Gonzaga University Senior Design Project

School of Engineering

Project Plan for:
Gonzaga University Parking Structure Design Project

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Center for Engineering Design

April 22nd, 2010
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*Prepared by:*

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Executive Summary:
With the expansion of Gonzaga University’s student body, the need for additional parking beyond that currently available has become apparent. Gonzaga has added three residence halls in the last four years as well as dramatically increased the enrollment. This has led to an increase of on-campus residence and a decrease in the availability for parking. The structure to be added will be a 214,000 plus square foot open roof parking structure capable of accommodating over 600 vehicles.

An additional component of this parking structure project is the addition of a two story retail facility lining Hamilton Street. This building will eventually earn Gonzaga a return on the project by housing tenants. In the short term, it can serve as the interim location for the Gonzaga Bookstore and Dining Hall while the University demolishes the current bookstore and dining hall to construct a new facility.

This project is formatted as a design build with extensive discussion and review of the design throughout its conception and design. ALSC Architects was awarded the project by Gonzaga University and ALSC hired the design team to perform the structural analysis.

The geotechnical data was attained through a site visit along with seven borings. The usage of the buildings was determined by ALSC Architects. They obtained this information through various interviews and meetings with Gonzaga University. This usage information along with the buildings’ location drove the buildings’ load data which was attained through the use of ASCE manual 7-05 and IBC 2006.

The design team will evaluate the appropriate dead, live, and environmental loads associated with the requirements of the structure as well as include the necessary measures pertaining to the design criteria specified by the City of Spokane. We will evaluate both structures individually to help us determine an efficient structural system. We will accomplish this by following our scope of work as outlined below.

Scope of Work
- Verify the local code requirements for the project
- Select a structural material that will be most appropriate for the project and site
- Design Structure
  - Determine live loads and dead loads of the material and design accordingly
  - Determine an appropriate structural system
  - Calculate structural elements (roof, lateral system, foundations, etc)
- Submit drawings and typical details supported by calculations
Deliverables
- AutoCAD drawing of the;
  - Foundation plan
  - Floor and roof framing plans
  - Lateral force resisting elements
  - Framing details
- Prepare a final report which includes;
  - Description of design system
  - Drawings
  - Calculations
  - Project impact evaluation

Materials Received
- Architectural drawings provided by ALSC Architects
- Geotechnical engineering study provided by GeoEngineers
- Specific Design Codes from the City of Spokane Building Official
- Various parts catalogs
- ASCE 7-05
- AISC Steel Construction Manual (13th Ed)

Social Impacts of the Project
Because the structures border Hamilton Street, traffic signal adjustments may need to be taken to deal with the new facility. The nearest stop light on Hamilton is two blocks north at E Sharp Avenue. A new stop light may be needed to be placed at the intersection of E Desmet Avenue and Hamilton Street or modification and/or re-timing of the light at the corner of Hamilton Street and E Sharp Avenue in order to deal with the large increase in turning traffic into and out of the parking structure or retail center.

However, the buildings should overall serve to reduce congestion and improve safety in the neighborhood areas around Gonzaga. The parking structure should reduce the number of students parking on the local residential streets, pull commuter traffic searching for parking out of side-streets, offer event parking, and help improve student and vehicle safety by keeping students and vehicles concentrated around a centralized more easily patrolled facility very close to both the Campus Security Offices and the local Spokane City Police post.
Aesthetically, the buildings should be an improvement over the current lot usage. Buildings are
designed with looks in mind and will mesh well with the architecture currently in the area.
Proof of the attention to building aesthetics can be found in the particular care in design of the
exterior of the parking structure to ensure that it is a pleasing structure to behold and not
merely a drab gray parking garage. Considerations also include the impact of light pollution on
the neighborhood east of Hamilton Street.

**Special Considerations**
The parking structure will have a penetrating sealer to prevent corrosion from anti-icing salt
that is brought in by cars during the snow months. This will have to be reapplied every 5 years
to ensure the protection and longevity of the concrete. Also corrosion inhibiting admixer can
be added to the concrete mix to increase the lifetime of the concrete.

All construction personnel should follow Occupational Safety (OSHA) and Washington Industrial
Safety and Health Act (WISHA) precautions during the construction phase.

**Sustainable Design**
Alternative materials used for the GU parking structure and retail center have been evaluated
for the preliminary design of the structure. Because the main elements of the project will
consist of concrete and steel, alternative materials are limited. The easiest way to provide a
sustainable design is to select suppliers near the project site. This will reduce transportation
emissions and costs. It was found for concrete, choosing a company that has up to date
equipment can save up to 8% on carbon dioxide emissions. Additives like slag, fly ash, or silicon
fume can be used to reduce costly materials produced by the cement mill thus recycling
materials that would otherwise be discarded.

Inherently, steel is highly recyclable and thus the best way to incorporate sustainable steel
structures is in an efficient steel design. Other opportunities may be in the actual design of the
structure. Incorporating south facing façades, clerestories, and a good thermal envelope will
substantially increase sustainability and will save Gonzaga money. The building location and
orientation will be considered but is highly constricted by the proposed site location.

**Design Loads**
Snow Loads:
- Minimum Roof Design Snow Load 30 psf
- Ground Snow Load 39 psf
- Live Load Retail Building, both floors 100 psf
- Roof Load Retail Building 20 psf
Floor Load, Garage 40 psf
Wind Load
   Retail Building 12 psf
   Garage 13 psf
Wind Load, Uplift
   Retail Building 6 psf
   Garage 7 psf

The live load for a second story retail building is 75 psf. However due to the uncertainty of use, the design team elected to use 100 psf for the second floor retail building to cover for the option of using it for the temporary COG.

**Design Controls**

From a structural design standpoint, the architectural drawings as well as considerations about efficient design of the structure control the design of the building. The design team will collaborate with ALSC to develop structural drawings as ALSC improves their architectural drawings.

The features that controlled the architectural drawings were versatility of structures and construction options, outward appearance of structures, and usage requirements. Construction options were important and drove the buildings being designed independently despite sharing a common wall. This was done so that Gonzaga University could have the option of building just the parking structure, the parking structure and later the retail building, or both buildings at the same time. Versatility of the structure was particularly important with the retail structure because the building will serve as a temporary host of both the cafeteria and bookstore. Further versatility was needed because after this usage is served, the building tenants will likely vary in building usage needs.

Consideration also must be given to surface runoff of an open roof parking garage. The existing swales that help collect the surface runoff will be removed and new design of surface runoff collection system must be considered.

**Geotechnical Engineering Evaluation**

The geotechnical engineering study provided information for the foundation design of the parking structure and retail building. GeoEngineers was provided preliminary foundation loads by the design team.

- All foundations must be supported by 2 feet of structural backfill. This will require the removal and recompaction of the existing soil 2 feet below the proposed elevation of the footings. Additional excavation may be required if there is unsuitable material such
as soft soil or excessive organic material. Extra attention must be paid to excavation in the southwest corner of the site.

