In Plane Discontinuity in Vertical Lateral Force Resisting Element (12.3.3.3 for SDS B)</td>
<td>Exists if an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab</td>
<td>NO</td>
<td>The Lateral Force Resistant System (concrete shear walls) is continuous</td>
</tr>
<tr>
<td>5a</td>
<td>Discontinuity in Lateral Strength - Weak Story Irregularity (SDS, D, E, F)</td>
<td>Exists if story lateral strength is less than 80% of the story above. Story lateral strength = total lateral strength in all seismic resisting elements sharing the story shear for the direction under consideration</td>
<td>NO</td>
<td>The building structure is in SDS B</td>
</tr>
<tr>
<td>5b</td>
<td>Discontinuity in Lateral Strength - Extreme Weak Story Irregularity (Section 12.3.3.2 for SDS B)</td>
<td>Exists if story lateral strength is less than 65% of the story above. Story lateral strength = total lateral strength in all seismic resisting elements sharing the story shear for the direction under consideration</td>
<td>NO</td>
<td>The story lateral strength below a stiffer story is greater than 65% of the story above</td>
</tr>
</tbody>
</table>

The discontinuity in lateral strength (extreme weak story) irregularity was checked and verified with ETABs. A discontinuity in lateral strength exists between the building’s 17th or penthouse 1 (PH-1) floor and the transfer floor. This is because on the penthouse floor, the four perimeter stair case shear walls were discontinued to accommodate for an open terrace. However, this abrupt change does not
result in a discontinuity; the most significant change in stiffness occurs on the 18th or penthouse 3 (PH-3) story and the 17th or penthouse (PH-2) story. The stiffness of the 18th floor is 1551kip/in and the stiffness of the 17th floor is 1200kip/in according to ETABS.

\[
\frac{1200\text{kip/in}}{1551\text{kip/in}} = 0.787
\]

Hence, the story below has 78.7% of the lateral strength above and the extreme weak story irregularity due to a discontinuity in lateral strength does not apply.
A typical floor will be subjected to a dead load of 116psf and a live load of 40psf for private residential and 100 psf in the hallway. The slab thickness was 8 inches and to avoid calculating deflection, the clear column to column spacing is 20ft. The analysis of the slab was completed in SAFE. An 8 inch thick slab and columns were drawn. The shear wall was modeled to have a rigid zone above the wall and to not take any out of plane loads. The dead and live loads were applied and the column strips were checked to correspond with lines of zero shear (corresponding to the maximum moments). Punching or two way shear was checked for and there were no issues. If the slab had failed in punching shear, several options were possible; the slab could have been thicker, the compressive strength of the concrete increased, drop beams or panels can be added, the columns can be larger, additional shear reinforcement such as Lenton Steel Fortress Punching Shear can be used. Vertical and horizontal strips column and middles strips were designed for the top bars. For bottom bars, a uniform reinforcement such as #4 bars at 12” o.c. or #5 bars at 12” o.c. were used.

Check if the Direct Design Method is applicable:

1. There are at least three continuous spans in each direction.
2. The panels are square, and the ratio of long to short side is less than two.
3. Adjacent spans in each direction do not differ by more than one-third of the larger span.
4. Columns are not offset by more than 10 percent of the span length in the direction of offset.
5. All loads are due to gravity and uniform. Also, the ratio of unfactored live to dead loads does not exceed two.
6. There are no beams present along the side.

**Factored Load**

\[ w_u = 1.2D + 1.6L = 1.2(116\text{psf}) + 1.6(40\text{psf}) = 203.2\text{psf} \]

**Effective Depth (assume a 3/4” concrete cover and #5 bars)**

\[ d = h - d_b - \frac{3}{4} = 8" - 5/8" - 3/4" = 6.625" \]
Punching Shear at a distance $d/2$ from face of interior column

$$V_{u@d} = w_u \left( L_1 L_2 - \frac{x}{12} \frac{y}{12} \right) = 203.2 \text{psf} \times \left( 20 \text{ft} \times 20 \text{ft} - \frac{20 + \frac{d}{2}}{12} \times \frac{20 + \frac{d}{2}}{12} \right) = 80.5k$$

Perimeter of critical section

$$b_o = 2x + 2y = 2 \left( 20 + \frac{d}{2} \right) + 2 \left( 20 + \frac{d}{2} \right) = 93.25\text{in}$$

$$\beta = \frac{\text{long side}}{\text{short side}} = 1$$

Shear Strength of Slab

$$\phi V_{c1} = 0.75 \left[ 2 + \left( \frac{4}{\beta} \right) \sqrt{f'c} b_o d \right] = 0.75 \times \left[ 2 + \left( \frac{4}{1} \right) \sqrt{4\text{ksi}} \times 93.25\text{in} \times 6.625\text{in} \right] = 175.8k$$

$$\phi V_{c2} = 0.75 \left[ \frac{10x_d}{b_o} + 2 \right] \sqrt{f'c} b_o d = 0.75 \left[ \left( \frac{40 \times 6.625}{93.25} \right) + \sqrt{4\text{ksi}} \times 93.25\text{in} \times 6.625\text{in} \right] = 141.9k$$

$$\phi V_{c3} = \phi 4\lambda \sqrt{f'c} b_o d = 0.75 \times 4 \times \sqrt{4\text{ksi}} \times 93.25\text{in} \times 6.625\text{in} = 117.2k$$

$$\phi V_n = 117.2k > V_{u@d} \ (OK)$$

One Way Shear at distance $d$

Consider a one foot strip

$$x = \frac{20ft}{2} - \frac{12\text{in}}{2} - \frac{d}{2} = 8.95\text{ft}$$

$$V_u = 203.2\text{psf} \times 8.95\text{ft} \times 1\text{ft} = 1.82k$$

$$\phi V_c = 0.75 \left[ 2\lambda \sqrt{f'c} bd \right] = 0.75 \times 2 \times \sqrt{4\text{ksi}} \times 12\text{in} \times 6.625\text{in} = 7.54k > V_u$$

Moment in the East-West Direction and North South direction

$$\text{Column Strip} = 0.25 \times 20\text{ft} + 0.25 \times 20\text{ft} = 10\text{ft}$$

$$\text{Middle Strip} = 20\text{ft} - 5\text{ft} - 5\text{ft} = 10\text{ft}$$

$$l_{n1} = 20\text{ft} - \frac{12\text{in}}{2} - \frac{12\text{in}}{2} = 19\text{ft}$$

$$M_{u \ EW} = \frac{q_u l_2 l_{n1}^2}{8} = \frac{203.2 \text{psf} \times 20\text{ft} \times (19\text{ft})^2}{8} = 183.4k\text{ft}$$
Column Strip Moments

*Interior and Exterior Negative Moment* = 0.75 * −0.65 * 183.4 kft = −89.4 kft

*Positive Moment* = 0.60 * 0.35 * 183.4 kft = +38.5 kft

**Middle Strip Moments**

*Negative Moment* = 0.25 * −0.65 * 183.4 kft = −29.8 kft
*Positive Moment* = 0.40 * −0.35 * 183.4 kft = +25.7 kft

**Minimum Reinforcement due to small moments**

\[
\rho_{\text{min}} = \frac{200}{f_y} = \frac{200}{60,000} = 0.0033
\]

\[A_s = \rho_{\text{min}}bd = 0.0033 \times 120\text{in} \times 6.625\text{in} = 2.62\text{in}^2\]

*Provide 10#5 bars, \(A_s = 3.10\text{in}^2\)*

\[A_{s,\text{shrinkage}} = 0.0018bh = 0.0018 \times 120\text{in} \times 8\text{in} = 1.73\text{in}^2\]

*Provide 7#5 bars, \(A_s = 2.17\text{in}^2\)*

**Spacing**

*Maximum Spacing* = 18" (ACI 318 – 11 Section 7.6.5)

*Use a spacing of 12" based on ACI 318 – 11 Section 13.3.8 of clear spans*

The procedure is the same for exterior columns, with the difference with \(\alpha_s = 30\).
Transfer floor beams will be utilized for the cellar floor to transfer the column loads from the ground floor; this is necessary because the parking garage underground requires a minimum amount of columns so that cars are able to maneuver. The transfer floor beam to be designed will support one column above (column 56). The dead and live load for column 56 and is 59.45k and 14.97k, respectively.

\[ P_u = 1.2(59.45k) + 1.6(14.97k) = 95.29k \]

The column is applied at 11.5 ft from the left edge of the beam and 13.5 ft from the right end of the beam.

**Maximum shear and Moment**

\[ V_u = \frac{P_u b}{L} = \frac{95.29k \times 13.5ft}{25ft} = 51.46k \]

\[ M_u = \frac{P_u ab}{L} = \frac{95.29k \times 11.5ft \times 13.5ft}{25ft} = 591.76kft \]

The size of the beam is 5’x3’. Assume 2.5 in of concrete spacing, and that #10 bars were used, the value of \( d \) is:

\[ d = 36in - 2.5in - \frac{10}{8}in = 32.25in \]

\[ d' = 2.5in + \frac{10}{8}in = 3.75in \]

Try 14#10 bottom bars \( (A_s = 17.18in^2) \) and 6#8 top bars \( (A_s = 4.71in^2) \), the moment strength assuming compressive steel yields is:

\[ a = \frac{(A_s - A_s')}{0.85f_y} \]

\[ \frac{0.85 \times 4kpsi \times 60in}{0.85 \times 1.0 \times \sqrt{4kpsi \times 60in} \times 30in} = 0.061in \]

\[ \phi M_n = \phi[(A_s - A_s')f_y(d - \frac{a}{2}) + A_s'f_y(d - d')] \]

\[ = 0.90[(17.18in^2 - 4.71in^2)(60kpsi)(32.25in - \frac{0.061in}{2}) + (4.71in^2)(60kpsi)(32.25in - 3.75in)] = 28,495kin = 2,412kft > M_u (OK) \]

**Shear Strength of Concrete**

\[ \phi V_c = \phi 2\lambda \sqrt{f'}bd = 0.75 \times 2 \times 1.0 \times \sqrt{4kpsi \times 60in \times 30in} = 170.8k \]

*The concrete strength is adequate for shear, however provide 4#4 stirrups at 3" anyways*
The following calculation is for column 3 under the 2nd floor load. The service dead and live load are 67.9k and 399.2k, respectively. Assume a reinforcement ratio of 0.003.

**Factored Loads**

\[ P_u = 1.2P_d + 1.6P_L = 1.2(67.9k) + 1.6(399.2k) = 720.2k \]

**Area Required**

\[ P_u = \Phi_cP_n = \Phi_cA_g[0.85f'c + \rho_g(f_y - 0.85f'c)] = 720.2k = 0.80A_g[0.85 \times 4\text{ksi} + 0.003(60\text{ksi} - 0.85 \times 4\text{ksi})] \]

Solving for \( A_g ... \)

\[ A_g = 158\text{in}^2 \]

Use a 12” x 24” sized rectangular concrete column with \( A_g = 288\text{in}^2 \) and reinforce the column with 4 # 8 bars with a total area of steel of \( A_s = 3.14\text{in}^2 \)

**Strength of Column**

\[ \Phi_cP_n = \Phi_c[0.85f'cA_g + A_s(f_y - 0.85f'c)] = 0.80[0.85 \times 4\text{ksi} \times 288\text{in}^2 + 3.14\text{in}^2(60\text{ksi} - 0.85 \times 4\text{ksi})] \]

\[ = 925.6k > P_u \text{ (OK)} \]

\[ \rho_g = \frac{A_s}{A_g} = \frac{3.14\text{in}^2}{288\text{in}^2} = 0.020 \]

**Tie Spacing**

Use #3 lateral ties and assume 2.5” of concrete cover

\[ \text{Spacing}_{\text{long direction}} = \left( \frac{24\text{in} - 2(2.5\text{in})}{2} \right) - 1.0\text{in} = 8.5\text{in} \]

\[ \text{Spacing}_{\text{short direction}} = \left( \frac{12\text{in} - 2(2.5\text{in})}{2} \right) - 1.0\text{in} = 2.5\text{in} \]
Assumptions:

The thickness of the basement wall is assumed to be 12 in or 1 ft. The height of the basement wall is equal to the height of the basement wall minus the parking roof slab thickness (12 in).

\[ h_{wall} = (24\text{ft}) - \left(12\text{in} \times \frac{ft}{12\text{in}}\right) = 23\text{ft} \]

The basement wall will be supported at the bottom of the footing and at the top by the parking roof slab and intermediately by the cellar slab. It will be assumed that the wall is laterally supported at these two slabs, hence the unsupported height is:

\[ h_{unsupported} = 23\text{ft} \]

The unit weight of soil backfill is 124 pcf and the angle of internal friction is 34° for a well graded sand. The soil surcharge, \( w_s \) is assumed to be 200 psf. The basement wall will be designed for a unit strip length of 1 ft.

Calculation:

The calculation of lateral earth pressure is determined using Rankine’s theory. Rankine’s theory neglects friction between the soil and the wall and assumes that the soil is homogeneous, incompressible, and cohesionless.

\[ K_A = \frac{1 - \sin(\emptyset)}{1 + \sin(\emptyset)} = \frac{1 - \sin(34^\circ)}{1 + \sin(34^\circ)} = 0.282 \]

Rankine’s Active Pressure

\[ P_A = \frac{1}{2} K_A \gamma H = \frac{1}{2} (0.282) \left(\frac{124\text{lb}}{ft^3}\right) 23\text{ft} = \frac{403.1\text{lb}}{ft^2} = 0.403\text{k/ft}^2 \]

Equivalent height of soil

\[ h_s = \frac{w_s}{\gamma_s} = \frac{200\text{psf}}{124\text{pcf}} = 1.61\text{ft} \]
Soil Surcharge Pressure

\[ P_s = K_A y h_s = 0.282 \left( \frac{124 \text{lb}}{\text{ft}^3} \right) 1.61\text{ft} = \frac{56.54\text{lb}}{\text{ft}^2} = 0.0565k/ft^2 \]

Pore Water Pressure (consider only half of the wall is subject to water pressure)

\[ P_w = 0.5 y h_{unsupported} = 0.5 \left( \frac{62.4\text{lb}}{\text{ft}^3} \right) 23\text{ft} = \frac{717.6\text{lb}}{\text{ft}^2} = 0.718k/ft^2 \]

Self Weight of Wall

\[ W = 150\text{pcf} (23\text{ft})(1\text{ft})(1\text{ft}) \text{k/1000lb} = 3.45k/ft \]

Axial Force

\[ P_u = 1.4 * \left( \frac{3.45k}{ft} \right) = 4.83k/ft \]

The basement wall is modeled as a continuous vertical beam with a fixed support at the footing and pinned supports at the slab levels (cellar and sub-cellar) assuming that the slabs have been constructed already. The building is fully loaded and the basement wall will experience lateral earth pressure and vertical (gravity) loads due to self weight.

Load distribution onto Continuous Wall due to 1.6H
In the moment diagram above, the point of maximum moment (11.215kft) is located at 15.98ft.

**Minimum thickness** (ACI 14.5.3)

For a wall used for exterior basement and foundation, the minimum wall thickness is 7.5\text{in}. Use a thickness of $h = 12\text{in} > 7.5\text{in}$ for the basement wall.

**Concrete Cover**

For nonprestressed concrete, a member cast against and permanently in contact with ground has a specified cover of 3\text{in}.

