1161 1st Avenue

Design Report

BCMK Project Engineers – Group 1

Mari Kobakhidze          Project Manager
Bryan Kwiatkowski        BIM Manager
Carlos Peña Del Valle    Structural Engineer
Kevin Tapia              Construction Manager/Assistant BIM Manager
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About BCMK

BCMK Engineering is the multi-functional engineering company that specializes in the design and construction of medium to high-rise office buildings. The company was established in 2018 and since then has obtained the respect of the construction field. BCMK provides various services to the clients. Starting from architectural and structural design, the company also provides construction management services. The team of highly qualified Professional Engineers have been working on many challenging projects and have proved that any structural design and construction project is possible.

Project Location

The building at 1161 1st Avenue is one of the priority projects for BCMK Engineering. The construction site is located on the Upper East Side of the Manhattan in New York City, having the following coordinates: Latitude: N 40.7625° and Longitude W 73.9603° (See Figure 1). The site is located in a densely populated area of New York City, between 1st and 2nd Avenues from east to west, and 64th and 63rd Streets from north to south, respectively. The area is a very popular location amongst developers and hosts many office buildings as well as luxurious residential condominiums.
Figure 1: Location of the Construction Site at 1161 1st Avenue
Project Purpose

The purpose of the project is to provide flexible office and commercial spaces to the tenants. It is our professional responsibility to our client to have the project completed with minimum interruptions and delays, and to finish the project within the given schedule. It is also our ethical responsibility to deliver a structurally sound and safe building.

Project Scope

For this project BCMK Engineering was given specific site conditions and requirements to fulfill. In regard to the site conditions, the top 25 feet of soil consists of well graded sand (SW), an allowable bearing capacity of 12,000 pounds per square foot, and a percolation rate of 4 inches per hour. Underneath the sand layer there is also a rock layer that has an allowable bearing capacity of 24,000 pounds per square foot.

As for the building itself, BCMK Engineering has decided to design an office building using steel as the primary structural material. According to our clients, the requirements for the building project include one building structure with a minimum footprint of 12,000 square feet and minimum of 16 stories. Additionally, setbacks on the top two stories must also be provided for all sides of the building and two underground levels have to be used for car parking. The total number of parking for both floors have to reach a minimum of 60. Other requirements for this project include: a green roof system to be investigated and proposed, an energy exterior wall system must be implemented, and the building shall house tenant amenities.

The deliverables for this project will be split into different phases that will include drawings and specifications where applicable. Our first phase of the project will consist of the architecture of our building. The architectural plans will include the layout for typical floors, amenities provided, design of means of egress, all in accordance with the International Building Code.

Afterward, the next phase will consist of the framing layout, detailing our structural system for both gravity and lateral loads. The following phase will include an analysis of the building codes corresponding to our project along with calculations of all gravity and lateral loads applicable using structural analysis. The analysis and design of our project will be performed using computer aided structural analysis and design software. In order to verify results from said software, sample calculations will be provided. Drawings such as a foundation plan, a footing
schedule, foundation details, all floor framing plans, etc… is the next phase once all the calculations are complete. This phase ultimately will provide all of our submission drawings upon completion. As requested by the owner, the last phase of the project will consist of an engineering cost estimate as well as a construction schedule for our project.

1.0 Architectural Design

Following section describes the architectural design and the inspiration of the building layout as well as reviews each typical floors to have better overview of the building.

1.1 Architectural Design Inspiration

The architectural design for 1161 1st Avenue was predominantly influenced by the location of the proposed building site. Given that the site is located on the upper east side of Manhattan, in close proximity to the East River, as reflected in Figure 1, the building was designed to have multiple terraces at different elevations with a direct line of sight to the East River, and the surrounding Manhattan skyline. Moreover, to optimize the usage of the available terrace and roof space, a green terrace and roof system was implemented, that seamlessly doubles as a rain water retention system, and an aesthetic non-structural component for all building occupants. Additionally, the architectural design was inspired by asymmetrical office buildings previously designed such as 2 World Trade Center in New York City. The building design finds balance while maintaining a state of imbalance from certain geographical perspectives. Ultimately, the goal of the architectural design is to create an environment that is favorable to the recurring occupants who will spend most of their time working in the building, by optimizing productivity and the quality of the work environment.