- It is recommended that the building foundations be built at least 24 inches below finished grade to protect against frost heave.
- The following table presents the allowable soil bearing pressures for wall or column footings for dead plus long-term live loads. The values in the table may be increase by one-third to account for short term loads such as wind or seismic forces.

<table>
<thead>
<tr>
<th>Footing Width (ft)</th>
<th>Allowable Bearing Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 or less</td>
<td>3,000</td>
</tr>
<tr>
<td>2 to 6</td>
<td>4,000</td>
</tr>
<tr>
<td>6 to 10</td>
<td>5,000</td>
</tr>
<tr>
<td>Greater than 10</td>
<td>6,000</td>
</tr>
</tbody>
</table>

- GeoEngineers has estimated that the settlements under the new footings on structural fill will be less than one inch. Differential settlement between columns or along 50 feet of continuous wall footings will be less than ½ inch. This settlement will occur as loads are applied. Post-construction settlement will be minor.
- The coefficient of friction for lateral loads is to be 0.35 applied to the vertical dead loads
- The on-site soil is classified as Class-D soil.

**Foundations**

We decided to choose gravel for our structural fill. Gravel has a high shear resistance which is ideal for foundations. The gravel should have less than 5% fines and no material greater than 6 inches. The column loadings were found by taking the tributary areas to each column placed in the parking structure by the architect. The loadings from each floor were added up and a total load per column was determined. The reinforced concrete weight is to be 150 pcf and we will use a 6 inch slab for the elevated slabs at each level. The live load on the floor will be 40 psf on the slab at each floor level.

We have decided to place the footings at 5 ½ feet below grade. We choose this location because the borings show consistently the soil changes from the loose fill material to the medium silty gravel at approximately 5 ½ to 6 ½ feet. This would require us to excavate 7 ½ feet and fill the bottom 2 feet in with gravel. While in practice, we would contact GeoEngineers and find ways to improve the soil to avoid excavating 7 ½ feet. However for the purpose of this exercise, the design team decided to excavate all the necessary soil.
GeoEngineers has given us allowable bearing pressures to size footings assuming the soil is prepared as outline above. We decided to calculate our own numbers using soil properties at 6 feet determined by the Standard Penetration Test. We used a phi angle of 30° and a unit weight of 115 pcf. Using these numbers we calculated the allowable bearing pressure on the soil and compared it to the allowable given to us by GeoEngineers. The calculated allowable were greater than those given by the soil report, generally by about 25-30%. Due to the uncertainty of our numbers, we chose to go with the values presented to us by GeoEngineers. The main difficulty we ran into was the inconclusiveness of the soil report. The report did not show much more than the soil type and the blows per foot. This made it difficult to understand where they arrived at their calculated numbers for allowable bearing pressure.

Using the table provided by GeoEngineers, we designed the footings for all the columns in both buildings. We choose square footings for the columns and continuous footings for the shear walls. The footing widths were rounded up to 7 different sizes to keep similar sizes for construction.

<table>
<thead>
<tr>
<th>Schedule</th>
<th>$d$ (in)</th>
<th>$A_{req'd}$ (in$^2$)</th>
<th>Steel Selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>6x6</td>
<td>10.5</td>
<td>2.49</td>
<td>6 #6s Both ways</td>
</tr>
<tr>
<td>7x7</td>
<td>14</td>
<td>3.88</td>
<td>7 #7s Both ways</td>
</tr>
<tr>
<td>8x8</td>
<td>15.5</td>
<td>4.91</td>
<td>7 #8s Both ways</td>
</tr>
<tr>
<td>9x9</td>
<td>18</td>
<td>6.42</td>
<td>7 #9s Both ways</td>
</tr>
<tr>
<td>10x10</td>
<td>27</td>
<td>10.69</td>
<td>9 #10s Both ways</td>
</tr>
<tr>
<td>12x12</td>
<td>24.5</td>
<td>11.64</td>
<td>8 #11s Both ways</td>
</tr>
<tr>
<td>14x14</td>
<td>29.5</td>
<td>16.35</td>
<td>11 #11s Both ways</td>
</tr>
</tbody>
</table>

The depths off the footings were calculated by checking for both one way and two way shear. Since the footings were square, two way shear governed in each case. Due to our inexperience with concrete foundations, the design team did not factor any moments into our calculations.

When designing the area of steel needed in our footings, we needed to consider the bending moment that would occur from the bearing stress in the soil. Due to symmetry, the bending moment would be the same on either axis for a square column. Taking the critical section as the distance between the edge of the footing and the face of the column, finding the maximum moment and comparing it to a corresponding $\rho_{min}$ value for flexure, an area of steel was calculated. This area of steel, because of the noted symmetry, will span both directions equally in the bottom of the footing.
Garage Design
Post Tension Slab

Due to the design team’s inexperience with post tension slabs, we used a program called PT Data to design the slab. The maximum concrete slab span is 24 feet. From this, we found that a concrete slab depth of 6 inches would be needed. The slab will be cast in place on top of the precast beams and girders. The design team has put together a slab casting schedule for the contractor. We estimate that a typical contractor could place 200,000 square feet per day of low shrinkage concrete. The slab casting schedule will show the cold joints above the shear walls to reduce shearing stress on the columns caused by thermal expansion and contraction. The force from the shrinkage is located on the shear wall rather than a less stiff column. See Appendix G for an illustration of the slab placement schedule. Approximately 198,000 square feet are poured on the first and third day and approximately 13,500 square feet are poured on the second day. The contractor is responsible for ensuring the concrete is poured and tensioned correctly.

The slab design will balance 75% of the dead load to reduce the unloaded slab sag. Each tendon will withstand 26.5 ksi and will be spaced at about 13”, or .88 tendons per foot. Secondary reinforcement will be used and was calculated for us using the program PTData.

Post Tension Beams and Girders

While calculating dimensions for a cast in place Beams and Girders to determine the depth, the design team found that an impractical depth of over 6 feet would be necessary to support loads on the beam and girders. As a result, team decided to use precast-post tensioned beams and girders for the structural components supporting the concrete slab in the parking garage. Precast beams and girders are also more consistent than cast in place would be because they are formed under controlled lab conditions. Due to the decrease in depth and the increase in quality of the beams and columns, there will be savings on material costs and maintenance costs giving the design team confidence in the decision to adopt post tension members in our design.

Precast Columns

The design team has decided to go with precast columns for a variety of reasons. First the precast columns, similar to the beams and girders, are more consistent and of higher quality because of the controlled conditions they are fabricated in. Also, speed is of the essence to this project because Gonzaga only has a summer to complete the project as the population on campus declines for a little over three months. The precast beam, girder, and columns with a cast in place concrete slab design is often referred to as “hybrid construction”. The hybrid
construction is highly beneficial for a tight schedule because of the speed that the design allows for. This quickens the construction time significantly because the contractor does not have to form the columns and wait for curing. To get the initial skeleton off the ground is simply a matter of putting the parts together. This cuts the time restraints on erection time drastically as well. After the skeleton is erected, the concrete slab will be cast in place with form work supported by the structures skeleton.