**Design for Shear**

Using # 5 bars and a spacing of $S = 8\text{in}$.

$$b = 12\text{in}/\text{ft}$$
\[ d_t = d = h - 3.0\text{ in} - \frac{5}{8}\text{ in} = 8.375\text{ in} \]

\[ \varnothing V_c = 0.75 \times 2\lambda \sqrt{f'_c \cdot h} = 0.75 \times 2 \times 1.0 \times \sqrt{4\text{ksi}} \times \frac{12\text{ in}}{f_t} \times 8.375\text{in} = 9.82k \]

\[ V_s = \frac{V_u - \varnothing V_c}{\varnothing} = \frac{22.7k - 9.82k}{0.75} = 17.175k \]

\[ A_v = \frac{SV_c}{f_s \cdot d} = \frac{8\text{ in} \times 17.175k}{60 \times 8.375\text{in}} = 0.210\text{in}^2 \]

\[ \varnothing V_c + V_s = 9.534k + 17.175k = 27.09k > V_u = 22.7k \]

**Minimum Horizontal Shear Reinforcement (ACI 11.9.9.1, 11.9.9.2)**

*Use #5 bars in two layers spaced at 12in apart*

\[ \rho_t = \frac{A_v}{bh} = \frac{2 \times (0.31\text{in}^2)}{bh} = 0.00426 \geq 0.0025 \text{ (OK)} \]

**Spacing of Horizontal Shear Reinforcement (ACI 11.9.9.3)**

\[ S_{\text{max}} \leq \min \left( \frac{l_w}{5}, 3h, 18\text{ in} \right) = \min \left( \frac{250\text{ ft}}{5}, \frac{12\text{ in}}{f_t}, 3 \times 12\text{ in}, 18\text{ in} \right) = 18\text{ in} \]

\[ S = 8\text{ in} < 18\text{ in} \text{ (OK)} \]

**Minimum Vertical Reinforcement (ACI 11.9.9.4)**

\[ \rho_t = \max (0.0025 + 0.5 \left( 2.5 - \frac{h_w}{l_w} \right) (\rho_t - 0.0025), 0.0025) \leq 0.0015bh \]

\[ \rho_t = \max \left( 0.0025 + 0.5 \left( 2.5 - \frac{23\text{ ft}}{250\text{ ft}} \right) (0.00426 - 0.0025), 0.0025 \right) = 0.00426 \leq 0.0015 \times 12\text{ in} \times 12\text{ in} \]

\[ = 0.216 \text{ (OK)} \]

**Spacing of Vertical Shear Reinforcement (ACI 11.9.9.5)**

\[ S_{\text{max}} \leq \min \left( \frac{l_w}{3}, 3h, 18\text{ in} \right) = \min \left( \frac{250\text{ ft}}{3}, \frac{12\text{ in}}{f_t}, 3 \times 12\text{ in}, 18\text{ in} \right) = 18\text{ in} \]
Design for Flexure and Axial Force

Assume $\Phi = 0.90$ and a tension control section with uniform reinforcement

$$M_u = 11.215\, \text{kft}$$

$$M_u = \Phi A_s f_y \left( d - \frac{A_s f_y}{1.7 f'_c b} \right) = 0.90 \times A_s \times 60\, \text{ksi} \times \left( 8.375\, \text{in} - \frac{A_s \times 60\, \text{ksi}}{1.7 \times 4\, \text{ksi} \times 12\, \text{in}} \right)$$

$\rightarrow$ solving the quadratic equation for $A_s$ will yield $A_s = 0.306 \, \text{in}^2$

Use #6 bar ($A_s = 0.44\, \text{in}^2$) in two layers of reinforcement

$$\rho_t = \frac{0.44\, \text{in}^2}{12\, \text{in} \times 12\, \text{in}} = 0.00307 \geq 0.0025 \, (OK)$$

Secondary Reinforcement for Shrinkage and Temperature

$$A_s = 0.0018bh = 0.0018 \times 12\, \text{in} \times 12\, \text{in} = 0.259 \, \text{in}^2$$

Use #4 bars spaced at 6 in apart, $A_s = 0.39\, \text{in}^2 > 0.259 \, \text{in}^2 \, (OK)$

Check for Wall slenderness

$$r = 0.3h = 0.3 \times 12\, \text{in} = 3.6\, \text{in} = 0.3\, \text{ft}$$

$$k = 0.80 \, (fixed \, pinned \, condition)$$

$$M_1 = 0 \, because \, a \, pinned \, connection \, is \, assumed \, at \, the \, floor \, slab \, level \, at \, the \, wall$$

$$\frac{kl_u}{r} < 34 - 12 \frac{M_1}{M_2} \leq 40 = \frac{0.80 \times 9\, \text{ft}}{0.3\, \text{ft}} = 29.33 < 34 - 12(0) \leq 40 \, \therefore \, wall \, is \, not \, slender$$

Depth of compressive block

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.44\, \text{in}^2 \times 60\, \text{ksi}}{0.85 \times 4\, \text{ksi} \times 12\, \text{in}} = 0.450\, \text{in}$$

$$c = \frac{a}{\beta_t} = \frac{0.450\, \text{in}}{0.85} = 0.529\, \text{in}$$
Flexural strength

\[ \Phi M_n = \Phi \left[ 0.85 f' \text{c} \left( \frac{h}{2} - \frac{a}{2} \right) + A_s f_y \left( d - \frac{h}{2} \right) \right] \]

\[ = 0.90 \left[ 0.85 \times 4 \text{ksi} \times 0.45 \text{in} \times 12 \text{in} + 0.44 \text{in}^2 \times 60 \text{ksi} \times \left( \frac{8.375 \text{in} - 12 \text{in}}{2} \right) \right] \]

\[ = 174,606 \text{lb - in} = 14.55 \text{kft} \geq M_u \]

Max spacing = min(3h, 18 in) = min(3 \times 12 in, 18 in) = 18 in

Use a spacing of 12 in

Dowel Length

According to ACI, the development length should be the larger of the lap splice in tension of the thinner bar or the development length of the thicker bar.

**Basic Tension-Development Equation**

\[ l_d = \frac{3}{40 \lambda \sqrt{f'c} \left( \frac{c_b + k_{tr}}{d_b} \right)} \psi_1 \psi_2 \psi_3 d_b \geq 12 \text{ in} \quad (\text{ACI Eq. 12-1}) \]

\( \psi_1 \): bar-location factor (ACI 12.2.4) = 1.0

\( \psi_2 \): epoxy coating factor (ACI 12.2.4) = 1.0

\( \psi_3 \): bar-size factor (ACI 12.2.4) = 1.0

\( \lambda \): lightweight concrete factor (ACI 12.2.4(d)) = 1.0

\( C_b \): min (smallest distance measured from the surface of the concrete to the center of a bar being developed, one half of the center-to-center spacing of the bars or wires being developed)

\( K_{tr} \): transverse reinforcement factor (ACI 12.2.3)

\( \left( \frac{c_b + k_{tr}}{d_b} \right) \) is limited to 2.5 or smaller, to prevent pull-out bond failures
\[ l_d = \frac{3}{40} \times \frac{60 \text{ksi} \times 1.0 \times 1.0}{\sqrt{4 \text{ksi}}} \times \left( \frac{6 \text{ in}}{8 \text{ in}} \right) = 28.46 \text{ in} \geq 12 \text{ in} \]

For a tension lap splice:

\[ l_d = 1.3l_d = 1.3 \times 28.46 \text{ in} = 37.0 \text{ in} \rightarrow \text{use 40 in} \]

*Use # 6 bars for dowels*

\[ l_d = \frac{f_y \psi_t \psi_e d_b}{25 \lambda \sqrt{f_c}} = \frac{60 \text{ksi} \times 1.0 \times 1.0}{\sqrt{4 \text{ksi}}} \left( \frac{6 \text{ in}}{8 \text{ in}} \right) = 28.46 \text{ in} \]

**Axial Strength**

If the resultant of all factored loads is located within the middle third of the thickness of a solid wall with a rectangular section, the nominal axial strength is calculated by ACI 14.5.2.

\[
R_n = 0.55f'cA_0\left[1 - \left(\frac{k_L}{32h}\right)^2\right] = 0.55 \times \frac{4 \text{ksi}}{1} \times (12 \text{ in} \times 12 \text{ in}) \times [1 - \left(\frac{0.80 \times 9 \text{ ft}}{32 \times 1 \text{ ft}}\right)^2] = 25.06 \text{k/ft}
\]

\[
\phi R_n = 0.65 \times \frac{25.06 \text{k/ft}}{\text{ft}} = \frac{16.29 \text{k}}{\text{ft}} > \frac{4.83 \text{k}}{\text{ft}} \quad (\text{OK})
\]

**Foundation Wall Footing Design**

Assume a depth of footing, \( h = 24 \text{ in} \) and that the bottom of the footing is 25 ft below grade.

\[
d = h - 3.5\text{ in} = 24\text{ in} - 3.5\text{ in} = 20.5\text{ in}
\]

The unit weight of concrete is 150pcf and the unit weight of soil is assumed to be 100pcf. The allowable bearing capacity of the underlain rock is 25,000psf.

\[
\text{Effective soil pressure} = q_e = 25,000\text{psf} - (150\text{pcf} \times 2\text{ft}) - (100\text{pcf} \times (25\text{ft} - 2\text{ft})) = 21,400\text{psf}
\]

\[
\text{Width of Footing} = \frac{\text{Service Loads}}{q_e} = \frac{3.45 \text{k}}{\text{ft} \times 1000} = \frac{3.45 \text{k}}{21,400 \text{psf}} = 0.16\text{ft}
\]

Use a 6ft wide footing

\[
P_u = 14 \times \frac{3.45 \text{k}}{\text{ft}} = 4.83 \text{k/ft}
\]
Net Upward Pressure per 1ft width

\[ q_u = \frac{4.83k/ft}{10ft} = 0.483k/ft \]

One Way Shear \( (c = \text{width of wall}) \)

\[ V_u = q_u \left( \frac{B}{2} - d - \frac{c}{2} \right) = \frac{0.483k}{ft} \left( \frac{6ft}{2} - 20.5in \times \frac{12in}{ft} - \frac{12in}{2x\frac{12in}{ft}} \right) = 0.637k \]

\[ V_u = 22.7k \text{ will control the design} \]

Required \( d \)

\[ d = \frac{V_u}{\phi 2\lambda \sqrt{f'c}bd} = \frac{22.7k \times 1000lb/k}{0.75 \times 2 \times 1.0\sqrt{4ksi} \times 12in/ft} = 19.94\text{in} \]

\[ d = 20.5in > 19.94\text{in} \text{ (OK)} \]

\[ \phi V_c = \phi 2\lambda \sqrt{f'c}bd = 0.75 \times 2 \times 1.0\sqrt{4ksi} \times \frac{12in}{ft} \times 20.5in = \frac{23.34k}{ft} > \frac{22.7k}{ft} \text{ (OK)} \]

Moment

\[ M_u = \frac{q_u (B - c)^2}{2} \left( \frac{6ft}{2} - 20.5in \times \frac{12in}{ft} \right)^2 = 0.568k - ft \]

\[ M_u = 11.215k - ft \text{ will control the design} \]

\[ \rho_{\text{min}} = \frac{200}{f_y} = \frac{200}{60,000} = 0.0033 \]

Use a \( \rho \) of 0.004

\[ A_s = \rho bd = 0.04 \times 12in \times 20.5in = 0.984in^2 \]

Use #8 bars spaced at 9in apart, \( A_s = 1.05in^2 \)

\[ a = \frac{A_s f_y}{0.85 f_m b} = \frac{1.05in^2 \times 60ksi}{0.85 \times 4 \text{ ksi} \times 12in} = 0.904\text{in} \]
\[ \varphi M_n = \varphi A_s f_y \left( d - \frac{c}{2} \right) = 0.90 \times 1.05 \text{in}^2 \times 60 \text{ksi} \left( \frac{20.5 \text{in} - \frac{0.905 \text{in}}{2}}{12 \text{in}} \right) \times \frac{ft}{12 \text{in}} = 94.73kft > M_u \]

\[ c = \frac{a}{0.85} = \frac{0.904 \text{in}}{0.85} = 1.063 \text{in} \]

\[ \frac{c}{d} < 0.375 = \frac{1.063}{20.5} = 0.052 < 0.375 \text{ (tension controlled)} \]

**Development Length**

\[ l_d = \max \left( B - \frac{c}{2} - 3 \text{in of concrete cover, } 48d_b \right) = \max \left( \frac{10ft \times 12\text{in}}{2} - \frac{12\text{in}}{2} - 3\text{in} \times 48 \times \left( \frac{8\text{in}}{8\text{in}} \right) \right) = 51 \text{ in} \]

**Secondary Reinforcement for Shrinkage and Temperature**

\[ A_s = 0.0018bh = 0.0018 \times 12\text{in} \times 24\text{in} = 0.518 \text{ in}^2 \]

*Use #5 bars spaced at 6 in apart, } A_s = 0.61\text{in}^2 > 0.518 \text{in}^2 *

**Penthouse Joist Design**

Data contained in the Marino\WARE catalog is based on allowable strength design (ASD) of the 2012 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-12 (S100), therefore design loads are not factored for strength design.

**Loads Summary**

Table 6, Penthouse Joist Load Summary:

<table>
<thead>
<tr>
<th>Level</th>
<th>Dead Load (psf)</th>
<th>Live Load (psf)</th>
<th>Snow Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>44</td>
<td>38</td>
<td>53</td>
</tr>
<tr>
<td>17th - 18th Floor</td>
<td>44</td>
<td>40 (private) 100 (public)</td>
<td></td>
</tr>
</tbody>
</table>
Joist spans 24 ft between exterior wall and intermediate wall. Joist are spaced at 16 in. on center. ASD load combinations used for strength design.

**Strength requirements**

\[ w = (44 + 53) \left(\frac{16}{12}\right) = 129.3 \text{ lb/ft} \]

\[ M_u = 9.31 k \cdot ft = 111.7 k \cdot \text{in} \]

\[ V_u = 1550 \text{ lb} \]

**Serviceability requirements**

\[ I_x \text{ for L/240 for Total Load (DL+SL)} \]

\[ I_{x,TL} = 27.74 \text{ in}^4 \]

\[ I_x \text{ for L/360 for Live Load (DL+SL)} \]

\[ I_{x,LL} = 16.30 \text{ in}^4 \]

Use 12J12 (1200S200-97) Joist spaced at 16 in. on center.

| 12J12 | \( I_x = 33.7 \text{ in}^4 \) | \( M_n = 135 k \cdot \text{in} \) | \( V_n = 7410 \text{ lb} \) |

Joist on the opposite side of intermediate wall spans 16 ft between exterior wall and intermediate wall. Joist are spaced at 16 in. on center.