1.2 Architectural Layout Description

The initial shape of the building in terms of geometry consists of four square corners attached to a rectangular core (for levels 2-6). The layout of these floors will consist of having rentable office space within these four corners while the core of the building contains the utilities. Once you’ve reached the 7th level containing the first few terraces the layout changes to incorporate these terraces where the setbacks of the building have been decided. In terms of the geometry, there is a shift from having four equal squares for rentable space to only two with L shaped terraces to replace the lost office space. After the first setback the shape of the building changes (for levels
8-14) to having only two corner squares attached to a rectangular core. Similar to levels 2-6 these square corners will still be the designated areas for rentable office space with utilities being placed at the rectangular core. For level 15 the layout changes to incorporate the terraces. Lastly, the second setback changes the geometry of the building to a simple square to fit the roof and bulkhead (levels 16-17).

1.2.1 Ground Floor

The ground floor of the building is multifunctional. Two main entrances to the building are located on the east and west wings of the building. The security desk and personnel are provided in the lobby by the main entrances. Office employees need to go through security turnstiles in order to access the elevators and get to their offices. The ground floor also provides amenities to the building tenants as well as to public users. The north part of the floor layout, around 7,200 square feet of space, is leased to a gymnasium and the south part, 5,000 square feet of spaces is used for a restaurant. Ground floors leads to the typical floors above and two (2) cellar levels below.

1.2.2 Typical Floors

Typical floors are mainly consisting of office spaces. Around 17,500 square feet of open space, provides flexibility for tenants to arrange their office or commercial layout the way they would like. On a typical floor from levels 2-6 the total amount of rentable space reaches 17,468 square feet. From levels 8-14 the amount of rentable space is reduced to 11,686 square feet due to the setback. For terraces on level 7 and 15, the square footage provided for rentable space is 11,868 square feet and 3,417 square feet respectively.

1.2.3 Green Roof

Green roofs will be made accessible for tenant use on the levels that contain a terrace. For our project these levels include floors 7, 15, and 17. The green roofs will be accessible directly from the office via doors that open onto the roof sections. Tenants will be able to utilize these terraces during their hours of operations. Each one of these green roofs will be designed to have vegetation that cover the majority of the terrace with narrow walkways for tenants to enjoy. For aesthetic purposes this vegetation was placed symmetrically on each terrace. To comply with the
code and ensure the safety of the tenants, parapets were also placed along the border of the terraces and roof sections.

### 1.2.4 Cellar Levels and Parking layout

The cellar levels of the building will consist of a two-story garage with the entrance being accessible through a ramp on the ground level. In total these two floors will have 92 available parking spots with 4 of these spots being handicapped, as well as 12 motorcycle parking spots. For tenant convenience, the handicap parking spots are located on the first cellar level and close to the center of the building. The layout of the parking consists of having the majority of parking around the edges of the building while the handicap parking is situated closer to the center of the building. From the center of both cellar levels, tenants will be able to access the elevators and staircases. For each cellar level of the building the ADA requirements were met.

### 1.3 Means of Egress

The means of egress for our building are in accordance with Chapter 10 of the New York City Building Code (2014 edition). To satisfy the requirements of the code, two stair cases are provided on every floor with access to the roof of the building via both staircases. These stair cases have been placed where all occupants of the building are within 100 ft of access to them. Additionally, the code calls for interior stairways with a minimum of 2-hr fire resistance rating, which have been provided. Five elevators have been provided within the center of the building along with a single service elevator, which have been placed on all floors of the building. In terms of access to building, two revolving doors have been placed on both the east and west side of the building. Once occupants are inside there are security turnstiles placed equidistant from the revolving doors to access the elevators.