The responsibility to design the columns themselves will rest on the manufacturer. An engineer would relay column dimensions, axial loads, and calculated moments to the column fabricator where then they would design appropriately. The columns will contain corbels to rest the beams upon.

As a comparison, we designed cast in place columns. For our project, the architect calls for 12”x18” columns, so our column design was centered on meeting these aesthetic requirements. For our calculations, the columns used for the concrete project were designed for axial loads only. Considering the moment that occurs in every building, the design team expects to find that a much larger area of steel would need to be used but again, this would be determined by the column fabricator and the appropriate changes would need to be made.

**Garage Lateral Design**

Due to the nature of the building, the design team decided on the use of shear walls to pick up the lateral force on the garage building. The shear walls were designed using the rigid floor analysis. The four large shear walls were responsible for picking up the shear from the North and South facing walls. Due to the absence of large shear walls to pick up the wind on the west wall, the design team selected various walls to pick up the shear force. These walls were either elevator shafts or stair wells since they would be consistent through the building and not be altered during the life of the building. Wind loads were the lateral forces considered.

Due to the non-symmetry of the building, the design team calculated a wind force on each section of the building and summed moments to determine the center of load. The force into each shear from the direct force was found for all shear walls.

The design team calculated rigidity for each shear wall and found the center of rigidity. With the center of load and rigidity, the design team was able to find the moment caused by the wind.

Displayed in the table below are the results for the wind onto each shear wall.
<table>
<thead>
<tr>
<th>Wall</th>
<th>North/South</th>
<th>Total Force (k) (N/S)</th>
<th>Total Force (k) (W/E)</th>
<th>Design Strength (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27.06</td>
<td>-0.74</td>
<td>27.06</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>27.06</td>
<td>-0.74</td>
<td>27.06</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>15.64</td>
<td>0.45</td>
<td>15.64</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>35.44</td>
<td>1.02</td>
<td>35.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>East/West</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A</td>
<td>0.44</td>
<td>12.08</td>
<td>12.08</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.51</td>
<td>15.60</td>
<td>15.60</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-0.11</td>
<td>10.40</td>
<td>10.40</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>-0.57</td>
<td>58.84</td>
<td>58.84</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>1.55</td>
<td>40.08</td>
<td>40.08</td>
</tr>
<tr>
<td></td>
<td>F</td>
<td>0.18</td>
<td>4.45</td>
<td>4.45</td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>0.18</td>
<td>4.45</td>
<td>4.45</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>-0.07</td>
<td>4.40</td>
<td>4.40</td>
</tr>
<tr>
<td></td>
<td>I</td>
<td>-0.07</td>
<td>4.40</td>
<td>4.40</td>
</tr>
</tbody>
</table>

The shear force into each wall was calculated using ACI Equation 11-29. Due to the large size of Walls 1-4, no horizontal reinforcing is needed to pick up shear force however they were required due to axial load. For Walls A-I, stirrups are required. $S_{\text{min}}$ governs for stirrup spacing due to the low shear not picked up by the concrete. The vertical reinforcing required to handle the moment for each shear wall was more than enough to pick up the vertical axial load.

**Retail Design**

**Retail Gravity Design**

The material selection of steel for the retail section of the parking structure was dependent on a few factors. The fact that the building is going to have so many uses and it might need to be retrofitted at some point pointed us towards steel and away from concrete. Also, the building plans from the architect pointed us towards steel to match the building’s aesthetics.

We analyzed a few options when it came to picking up the gravity loads in the building. Steel tubes were the obvious choice for the columns in the building because of not only their strength to weight ratio but also for their framing versatility. When it came to the beam and girder selection we experimented with wide flanges for beams and girders, open steel joist for both beams and girders, and open steel joist for the beams and wide flanges for the girders. After analysis it turned out that for the roof framing that the steel joists for both the beams and girders was the lightest and most cost effective with beams spacing at 8 ft. on center. The savings in cost between the wide flange girders and steel joist girders was over $40,000 per bay.
For the second floor framing, steel wide flanges were used because of higher design live load of 100 psf; we need a higher strength section with a reasonable depth/ strength of the wide flanges with a reasonable depth. After analysis, the concrete for the second floor would be laid on 18 gauge steel decking with a slab thickness of 3 1/2” on wide flanges spaced 8 ft. on center. This choice is made from the allowable design loads from the steel decking made by Verco Manufacturing for means spacing of 8 ft. on center. When we started analyzing the floor thickness we also started considering the idea of using composite beam systems in the retail section of the parking structure. This is beneficial because it can up the strength of our building while limiting the beam sizes as well as the slab thickness. With the concrete working in compression and the steel in tension, this makes for a more efficient system. Cambering the beams might also help shrink the beams sizes.

Retail Lateral Design
North-South Direction
In the north-south direction, the building has a rigid diaphragm for the second floor. The roof diaphragm is not fully rigid since it is only 3” deep steel deck. However, because of the dimensions of the building (it is 280’ deep and only 80’ wide in this direction), the diaphragm can be approximated as rigid. The direct force from the lateral load in this direction is resisted by the CMU fire wall that runs the entire length of the west side of the building. This fire wall is a barrier between the retail building and the parking garage.

Since the direct force resisting element (fire wall) is located off center from the lateral load, a torsional moment is created. This torsional moment is picked up by the direct lateral force resisting elements in the east-west direction.

The wind pressures act on a surface 80’ wide and 42’ tall. The building is two floors with each floor 15 feet high. The building then has a maximum height of architectural features above the roof of the building is 12’. This maximum height was applied about the entire structure to be conservative in initial lateral system calculations.

The wind pressures used in design were calculated using ASCE Method 1 (Simplified Procedure). This method produced a resultant total load of about 25 kips. These loads however, were not what controlled for lateral system design.

The seismic loads governed for the lateral system design. The design method employed for seismic loadings is the Equivalent Lateral Force Procedure outlined in ASCE Section 12.8. This method produced a total lateral load of around 210 kips. Since this value is significantly greater than the lateral wind load, the seismic forces governed the lateral system design in the north-south direction.
East-West Direction

In the east-west direction, the building has a rigid diaphragm for the second floor. The roof diaphragm is flexible since it is 3” deep steel deck. The direct force from the lateral load in this direction is resisted by a series of four steel cross-braced shear walls. This cross-bracing is currently made up of Square HSS tubes joined in the center with knife plates.

Since the direct force resisting elements (cross-braced walls) are located off center from the lateral load, a torsional moment is created. This torsional moment is picked up by the direct lateral force resisting element in the north-south direction (fire wall).

The wind pressures act on a surface 280’ wide and 42’ tall. The building is two floors with each floor 15 feet high. The building then has a maximum height of architectural features above the roof of the building is 12’. This maximum height was applied about the entire structure to be conservative in initial lateral system calculations.

The wind pressures used in design were calculated using ASCE Method 1 (Simplified Procedure). The resulting total lateral load from the wind pressure is around 75 kips. These loads were what ended up controlling lateral system design.