**Strength requirements**

\[ w = (44 + 53) \left(\frac{16}{12}\right) = 129.3 \text{ lb/ft} \]

\[ M_u = 4.14 k \cdot ft = 49.7 k \cdot \text{in} \]

\[ V_u = 1030 \text{ lb} \]

**Serviceability requirements**

\[ I_x \text{ for L/240 for Total Load (DL+SL)} \]

\[ I_{x,TL} = 8.22 \text{ in}^4 \]

\[ I_x \text{ for L/360 for Live Load (DL+SL)} \]

\[ I_{x,LL} = 4.83 \text{ in}^4 \]

Use 10J14 (1000S200-68) or 12J16 (1200S200-54) joist spaced at 16 in. on center or 12J12 (1200S200-97) joist spaced at 32 in. on center.

| 10J14 | \( I_x = 13.6 \text{ in}^4 \) | \( M_n = 64.5 k \cdot \text{in} \) | \( V_n = 3345 \text{ lb} \) |
| 12J16 | \( I_x = 16.1 \text{ in}^4 \) | \( M_n = 54.8 k \cdot \text{in} \) | \( V_n = 1377 \text{ lb} \) |
Header beam spans 16 ft between exterior wall and concrete core wall is subject to a linearly increasing distribution due to the triangular tributary area. Tributary width linearly increases from 0 ft to 5’-6”.

**Strength requirements**

- \( w_1 = 0 \)
- \( w_2 = (44 + 53)(5.5) = 533.5 \text{ lb/ft} \)
- \( M_u = 8.76 \text{ k-ft} = 105.1 \text{ k-in} \)
- \( V_u = 2850 \text{ lb} \)

**Serviceability requirements**

- \( I_x \) for \( L/240 \) for Total Load (DL+SL)
- \( I_{x,TL} = 16.98 \text{ in}^4 \)
- \( I_s \) for \( L/360 \) for Live Load (DL+SL)
- \( I_{s,LL} = 9.98 \text{ in}^4 \)

Use (2) 12J16 (1200S200-54) Box Beam.

| (2) 12J16 | \( I_x = 32.2 \text{ in}^4 \) | \( M_n = 109.6 \text{ k-in} \) | \( V_n = 2754 \text{ lb} \) |

Joists spanning between concrete core wall and the header beam, should be same size as those joists spanning 16 ft. between exterior wall and intermediate wall for economy. The reaction on the concrete core wall is calculated as follows:

- Joist span = 12 ft \( \Rightarrow R_{wall} = (44 + 53)(\frac{12}{2}) = 582 \text{ lb/ft} \)

**Concrete wall anchorage**

Joists will be mounted to the concrete core wall using a light-gage steel track. A 12T14 (1200T200-68) should be used as the ledger which has a shear capacity of 2712 lb.

The track will be anchored to the wall using Hilti Powder Actuated Fasteners. Hilti X-U Powder actuated Fasteners with 1” embedment into 4,000 psi concrete develop shear strength of 225 lb. Using (2) fasteners with 8 in. on center spacing provides a shear capacity of 675 lb/ft which is sufficient to resist 582 lb/ft.
Design of 17th & 18th Floor Joists

Joist spans 24 ft between exterior wall and intermediate wall. Joist are spaced at 16 in. on center. ASD load combinations used for strength design.

Strength requirements

\[ w = (44 + 40) \left( \frac{16}{12} \right) = 112.0 \text{ lb/ft} \]
\[ M_u = 8.06 \text{k ft} = 96.8 \text{k in} \]
\[ V_u = 1340 \text{ lb} \]

Serviceability requirements

\[ I_x \text{ for } L/240 \text{ for Total Load (DL+SL)} \]
\[ I_{x,TL} = 24.03 \text{ in}^4 \]
\[ I_x \text{ for } L/360 \text{ for Live Load (DL+SL)} \]
\[ I_{x,LL} = 17.16 \text{ in}^4 \]

Use 12J12 (1200S200-97) Joist spaced at 16 in. on center.

| 12J12 | \( I_x = 33.7 \text{ in}^4 \) | \( M_n = 135 \text{ k in} \) | \( V_n = 7410 \text{ lb} \) |

Joist on the opposite side of intermediate wall spans 16 ft between exterior wall and intermediate wall. Joist are spaced at 16 in. on center.

Strength requirements

\[ w = (44 + 40) \left( \frac{16}{12} \right) = 112.0 \text{ lb/ft} \]
\[ M_u = 3.58 \text{k ft} = 43.01 \text{k in} \]
\[ V_u = 900 \text{ lb} \]

Serviceability requirements

\[ I_x \text{ for } L/240 \text{ for Total Load (DL+SL)} \]
\[ I_{x,TL} = 7.12 \text{ in}^4 \]
\[ I_x \text{ for } L/360 \text{ for Live Load (DL+SL)} \]
\[ I_{x,LL} = 5.08 \text{ in}^4 \]

Use 10J16 (1000S200-54) or 12J16 (1200S200-54) joist spaced at 16 in. on center or 12J12 (1200S200-97) joist spaced at 32 in. on center.

| 10J16 | \( I_x = 10.7 \text{ in}^4 \) | \( M_n = 46.6 \text{ k in} \) | \( V_n = 1660 \text{ lb} \) |
| 12J16 | \( I_x = 16.1 \text{ in}^4 \) | \( M_n = 54.8 \text{ k in} \) | \( V_n = 1377 \text{ lb} \) |
Header beam spans 16 ft between exterior wall and concrete core wall is subject to a linearly increasing distribution due to the triangular tributary area. Tributary width linearly increases from 0 ft to 5’-6”.

**Strength requirements**

\[ w_1 = 0 \quad w_2 = (44 + 40)(5.5) = 462.0 \text{ lb/ft} \]

\[ M_u = 7.59 \text{ k · ft} = 91.05 \text{ k · in} \]

\[ V_u = 2460 \text{ lb} \]

**Serviceability requirements**

\[ l_x \text{ for } L/240 \text{ for Total Load (DL+SL)} \]

\[ l_{x,TL} = 14.71 \text{ in}^4 \]

\[ l_x \text{ for } L/360 \text{ for Live Load (DL+SL)} \]

\[ l_{x,LL} = 10.51 \text{ in}^4 \]

Use (2) 12J16 (1200S200-54) Box Beam.

| (2) 12J16 | \( l_x = 32.2 \text{ in}^4 \) | \( M_n = 109.6 \text{ k · in} \) | \( V_n = 2754 \text{ lb} \) |

Joists spanning between concrete core wall and the header beam, should be same size as those joists spanning 16 ft. between exterior wall and intermediate wall for economy. The reaction on the concrete core wall is calculated as follows:

\[ \text{Joist span} = 12 \text{ ft} \rightarrow R_{wall} = (44 + 40) \left( \frac{12}{2} \right) = 504 \text{ lb/ft} \]

**Concrete wall anchorage**

Joists will be mounted to the concrete core wall using a light-gage steel track. A 12T14 (1200T200-68) should be used as the ledger which has a shear capacity of 2712 lb.

The track will be anchored to the wall using Hilti Powder Actuated Fasteners. Hilti X-U Powder actuated Fasteners with 1” embedment into 4,000 psi concrete develop shear strength of 225 lb. Using (2) fasteners with 8 in. on center spacing provides a shear capacity of 675 lb/ft which is sufficient to resist 504 lb/ft.
Data contained in the Marino\WARE catalog is based on allowable strength design (ASD) of the 2012 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-12 (S100), therefore design loads are not factored for strength design.

### Loads Summary

Table 7, Load Summary

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<td>38</td>
<td>53</td>
<td>41.1</td>
</tr>
<tr>
<td>18th Floor</td>
<td>44</td>
<td>40 (private)</td>
<td>-</td>
<td>40.7</td>
</tr>
<tr>
<td>17th Floor</td>
<td>44</td>
<td>40 (private)</td>
<td>-</td>
<td>40.3</td>
</tr>
</tbody>
</table>

### Header Design (typical double windows on roof level)

The length of the door is 4.60ft. The tributary width is 12 ft. A boxed beam with 2 studs will be used as a header, thus each header will carry half of the gravity loads from above.

#### Strength requirements

**ASD Load Combination:** \( D + 0.75L + 0.75S \)

\[
w_u = \frac{1}{2} (44 + 0.75(38) + 0.75(53))(12\text{ft})\]

\[
= \frac{k}{1000lb} = 0.687\text{ k/ft}
\]

\[
M_u = \frac{0.687 \times (4.60\text{ft})^2}{8} = 1.78\text{ kft}
\]

\[
= 21.36\text{ k} - \text{in}
\]

\[
V_u = \frac{0.678k\text{ ft}}{2} \times \frac{4.60\text{ft}}{2} = 1.55k = 1550\text{lb}
\]

**Serviceability requirements**

\( I_s \) for L/240 for Total Load (DL+LL)

\( I_{x,TL} = 1.00\text{ in}^4 \)

\( I_s \) for L/360 for Live Load (LL)

\( I_{x,LL} = 1.50\text{ in}^4 \)

Use 2 6J16 (600S200-54) for the header.
6J16  \( I_x = 2.86 \text{ in}^4 \)  \( M_n = 25.9 \text{ k \cdot in} \)  \( V_n = 1947 \text{ lb} \)

**Header Design (typical door)**

The length of the double windows is 5.46ft. The tributary width is 12 ft.

### Strength requirements

\[
\begin{align*}
    w_u &= 0.5(44 + 40)(12\text{ft}) \cdot \frac{k}{1000\text{lb}} \\
    &= 0.504 \text{ k/ft} \\
    M_u &= \frac{0.504 \cdot (5.46\text{ft})^2}{8} = 1.875 \text{ kft} \\
    &= 22.5k \text{ in} \\
    V_u &= \frac{0.504k}{\text{ft}} \cdot \frac{5.46\text{ft}}{2} = 1.375k = 1375\text{lb}
\end{align*}
\]

### Serviceability requirements

\[
\begin{align*}
    I_x \text{ for } L/240 \text{ for Total Load (DL+LL)} \\
    I_x,TL &= 1.25 \text{ in}^4 \\
    I_x,LL &= 1.88 \text{ in}^4
\end{align*}
\]

Use 2 6J16 (600S200-97) for the header

6J16  \( I_x = 2.86 \text{ in}^4 \)  \( M_n = 25.9 \text{ k \cdot in} \)  \( V_n = 1947 \text{ lb} \)

**Jamb Design**

The jamb is a vertical element that the header will frame into and will resist both the gravity loads from the walls and floors above and a lateral wind pressure. A 40 psf wind pressure will be used for design. The tributary width for each jamb (two on each side) is half the width of the header for a door and half the spacing of the floor joists.

### Jamb Loads

\[
\begin{align*}
    \text{Tributary width} &= 0.5(5.46\text{ft}) + 0.5(1\text{ft}) = 3.23\text{ft} \\
    \text{Tributary area} &= 3.23\text{ft} \cdot 12\text{ft} = 38.76\text{ft}^2
\end{align*}
\]
Roof Axial Load = \((44 + 0.75(38) + 0.75(53)) \times 38.76\text{ft}^2 \times \frac{k}{1000\text{lb}} = 4.35k\)

\[17th, 18th\text{ floor Axial Load} = (44 + 38) \times 38.76\text{ft}^2 \times \frac{k}{1000\text{lb}} = 3.178k\]

Total \(P_u = 4.35k + 2 \times 3.178k = 10.71k\)

Wall height is 10 ft and stud spacing is 12 ft. The wind load is reduced by 0.6 per 2012 IBC (0.6*40psf = 24 psf), thus 25 psf tables will be used.

| 600S200-97 (50ksi) | \(P_{capacity} = 14.1k\) |

**Penthouse Stud Wall Design**

Data contained in the Marino\WARE catalog is based on allowable strength design (ASD) of the 2012 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-12 (S100), therefore design loads are not factored for strength design.

**Loads Summary**

Table 8, Penthouse Stud Wall Load Summary

<table>
<thead>
<tr>
<th>Level</th>
<th>Dead Load (psf)</th>
<th>Live Load (psf)</th>
<th>Snow Load (psf)</th>
<th>Wind Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>44</td>
<td>38</td>
<td>53</td>
<td>41.1</td>
</tr>
<tr>
<td>18th Floor</td>
<td>44</td>
<td>40 (private)</td>
<td>-</td>
<td>40.7</td>
</tr>
<tr>
<td>17th Floor</td>
<td></td>
<td></td>
<td></td>
<td>40.3</td>
</tr>
</tbody>
</table>

**18th Floor - Roof**

**Exterior Stud Wall Design**

Wall height is 10 ft. The wall into which joists spanning 24 ft. will govern the design due to larger tributary width. ASD load combinations used for strength design (0.6WL per 2012 IBC). For deflection calculations, wind pressures have been reduced by 0.70 as per 2012 IBC.
Intermediate Stud Wall Design

Wall height is 10 ft. and is subject only to axial load.

\[ P_u = (44 + 53) \left( \frac{24 + 16}{2} \right) = 1.94 \text{ k/ft} \]

For 16 in. spacing \( P_u = 1.94 \left( \frac{16}{12} \right) = 2.59 \text{ kip} \)

Use 362S200-43 studs spaced at 12 in. on center have nominal axial capacity of 1.53 kip with 25 psf lateral load. Deflection meets L/720.

\[ WL = 0.6(41.1) = 24.66 \text{ psf (Use 25 psf table)} \]

17th Floor - 18th Floor

Exterior Stud Wall Design

Wall height is 10 ft. The wall into which joists spanning 24 ft. will govern the design due to larger tributary width.

\[ P_u = 1160 + (44 + 40) \left( \frac{24}{2} \right) = 2.17 \text{ k/ft} \]

\[ WL = 0.6(40.7) = 24.42 \text{ psf (Use 25 psf table)} \]

Use 362S162-54 studs spaced at 12 in. on center have nominal axial capacity of 2.42 kip with 25 psf lateral load. Deflection meets L/720.

Intermediate Stud Wall Design

Wall height is 10 ft. and is subject only to axial load.

\[ P_u = 1940 + (44 + 40) \left( \frac{24 + 16}{2} \right) = 3.62 \text{ k/ft} \]

For 16 in. spacing \( P_u = 3.62 \left( \frac{16}{12} \right) = 4.83 \text{ kip} \)

Use 362S250-54 studs spaced at 16 in. on center have nominal axial capacity of 4.95 kip with 5 psf lateral load. Deflection meets L/720.
Exterior Stud Wall Design

Wall height is 10 ft. The wall into which joists spanning 24 ft. will govern the design due to larger tributary width.

\[ P_u = 2170 + (44 + 40) \left( \frac{24}{2} \right) = 3.18 \text{ kip/ft} \]
\[ WL = 0.6(40.3) \]
\[ = 24.18 \text{ psf (Use 25 psf table)} \]

Concrete Slab Anchorage

\[ R_{slab} = 40.3 \left( \frac{10}{2} \right) = 201.5 \text{ lb/ft} \]

Studs will be anchored to the concrete floor slab using a light-gage steel track. A 358T16 (362T250-54) should be used as the ledger which has a shear capacity of 2141 lb.

The track will be anchored to the wall using Hilti Powder Actuated Fasteners. Hilti X-U Powder actuated Fasteners with 1” embedment into 4,000 psi concrete develop shear strength of 225 lb. Using staggered fasteners with 6 in. on center spacing provides a shear capacity of 450 lb/ft which is sufficient to resist 201.5 lb/ft.