### 2.0 Codes and Manuals

The design is governed by the local building code issued in 2014: New York City Building Code (NYCBC 2014). The code is a local version of International Building Code (IBC) issued in 2012. The NYCBC provides slight changes to the IBC 2012 in order to fit its standards to the New York City requirements. The design loads are obtained from ASCE 7-10: Minimum Design Loads for Buildings and Other Structures. The code dictates the loads that need to be applied to the
building based on its risk category. According to ASCE 7-10, Table 1.5-1 the building is categorized as Risk Category II. NYCBC 2014 classifies our building as Business Group B (Section 304.1).

The structural design will be guided by the Steel Construction Manual: AISC 360-10. As well as AISC Design Guides Series and AISC Design Example-Version 14. The manual will simplify the design process by providing tabulated values for the steel components used to frame the building. A list of the codes to be used are listed below:

- NYCBC 2014 (New York City Building Code)
- ASCE 7-10 (Minimum Design Loads for Buildings and Other Structures)
- ICC A117.1 (Accessible and Usable Buildings and Facilities)
- AISC 360-10 (Steel Construction Manual)
- AISC Design Example, Version 14
- AISC Steel Design Guide 5
- AISC Steel Design Guide 3

3.0 Load Calculations

The two main types of loading acting on the building are gravity loads, such as the self-weight of materials and lateral loads, such as wind and seismic loads. Moreover, gravity and lateral loads can further be sub-categorized into dead loads (DL), superimposed dead loads (SDL) and live loads (LL). This section quantifies the loading for each of these sub-categories, which cumulatively represents the total loading on the structural system.

3.1 Dead Loads (DL)

Dead loads represent the loading from the self-weight of the structural system. For the proposed building structure, this consists of the self-weight of beams, columns, composite metal deck and foundation. The surface area dead loads were determined for unique bays. There are three distinct bay arrangements or the proposed structural framing system, illustrated in the following Figures. Then, Table 1 depicts the calculated values for the surface area dead loads of each unique bay. Sample calculations for the dead load of a typical bay are attached in the Appendix A: Dead-Load Sample Calculations.
Table 1: Surface Area Dead Load for Bays

<table>
<thead>
<tr>
<th>Bay Type</th>
<th>DL (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical</td>
<td>7.47</td>
</tr>
<tr>
<td>Elevator (Core)</td>
<td>9.10</td>
</tr>
<tr>
<td>Stair (Core)</td>
<td>9.24</td>
</tr>
</tbody>
</table>

Figure 2: Typical Bay

Figure 3: Elevator (Core) Bay
3.2 Live Loads ($LL$)

Live loads are moving loads that will be applied onto the structure after the installation of structural components, which is largely based on the occupants of the building. Since the tenants of the building and the final architectural arrangement of individual office floors are unknown, the live loads for different occupancies were obtained in accordance with Table 4.1 in ASCE 7-10, as well as Table C4.1 in the commentary section of code. The occupancy loads representative of the proposed building design are tabulated in Table 2.
Table 2: Occupancy Live Loads

<table>
<thead>
<tr>
<th>Occupancy</th>
<th>LL (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevator Equipment</td>
<td>150</td>
</tr>
<tr>
<td>Gym</td>
<td>100</td>
</tr>
<tr>
<td>IT Room</td>
<td>150</td>
</tr>
<tr>
<td>Kitchen</td>
<td>150</td>
</tr>
<tr>
<td>Lobby</td>
<td>100</td>
</tr>
<tr>
<td>Mechanical</td>
<td>125</td>
</tr>
<tr>
<td>Office</td>
<td>50</td>
</tr>
<tr>
<td>Parking</td>
<td>40</td>
</tr>
<tr>
<td>Restaurant</td>
<td>100</td>
</tr>
<tr>
<td>Restroom</td>
<td>60</td>
</tr>
<tr>
<td>Roof (Green)</td>
<td>100</td>
</tr>
<tr>
<td>Roof (Mechanical)</td>
<td>125</td>
</tr>
<tr>
<td>Roof (Ordinary)</td>
<td>20</td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
</tr>
<tr>
<td>Storage</td>
<td>125</td>
</tr>
<tr>
<td>Terrace (Green)</td>
<td>100</td>
</tr>
</tbody>
</table>

3.3 Superimposed Dead Loads (SDL)

Superimposed dead loads are additional dead loads added onto the structural system; superimposing dead loads on the current dead loads present. This type of dead load commonly arises from the installation of non-structural components of the building such as the addition of the sprinkler/lighting system of the building, or the addition of flooring and ceiling finishes. For the proposed building structure, the superimposed dead loads were determined according to the materials specified in the architectural plans, and using Table C3.1 and Table C3.2 in ASCE 7-10, as well as specifications and details from selected companies.