The design method employed for seismic loadings is the Equivalent Lateral Force Procedure outlined in ASCE Section 12.8. This method produced a total lateral load of around 150 kips. Since this value is double the lateral wind load, the seismic forces govern the lateral system design in the east-west direction.
Works Cited


Appendix A- Calculations

A-1 Foundation Width

Allowable stress on soil, column footings

\[ q_{allowable} = \frac{1.3c' \cdot N_c + qN_q + .4 \cdot \gamma \cdot B \cdot N_F}{FS} \]

For Gravel:

\[ \gamma = 115 \quad c' = 0 \quad \varnothing = 30^\circ \quad D_F = 5' \]

\[ N_c = 0 \quad N_q = 22.46 \quad N_F = 19.13 \quad FS = 3.0 \]

\[ q_{allowable} = \frac{(115 \cdot 5')(22.46) + 4(115)(7.25)(19.13)}{3.0} \quad q_{allowable} = 6.34 \text{ ksf} \]

\[ q_{actual} = \frac{254.22 \text{ kip}}{7.25 \cdot 7.25} \quad q_{actual} = 4.84 \text{ ksf} \]

Allowable stress on soil, continuous footing

\[ q_{allowable} = c' \cdot N_c + q \cdot N_q + .5 \cdot \gamma \cdot B \cdot N_F \]

\[ q_{allowable} = \frac{(115 \cdot 5) + 22.46 + .5 \cdot 115 \cdot 5.75 \cdot 19.13}{3.0} \quad q_{allowable} = 6.84 \text{ ksf} \]

\[ q_{actual} = \frac{770.27 \text{ kip}}{5.25 \cdot 27.42} \quad q_{actual} = 4.89 \text{ ksf} \]

The design team will use the allowable stress provided to us in the geotechnical report.
A-2 Foundation Depth

Two Way Shear

\[ V_{u2} = P - d \times q_u \quad b_o = 4(a + d) \]

\[ d = \frac{V_{u2}}{4 \times \phi \times \lambda \times \sqrt{f'c} \times b_o} \quad (ACI \ Equation\ 11-35) \]

\[ d = \frac{V_{u2}}{\phi \times (\frac{a + d}{b_o}) \times \sqrt{f'c} \times b_o} \quad (ACI\ \ Equation\ 11-34) \]

\[ d=\text{depth of footing} \]

\[ V_{u2} = \text{Point load on foundation for two way shear} \]

\[ q_u = \text{soil bearing pressure} \]

\[ b_o = \text{perimeter around punching area} \]

\[ a = \text{width of column} \]

\[ \phi = \text{Safety factor (0.75 for shear)} \]

\[ \lambda = \text{lightweight concrete modification factor. (1.0 for our design)} \]

\[ f'c = \text{concrete strength (4,000 psi)} \]

\[ \alpha_s = \text{modification factor for columns. 40 for interior, 30 for edge, and 20 for corner} \]

Using these equations, we selected a depth of footing to calculate \( V_{u2} \) and \( b_o \). With those values we then found depth and iterated until the depth matched our values.

Use \( d=14" \)

\[ q_u = \frac{254.22 \text{ kip}}{7.25 \text{ ft} + 7.25 \text{ ft}} \quad q_u = 4.84 \text{ ksf} \]

\[ b_o = 4 \times (10" + 14") \quad b_o = 96" \]

\[ V_{u2} = 254.22 \text{ kip} - \frac{14"}{12"} \times 4.84 \text{ ksf} \quad V_u = 247.634 \text{ kips} \]

\[ d = \frac{247,634 \text{ lbs}}{0.75 + 4 \times \sqrt{4000 \text{ psi} \times 96"}} \quad d = 13.6" < 14" \]

\[ d = \frac{247,634 \text{ lbs}}{0.75 \times (\frac{40 + 14"}{96"}) \times \sqrt{4000 \text{ psi} \times 96"}} \quad d = 8.53" < 14" \]

One Way Shear

\[ V_{u1} = b_w \times (\frac{1}{2} \times \frac{a}{2} - d) \times q_u \]

\[ d = \frac{V_{u1}}{2 \times \phi \times \sqrt{f'c} \times b_w} \]

\[ b_w = \text{whole width of footing} \]

\[ l = \text{length of footing} \]

For one way shear, we used the \( d \) found for two way shear to calculated \( V_{u1} \) and ensured it was sufficient.

\[ V_{u1} = (7.25" \times 12") \times (\frac{7.25" \times 10"}{2} - \frac{14"}{2} - 14") \times 4.84 \text{ ksf} \times \frac{1000 \text{ lbs}}{144 \text{ in}^2} \]

\[ V_{u1} = 71,597 \text{ lbs} \]

\[ d = \frac{71,597 \text{ lbs}}{0.75 \times 2 \times \sqrt{4000 \text{ psi} \times 96"}} \quad d = 7.86" < 14" \]
A-3 Foundation Area of Steel

Area of Steel Selection

The moment capacity of the foundation in flexure must be calculated in order to find the correct $\rho$ that would correlate with the necessary area of steel needed.

$$M_u = \left(\frac{7.25' - 1.5'}{2}\right) \times (7.25') \times 4.84 ks f \times \frac{\left(\frac{7.25'}{2} - 1.5'\right)}{2} \times \frac{144 ft^2}{1000 lb}$$

$$M_u = 204.3 \text{ ft} - k$$

$$\frac{M_u}{\phi b d^2} = \frac{204.3 ft - k}{.9 \times 87'' \times 14''^2}$$

$$\frac{M_u}{\phi b d^2} = 159.7 \text{ psi}$$

From Table A.13 in *Design of Reinforced Concrete 8th Edition*, select the correlating $\rho$ for, an \(\frac{M_u}{\phi b d^2}\) equal to 159.7 psi and compare it to $\rho_{\text{min}}$ for flexure (.0033)

\[ \rho = .0027 < \rho_{\text{min}} = .0033 \quad \text{there for; } \quad \rho_{\text{design}} = .0033 \]

$$A_s = \rho_{\text{design}} \times b \times d$$

$$A_s = .0033 \times 7.25' \times 12'' \times 14'' = 4.02 \text{ in}^2$$

*Use 7 #7s bars in both directions (4.21in$^2$)*
A-4 Shear Wall

**Rigidity**

\[ \Delta_F = 0.1 \left( \frac{h}{d} \right)^3 + 0.3 \left( \frac{h}{d} \right) \]

\[ R_1 = \frac{1}{\Delta_F} \]

\[ \Delta_F = 0.783 \]

\[ R_1 = \frac{1}{0.783} \]

\[ R_1 = 1.28 \]

**Center of Resistance**

\[ CR = (\bar{x}, \bar{y}) \]

\[ \bar{x} = \frac{\Sigma R_{y_i} \cdot x_i}{\Sigma R_{y_i}} \]

\[ \bar{y} = \frac{\Sigma R_{x_i} \cdot y_i}{\Sigma R_{x_i}} \]

\[ \bar{x} = 78.32' \]

\[ CR = (78.32, 145.87) \]