Intermediate Stud Wall Design

Wall height is 10 ft. and is subject only to axial load.

\[ P_u = 3620 + (44 + 40) \left( \frac{24 + 16}{2} \right) = 5.30 \text{ kip/ft} \]

For 16 in. spacing \[ P_u = 5.30 \left( \frac{16}{12} \right) = 7.07 \text{ kip} \]

Use 400S250-68 studs spaced at 16 in. on center have nominal axial capacity of 7.64 kip with 5 psf lateral load. Deflection meets L/720.
Combined footing details
Length of combined footing; \( L = 6.000 \text{ ft} \)
Width of combined footing; \( B = 6.000 \text{ ft} \)
Area of combined footing; \( A = L \times B = 36.000 \text{ ft}^2 \)
Depth of combined footing; \( h = 24.000 \text{ in} \)
Depth of soil over combined footing; \( h_{soil} = 36.000 \text{ in} \)
Density of concrete; \( \rho_{conc} = 150.0 \text{ lb/ft}^3 \)

Column details
Column base length; \( l_A = 18.000 \text{ in} \)
Column base width; \( b_A = 18.000 \text{ in} \)
Column eccentricity in x; \( e_{PxA} = 0.000 \text{ in} \)
Column eccentricity in y; \( e_{PYA} = 0.000 \text{ in} \)

Soil details
Density of soil; \( \rho_{soil} = 120.0 \text{ lb/ft}^3 \)
Angle of internal friction; \( \phi' = 25.0 \text{ deg} \)
Design base friction angle; \( \delta = 19.3 \text{ deg} \)
Coefficient of base friction; \( \tan(\delta) = 0.350 \)
Allowable bearing pressure; \( P_{bearing} = 24.000 \text{ ksf} \)

Axial loading on column
Dead axial load on column; \( P_{OA} = 500.000 \text{ kips} \)
Live axial load on column; \( P_{QA} = 100.000 \text{ kips} \)
Wind axial load on column; \( P_{WA} = 0.000 \text{ kips} \)
Total axial load on column; \( P_A = 600.000 \text{ kips} \)
## Foundation loads

- **Dead surcharge load;**
  \[ F_{Gsur} = 0.100 \text{ ksf} \]

- **Live surcharge load;**
  \[ F_{Qsur} = 0.100 \text{ ksf} \]

- **Footing self weight;**
  \[ F_{swt} = h \times \rho_{conc} = 0.300 \text{ ksf} \]

- **Soil self weight;**
  \[ F_{soil} = h_{soil} \times \rho_{soil} = 0.360 \text{ ksf} \]

**Total foundation load;**
\[ F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = 30.960 \text{ kips} \]

### Calculate base reaction

**Total base reaction;**
\[ T = F + P_A = 630.960 \text{ kips} \]

**Eccentricity of base reaction in x;**
\[ e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = 0.000 \text{ in} \]

**Eccentricity of base reaction in y;**
\[ e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T = 0.000 \text{ in} \]

### Check base reaction eccentricity

\[ \text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = 0.000 \]

*Base reaction acts within middle third of base*

### Calculate base pressures

- **17.527 ksf**
  \[ q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \]

- **17.527 ksf**
  \[ q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \]

- **17.527 ksf**
  \[ q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \]

- **17.527 ksf**
  \[ q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \]

**Minimum base pressure;**
\[ q_{min} = \min(q_1, q_2, q_3, q_4) = 17.527 \text{ ksf} \]

**Maximum base pressure;**
\[ q_{max} = \max(q_1, q_2, q_3, q_4) = 17.527 \text{ ksf} \]

*PASS - Maximum base pressure is less than allowable bearing pressure*
Load combination factors for loads
-Load combination factor for dead loads; \(\gamma_G = 1.20\)
-Load combination factor for live loads; \(\gamma_Q = 1.60\)
-Load combination factor for wind loads; \(\gamma_W = 0.00\)

Strength reduction factors
- Flexural strength reduction factor; \(\phi_f = 0.90\)
- Shear strength reduction factor; \(\phi_s = 0.75\)

Ultimate axial loading on column
Ultimate axial load on column; \(P_{UA} = P_GA \times \gamma_G + P_QA \times \gamma_Q + P_{WA} \times \gamma_W = 760.000\) kips

Ultimate foundation loads
Ultimate foundation load; \(F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_G + F_{Qsur} \times \gamma_Q] = 38.592\) kips

Ultimate horizontal loading on column
Ultimate horizontal load in x direction; \(H_{uxA} = H_{GxA} \times \gamma_G + H_{QxA} \times \gamma_Q + H_{WxA} \times \gamma_W = 0.000\) kips
Ultimate horizontal load in y direction; \(H_{uyA} = H_{GyA} \times \gamma_G + H_{QyA} \times \gamma_Q + H_{WyA} \times \gamma_W = 0.000\) kips

Ultimate moment on column
Ultimate moment on column in x direction; \(M_{uxA} = M_{GxA} \times \gamma_G + M_{QxA} \times \gamma_Q + M_{WxA} \times \gamma_W = 0.000\) kip_ft
Ultimate moment on column in y direction; \(M_{uyA} = M_{GyA} \times \gamma_G + M_{QyA} \times \gamma_Q + M_{WyA} \times \gamma_W = 0.000\) kip_ft

Calculate ultimate base reaction
Ultimate base reaction; \(T_u = F_u + P_{UA} = 798.592\) kips
Eccentricity of ultimate base reaction in x; \(e_{Txu} = (P_{UA} \times e_{PxA} + M_{uxA} + H_{uxA} \times h) / T_u = 0.000\) in
Eccentricity of ultimate base reaction in y; \(e_{Tyu} = (P_{UA} \times e_{PYA} + M_{uyA} + H_{yuA} \times h) / T_u = 0.000\) in

Calculate ultimate base pressures
- \(q_{1u} = T_u / A - 6 \times T_u \times e_{Txu} / (L \times A) - 6 \times T_u \times e_{Pyu} / (B \times A) = 22.183\) ksf
- \(q_{2u} = T_u / A - 6 \times T_u \times e_{Txu} / (L \times A) + 6 \times T_u \times e_{Tyu} / (B \times A) = 22.183\) ksf
- \(q_{3u} = T_u / A + 6 \times T_u \times e_{Txu} / (L \times A) - 6 \times T_u \times e_{Tyu} / (B \times A) = 22.183\) ksf
- \(q_{4u} = T_u / A + 6 \times T_u \times e_{Txu} / (L \times A) + 6 \times T_u \times e_{Tyu} / (B \times A) = 22.183\) ksf

Minimum ultimate base pressure;
Maximum ultimate base pressure;
\(q_{\text{minu}} = \text{min}(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 22.183\) ksf
\(q_{\text{maxu}} = \text{max}(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 22.183\) ksf

Calculate rate of change of base pressure in x direction
Left hand base reaction; \(f_{ul} = (q_{1u} + q_{2u}) \times B / 2 = 133.099\) kips/ft
Right hand base reaction; \(f_{ur} = (q_{3u} + q_{4u}) \times B / 2 = 133.099\) kips/ft
Length of base reaction; \(L_s = L = 72.000\) in
Rate of change of base pressure; \(C_x = (f_{ur} - f_{ul}) / L_s = 0.000\) kips/ft

Calculate footing lengths in x direction
Left hand length; \(L_L = L / 2 + e_{PxA} = 3.000\) ft
Right hand length; \(L_R = L / 2 - e_{PxA} = 3.000\) ft

Calculate ultimate moments in x direction
Ultimate moment in x direction; \(M_x = f_{ul} \times L_s^2 / 2 + C_x \times L_s^2 / 6 - F_u \times L_s^2 / (2 \times L) = 570.000\) kip_ft

Calculate rate of change of base pressure in y direction
Top edge base reaction; \(f_{uT} = (q_{2u} + q_{4u}) \times L / 2 = 133.099\) kips/ft
Bottom edge base reaction; \(f_{ub} = (q_{1u} + q_{3u}) \times L / 2 = 133.099\) kips/ft
Length of base reaction; $L_y = B = 6.000$ ft
Rate of change of base pressure; $C_y = (f_{ub} - f_{at}) / L_y = 0.000$ kips/ft/ft

**Calculate footing lengths in y direction**
- Top length; $L_T = B / 2 + e_{py}A = 3.000$ ft
- Bottom length; $L_b = B / 2 - e_{py}A = 3.000$ ft

**Calculate ultimate moments in y direction**
- Ultimate moment in y direction; $M_y = f_{ut} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) = 570.000$ kip_ft

**Material details**
- Compressive strength of concrete; $f_c = 4000$ psi
- Yield strength of reinforcement; $f_y = 60000$ psi
- Cover to reinforcement; $c_{nom} = 2.000$ in
- Concrete type; Normal weight
- Concrete modification factor; $\lambda = 1.00$

**Moment design in x direction**
- Reinforcement provided; 11 No. 7 bars bottom
- Depth of tension reinforcement; $d_x = h - c_{nom} - \phi_t \times \phi_b / 2 = 21.563$ in
- Area of tension reinforcement provided; $A_{s_x} = \pi \times \phi_b \times 2 / 4 = 6.615$ in$^2$
- Area of compression reinforcement provided; $A_{s_c} = \pi \times \phi_b \times 2 / 4 = 0.000$ in$^2$
- Minimum area of reinforcement; $A_{s,x,min} = 0.0018 \times h \times B = 3.110$ in$^2$
- Spacing of reinforcement; $s_{zb} = (B - 2 \times c_{nom}) / \max(N_{vb} - 1, 1) = 6.800$ in
- Maximum spacing of reinforcement; $s_{max} = \min(3 \times h, 18\text{in}) = 18.000$ in

**PASS - Reinforcement provided exceeds minimum reinforcement required**
- Depth of compression block $a_x = A_{s,zb,prov} \times f_y / (0.85 \times f'_c \times B) = 1.62$ in
- Neutral axis factor $\beta_1 = 0.85$
- Depth to the neutral axis $c_{na,x} = a_x / \beta_1 = 1.91$ in
- Strain in reinforcement $\varepsilon_{L,x} = 0.003 \times (d_x - c_{na,x}) / c_{na,x} = 0.03092$

**PASS - The section has adequate ductility (cl. 10.3.5)**

**Moment design in y direction**
- Reinforcement provided; 11 No. 7 bars bottom
- Depth of tension reinforcement; $d_y = h - c_{nom} - \phi_t \times \phi_b / 2 = 20.688$ in
- Area of tension reinforcement provided; $A_{s,y} = \pi \times \phi_b \times 2 / 4 = 6.615$ in$^2$
- Area of compression reinforcement provided; $A_{s,c} = \pi \times \phi_b \times 2 / 4 = 0.000$ in$^2$
- Minimum area of reinforcement; $A_{s,y,min} = 0.0018 \times h \times L = 3.110$ in$^2$
- Spacing of reinforcement; $s_{yb} = (L - 2 \times c_{nom}) / \max(N_{vb} - 1, 1) = 6.800$ in
- Maximum spacing of reinforcement; $s_{max} = \min(3 \times h, 18\text{in}) = 18.000$ in

**PASS - Reinforcement provided exceeds minimum reinforcement required**
- Depth of compression block $a_y = A_{s,yb,prov} \times f_y / (0.85 \times f'_c \times L) = 1.62$ in
Neutral axis factor

Depth to the neutral axis

Strain in reinforcement

Nominal moment strength required;

Moment capacity of base;

Shear design at d from top face of column

Strength reduction factor in shear;

Nominal shear strength;

Concrete shear strength;

Calculate ultimate punching shear force at perimeter of d / 2 from face of column

Punching shear stresses at perimeter of d / 2 from face of column

PASS - Nominal shear strength is less than concrete shear strength
Combined footing details
Length of combined footing; \( L = 8.000 \) ft
Width of combined footing; \( B = 8.000 \) ft
Area of combined footing; \( A = L \times B = 64.000 \) ft\(^2\)
Depth of combined footing; \( h = 30.000 \) in
Depth of soil over combined footing; \( h_{\text{soil}} = 24.000 \) in
Density of concrete; \( \rho_{\text{conc}} = 150.0 \) lb/ft\(^3\)

Column details
Column base length; \( l_A = 18.000 \) in
Column base width; \( b_A = 18.000 \) in
Column eccentricity in \( x \); \( e_{PxA} = 0.000 \) in
Column eccentricity in \( y \); \( e_{PYA} = 0.000 \) in

Soil details
Density of soil; \( \rho_{\text{soil}} = 120.0 \) lb/ft\(^3\)
Angle of internal friction; \( \phi' = 25.0 \) deg
Design base friction angle; \( \delta = 19.3 \) deg
Coefficient of base friction; \( \tan(\delta) = 0.350 \)
Allowable bearing pressure; \( P_{\text{bearing}} = 24.000 \) ksf

Axial loading on column
Dead axial load on column; \( P_{OA} = 900.000 \) kips
Live axial load on column; \( P_{QA} = 150.000 \) kips
Wind axial load on column; \( P_{WA} = 0.000 \) kips
Total axial load on column; \( P_A = 1050.000 \) kips
Foundation loads

Dead surcharge load;  
\( F_{Gsur} = 0.100 \text{ ksf} \)

Live surcharge load;  
\( F_{Qsur} = 0.100 \text{ ksf} \)

Footing self weight;  
\( F_{swt} = h \times \rho_{conc} = 0.375 \text{ ksf} \)

Soil self weight;  
\( F_{soil} = h_{soil} \times \rho_{soil} = 0.240 \text{ ksf} \)

Total foundation load;  
\( F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = 52.160 \text{ kips} \)

Calculate base reaction

Total base reaction;  
\( T = F + P_A = 1102.160 \text{ kips} \)

Eccentricity of base reaction in \( x \);  
\( e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = 0.000 \text{ in} \)

Eccentricity of base reaction in \( y \);  
\( e_{Ty} = (P_A \times e_{PxA} + M_{yA} + H_{yA} \times h) / T = 0.000 \text{ in} \)

Check base reaction eccentricity

\[ \text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = 0.000 \]

Base reaction acts within middle third of base

Calculate base pressures

17.221 ksf  
\( q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \)

17.221 ksf  
\( q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \)

17.221 ksf  
\( q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \)

17.221 ksf  
\( q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \)

Minimum base pressure;  
\( q_{min} = \min(q_1, q_2, q_3, q_4) = 17.221 \text{ ksf} \)

Maximum base pressure;  
\( q_{max} = \max(q_1, q_2, q_3, q_4) = 17.221 \text{ ksf} \)

PASS - Maximum base pressure is less than allowable bearing pressure
Load combination factors for loads

Load combination factor for dead loads; \( \gamma_G = 1.20 \)
Load combination factor for live loads; \( \gamma_Q = 1.60 \)
Load combination factor for wind loads; \( \gamma_W = 0.00 \)

Strength reduction factors

Flexural strength reduction factor; \( \phi_f = 0.90 \)
Shear strength reduction factor; \( \phi_s = 0.75 \)

Ultimate axial loading on column

Ultimate axial load on column; \( P_{UA} = P_GA \times \gamma_G + P_QA \times \gamma_Q + P_{WA} \times \gamma_W = 1320.000 \text{ kips} \)

Ultimate foundation loads

Ultimate foundation load; \( F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_G + F_{Qsur} \times \gamma_Q] = 65.152 \text{ kips} \)

Ultimate horizontal loading on column

Ultimate horizontal load in x direction; \( H_{xuA} = H_{Gxu} \times \gamma_G + H_{Qxu} \times \gamma_Q + H_{Wxu} \times \gamma_W = 0.000 \text{ kips} \)
Ultimate horizontal load in y direction; \( H_{yuA} = H_{Gyu} \times \gamma_G + H_{Qyu} \times \gamma_Q + H_{WyA} \times \gamma_W = 0.000 \text{ kips} \)