The superimposed dead loads were determined for three unique regions in the building floor plan that had significant SDL variations. The first region is for a typical floor layout, such as office space. Then, the second region is for a floor layout with mechanical, IT, storage occupancies, which requires additional equipment to accommodate the usage of the space. Lastly, the third region word is for a floor layout with a green space, which also requires additional equipment to accommodate the usage of the space. Breakdowns of the individual loadings for each unique region are tabulated in Table 3 to Table 5.
Table 3: Superimposed Dead Loads (Typical)

<table>
<thead>
<tr>
<th>Materials</th>
<th>SDL (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Fill Finish (2”)</td>
<td>1.17</td>
</tr>
<tr>
<td>Liquid Applied Membrane</td>
<td>0.05</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>10</td>
</tr>
<tr>
<td>Ceiling</td>
<td>10</td>
</tr>
<tr>
<td>Sprinkler system</td>
<td>3</td>
</tr>
<tr>
<td>Lighting system</td>
<td>4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>35</strong></td>
</tr>
</tbody>
</table>

Table 4: Superimposed Dead Loads (Mechanical/IT/Storage)

<table>
<thead>
<tr>
<th>Materials</th>
<th>SDL (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Fill Finish (2”)</td>
<td>1.17</td>
</tr>
<tr>
<td>Liquid Applied Membrane</td>
<td>0.05</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>10</td>
</tr>
<tr>
<td>Ceiling</td>
<td>10</td>
</tr>
<tr>
<td>Sprinkler system</td>
<td>3</td>
</tr>
<tr>
<td>Lighting system</td>
<td>4</td>
</tr>
<tr>
<td>IT/Mechanical/Storage</td>
<td>15</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>50</strong></td>
</tr>
</tbody>
</table>

Table 5: Superimposed Dead Loads (Green)

<table>
<thead>
<tr>
<th>Materials</th>
<th>SDL (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Fill Finish (2”)</td>
<td>1.17</td>
</tr>
<tr>
<td>Liquid Applied Membrane</td>
<td>0.05</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>10</td>
</tr>
<tr>
<td>Green Roof</td>
<td>29</td>
</tr>
<tr>
<td>Ceiling</td>
<td>10</td>
</tr>
<tr>
<td>Sprinkler system</td>
<td>3</td>
</tr>
<tr>
<td>Lighting system</td>
<td>4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>60</strong></td>
</tr>
</tbody>
</table>

The final SDL corresponding to individual occupancies of the proposed building structure, based on region specific loading, depicted in the aforementioned tables, is reflected in Table 6.
3.4 Snow Loads (S) and Snow Drift

Snow loads that act on a structure are loads that are caused by the self-weight of the snow that settles on top of flat exposed portions of the roof. The snow loads were calculated in accordance with the NYCBC 2014, predominantly using the ASCE 7-10 design manual. The relevant snow load and drift information is tabulated in Table 8. Table 7 shows site and building parameters corresponding to snow loads that were obtained from ASCE 7-10.

<table>
<thead>
<tr>
<th>Description</th>
<th>Coefficient</th>
<th>Values</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category</td>
<td>$-$</td>
<td>III</td>
<td>ASCE 7-10 Table 1.5-1</td>
</tr>
<tr>
<td>Exposure Factor</td>
<td>$C_e$</td>
<td>1.0</td>
<td>ASCE 7-10 Table 7-2</td>
</tr>
<tr>
<td>Thermal Factor</td>
<td>$C_t$</td>
<td>1.0</td>
<td>ASCE 7-10 Table 7-3</td>
</tr>
<tr>
<td>Roof Slope Factor</td>
<td>$C_s$</td>
<td>1.0</td>
<td>ASCE 7-10 Figure 7-2a</td>
</tr>
<tr>
<td>Importance Factor</td>
<td>$I_s$</td>
<td>1.1</td>
<td>ASCE 7-10 Table 1.5-2</td>
</tr>
<tr>
<td>Ground Snow Load</td>
<td>$p_g$</td>
<td>25 psf</td>
<td>NYCBC 2014 Ch. 16</td>
</tr>
</tbody>
</table>