**Direct Wind Load into Each Wall**

\[ F_1 = \frac{P}{\Sigma R_{y_i}} \cdot R_1 \]

\[ F_1 = \frac{105.21}{4.405} \cdot 1.28 \]

\[ F_1 = 30.57 \]

**Distance to Center of Resistance**

\[ d_1 = x - \bar{x} \]

\[ d_1 = 0' - 78.32' \]

\[ d_1 = -78.32' \]

**Moment caused by Wind Force**

\[ M = P_y \cdot e_x \]

\[ M = 105.21 \cdot (90.8 - 78.32) \]

\[ M = 1312.65 \text{ ft} \cdot k \]

**Force from Rotation of Rigid Floor**

\[ F_1 = M \cdot \left( \frac{R_i \cdot d_i}{\Sigma R_i \cdot d_i^2} \right) \]

\[ F_1 = 1,312.65 \cdot \frac{1.25 \cdot (-78.32)}{37492.02} \]

\[ F_1 = -3.51k \]
A-5 Concrete Column  
Required Steel, Axial Forces only

\[
\phi P_n = \phi \times 0.8[0.85f'c (A_g - A_{st}) + f_y A_{st}]
\]

For our Columns
\[\phi = 0.65\]
\[A_g = (12" \times 18") = 216" \text{ (as per architect’s request)}\]
\[\phi P_n = \text{given}\]
\[f_y = 60 \text{ ksi}\]
\[f_u = 4 \text{ ksi}\]

Manipulation gives a required area of steel. An example:

For column MM-1:

Total Load, \(\phi P_n = 492.6 \text{ kips and } \phi = 0.65\)

\[
492.6 \text{ k} = 0.65 \times 0.8[0.85(4\text{ksi})(216" - A_{st}) + (60\text{ksi})A_{st}]
\]

\[A_{st} = 3.76 \text{ in}^2 \text{ Use 6 #8 bars (A_g=4.71 in^2)}\]
A-6 Steel Column

Square HSS

**Material Properties**

\[ f_y := 46000 \text{PSI} \]

** Loads **

**Roof Loads**

- 4 Ply Felt & Gravel Roofing \[ \text{wfelt} := 5.5 \text{ psf} \]
- 18 Gauge Verco W2 \[ \text{wdeck} := 3.3 \text{ psf} \]
- Miscellaneous \[ \text{wmiscroof} := 1.2 \text{ psf} \]
- Joist Weight \[ \text{wjoist} := 2.3625 \text{ psf} \]
- Joist Girder Weights \[ \text{wjoistgirder} := .925 \text{ psf} \]

**Total Dead Load**

\[ \text{wdeadroof} :=(\text{wfelt} \ + \text{wdeck} \ + \text{wmiscroof}) = 10 \text{ psf} \]

**Total Live Load**

\[ \text{wliveroof} := 20 \text{ psf} \]

**Total Snow Load**

\[ \text{wsnowroof} := 25 \text{ psf} \]

**Total Load (ASD)**

\[ \text{wroof} := \text{wdeadroof} \ + \text{wliveroof} \ + \text{wsnowroof} = 55 \text{ psf} \]

**Second Floor Loads**

**Dead Loads**

- Normal Weight Concrete \[ \text{wconc} := 150 \text{ pcf} \]
- Slab (4" Deep) \[ \text{tslab} := 4 \text{ in} \]
- Slab Weight (from VERCO Catalog) \[ \text{wslab} := 36.6 \text{ psf} \]

**Beam Weight**

\[ \text{wbeam} := \left[ \frac{199.40 \times 2}{40 \times 80} \right] + 3.475 = 5.95 \text{ psf} \]

**Miscellaneous**

\[ \text{wmisc} := 4.45 \text{ psf} \]

**Total Dead Load**

\[ \text{wdead} := \text{wslab} + \text{wbeam} + \text{wmisc} = 47 \text{ psf} \]

**Total Live Load**

\[ \text{wlive} := 100 \text{ psf} \]

**Total Load (ASD)**

\[ \text{wu} := \text{wdead} + \text{wlive} = 147 \text{ psf} \]

**Column Tributary Areas**

**Center Columns**

\[ \text{Atribcenter} := (40 \times 40) = 1.6 \times 10^3 \text{ ft}^2 \]

**Edge Columns**

\[ \text{Atribedge} := (40 \times 20) = 800 \text{ ft}^2 \]

Use same trib.

Area for edge columns

To add uniformity
Column Loads

Center Loads

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<thead>
<tr>
<th></th>
<th>Proof</th>
<th>Psecond</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Load</td>
<td>8.8📚 4</td>
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<td></td>
</tr>
<tr>
<td>2nd Floor Load</td>
<td>2.352📚 5</td>
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<td></td>
</tr>
<tr>
<td>Total Load</td>
<td>3.232📚 5</td>
<td></td>
<td></td>
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</tbody>
</table>

Edge Loads

<table>
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<tr>
<th></th>
<th>Proof</th>
<th>Psecond</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Load</td>
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<td></td>
</tr>
<tr>
<td>2nd Floor Load</td>
<td>1.176📚 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Load</td>
<td>1.616📚 5</td>
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<td></td>
</tr>
</tbody>
</table>

Column Selection

From AISC Table 4-4

Effective Length from Both is 15 feet

Center Columns

Square HSS 10x10x1/2"

Edge Columns

Square HSS 8x8x5/16"
A-7 Composite Beam Design

Material Properties
Concrete \( f_c = 4000 \text{ PSI} \)  
Beam Steel \( f_y = 50000 \text{ PSI} \)  
\( f_u = 65000 \text{ PSI} \)  
\( E = 29000000 \text{ PSI} \)

Loads
Dead Loads
Normal Weight Concrete \( w_{conc} = 150 \text{ pcf} \)  
Slab (4" Deep) \( t_{slab} = 4 \text{ in} \)  
Slab Weight (from VERCO Catalog) \( w_{slab} = 36.6 \text{ psf} \)  
Beam Weight (estimated) \( w_{beam} = 3 \text{ psf} \)  
Miscellaneous \( w_{misc} = 5.4 \text{ psf} \)

Live Loads
Live Load \( w_{live1} = 100 \text{ psf} \)

Since Beams Spaced at 10' O.C.
\( b_{spacing} = 10 \text{ ft} \)

Total Dead Load
\( w_{dead} = \frac{w_{slab} + w_{beam} + w_{misc}(10)}{1000} = 0.45 \text{ klf} \)

Total Live Load
\( w_{live} = \frac{w_{live1}(10)}{1000} = 1 \text{ klf} \)

Total Load (ASD)
\( w_u = w_{dead} + w_{live} = 1.45 \text{ klf} \)

Moment Created
Beam Length:
\( b_{length} = 40 \text{ ft} \)

Moment
\[ M_a = \frac{w_u b_{length}^2}{8} = 290 \text{ kip ft} \]

Effective Width
\[ b_{eff1} = \frac{b_{length}}{8} \cdot 2 = 10 \text{ ft} \]  
\[ b_{eff2} = \frac{b_{spacing}}{2} \cdot 2 = 10 \text{ ft} \]  
\[ b_{eff} = b_{eff1} = 10 \text{ ft} \]
Nominal Moment Arm