Ultimate moment on column

Ultimate moment on column in x direction; \( M_{xuA} = M_{Gxu} \times \gamma_G + M_{Qxu} \times \gamma_Q + M_{Wxu} \times \gamma_W = 0.000 \text{ kip-ft} \)
Ultimate moment on column in y direction; \( M_{yuA} = M_{Gyu} \times \gamma_G + M_{Qyu} \times \gamma_Q + M_{WyA} \times \gamma_W = 0.000 \text{ kip-ft} \)

Calculate ultimate base reaction

Ultimate base reaction; \( T_u = F_u + P_{uA} = 1385.152 \text{ kips} \)
Eccentricity of ultimate base reaction in x; \( e_{Txu} = (P_{uA} \times e_{PxA} + M_{xuA} + H_{xuA} \times h) / T_u = 0.000 \text{ in} \)
Eccentricity of ultimate base reaction in y; \( e_{Tyu} = (P_{uA} \times e_{Pya} + M_{yuA} + H_{yuA} \times h) / T_u = 0.000 \text{ in} \)

Calculate ultimate base pressures

\( q_{1u} = T_u/A - 6 \times T_u \times e_{Txu}/(L \times A) = 21.643 \text{ ksf} \)
\( q_{2u} = T_u/A - 6 \times T_u \times e_{Tyu}/(L \times A) = 21.643 \text{ ksf} \)
\( q_{3u} = T_u/A - 6 \times T_u \times e_{Txu}/(B \times A) = 21.643 \text{ ksf} \)
\( q_{4u} = T_u/A - 6 \times T_u \times e_{Tyu}/(B \times A) = 21.643 \text{ ksf} \)

Minimum ultimate base pressure; \( q_{minu} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 21.643 \text{ ksf} \)
Maximum ultimate base pressure; \( q_{maxu} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 21.643 \text{ ksf} \)

Calculate rate of change of base pressure in x direction

Left hand base reaction; \( f_{ul} = (q_{1u} + q_{2u}) \times B / 2 = 173.144 \text{ kips/ft} \)
Right hand base reaction; \( f_{ur} = (q_{3u} + q_{4u}) \times B / 2 = 173.144 \text{ kips/ft} \)
Length of base reaction; \( L_x = L = 96.000 \text{ in} \)
Rate of change of base pressure; \( C_x = (f_{ur} - f_{ul}) / L_x = 0.000 \text{ kips/ft} \)

Calculate footing lengths in x direction

Left hand length; \( L_L = L / 2 + e_{PxA} = 4.000 \text{ ft} \)
Right hand length; \( L_R = L / 2 - e_{PxA} = 4.000 \text{ ft} \)

Calculate ultimate moments in x direction

Ultimate moment in x direction; \( M_x = f_{ul} \times L_l^2 / 2 + C_x \times L_l^2 / 6 - F_u \times L_l^2 / (2 \times L) = 1320.000 \text{ kip-ft} \)

Calculate rate of change of base pressure in y direction

Top edge base reaction; \( f_{ut} = (q_{1u} + q_{3u}) \times L / 2 = 173.144 \text{ kips/ft} \)
Bottom edge base reaction; \( f_{ub} = (q_{1u} + q_{3u}) \times L / 2 = 173.144 \text{ kips/ft} \)
Calculate footing lengths in y direction
Top length; \[ L_T = \frac{B}{2} + \varepsilon_{p_y} = 4.000 \text{ ft} \]
Bottom length; \[ L_b = \frac{B}{2} - \varepsilon_{p_y} = 4.000 \text{ ft} \]

Calculate ultimate moments in y direction
Ultimate moment in y direction; \[ M_y = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) = 1320.000 \text{ kip_ft} \]

Material details
Compressive strength of concrete; \[ f_c = 4000 \text{ psi} \]
Yield strength of reinforcement; \[ f_y = 60000 \text{ psi} \]
Cover to reinforcement; \[ c_{nom} = 1.500 \text{ in} \]
Concrete type; Normal weight
Concrete modification factor; \[ \lambda = 1.00 \]

Moment design in x direction
Reinforcement provided; 15 No. 8 bars bottom
Depth of tension reinforcement; \[ d_x = h - c_{nom} - \psi_{B} / 2 = 28.000 \text{ in} \]
Area of tension reinforcement provided; \[ A_{x,T;\text{prov}} = N_{yB} \times \pi \times \psi_{B}^2 / 4 = 11.781 \text{ in}^2 \]
Area of compression reinforcement provided; \[ A_{x,C;\text{prov}} = N_{yT} \times \pi \times \psi_{T}^2 / 4 = 0.000 \text{ in}^2 \]
Minimum area of reinforcement; \[ A_{x,x;\text{min}} = 0.0018 \times h \times B = 5.184 \text{ in}^2 \]
Spacing of reinforcement; \[ s_{xB;\text{prov}} = (B - 2 \times c_{nom}) / \max(N_{yB} - 1, 1) = 6.643 \text{ in} \]
Maximum spacing of reinforcement; \[ s_{max} = \min(3 \times h, 18\text{in}) = 18.000 \text{ in} \]

PASS - Reinforcement provided exceeds minimum reinforcement required
Depth of compression block \[ a_x = A_{x,T;\text{prov}} \times f_y / (0.85 \times f'_c \times B) = 2.17 \text{ in} \]
Neutral axis factor \[ \beta_1 = 0.85 \]
Depth to the neutral axis \[ c_{na,x} = a_x / \beta_1 = 2.55 \text{ in} \]
Strain in reinforcement \[ \varepsilon_{L,x} = 0.003 \times (d_x - c_{na,x}) / c_{na,x} = 0.02997 \]

PASS - The section has adequate ductility (cl. 10.3.5)
Nominal moment strength required; \[ M_{nx;\text{abs}} = \phi_1 = 1466.667 \text{ kip_ft} \]
Moment capacity of base; \[ M_{capx} = A_{x,T;\text{prov}} \times f_y \times [d_x - (A_{x,T;\text{prov}} \times f_y / (1.7 \times f'_c \times B))] \]
\[ M_{capx} = 1585.553 \text{ kip_ft} \]

PASS - Moment capacity of base exceeds nominal moment strength required

Moment design in y direction
Reinforcement provided; 15 No. 8 bars bottom
Depth of tension reinforcement; \[ d_y = h - c_{nom} - \psi_{B} - \psi_{B} / 2 = 27.000 \text{ in} \]
Area of tension reinforcement provided; \[ A_{x,yB;\text{prov}} = N_{yB} \times \pi \times \psi_{B}^2 / 4 = 11.781 \text{ in}^2 \]
Area of compression reinforcement provided; \[ A_{x,yT;\text{prov}} = N_{yT} \times \pi \times \psi_{T}^2 / 4 = 0.000 \text{ in}^2 \]
Minimum area of reinforcement; \[ A_{x,y;\text{min}} = 0.0018 \times h \times L = 5.184 \text{ in}^2 \]
Spacing of reinforcement; \[ s_{yB;\text{prov}} = (L - 2 \times c_{nom}) / \max(N_{yB} - 1, 1) = 6.643 \text{ in} \]
Maximum spacing of reinforcement; \[ s_{max} = \min(3 \times h, 18\text{in}) = 18.000 \text{ in} \]

PASS - Reinforcement provided exceeds minimum reinforcement required
Depth of compression block \[ a_y = A_{x,yB;\text{prov}} \times f_y / (0.85 \times f'_c \times L) = 2.17 \text{ in} \]
\[ \beta_1 = 0.85 \]
\[ c_{na,y} = a_y / \beta_1 = 2.55 \text{ in} \]
\[ \epsilon_{ly} = 0.003 \times (d_y - c_{na,y}) / c_{na,y} = 0.02879 \]

**PASS - The section has adequate ductility (cl. 10.3.5)**

**Nominal moment strength required;**
\[ M_{ny} = \text{abs}(M_y) / \phi_t = 1466.667 \text{ kip ft} \]

**Moment capacity of base;**
\[ M_{cap} = A_s \gamma_B \times f_y \times [d_y - (A_s \gamma_B \times f_y / (1.7 \times f_c \times L))] \]
\[ M_{cap} = 1526.649 \text{ kip ft} \]

**PASS - Moment capacity of base exceeds nominal moment strength required**

**Calculate ultimate shear force at d from top face of column**
\[ q_{su} = (q_{1u} - C_y \times (B / 2 + e_{pyA} + b_y / 2 + d_y) / L + q_{su}) / 2 \]
\[ q_{su} = 21.643 \text{ ksf} \]
\[ A_s = L \times (B / 2 - e_{pyA} - b_y / 2 - d_y) = 8.000 \text{ ft}^2 \]

**Ultimate shear force at d from face of column;**
\[ V_{su} = A_s \times (q_{su} - F_u / A) = 165.000 \text{ kips} \]

**Shear design at d from top face of column**
\[ \phi_s = 0.75 \]

**Nominal shear strength;**
\[ V_{nsu} = V_{su} / \phi_s = 220.000 \text{ kips} \]

**Concrete shear strength;**
\[ V_{c,s} = 2 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi}) \times (L \times d_y)} = 327.865 \text{ kips} \]

**PASS - Nominal shear strength is less than concrete shear strength**

**Calculate ultimate punching shear force at perimeter of d / 2 from face of column**
\[ q_{puA} = q_{1u} \times [(L/2 + e_{pYA} - lA/2 - d/2)/2 + (lA + 2 \times d/2)/2] \times C_y / B - \]
\[ q_{puA} = 21.643 \text{ ksf} \]
\[ \bar{d} = (d_y + d_i) / 2 = 27.500 \text{ in} \]
\[ A_{pa} = (lA + 2 \times d/2) \times (b_y + 2 \times d/2) = 14.377 \text{ ft}^2 \]
\[ \text{Length of punching shear perimeter;} \]
\[ u_{pa} = 2 \times (lA + 2 \times d/2) + 2 \times (b_y + 2 \times d/2) = 15.167 \text{ ft} \]
\[ \text{Ultimate shear force at shear perimeter;} \]
\[ V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pa} = 1023.480 \text{ kips} \]

**Punching shear stresses at perimeter of d / 2 from face of column**
\[ V_{puAu} = V_{puA} / \phi_u = 1364.640 \text{ kips} \]

**Nominal shear strength;**
\[ V_{c,puA} = V_{puAu} \times \phi_u = 1364.640 \text{ kips} \]

**Ratio of column long side to short side;**
\[ \beta_A = \max(l_{ai}, b_y) / \min(l_{ai}, b_y) = 1.000 \]

**Column constant for interior column;**
\[ \alpha_{su} = 40 \]
\[ V_{c,p,i} = (2 + 4 / \beta_A) \times \lambda \times \sqrt{(f_c \times 1 \text{ psi}) \times u_{pa} \times d} = 1899.264 \]
\[ V_{c,p,ii} = (4 / \alpha_{su} \times \bar{d} / u_{pa} + 2) \times \lambda \times \sqrt{(f_c \times 1 \text{ psi}) \times u_{pa} \times d} = 2546.266 \text{ kips} \]
\[ V_{c,p,iii} = 2546.266 \text{ kips} \]

**Punching shear reinforcement**
\[ \phi_{sv} = 0.500 \text{ in} \]
\[ S_{pv,perp} = 12.000 \text{ in} \]
\[ S_{pv,par} = 8.000 \text{ in} \]
Area of shear links required;
Area of shear links provided;

\[ A_{pv,\text{req}} = \max(0.75 \times \sqrt{f'_c \times 1 \text{ psi}}, 50 \text{ psi}) / f_y = 0.120 \text{ in}^2/\text{ft}^2 \]

\[ A_{pv,\text{prov}} = \pi \times \phi_{pv}^2 / (4 \times s_{pv,\text{perp}} \times s_{pv,\text{par}}) = 0.295 \text{ in}^2/\text{ft}^2 \]

**PASS - Area of shear links provided exceeds area of shear links required**

15 No. 5 bars btm (1/4 c)

15 No. 6 bars btm (1/2 c)

:: One way shear at 3 btm column face

:: Two way shear at 3/2 from column face
Combined footing details
Length of combined footing; \( L = 9.000 \) ft
Width of combined footing; \( B = 9.000 \) ft
Area of combined footing; \( A = L \times B = 81.000 \) ft²
Depth of combined footing; \( h = 30.000 \) in
Depth of soil over combined footing; \( h_{soil} = 18.000 \) in
Density of concrete; \( \rho_{conc} = 150.0 \) lb/ft³

Column details
Column base length; \( l_A = 18.000 \) in
Column base width; \( b_A = 18.000 \) in
Column eccentricity in x; \( e_{PA} = 0.000 \) in
Column eccentricity in y; \( e_{PYA} = 0.000 \) in

Soil details
Density of soil; \( \rho_{soil} = 120.0 \) lb/ft³
Angle of internal friction; \( \phi' = 25.0 \) deg
Design base friction angle; \( \delta = 19.3 \) deg
Coefficient of base friction; \( \tan(\delta) = 0.350 \)
Allowable bearing pressure; \( P_{bearing} = 24.000 \) ksf

Axial loading on column
Dead axial load on column; \( P_{GA} = 1000.000 \) kips
Live axial load on column; \( P_{QA} = 200.000 \) kips
Wind axial load on column; \( P_{WA} = 0.000 \) kips
Total axial load on column; \( P_A = 1200.000 \) kips

**Foundation loads**
Dead surcharge load; \( F_{Gsur} = 0.100 \) ksf
Live surcharge load; \( F_{Qsur} = 0.100 \) ksf
Footing self weight; \( F_{swt} = h \times \rho_{conc} = 0.375 \) ksf
Soil self weight; \( F_{soil} = h_{soil} \times \rho_{soil} = 0.180 \) ksf
Total foundation load; \( F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = 61.155 \) kips

**Calculate base reaction**
Total base reaction; \( T = F + P_A = 1261.155 \) kips
Eccentricity of base reaction in x; \( e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = 0.000 \) in
Eccentricity of base reaction in y; \( e_{Ty} = (P_A \times e_{Pya} + M_{yA} + H_{yA} \times h) / T = 0.000 \) in

**Check base reaction eccentricity**
\[ \text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = 0.000 \]

*Base reaction acts within middle third of base*

**Calculate base pressures**
\( q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 15.570 \) ksf
\( q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 15.570 \) ksf
\( q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 15.570 \) ksf
\( q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 15.570 \) ksf

Minimum base pressure; \( q_{min} = \min(q_1, q_2, q_3, q_4) = 15.570 \) ksf
Maximum base pressure; \( q_{max} = \max(q_1, q_2, q_3, q_4) = 15.570 \) ksf

**PASS - Maximum base pressure is less than allowable bearing pressure**
Load combination factors for loads
Load combination factor for dead loads; \( \gamma_G = 1.20 \)
Load combination factor for live loads; \( \gamma_Q = 1.60 \)
Load combination factor for wind loads; \( \gamma_W = 0.00 \)

Strength reduction factors
Flexural strength reduction factor; \( \phi_f = 0.90 \)
Shear strength reduction factor; \( \phi_s = 0.75 \)

Ultimate axial loading on column
Ultimate axial load on column;
\[ P_{uA} = P_{G} \times \gamma_G + P_{Q} \times \gamma_Q + P_{W} \times \gamma_W = 1520.000 \text{ kips} \]