Table 8 shows the calculated snow load and snow drift based on the parameters in Table 7, which were used in the design of the building. Sample calculations for the snow load and
snowdrift parameters are shown in Appendix B: Snow Loads and Snow Drift Sample Calculations. Moreover, it is also relevant to note that since the ground snow load was greater than 20 psf, a rain on surcharge snow load does not have to be considered in the design of the proposed building, in accordance with ASCE 7-10.

### Table 8: Snow Load and Snow Drift Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Coefficient</th>
<th>Units</th>
<th>Leeward</th>
<th>Windward</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Roof Snow Load</td>
<td>$p_f$</td>
<td>(psf)</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>Balanced Snow Load Height</td>
<td>$h_b$</td>
<td>(ft)</td>
<td>1.16</td>
<td></td>
</tr>
<tr>
<td>Height of Snow Drift</td>
<td>$h_d$</td>
<td>(ft)</td>
<td>1.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Max. Intensity of Drift Surcharge</td>
<td>$(p_d)_{lon}$</td>
<td>(psf)</td>
<td></td>
<td>30.2</td>
</tr>
</tbody>
</table>

### 3.5 Wind Loads ($W$)

Wind loads were calculated and modelled following the procedure specified in ASCE 7-10 Chapters 26 and 27, in conjunction with the NYCBC 2014. The following parameters shown in were used to define the wind load pattern in the ETABS model, as well as verify the applied loads from ETABS using ASCE 7-10 Chapter 27. The sample calculations for the first two-wind load case step number are shown in Appendix D: Wind Load Calculations. Following Table 9 is a plot comparing the applied pressure on the building for the wind load case step numbers 1 and 2 from ETABS and ASCE 7-10 sample verification calculations.

### Table 9: Wind Load Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Variable</th>
<th>Value</th>
<th>Units</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic Wind Speed</td>
<td>$V$</td>
<td>124</td>
<td>[mph]</td>
<td>NYCBC 14 Sec.1609.3</td>
</tr>
<tr>
<td>Building Risk Category</td>
<td>$-$</td>
<td>II</td>
<td>$-$</td>
<td>NYCBC 14 Table 1604.5</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>$-$</td>
<td>C</td>
<td>$-$</td>
<td>NYCBC 14 Fig. 1609.4.3</td>
</tr>
<tr>
<td>Mean Roof Height</td>
<td>$h$</td>
<td>220</td>
<td>[ft]</td>
<td>$-$</td>
</tr>
<tr>
<td>Gust Effect Factor</td>
<td>$G_f$</td>
<td>0.89</td>
<td>$-$</td>
<td>ASCE 7-10 Sec. 26.9.5</td>
</tr>
<tr>
<td>Wind Directionality Factor</td>
<td>$K_d$</td>
<td>0.85</td>
<td>$-$</td>
<td>ASCE 7-10 Table 26.6-1</td>
</tr>
<tr>
<td>Wind Topographic Factor</td>
<td>$K_{zt}$</td>
<td>1.0</td>
<td>$-$</td>
<td>ASCE 7-10 Sec. 26.8.2</td>
</tr>
<tr>
<td>Gradient Height</td>
<td>$z_g$</td>
<td>900</td>
<td>[ft]</td>
<td>ASCE 7-10 Table 26.9-1</td>
</tr>
<tr>
<td>Internal Pressure Coefficient</td>
<td>$G_{Cp}$</td>
<td>0.18</td>
<td>$-$</td>
<td>ASCE 7-10 Table 26.11-1</td>
</tr>
</tbody>
</table>
Figure 5: Story Wind Applied Force (Step 2: X-Direction)