Assumed a \( a_{\text{assume}} := 2 \) in

\[ Y_{2\text{nom}} := \text{tslab} - \left( \frac{a_{\text{assume}}}{2} \right) = 3 \text{ in} \]

From AISC Table 3-19:

Using W18x40 (have to use PNA Location 5)

\[ \frac{M_n}{\Omega} = 299 \text{ ft}^k > 290 \text{ ft}^k \text{ ok} \]

\[ I_x := 612 \text{ in}^4 \]

\[ A_s := 11.8 \text{ in}^2 \]

Moment of Inertia Required

\[ I_{\text{req}} := \left( \frac{5}{384} \right) \left[ \left( \frac{w_{\text{dead}} \cdot 1000}{12} \right) \left( \frac{(\text{blength} \cdot 12)^4}{(E \cdot 2.5)} \right) \right] = 357.517 \text{ in}^4 \text{ ok} \]

Strength as Un-Shored Beam Under Wet Concrete

\[ w_{\text{beam1}} := 40 \text{ plf} \]

\[ w_d := \left( \frac{w_{\text{lab}} \cdot \text{bspacing} + w_{\text{beam1}}}{1000} \right) = 0.406 \text{ klf} \]

\[ M_d := \left( \frac{w_d \cdot (\text{blength}^2)}{8} \right) = 81.2 \text{ ft} \cdot \text{k} \text{ ok} \]

\[ 81.2 < 196(\text{PlasticMoment}) \]

Solving Camber (75% Dead Load)

\[ \delta_{\text{d1}} := \left( \frac{0.75 \cdot M_d \cdot (\text{blength}^2)}{161 \cdot I_x} \right) = 0.989 \text{ in} \]

Checking Moment Arm

\[ a := \frac{(A_s \cdot f_y)}{[0.85 \cdot f_c \cdot (\text{bEff} \cdot 12)]} = 1.446 \text{ in} \]

\[ Y_{21} := \text{tslab} - \left( \frac{a}{2} \right) = 3.277 \text{ in} \]

\[ Y_{2} := 3 \text{ in} \text{ ok} \]

\[ \delta_{\text{d1}} := 0.989 \text{ in} \]

\[ \delta_{\text{c}} := 1 \text{ in} \]

Shear Stud Design-Deck Perpendicular-Weak

1 Stud per Rib - Normal Wt. Concrete-3/4" DIA

\[ Q_{\text{nstud}} := 17.2 \text{ kips/stud} \]

\[ \text{Numstuds} := \left( \frac{2 \cdot Q_n}{Q_{\text{nstud}}} \right) = 31.628 \text{ 31 Studs Req'd} \]

With Verco B Formlock-Spacings of 6" possible only-Therefore, will use 40 studs spaced 12" o.c.

\[ W_{18X40(40)} \text{ C} = 1 \text{ lin} \]
A-8 Composite Girder Design

Material Properties
Concrete  \( f_{c} := 4000 \text{ PSI} \)  Beam Steel  \( f_{y} := 50000 \text{ PSI} \)  \( f_{u} := 65000 \text{ PSI} \)

\( E := 29000000 \text{ PSI} \)

Loads

Dead Loads
- Normal Weight Concrete  \( w_{\text{conc}} := 150 \text{ pcf} \)
- Slab (4" Deep)  \( t_{\text{slab}} := 4 \text{ in} \)
- Slab Weight (from VERCO Catalog)  \( w_{\text{slab}} := 36.6 \text{ psf} \)
- Beam Weight (estimated)  \( w_{\text{beam}} := 3.475 \text{ psf} \)
- Miscellaneous  \( w_{\text{misc}} := 4.925 \text{ psf} \)

Live Loads
- Live Load  \( w_{\text{live1}} := 100 \text{ psf} \)

Since Beams Spaced at 10' O.C.
- \( b_{\text{spacing}} := 40 \text{ ft} \)

Total Dead Load
\[
w_{\text{dead}} := \frac{(w_{\text{slab}} + w_{\text{beam}} + w_{\text{misc}})(40)}{1000} = 1.8 \text{ klf}
\]

Total Live Load
\[
w_{\text{live}} := \frac{(w_{\text{live1}})(40)}{1000} = 4 \text{ klf}
\]

Total Load (ASD)
\[w_{u} := w_{\text{dead}} + w_{\text{live}} = 5.8 \text{ klf}\]

Moment Created
- Beam Length:  \( b_{\text{length}} := 40\text{ ft} \)

Moment
\[
M_{a} := \frac{8(w_{u}b_{\text{length}}^2)}{8} = 1.16 \times 10^{3} \text{ kip ft}
\]

Effective Width
\[
\begin{align*}
\text{befl} & := \left(\frac{b_{\text{length}}}{8}\right) \cdot 2 = 10 \text{ ft} \\
\text{beff} & := \left(\frac{b_{\text{spacing}}}{2}\right) \cdot 2 = 40 \text{ ft} \\
\text{beff} & := \text{beff} = 10 \text{ ft}
\end{align*}
\]
Nominal Moment Arm

Assumed $a_{assumed} = 1$ in

$Y_{2, nom} := t_{slab} - \left( \frac{a_{assumed}}{2} \right) = 3.5$ in

From AISC Table 3-19:

Using W30X99 (have to use PNA Location 5)

$$\frac{M_n}{\Omega} = \begin{cases} 1200 \text{ ft} \cdot \text{k} > 1160 \text{ ft} \cdot \text{k} & \text{ok} \\ 1160 \text{ ft} \cdot \text{k} & \text{ok} \end{cases}$$

Moment of Inertia Required

$$I_{req} := \left( \frac{5}{384} \right) \left[ \left( \frac{w_{dead} \cdot 1000}{12} \right) \left( \frac{b_{length} \cdot 12}{E \cdot 2.5} \right) \right] = 1.43 \times 10^3 \text{ in}^4 \text{ ok}$$

Strength as Un-Shored Beam Under Wet Concrete

$$w_{beam} := 99 \text{ plf}$$

$$w_d := \left( \frac{(w_{slab} \cdot b_{spacing}) + w_{beam}}{1000} \right) = 1.563 \text{ klf}$$

$$M_d := \left( \frac{w_d b_{length}^2}{8} \right) = 312.6 \text{ ft} \cdot \text{k} \text{ ok}$$

Solving Camber (75% Dead Load)

$$deltad := \left( \frac{.75 M_d \cdot \left( b_{length}^2 \right)}{161 I_x} \right) = 0.584 \text{ in}$$

Checking Moment Arm

$$a := \left( \frac{\left( A_s \cdot f_y \right)}{[0.85 f_c \cdot (b_{eff} \cdot 12)]} \right) = 3.566 \text{ in}$$

$Y_{21} := t_{slab} - \left( \frac{a}{2} \right) = 2.217 \text{ in}$

$Y_2 := 2.5 \text{in} \quad < Y_{nom} \text{ ok}$

Shear Req'd = $Q_n := 754$ kips

Shear Stud Design-Deck Parallel-Weak

1 Stud Per Rib- Normal Wt. Concrete-3/4" DIA

$$Q_{n, stud} := 18.3 \frac{\text{kips}}{\text{stud}}$$

$$\text{Num studs} := \left( \frac{2 \cdot Q_n}{Q_{n, stud}} \right) = 82.404 \quad 82 \text{ Studs Req'd}$$

Will use 84 studs spaced 5 3/4" o.c.