Ultimate foundation loads
Ultimate foundation load;
\[ F_u = A \times [(F_{G\text{surf}} + F_{\text{swt}} + F_{\text{soil}}) \times \gamma_G + F_{G\text{surf}} \times \gamma_Q] = 76.626 \text{ kips} \]

Ultimate horizontal loading on column
Ultimate horizontal load in x direction;
\[ H_{xuA} = H_{Gx} \times \gamma_G + H_{Qx} \times \gamma_Q + H_{Wx} \times \gamma_W = 0.000 \text{ kips} \]
Ultimate horizontal load in y direction;
\[ H_{yuA} = H_{Gy} \times \gamma_G + H_{Qy} \times \gamma_Q + H_{Wy} \times \gamma_W = 0.000 \text{ kips} \]

Ultimate moment on column
Ultimate moment on column in x direction;
\[ M_{xuA} = M_{Gx} \times \gamma_G + M_{Qx} \times \gamma_Q + M_{Wx} \times \gamma_W = 0.000 \text{ kip ft} \]
Ultimate moment on column in y direction;
\[ M_{yuA} = M_{Gy} \times \gamma_G + M_{Qy} \times \gamma_Q + M_{Wy} \times \gamma_W = 0.000 \text{ kip ft} \]

Calculate ultimate base reaction
Ultimate base reaction;
\[ T_u = F_u + P_{uA} = 1596.626 \text{ kips} \]
Eccentricity of ultimate base reaction in x;
\[ \epsilon_{Txu} = (P_{uA} \times \epsilon_{PyA} + M_{xuA} + H_{xuA} \times h) / T_u = 0.000 \text{ in} \]
Eccentricity of ultimate base reaction in y;
\[ \epsilon_{Tyu} = (P_{uA} \times \epsilon_{PyA} + M_{yuA} + H_{yuA} \times h) / T_u = 0.000 \text{ in} \]

Calculate ultimate base pressures
\[ q_{1u} = T_u / A - 6 \times T_u \times \epsilon_{Txu} / (L \times A) - 6 \times T_u \times \epsilon_{Tyu} / (B \times A) = 19.711 \text{ ksf} \]
\[ q_{2u} = T_u / A - 6 \times T_u \times \epsilon_{Txu} / (L \times A) + 6 \times T_u \times \epsilon_{Tyu} / (B \times A) = 19.711 \text{ ksf} \]
\[ q_{3u} = T_u / A + 6 \times T_u \times \epsilon_{Txu} / (L \times A) - 6 \times T_u \times \epsilon_{Tyu} / (B \times A) = 19.711 \text{ ksf} \]
Minimum ultimate base pressure;  
Maximum ultimate base pressure;  

\[ q_u = \frac{T_u}{A} + 6 \times \frac{T_{ux}}{e_T} / (L \times A) + 6 \times \frac{T_{uy}}{e_T} / (B \times A) = 19.711 \text{ ksf} \]

\[ q_{min} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 19.711 \text{ ksf} \]

\[ q_{max} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 19.711 \text{ ksf} \]

**Calculate rate of change of base pressure in x direction**

Left hand base reaction;  
Right hand base reaction;  
Length of base reaction;  
Rate of change of base pressure;  

\[ f_{ul} = (q_{1u} + q_{2u}) \times B / 2 = 177.403 \text{ kips/ft} \]

\[ f_{ur} = (q_{3u} + q_{4u}) \times B / 2 = 177.403 \text{ kips/ft} \]

\[ L_x = L = 108.000 \text{ in} \]

\[ C_x = \frac{(f_{ur} - f_{ul})}{L_x} = 0.000 \text{ kips/ft/ft} \]

**Calculate footing lengths in x direction**

Left hand length;  
Right hand length;  

\[ L_L = L / 2 + 6 \times e_{pA} = 4.500 \text{ ft} \]

\[ L_R = L / 2 - 6 \times e_{pA} = 4.500 \text{ ft} \]

**Calculate ultimate moments in x direction**

Ultimate moment in x direction;  

\[ M_x = f_{ul} \times L_x^2 / 2 + C_x \times L_x \times (L_x / 2) - (F_u \times L_x^2 / (2 \times L_x)) = 1710.000 \text{ kip ft} \]

**Calculate rate of change of base pressure in y direction**

Top edge base reaction;  
Bottom edge base reaction;  
Length of base reaction;  
Rate of change of base pressure;  

\[ f_{ut} = (q_{2u} + q_{3u}) \times L / 2 = 177.403 \text{ kips/ft} \]

\[ f_{ub} = (q_{1u} + q_{4u}) \times L / 2 = 177.403 \text{ kips/ft} \]

\[ L_y = B = 9.000 \text{ ft} \]

\[ C_y = \frac{(f_{ub} - f_{ut})}{L_y} = 0.000 \text{ kips/ft/ft} \]

**Calculate footing lengths in y direction**

Top length;  
Bottom length;  

\[ L_T = B / 2 + 6 \times e_{pA} = 4.500 \text{ ft} \]

\[ L_B = B / 2 - 6 \times e_{pA} = 4.500 \text{ ft} \]

**Calculate ultimate moments in y direction**

Ultimate moment in y direction;  

\[ M_y = f_{ut} \times L_T^2 / 2 + C_y \times L_T \times (L_T / 2) - (F_u \times L_T^2 / (2 \times L_T)) = 1710.000 \text{ kip ft} \]

**Material details**

Compressive strength of concrete;  
Yield strength of reinforcement;  
Cover to reinforcement;  
Concrete type;  
Concrete modification factor;  

\[ f_c = 4000 \text{ psi} \]

\[ f_y = 60000 \text{ psi} \]

\[ c_{nom} = 1.500 \text{ in} \]

\[ \lambda = 1.00 \]

**Moment design in x direction**

Reinforcement provided;  
Depth of tension reinforcement;  
Area of tension reinforcement provided;  
Area of compression reinforcement provided;  
Minimum area of reinforcement;  
Spacing of reinforcement;  
Maximum spacing of reinforcement;  

\[ d_x = h - c_{nom} - \phi \beta / 2 = 28.000 \text{ in} \]

\[ A_{x,x_b,prov} = N_{x_b} \times \pi \times \phi \beta^2 / 4 = 14.923 \text{ in}^2 \]

\[ A_{x,x_T,prov} = N_{x_T} \times \pi \times \phi \beta^2 / 4 = 0.000 \text{ in}^2 \]

\[ A_{x,x,min} = 0.0018 \times h \times B = 5.832 \text{ in}^2 \]

\[ s_{x_b,prov} = (B - 2 \times c_{nom}) / \max(N_{x_b} - 1, 1) = 5.833 \text{ in} \]

\[ s_{max} = \min(3 \times h, 18 \text{in}) = 18.000 \text{ in} \]

**PASS - Reinforcement provided exceeds minimum reinforcement required**

Depth of compression block  
\[ a_x = A_{x,x_b,prov} \times f_y / (0.85 \times f_c \times B) = 2.44 \text{ in} \]

Neutral axis factor  
\[ \beta_1 = 0.85 \]
Depth to the neutral axis

Strain in reinforcement

Nominal moment strength required;

Moment capacity of base;

PASS - The section has adequate ductility (cl. 10.3.5)

Moment design in y direction

Reinforcement provided;

Depth of tension reinforcement;

Area of tension reinforcement provided;

Area of compression reinforcement provided;

Minimum area of reinforcement;

Spacing of reinforcement;

Maximum spacing of reinforcement;

PASS - Moment capacity of base exceeds nominal moment strength required

Depth of compression block

Neutral axis factor

Depth to the neutral axis

Strain in reinforcement

Nominal moment strength required;

Moment capacity of base;

PASS - The section has adequate ductility (cl. 10.3.5)

Calculate ultimate shear force at d from top face of column

Ultimate pressure for shear d from face of column;

Area loaded for shear d from face of column;

Ultimate shear force at d from face of column;

Shear design at d from top face of column

Strength reduction factor in shear;

Nominal shear strength;

Concrete shear strength;

PASS - Nominal shear strength is less than concrete shear strength

Calculate ultimate punching shear force at perimeter of d / 2 from face of column

Ultimate pressure for punching shear;

Average effective depth of reinforcement;

Area loaded for punching shear at column;
Length of punching shear perimeter; \[ u_{PA} = 2 \times (l_A + 2 \times d/2) + 2 \times (b_A + 2 \times d/2) = 15.167 \text{ ft} \]

Ultimate shear force at shear perimeter; \[ V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = 1250.214 \text{ kips} \]

**Punching shear stresses at perimeter of \( d / 2 \) from face of column**

Nominal shear strength; \[ V_{puA} = V_{pu} / \phi_s = 1666.952 \text{ kips} \]

Ratio of column long side to short side; \[ \beta_A = \text{max}(l_A, b_A) / \text{min}(l_A, b_A) = 1.000 \]

Column constant for interior column; \[ \alpha_{sA} = 40 \]

Concrete shear strength; \[ V_{c,p,i} = (2 + 4 / \beta_A) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times u_{PA} \times d = 1899.264 \text{ kips} \]

2546.266 kips

**Punching shear reinforcement**

Shear reinforcement bar designation; \[ 4 \]

Diameter of shear reinforcement; \[ \phi_{pv} = 0.500 \text{ in} \]

Spacing perpendicular to column face; \[ s_{pv,perp} = 12.000 \text{ in} \]

Spacing parallel to column face; \[ s_{pv,par} = 8.000 \text{ in} \]

Area of shear links required; \[ A_{pv,req} = (V_{puA} - V_{c,p}) / (f_y \times u_{PA} \times d) = 0.192 \text{ in}^2/\text{ft}^2 \]

Area of shear links provided; \[ A_{pv,prov} = \pi \times \phi_{pv}^2 / (4 \times s_{pv,perp} \times s_{pv,par}) = 0.295 \text{ in}^2/\text{ft}^2 \]

**PASS** - Area of shear links provided exceeds area of shear links required

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One way shear at 3 / 2 from column face

Two way shear at 3 / 2 from column face
Combined footing details

Length of combined footing; \( L = 10.000 \) ft
Width of combined footing; \( B = 10.000 \) ft
Area of combined footing; \( A = L \times B = 100.000 \) ft\(^2\)
Depth of combined footing; \( h = 36.000 \) in
Depth of soil over combined footing; \( h_{\text{soil}} = 18.000 \) in
Density of concrete; \( \rho_{\text{conc}} = 150.0 \) lb/ft\(^3\)

Column details

Column base length; \( l_A = 24.000 \) in
Column base width; \( b_A = 24.000 \) in
Column eccentricity in \( x \); \( e_{PA} = 0.000 \) in
Column eccentricity in \( y \); \( e_{PYA} = 0.000 \) in

Soil details

Density of soil; \( \rho_{\text{soil}} = 120.0 \) lb/ft\(^3\)
Angle of internal friction; \( \phi' = 25.0 \) deg
Design base friction angle; \( \delta = 19.3 \) deg
Coefficient of base friction; \( \tan(\delta) = 0.350 \)
Allowable bearing pressure; \( P_{\text{bearing}} = 24.000 \) ksf

Axial loading on column

Dead axial load on column; \( P_{GA} = 1850.000 \) kips
Live axial load on column; \( P_{QA} = 350.000 \) kips
Wind axial load on column; 
Total axial load on column; 

**Foundation loads**
Dead surcharge load; 
Live surcharge load; 
Footing self weight; 
Soil self weight; 
Total foundation load; 

**Calculate base reaction**
Total base reaction; 
Eccentricity of base reaction in x; 
Eccentricity of base reaction in y; 

**Check base reaction eccentricity**

**Calculate base pressures**

<table>
<thead>
<tr>
<th>q1</th>
<th>22.830 ksf</th>
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<tbody>
<tr>
<td>q2</td>
<td>22.830 ksf</td>
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<tr>
<td>q3</td>
<td>22.830 ksf</td>
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<td>q_{min}</td>
<td>22.830 ksf</td>
</tr>
<tr>
<td>q_{max}</td>
<td>22.830 ksf</td>
</tr>
</tbody>
</table>

PASS - Maximum base pressure is less than allowable bearing pressure
Load combination factors for loads

Load combination factor for dead loads; \( \gamma_G = 1.20 \)
Load combination factor for live loads; \( \gamma_Q = 1.60 \)
Load combination factor for wind loads; \( \gamma_W = 0.00 \)

Strength reduction factors

Flexural strength reduction factor; \( \phi_f = 0.90 \)
Shear strength reduction factor; \( \phi_s = 0.75 \)

Ultimate axial loading on column

Ultimate axial load on column;
\[ P_{UA} = P_{GA} \times \gamma_G + P_{QA} \times \gamma_Q + P_{WA} \times \gamma_W = 2780.000 \text{ kips} \]

Ultimate foundation loads

Ultimate foundation load;
\[ F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_G + F_{Qsur} \times \gamma_Q] = 103.600 \text{ kips} \]

Ultimate horizontal loading on column

Ultimate horizontal load in x direction;
\[ H_{xuA} = H_{GxA} \times \gamma_G + H_{QxA} \times \gamma_Q + H_{WxA} \times \gamma_W = 0.000 \text{ kips} \]
Ultimate horizontal load in y direction;
\[ H_{yuA} = H_{GyA} \times \gamma_G + H_{QyA} \times \gamma_Q + H_{WyA} \times \gamma_W = 0.000 \text{ kips} \]

Ultimate moment on column

Ultimate moment on column in x direction;
\[ M_{xuA} = M_{GxA} \times \gamma_G + M_{QxA} \times \gamma_Q + M_{WxA} \times \gamma_W = 0.000 \text{ kip}\_ft \]
Ultimate moment on column in y direction;
\[ M_{yuA} = M_{GyA} \times \gamma_G + M_{QyA} \times \gamma_Q + M_{WyA} \times \gamma_W = 0.000 \text{ kip}\_ft \]

Calculate ultimate base reaction

Ultimate base reaction;
\[ T_u = F_u + P_{UA} = 2883.600 \text{ kips} \]

Eccentricity of ultimate base reaction in x;
\[ \varepsilon_{Txu} = (P_{UA} \times \varepsilon_{PyA} + M_{ruA} + H_{ruA} \times h) / T_u = 0.000 \text{ in} \]

Eccentricity of ultimate base reaction in y;
\[ \varepsilon_{Tyu} = (P_{UA} \times \varepsilon_{PyA} + M_{ryA} + H_{ryA} \times h) / T_u = 0.000 \text{ in} \]

Calculate ultimate base pressures

\( q_{1u} = T_u / A - 6 \times T_u \times \varepsilon_{Txu} / (L \times A) - 6 \times T_u \times \varepsilon_{Tyu} / (B \times A) = 28.836 \text{ ksf} \)
\( q_{2u} = T_u / A - 6 \times T_u \times \varepsilon_{Txu} / (L \times A) + 6 \times T_u \times \varepsilon_{Tyu} / (B \times A) = 28.836 \text{ ksf} \)
Minimum ultimate base pressure; 
\[ q_{\text{min}} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 28.836 \text{ ksf} \]

Maximum ultimate base pressure; 
\[ q_{\text{max}} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 28.836 \text{ ksf} \]