$W30X99(84) := \frac{5}{8}$ in
A-9 Roof Open Web Steel Joist and Open Web Steel Joist Girder Design

Open Web Steel Joists

Loads
Dead Loads
4 Ply Felt & Gravel Roofing \( w_{\text{felt}} := 5.5 \text{ psf} \)
18 Gauge Verco W2 \( w_{\text{deck}} := 3.3 \text{ psf} \)
Miscellaneous \( w_{\text{misc}} := 1.2 \text{ psf} \)
Live Loads
Roof live Load \( w_{\text{live1}} := 20 \text{ psf} \)
Snow Loads
Roof Snow Load \( w_{\text{snow1}} := 25 \text{ psf} \)
Since Joists Spaced at 10' O.C.
\( b_{\text{spacing}} := 10 \text{ ft} \)
Total Dead Load
\( w_{\text{dead}} := \frac{[w_{\text{felt}} + w_{\text{deck}} + w_{\text{misc}}] \cdot b_{\text{spacing}}}{1000} = 0.1 \text{ klf} \)
Total Live Load
\( w_{\text{live}} := \frac{[w_{\text{live1}}] \cdot b_{\text{spacing}}}{1000} = 0.2 \text{ klf} \)
Total Snow Load
\( w_{\text{snow}} := \frac{[w_{\text{snow1}}] \cdot b_{\text{spacing}}}{1000} = 0.25 \text{ klf} \)
Total Load (ASD)
\( w_{u} := w_{\text{dead}} + w_{\text{live}} + w_{\text{snow}} = 0.55 \text{ klf} \)

Moment Created

Beam Length
\( b_{\text{length}} := 40 \text{ ft} \)
Moment
\( M_{a} := \left[ \frac{w_{u} \left( b_{\text{length}}^2 \right)}{8} \right] = 110 \text{ k-ft} \)

Joist Selected

From Joist Catalog
Select 24LH09 21 \( w_{\text{self}} := .021 \text{ klf} \)

Open Web Steel Joist Girder

Joist Spaces: 4N
Total Beam Load (ASD)
\( w_{\text{beam}} := w_{\text{dead}} + w_{\text{live}} + w_{\text{snow}} + w_{\text{self}} = 0.571 \text{ klf} \)
End Reaction
\( r_{\text{end1}} := \frac{\left( w_{\text{beam}} \cdot b_{\text{length}} \right)}{2} = 11.42 \text{ kips} \)
From Joist Catalog
Select 32G 4N \( w_{\text{selfgirder}} := .037 \text{ klf} \)
A-10 Shear Tab Calculation

Typical Detail #4 - W18X40 Beam onto HSS 10X10X1/2” Column

Material Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Wall Thickness</td>
<td>t_{column} := 0.465in</td>
</tr>
<tr>
<td>Beam Web Thickness</td>
<td>t_{web} := 0.315in</td>
</tr>
<tr>
<td>Usable Beam Web Depth</td>
<td>T := 15.5in</td>
</tr>
</tbody>
</table>

End Reactions

Dead Load Reaction

\[ P_{\text{dead}} = \left( \frac{36.6 \times 10 - 40}{2 \times 1000} \right) + \left( \frac{40 - 40}{2 \times 1000} \right) = 8.12 \text{ kips} \]

Live Load Reaction

\[ P_{\text{live}} = \left( \frac{100 \times 10 - 40}{2 \times 1000} \right) = 20 \text{ kips} \]

Total

\[ P_{\text{tot}} := P_{\text{dead}} + P_{\text{live}} = 28.12 \text{ kips} \]

Connection Selection

Using A325-N STD 3/4" Bolts ASD

n := 4

\[ L := 11.5 \text{ in} \]

Maximum Capacity=34.8 kips

\[ t_{\text{plate}} := \frac{1}{4} \text{ in} \]

WeldSize := \( \frac{3}{16} \) in

Since \( t_{\text{web}} > t_{\text{plate}} \), therefore beam web is strong enough

Max Weld Size

\[ \text{Weldmax} := t_{\text{column}} - \left( \frac{1}{16} \right) = 0.403 \text{ in} \]

\( \Rightarrow \frac{3}{16} \text{ in} \) ok

\[ \text{Leh} := 2 \left( \frac{3}{4} \right) = 1.5 \text{ in} \]

\[ a := \text{Leh} + \frac{1}{2} = 2 \text{ in} \]

\[ \text{Lev} := 1 + \frac{1}{4} = 1.25 \text{ in} \]

AISC Table J3.4
Appendix B: Project Schedule
### Appendix C: Foundation Schedule

#### Final Schedule

<table>
<thead>
<tr>
<th>Quadrant A</th>
<th>Column</th>
<th>Rounded Widths (ft)</th>
<th>q(_{\text{allowable}})</th>
<th>q(_{\text{actual}})</th>
<th>Final Sizing (ftxft)</th>
<th>d (in)</th>
<th>(A_s) (in(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>NN-7</td>
<td>8</td>
<td>5.00</td>
<td>4.84</td>
<td>8x8</td>
<td>14</td>
<td>4.44</td>
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<tr>
<td>NN-8</td>
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<td>3.93</td>
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<td>10</td>
<td>2.38</td>
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<td>NN-9</td>
<td>5</td>
<td>4.12</td>
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<tr>
<td>MM-9</td>
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<td>5.44</td>
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<td>LL-9</td>
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<td>5.51</td>
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**Retail Building**

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Appendix D- General Requirements for Shear Walls

Minimum Spacing for horizontal reinforcing, $s_{min} = \begin{cases} \frac{l_w}{5} \\ 3h \\ 18" \end{cases}$  

Section 11.9.9

$\rho_{t, min} = 0.0025$

If $V_u < \frac{\Theta V_c}{2}$, no shear wall reinforcing is needed  

Section 11.9.9.3

$V_c = 3.3 * \lambda * \sqrt{\frac{f'_c}{h}} * h + \frac{N_{ud}}{A_{lw}}$  

ACI Equation 11 - 29

Shear Reinforcing Spacing, $s = \frac{A_v f_y d}{V_s}$  

ACI Equation 11 - 31

If:

Height to Length Ratio $\leq 0.5$, vertical shear reinforcing is equal to horizontal shear reinforcing

Height to Length Ratio $> 2.5$, $\rho_t = 0.0025$

Otherwise:

$\rho_t = 0.0025 + 0.5 \left( 2.5 - \frac{h_w}{l_w} \right) \left( \rho_h - 0.0025 \right)$  

ACI Equation 11 - 32

Minimum Spacing for vertical shear reinforcing, $s_{min} = \begin{cases} \frac{l_w}{3} \\ 3h \\ 18" \end{cases}$  