Calculate rate of change of base pressure in x direction
Left hand base reaction; 
\[ f_{\text{LU}} = (q_{1u} + q_{2u}) \times B / 2 = 288.360 \text{ kips/ft} \]
Right hand base reaction; 
\[ f_{\text{RU}} = (q_{3u} + q_{4u}) \times B / 2 = 288.360 \text{ kips/ft} \]
Length of base reaction; 
\[ L_x = L = 120.000 \text{ in} \]
Rate of change of base pressure; 
\[ C_x = (f_{\text{RU}} - f_{\text{LU}}) / L_x = 0.000 \text{ kips/ft} \]

Calculate footing lengths in x direction
Left hand length; 
\[ L_L = L / 2 + e_{\text{PA}} = 5.000 \text{ ft} \]
Right hand length; 
\[ L_R = L / 2 - e_{\text{PA}} = 5.000 \text{ ft} \]

Calculate ultimate moments in x direction
Ultimate moment in x direction; 
\[ M_x = f_{\text{LU}} \times L_L^2 / 2 + C_x \times L_L^3 / 6 - F_u \times L_L^2 / (2 \times L) = 3475.000 \text{ kip-ft} \]

Calculate rate of change of base pressure in y direction
Top edge base reaction; 
\[ f_{\text{TU}} = (q_{2u} + q_{4u}) \times L / 2 = 288.360 \text{ kips/ft} \]
Bottom edge base reaction; 
\[ f_{\text{BU}} = (q_{1u} + q_{3u}) \times L / 2 = 288.360 \text{ kips/ft} \]
Length of base reaction; 
\[ L_y = B = 10.000 \text{ ft} \]
Rate of change of base pressure; 
\[ C_y = (f_{\text{BU}} - f_{\text{TU}}) / L_y = 0.000 \text{ kips/ft} \]

Calculate footing lengths in y direction
Top length; 
\[ L_T = B / 2 + e_{\text{PYA}} = 5.000 \text{ ft} \]
Bottom length; 
\[ L_B = B / 2 - e_{\text{PYA}} = 5.000 \text{ ft} \]

Calculate ultimate moments in y direction
Ultimate moment in y direction; 
\[ M_y = f_{\text{TU}} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) = 3475.000 \text{ kip-ft} \]

Material details
Compressive strength of concrete; 
\[ f'_c = 4000 \text{ psi} \]
Yield strength of reinforcement; 
\[ f_y = 60000 \text{ psi} \]
Cover to reinforcement; 
\[ c_{\text{nom}} = 1.500 \text{ in} \]
Concrete type; 
Normal weight
Concrete modification factor; 
\[ \lambda = 1.00 \]

Moment design in x direction
Reinforcement provided; 
25 No. 9 bars bottom
Depth of tension reinforcement; 
\[ d_x = h - c_{\text{nom}} - \phi_B / 2 = 33.936 \text{ in} \]
Area of tension reinforcement provided; 
\[ A_{x,T,\text{prov}} = N_{x,T} \times \pi \times \phi_B^2 / 4 = 24.983 \text{ in}^2 \]
Area of compression reinforcement provided; 
\[ A_{x,C,\text{prov}} = N_{x,C} \times \pi \times \phi_C^2 / 4 = 0.000 \text{ in}^2 \]
Minimum area of reinforcement; 
\[ A_{x,x,min} = 0.0018 \times h \times B = 7.776 \text{ in}^2 \]
Spacing of reinforcement; 
\[ s_{x,T,\text{prov}} = (B - 2 \times c_{\text{nom}}) / \max(N_{x,T} - 1, 1) = 4.875 \text{ in} \]
Maximum spacing of reinforcement; 
\[ s_{x,T,max} = \min(3 \times h, 18\text{in}) = 18.000 \text{ in} \]

**PASS - Reinforcement provided exceeds minimum reinforcement required**

Depth of compression block 
\[ a_x = A_{x,x,T,\text{prov}} \times f_y / (0.85 \times f'_c \times B) = 3.67 \text{ in} \]
Neutral axis factor 
\[ \beta_1 = 0.85 \]
Depth to the neutral axis

Strain in reinforcement

Nominal moment strength required;

Moment capacity of base;

PASS - The section has adequate ductility (cl. 10.3.5)

Moment design in y direction

Reinforcement provided;

Depth of tension reinforcement;

Area of tension reinforcement provided;

Area of compression reinforcement provided;

Minimum area of reinforcement;

Spacing of reinforcement;

Maximum spacing of reinforcement;

PASS - Moment capacity of base exceeds nominal moment strength required

Depth of compression block

Neutral axis factor

Depth to the neutral axis

Strain in reinforcement

Nominal moment strength required;

Moment capacity of base;

PASS - The section has adequate ductility (cl. 10.3.5)

Calculate ultimate shear force at d from top face of column

Ultimate pressure for shear d from face of column;

Area loaded for shear at d from face of column;

Shear design at d from top face of column

Strength reduction factor in shear;

Nominal shear strength;

Concrete shear strength;

PASS - Nominal shear strength is less than concrete shear strength

Calculate ultimate punching shear force at perimeter of d / 2 from face of column

Ultimate pressure for punching shear;

Average effective depth of reinforcement;

Area loaded for punching shear at column;
Length of punching shear perimeter; \( u_{PA} = 2 \times (l_A + 2 \times d/2) + 2 \times (b_A + 2 \times d/2) = 19.124 \) ft

Ultimate shear force at shear perimeter; \( V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = 2144.549 \) kips

**Punching shear stresses at perimeter of \( d/2 \) from face of column**

Nominal shear strength; \( V_{npuA} = V_{puA} / \phi_s = 2859.398 \) kips

Ratio of column long side to short side; \( \beta_A = \max(l_A, b_A) / \min(l_A, b_A) = 1.000 \)

Column constant for interior column; \( \alpha_{sA} = 40 \)

Concrete shear strength; \( V_{c,p,\text{i}} = (2 + 4 / \beta_A) \times \lambda \times \sqrt{f'c \times 1 \text{ psi}} \times u_{pA} \times d = 2906.186 \) kips

\( V_{c,p,\text{ii}} = (\alpha_{sA} \times d / u_{pA} + 2) \times \lambda \times \sqrt{f'c \times 1 \text{ psi}} \times u_{pA} \times d = 1937.458 \) kips

\( V_{c,p,\text{iii}} = 4 \times \lambda \times \sqrt{f'c \times 1 \text{ psi}} \times u_{pA} \times d = 1937.458 \) kips

\( V_{c,p} = \min(V_{c,p,\text{i}}, V_{c,p,\text{ii}}, V_{c,p,\text{iii}}) = 1937.458 \) kips

\( V_{npuA} > V_{c,p} - Shear \text{ reinforcement is required} \)

**Punching shear reinforcement**

Shear reinforcement bar designation; \( \phi_{pv} = 0.500 \) in

Diameter of shear reinforcement; \( \phi_{pv} = 0.500 \) in

Spacing perpendicular to column face; \( s_{pv,\text{perp}} = 12.000 \) in

Spacing parallel to column face; \( s_{pv,\text{par}} = 8.000 \) in

Area of shear links required; \( A_{pv,\text{req}} = (V_{npuA} - V_{c,p}) / (f_y \times u_{pA} \times d) = 0.289 \) in²/ft²

Area of shear links provided; \( A_{pv,\text{prov}} = \pi \times \phi_{pv}^2 / (4 \times s_{pv,\text{perp}} \times s_{pv,\text{par}}) = 0.295 \) in²/ft²

**PASS - Area of shear links provided exceeds area of shear links required**
Combined footing details
Length of combined footing; \( L = 17.000 \) ft
Width of combined footing; \( B = 8.000 \) ft
Area of combined footing; \( A = L \times B = 136.000 \) ft\(^2\)
Depth of combined footing; \( h = 30.000 \) in
Depth of soil over combined footing; \( h_{\text{soil}} = 18.000 \) in
Density of concrete; \( \rho_{\text{conc}} = 150.0 \) lb/ft\(^3\)

Column details
Column A
Column base length; \( l_A = 12.000 \) in
Column base width; \( b_A = 30.000 \) in
Column eccentricity in \( x \); \( e_{p_A} = 49.000 \) in
Column eccentricity in \( y \); \( e_{p_A} = 0.000 \) in

Column B
Column base length; \( l_B = 12.000 \) in
Column base width; \( b_B = 30.000 \) in
Column eccentricity in \( x \); \( e_{p_B} = -49.000 \) in
Column eccentricity in \( y \); \( e_{p_B} = 0.000 \) in

Soil details
Density of soil; \( \rho_{\text{soil}} = 120.0 \) lb/ft\(^3\)
Angle of internal friction; \( \phi' = 25.0 \) deg
Design base friction angle; \( \delta = 19.3 \) deg
Coefficient of base friction; \( \tan(\delta) = 0.350 \)
Allowable bearing pressure; \( P_{\text{bearing}} = 24.000 \) ksf

Axial loading on columns
Column A
Dead axial load on column; \( P_{GA} = 800.000 \) kips
Live axial load on column; \( P_{QA} = 180.000 \) kips
Wind axial load on column; \( P_{WA} = 0.000 \) kips
Total axial load on column; \( P_A = 980.000 \) kips

Column B
Dead axial load on column; \( P_{GB} = 800.000 \) kips
Live axial load on column; \( P_{QB} = 180.000 \) kips
Wind axial load on column; \( P_{WB} = 0.000 \) kips
Total axial load on column; \( P_B = 980.000 \) kips

Foundation loads
Dead surcharge load; \( F_{\text{Gs}} = 0.100 \) ksf
Live surcharge load; \( F_{\text{Os}} = 0.100 \) ksf
Footing self weight; \( F_{\text{sw}} = h \times \rho_{\text{conc}} = 0.375 \) ksf
Soil self weight; \( F_{\text{soil}} = h_{\text{soil}} \times \rho_{\text{soil}} = 0.180 \) ksf
Total foundation load; \( F = A \times (F_{\text{Gs}} + F_{\text{Os}} + F_{\text{sw}} + F_{\text{soil}}) = 102.680 \) kips
Calculate base reaction
Total base reaction;
\[ T = F + P_A + P_B = 2062.680 \text{ kips} \]
Eccentricity of base reaction in \( x \);
\[ e_{Tx} = (P_A \times e_{PxA} + P_B \times e_{PxB} + M_{xA} + M_{xB} + (H_{xA} + H_{xB}) \times h) / T \]
Eccentricity of base reaction in \( y \);
\[ e_{Ty} = (P_A \times e_{Pya} + P_B \times e_{Pyb} + M_{yA} + M_{yB} + (H_{yA} + H_{yB}) \times h) / T \]

Check base reaction eccentricity
\[ \text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = 0.000 \]

Base reaction acts within middle third of base

Calculate base pressures
\[ q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 15.167 \text{ ksf} \]
\[ q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 15.167 \text{ ksf} \]
\[ q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 15.167 \text{ ksf} \]
\[ q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 15.167 \text{ ksf} \]

Minimum base pressure;
\[ q_{\text{min}} = \min(q_1, q_2, q_3, q_4) = 15.167 \text{ ksf} \]
Maximum base pressure;
\[ q_{\text{max}} = \max(q_1, q_2, q_3, q_4) = 15.167 \text{ ksf} \]

Load combination factors for loads
Load combination factor for dead loads;
\[ \gamma_G = 1.20 \]
Load combination factor for live loads;
\[ \gamma_Q = 1.60 \]
Load combination factor for wind loads;
\[ \gamma_W = 0.00 \]

Strength reduction factors
Flexural strength reduction factor;
\[ \phi_f = 0.90 \]
Shear strength reduction factor;
\[ \phi_s = 0.75 \]

Ultimate axial loading on columns
Ultimate axial load on column \( A \);
\[ P_{UA} = P_{GA} \times \gamma_G + P_{QA} \times \gamma_Q + P_{WA} \times \gamma_W = 1248.000 \text{ kips} \]
Ultimate axial load on column B;

**Ultimate foundation loads**
Ultimate foundation load; kips

**Ultimate horizontal loading on column A**
Ultimate horizontal load in x direction; kips
Ultimate horizontal load in y direction;

**Ultimate horizontal loading on column B**
Ultimate horizontal load in x direction; kips
Ultimate horizontal load in y direction;

**Ultimate moment on column A**
Ultimate moment on column in x direction; kips
Ultimate moment on column in y direction;

**Ultimate moment on column B**
Ultimate moment on column in x direction; kips
Ultimate moment on column in y direction;

**Calculate ultimate pad base reaction**
Ultimate base reaction;
Eccentricity of ultimate base reaction in x; 0.000 in
Eccentricity of ultimate base reaction in y; 0.000 in

**Calculate ultimate base pressures**

**Calculate rate of change of base pressure in x direction**
Left hand base reaction; kips/ft
Right hand base reaction; kips/ft
Length of base reaction; in
Rate of change of base pressure; kips/ft/in

**Calculate pad lengths in x direction**
Left hand length; in
Middle length; in
Right hand length; in

**Calculate shear forces in x direction**
Shear at left hand column; kips
Shear at right hand column; kips

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\[ P_{UB} = P_{GB} \times \gamma_{IG} + P_{GB} \times \gamma_{IQ} + P_{WB} \times \gamma_{W} = 1248.000 \text{ kips} \]

\[ F_{u} = A \times ([F_{Gsurf} + F_{swt} + F_{soil}] \times \gamma_{IG} + F_{Gsurf} \times \gamma_{Q}) = 128.656 \]

\[ H_{uA} = H_{OXA} \times \gamma_{IG} + H_{OXA} \times \gamma_{IQ} + H_{WXY} \times \gamma_{W} = 0.000 \text{ kips} \]

\[ H_{uA} = H_{OYA} \times \gamma_{IG} + H_{OYA} \times \gamma_{IQ} + H_{WYA} \times \gamma_{W} = 0.000 \text{ kips} \]

\[ H_{uB} = H_{OXB} \times \gamma_{IG} + H_{OXB} \times \gamma_{IQ} + H_{WXB} \times \gamma_{W} = 0.000 \text{ kips} \]

\[ H_{uB} = H_{OYB} \times \gamma_{IG} + H_{OYB} \times \gamma_{IQ} + H_{WYB} \times \gamma_{W} = 0.000 \text{ kips} \]

\[ M_{uA} = M_{OXA} \times \gamma_{IG} + M_{OXA} \times \gamma_{IQ} + M_{WXA} \times \gamma_{W} = 0.000 \text{ kip} \text{ ft} \]

\[ M_{uA} = M_{OYA} \times \gamma_{IG} + M_{OYA} \times \gamma_{IQ} + M_{WYA} \times \gamma_{W} = 0.000 \text{ kip} \text{ ft} \]

\[ M_{uB} = M_{OXB} \times \gamma_{IG} + M_{OXB} \times \gamma_{IQ} + M_{WXB} \times \gamma_{W} = 0.000 \text{ kip} \text{ ft} \]

\[ M_{uB} = M_{OYB} \times \gamma_{IG} + M_{OYB} \times \gamma_{IQ} + M_{WYB} \times \gamma_{W} = 0.000 \text{ kip} \text{ ft} \]

\[ T_{u} = F_{u} + P_{uA} + P_{uB} = 2624.656 \text{ kips} \]

\[ \epsilon_{T_{u}} = \epsilon_{TPA} \times (P_{uA} + P_{uB}) + \epsilon_{TPB} = 0.000 \text{ in} \]