Section 11.9.9.5
## Appendix E - Shear Wall Forces

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## Appendix F- Design Value Calculations

### Live Loads

#### Floor Load

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#### Garage

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### Roof Load

#### Retail

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### Snow Loads

#### Retail and Garage

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<td>Table 7-3</td>
<td>Thermal Factor</td>
<td>$C_t = 0.9$</td>
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<td>Table 7-2</td>
<td>Exposure Factor</td>
<td>$C_e = 1$</td>
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<td>Ground Snow Load</td>
<td>$p_g = 39$ psf</td>
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<td>ASCE 7-05</td>
<td>Table 7-4</td>
<td>Exposure Category B</td>
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<tr>
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<td>Table 7-1</td>
<td>Roof Snow Load</td>
<td>$p_r = 30$ psf</td>
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### Wind Load

#### Retail

<table>
<thead>
<tr>
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<th>Table/Section</th>
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<th>Calculation</th>
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<td>Exposure Category</td>
<td>B</td>
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<td>Table 6-2</td>
<td>Occupancy Category</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Exposure Category B, mean roof height 35 ft</td>
<td>$\lambda_b = 1.05$</td>
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<td>Building not located on a hil, ridge. Unexposed</td>
<td>$K_{zt} = 1$</td>
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<td>Figure 6-2</td>
<td>Horizontal Pressure</td>
<td>$p_{S30} = 11.5$ psf</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Vertical Pressure</td>
<td>$p_{S30} = -6.1$ psf</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Adjusted Horizontal Pressure</td>
<td>$p_{net} = 12$ psf</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Adjusted Vertical Pressure</td>
<td>$p_{net} = -6$ psf</td>
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#### Garage

<table>
<thead>
<tr>
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<th>Table/Section</th>
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<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Exposure Category B, mean roof height 35 ft</td>
<td>$\lambda_b = 1.09$</td>
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<td>Figure 6-2</td>
<td>Horizontal Pressure</td>
<td>$p_{S30} = 11.5$ psf</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Vertical Pressure</td>
<td>$p_{S30} = -6.1$ psf</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Adjusted Horizontal Pressure</td>
<td>$p_{net} = 13$ psf</td>
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<tr>
<td>ASCE 7-05</td>
<td>Figure 6-2</td>
<td>Adjusted Vertical Pressure</td>
<td>$p_{net} = -7$ psf</td>
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</table>
### Seismic Design Criteria

**Retail**

<table>
<thead>
<tr>
<th>ASCE 7-05</th>
<th>Table 11.6-1</th>
<th>Seismic Design Category</th>
<th>$S_{DC}$</th>
<th>C</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Seismic Force Resisting System</td>
<td>North-South</td>
<td>Ordinary Reinforced Masonry Shearwalls</td>
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<tr>
<td></td>
<td></td>
<td>Seismic Force Resisting System</td>
<td>East-West</td>
<td>Eccentrically Braced Frame with Non-moment resisting connections</td>
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<table>
<thead>
<tr>
<th>IBC</th>
<th>Table 1613.5.2</th>
<th>Site class (Provided by GeoEngineers Report)</th>
<th>Site Class: D</th>
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<tbody>
<tr>
<td>ASCE 7-05</td>
<td>Fig 22-1</td>
<td>Spectral Response Acceleration (Short Period)</td>
<td>$S_s = 0.400$</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Fig 22-2</td>
<td>Spectral Response Acceleration (Second Period)</td>
<td>$S_s = 0.100$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Table 11.4-1</td>
<td>Site Coefficient</td>
<td>$F_a = 1.480$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Table 11.4-2</td>
<td>Site Coefficient</td>
<td>$F_v = 2.400$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Eqn (11.4-1)</td>
<td>MCE Spectral Response Acceleration for Short Periods</td>
<td>$S_{Ms} = 0.592$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Eqn (11.4-2)</td>
<td>MCE Spectral Response at 1sec</td>
<td>$S_{M1} = 0.240$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Eqn (11.4-3)</td>
<td>Spectral Response Acceleration for Short Periods</td>
<td>$S_a = 0.395$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Eqn (11.4-4)</td>
<td>Spectral Response at 1sec</td>
<td>$S_{D1} = 0.160$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Eq. 12.8-7</td>
<td>Period of Steel (E/W)</td>
<td>$T_{steel} = 3.300$ sec</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Eq. 12.8-8</td>
<td>Period of CMU (N/S)</td>
<td>$T_{concrete} = 4.950$ sec</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Sect. 11.4.5</td>
<td>$0.2(S_{D1}/S_{DS})$</td>
<td>$T_0 = 0.081$ sec</td>
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<tr>
<td>ASCE 7-05</td>
<td>Sect. 11.4.6</td>
<td>$(S_{D1}/S_{DS})$</td>
<td>$T_s = 0.405$ sec</td>
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<tr>
<td>ASCE 7-05</td>
<td>Fig. 22-15</td>
<td>Long Transition Period</td>
<td>$T_l = 16.000$ sec</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Eq. 11.4-7</td>
<td>$S_a = [S_{D1}<em>T_l]/(T</em>T)$ ; $T &gt; T_0, T &lt; T_l$</td>
<td>$S_a_{steel} = 0.235$</td>
</tr>
<tr>
<td>ASCE 7-05</td>
<td>Eq. 11.4-8</td>
<td>$S_a = [S_{D1}<em>T_l]/(T</em>T)$ ; $T &gt; T_0, T &lt; T_l$</td>
<td>$S_a_{concrete} = 0.105$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Table 11.5-1</td>
<td>Importance Factor</td>
<td>$I = 1.000$</td>
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<tr>
<td>ASCE 7-05</td>
<td>Table 11.6-1</td>
<td>Seismic Design Category</td>
<td>$0.33 &lt; S_{DS} &lt; 0.5$ Design Category C</td>
</tr>
</tbody>
</table>

### Retail System

<table>
<thead>
<tr>
<th>CMU</th>
<th>Steel</th>
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<tbody>
<tr>
<td>2</td>
<td>7</td>
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</table>

**Note:** The seismic response coefficient is taken as Eq. 12.8-2. It will not exceed Eq. 12.8-5 nor less than 12.8-3.

**Note:** For design, considering the retail can have multiple uses, the larger $L_0$ value is used.
Appendix G- Pour Schedule
Appendix H- Structural Drawings
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S12.1 FOUNDATION PLAN
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S22.1 FOUNDATION PLAN
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S26.1 CONCRETE SLAB PLACEMENT SCHEDULE
1. Typical Beam to Outer Column
2. Typical Girder to Outer Column
3. Typical Girder to Inner Column
4. Typical Beam to Inner Column
5. Typical Beam to Girder
6. Typical Girder to CMU Column
7. Typical 2nd Floor Diaphragm to CMU Wall
8. Typical Roof Diaphragm to CMU Wall
9. Typical Inner Column Baseplate
10. Typical Outer Column Baseplate

Typical Details
Scale: 3/8" = 1'-0"
S11.1