\[ \epsilon_{T_{u}} = \epsilon_{TPA} \times (P_{uA} + P_{uB}) + \epsilon_{TPB} = 0.000 \text{ in} \]

\[ T_{u} = F_{u} + P_{uA} + P_{uB} = 2624.656 \text{ kips} \]

\[ \epsilon_{T_{u}} = \epsilon_{TPA} \times (P_{uA} + P_{uB}) + \epsilon_{TPB} = 0.000 \text{ in} \]

\[ T_{u} = F_{u} + P_{uA} + P_{uB} = 2624.656 \text{ kips} \]

\[ \epsilon_{T_{u}} = \epsilon_{TPA} \times (P_{uA} + P_{uB}) + \epsilon_{TPB} = 0.000 \text{ in} \]

\[ q_{1u} = T_{u}/A - 6 \times T_{u} \times \epsilon_{TPA} / (L \times A) - 6 \times T_{u} \times \epsilon_{TPA} / (B \times A) = 19.299 \text{ ksf} \]

\[ q_{2u} = T_{u}/A - 6 \times T_{u} \times \epsilon_{TPB} / (L \times A) + 6 \times T_{u} \times \epsilon_{TPB} / (B \times A) = 19.299 \text{ ksf} \]

\[ q_{3u} = T_{u}/A - 6 \times T_{u} \times \epsilon_{TPA} / (L \times A) - 6 \times T_{u} \times \epsilon_{TPB} / (B \times A) = 19.299 \text{ ksf} \]

\[ q_{4u} = T_{u}/A - 6 \times T_{u} \times \epsilon_{TPA} / (L \times A) + 6 \times T_{u} \times \epsilon_{TPB} / (B \times A) = 19.299 \text{ ksf} \]

\[ q_{\text{minu}} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 19.299 \text{ ksf} \]

\[ q_{\text{maxu}} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = 19.299 \text{ ksf} \]

\[ f_{uL} = (q_{1u} + q_{2u}) \times B / 2 = 154.392 \text{ kips/ft} \]

\[ f_{uR} = (q_{3u} + q_{4u}) \times B / 2 = 154.392 \text{ kips/ft} \]

\[ L_{x} = L = 204.000 \text{ in} \]

\[ C_{x} = (f_{uR} - f_{uL}) / L_{x} = 0.000 \text{ kips/ft} \]

\[ L_{L} = L / 2 + \min(\epsilon_{TPA}, \epsilon_{TPB}) = 53.000 \text{ in} \]

\[ L_{M} = \max(\epsilon_{TPA}, \epsilon_{TPB}) = 98.000 \text{ in} \]

\[ L_{R} = L / 2 - \max(\epsilon_{TPA}, \epsilon_{TPB}) = 53.000 \text{ in} \]

\[ S_{L} = f_{uL} \times L_{u} + C_{x} \times L_{u}^{2} / 2 - F_{u} \times L_{u} / L = 648.471 \text{ kips} \]

\[ S_{R} = f_{uR} \times (L_{L} + L_{M}) + C_{x} \times (L_{L} + L_{M})^{2} / 2 - P_{uB} - F_{u} \times (L_{L} + L_{M}) \]
Calculate ultimate moments in x direction

Ultimate moment in x direction;
\[ M_x = f_{ub} \times L_1^2 / 2 + C_v \times L_1^3 / 6 - F_u \times L_1^2 / (2 \times L) = 1432.039 \text{ kip}_\text{ft} \]

Calculate rate of change of base pressure in y direction

Top edge base reaction;
\[ f_{ut} = (q_{tu} + q_{w}) \times L / 2 = 328.082 \text{ kips/ft} \]
Bottom edge base reaction;
\[ f_{ub} = (q_{tu} + q_{w}) \times L / 2 = 328.082 \text{ kips/ft} \]
Length of base reaction;
\[ L_y = B = 8.000 \text{ ft} \]
Rate of change of base pressure;
\[ C_y = (f_{ub} - f_{ut}) / L_y = 0.000 \text{ kips/ft} \]

Calculate footing lengths in y direction

Top length;
\[ L_T = B / 2 + \epsilon_{PyA} = 4.000 \text{ ft} \]
Bottom length;
\[ L_B = B / 2 - \epsilon_{PyA} = 4.000 \text{ ft} \]

Calculate ultimate moments in y direction

Ultimate moment in y direction;
\[ M_y = f_{ut} \times L_T^2 / 2 + C_v \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) = 2496.000 \text{ kip}_\text{ft} \]

Material details

Compressive strength of concrete; \( f'_c = 4000 \text{ psi} \)
Yield strength of reinforcement; \( f_y = 60000 \text{ psi} \)
Cover to reinforcement; \( c_{nom} = 1.500 \text{ in} \)
Concrete type; Normal weight
Concrete modification factor; \( \lambda = 1.00 \)

Moment design in x direction

Reinforcement provided; 12 No. 9 bars bottom
Depth of tension reinforcement;
\[ d_x = h - c_{nom} \times \phi_{yb} / 2 = 27.936 \text{ in} \]
Area of tension reinforcement provided;
\[ A_{s,xb,prov} = N_{ub} \times \pi \times \phi_{yb}^2 / 4 = 11.992 \text{ in}^2 \]
Area of compression reinforcement provided;
\[ A_{s,xt,prov} = N_{xt} \times \pi \times \phi_{xt}^2 / 4 = 0.000 \text{ in}^2 \]
Minimum area of reinforcement;
\[ A_{s,x,min} = 0.0018 \times h \times B = 5.184 \text{ in}^2 \]
Spacing of reinforcement;
\[ s_{xb,prov} = (B - 2 \times c_{nom}) / \max(N_{ub} - 1, 1) = 8.455 \text{ in} \]
Maximum spacing of reinforcement;
\[ s_{max} = \min(3 \times h, 18\text{in}) = 18.000 \text{ in} \]

**PASS - Reinforcement provided exceeds minimum reinforcement required**

Depth of compression block
\[ a_x = A_{s,xb,prov} \times f_y / (0.85 \times f'_c \times B) = 2.20 \text{ in} \]
Neutral axis factor
\[ \beta_1 = 0.85 \]
Depth to the neutral axis
\[ c_{na,x} = a_x / \beta_1 = 2.59 \text{ in} \]
Strain in reinforcement
\[ \epsilon_{L,x} = 0.003 \times (d_x - c_{na,x}) / c_{na,x} = 0.02932 \]

**PASS - The section has adequate ductility (cl. 10.3.5)**

Nominal moment strength required;
\[ M_{nx} = \text{abs}(M_x) / \phi_\ell = 1591.155 \text{ kip}_\text{ft} \]
Moment capacity of base;
\[ M_{capx} = A_{s,xb,prov} \times f_y \times [d_x - (A_{s,xb,prov} \times f_y / (1.7 \times f'_c \times B))] \]
\[ M_{capx} = 1608.946 \text{ kip}_\text{ft} \]

**PASS - Moment capacity of base exceeds nominal moment strength required**

Moment design in y direction

Reinforcement provided; 26 No. 9 bars bottom
Depth of tension reinforcement;
\[ d_y = h - c_{nom} \times \phi_{yb} / 2 = 26.808 \text{ in} \]
Area of tension reinforcement provided;
\[ A_{s,yb,prov} = N_{yb} \times \pi \times \phi_{yb}^2 / 4 = 25.983 \text{ in}^2 \]
Area of compression reinforcement provided:
\[ A_{c,\text{yT,prov}} = N_{yT} \times \pi \times \phi_T^2 / 4 = 0.000 \text{ in}^2 \]

Minimum area of reinforcement:
\[ A_{c,\text{y,min}} = 0.0018 \times h \times L = 11.016 \text{ in}^2 \]

Spacing of reinforcement:
\[ s_{yB,\text{prov}} = (L - 2 \times c_{\text{rom}}) / \max(N_y - 1, 1) = 8.040 \text{ in} \]

Maximum spacing of reinforcement:
\[ S_{\text{max}} = \min(3 \times h, 18\text{ in}) = 18.000 \text{ in} \]

**PASS - Reinforcement provided exceeds minimum reinforcement required**

Depth of compression block
\[ a_y = A_{c,\text{yB,prov}} \times f_y / (0.85 \times f'_{c} \times L) = 2.25 \text{ in} \]

Neutral axis factor
\[ \beta_1 = 0.85 \]

Depth to the neutral axis
\[ c_{nax,y} = a_y / \beta_1 = 2.64 \text{ in} \]

Strain in reinforcement
\[ \varepsilon_{c,y} = 0.003 \times (d_y - c_{nax,y}) / c_{nax,y} = 0.02741 \]

**PASS - The section has adequate ductility (cl. 10.3.5)**

Nominal moment strength required:
\[ M_{ny} = \text{abs}(M_y) / \phi = 2773.333 \text{ kip ft} \]

Moment capacity of base:
\[ M_{capy} = A_{c,\text{yB,prov}} \times f_y \times [d_y - (A_{c,\text{yB,prov}} \times f_y / (1.7 \times f'_{c} \times L))] \]

\[ M_{capy} = 3336.701 \text{ kip ft} \]

**PASS - Moment capacity of base exceeds nominal moment strength required**

Calculate ultimate shear force at d from right face of column A

Ultimate pressure for shear d from face of column:
\[ q_{su} = (q_{tu} + C_x \times (L / 2 + \theta_{PAA} \times l_a / 2 + d_x) / B + q_{su}) / 2 \]

\[ q_{su} = 19.299 \text{ ksf} \]

Area loaded for shear at d from face of column:
\[ A_s = B \times \min(3 \times (L / 2 - \theta_{TA}), L / 2 - \theta_{PAA} - l_a / 2 - d_x) = 12.709 \text{ ft}^2 \]

Ultimate shear force at d from face of column:
\[ V_{su} = A_s \times (q_{su} - F_u / A) = 233.254 \text{ kips} \]

**Shear design at d from right face of column A**

Strength reduction factor in shear:
\[ \phi_s = 0.75 \]

Nominal shear strength:
\[ V_{nsu} = V_{su} / \phi_s = 311.005 \text{ kips} \]

Concrete shear strength:
\[ V_{c,s} = 2 \times \lambda \times \sqrt{(f'_{c} \times 1 \text{ psi})} \times (B \times d_x) = 339.231 \text{ kips} \]

**PASS - Nominal shear strength is less than concrete shear strength**

Calculate ultimate punching shear force at perimeter of d / 2 from face of column A

Ultimate pressure for punching shear:
\[ q_{puA} = q_{tu} + [(L/2 + \theta_{PAA} \times l_a / 2 - d / 2) + (l_a + 2 \times d / 2) / 2] \times C_y / [B / (B / 2) \times C_y / L] \]

\[ q_{puA} = 19.299 \text{ ksf} \]

Average effective depth of reinforcement:
\[ d = (d_x + d_y) / 2 = 27.372 \text{ in} \]

Area loaded for punching shear at column:
\[ A_{PA} = (l_a + 2 \times d / 2) \times B = 26.248 \text{ ft}^2 \]

Length of punching shear perimeter:
\[ l_{PA} = 2 \times B = 16.000 \text{ ft} \]

Ultimate shear force at shear perimeter:
\[ V_{puA} = P_{up} + (F_u / A \times q_{puA}) \times A_{PA} = 766.272 \text{ kips} \]

**Punching shear stresses at perimeter of d / 2 from face of column A**

Nominal shear strength:
\[ V_{npuA} = V_{puA} / \phi_s = 1021.696 \text{ kips} \]

Ratio of column long side to short side:
\[ \beta_A = \max(l_a, b_a) / \min(l_a, b_a) = 2.500 \]

Column constant for edge column:
\[ \alpha_{PA} = 30 \]

Concrete shear strength:
\[ V_{c,p, i} = (2 + 4 / \beta_A) \times \lambda \times \sqrt{(f'_{c} \times 1 \text{ psi})} \times u_{PA} \times d = 1196.576 \text{ kips} \]

\[ V_{c,p, ii} = (\alpha_{PA} \times d / u_{PA} + 2) \times \lambda \times \sqrt{(f'_{c} \times 1 \text{ psi})} \times u_{PA} \times d = 2086.322 \text{ kips} \]

\[ V_{c,p, iii} = 4 \times \lambda \times \sqrt{(f'_{c} \times 1 \text{ psi})} \times u_{PA} \times d = 1329.529 \text{ kips} \]

\[ V_{c,p} = \min(V_{c,p, i}, V_{c,p, ii}, V_{c,p, iii}) = 1196.576 \text{ kips} \]
PASS - Nominal shear strength is less than concrete shear strength

Calculate ultimate punching shear force at perimeter of d / 2 from face of column B

Ultimate pressure for punching shear;
\[ q_{puB} = q_{tu} + [(L/2 + \varepsilon_P B + l_B d/2) + (l_B + 2d/2)/2] C_y B [B/2] C_y / L \]
\[ q_{puB} = 19.299 \text{ ksf} \]

Average effective depth of reinforcement;
\[ d = (d_x + d_y) / 2 = 27.372 \text{ in} \]

Area loaded for punching shear at column;
\[ A_{pB} = (l_B + 2d/2) B = 26.248 \text{ ft}^2 \]

Length of punching shear perimeter;
\[ u_{pB} = 2B = 16.000 \text{ ft} \]

Ultimate shear force at shear perimeter;
\[ V_{puB} = P_{uB} + (F_u / A_pB) A_{pB} = 766.272 \text{ kips} \]

Punching shear stresses at perimeter of d / 2 from face of column B

Nominal shear strength;
\[ V_{npuB} = V_{puB} / \phi_s = 1021.696 \text{ kips} \]

Ratio of column long side to short side;
\[ \beta_B = \max(l_B, b_B) / \min(l_B, b_B) = 2.500 \]

Column constant for edge column;
\[ \alpha_{sB} = 30 \]

Concrete shear strength;
\[ V_{c,p,i} = (2 + 4 / \beta_B) \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times u_{pB} \times d = 1196.576 \text{ kips} \]
\[ V_{c,p,ii} = (\alpha_{sB} \times d / u_{pB} + 2) \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times u_{pB} \times d = 2086.322 \text{ kips} \]
\[ V_{c,p,iii} = 4 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times u_{pB} \times d = 1329.529 \text{ kips} \]
\[ V_{c,p} = \min(V_{c,p,i}, V_{c,p,ii}, V_{c,p,iii}) = 1196.576 \text{ kips} \]

PASS - Nominal shear strength is less than concrete shear strength

26 No. 5 bars (4 in.)

One way shear at d / 2 from column face
Two way shear at d / 2 from column face
Construction Schedule
## Construction Estimate

**RSMeans QuickCost Estimator**

- **Project Title:** Shuriken Tower
- **Model:** Apartment, 8-24 Story
- **Construction:** Ribbed Precast Concrete Panel / Steel Frame
- **Location:** BROOKLYN, NY
- **Stories:** 15
- **Story Height (l.f.):** 10.5
- **Floor Area (s.f.):** 513,000
- **Data Release:** Year 2012 Quarter 3
- **Wage Rate:** Union
- **Basement:** Not included

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<th>Cost Ranges</th>
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<th>Med</th>
<th>High</th>
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<td>Total</td>
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<td>Architectural Fees</td>
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*Costs are derived from a building model with basic components. Scope differences and market conditions can cause costs to vary significantly.*

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