Figure 6: Story Wind Applied Force (Step 2: Y-Direction)
3.6 Seismic Loads ($E$)

The Seismic load effects on the building was taken into the consideration. ASCE 7-10, Chapter 11 and 12, guided the design. The analyses procedure was chosen to be Equivalent Lateral Force Analysis (ASCE 7-10 Table 12.6-1). This method lumps the story mass together and based on base shear ($V$) and seismic response coefficient ($C_s$) distributes it on each story. Based on the Table 1.5-1 of ASCE 7-10, the building is classified as Risk Category II. Because the properties of the soil was not given to determine the site class, based on Section 11.4.2 the site class is D, since there is no geotechnical research showing it to be either E, or F. It is also has to be noted that, because we are not designing for special seismic detailing, the Seismic force resisting system form Table 12.2-1 from ASCE 7-10, was chosen to be $H$: Steel Systems not specifically detailed for seismic resistance. The parameters that were used in the design are as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Variable</th>
<th>Value</th>
<th>Units</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Risk Category</td>
<td>$-$</td>
<td>II</td>
<td>$-$</td>
<td>ASCE 7-10 Table 1.5-1</td>
</tr>
<tr>
<td>Site Class</td>
<td>$-$</td>
<td>D</td>
<td>$-$</td>
<td>ASCE 7-10 Sec. 11.4.2</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>$-$</td>
<td>C</td>
<td>$-$</td>
<td>ASCE 7-10 Sec. 11.6-1,2</td>
</tr>
<tr>
<td>Seismic Force Resisting System</td>
<td>$-$</td>
<td>H</td>
<td>$-$</td>
<td>ASCE 7-10 Table 12.2-1</td>
</tr>
<tr>
<td>Response Modification factor</td>
<td>$R$</td>
<td>3</td>
<td>$-$</td>
<td>ASCE 7-10 Table 12.2-1</td>
</tr>
<tr>
<td>Over strength Factor</td>
<td>$\Omega_0$</td>
<td>3</td>
<td>$-$</td>
<td>ASCE 7-10 Table 12.2-1</td>
</tr>
<tr>
<td>Deflection Amplification Factor</td>
<td>$C_d$</td>
<td>3</td>
<td>$-$</td>
<td>ASCE 7-10 Table 12.2-1</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>$I_e$</td>
<td>1</td>
<td>$-$</td>
<td>ASCE 7-10 Table 1.5-1</td>
</tr>
<tr>
<td>Site Coefficient</td>
<td>$F_a$</td>
<td>1.57</td>
<td>$-$</td>
<td>NYCBC 14 Table 1613.5.3 (1)</td>
</tr>
<tr>
<td>Site Coefficient</td>
<td>$F_v$</td>
<td>2.4</td>
<td>$-$</td>
<td>NYCBC 14 Table 1613.5.3 (2)</td>
</tr>
<tr>
<td>Building Height</td>
<td>$S_s$</td>
<td>0.281</td>
<td>$g$</td>
<td>NYCBC 14 Table 1613.5.1</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>$S_1$</td>
<td>0.073</td>
<td>$g$</td>
<td>NYCBC 14 Table 1613.5.1</td>
</tr>
<tr>
<td>Mapped long-period transition</td>
<td>$T_L$</td>
<td>6</td>
<td>$s$</td>
<td>NYCBC 14 Table 1613.5.1</td>
</tr>
<tr>
<td>Approximate Fundamental period</td>
<td>$T_a$</td>
<td>1.9</td>
<td>$s$</td>
<td>ETABS</td>
</tr>
<tr>
<td>Redundancy factor</td>
<td>$\rho$</td>
<td>1</td>
<td>1</td>
<td>ASCE 7-10 Sec. 12.4.2.1</td>
</tr>
</tbody>
</table>

The important parameters to calculate for the equivalent load determination are the spectral response acceleration parameter for short periods ($S_{MS}$) and at 1 second ($S_{M1}$), which then gives
the values of the design spectral response acceleration parameter at short periods \((S_{DS})\) and the design spectral response acceleration parameter at 1-s period \((S_{D1})\):

\[
S_{MS} = F_a S_S \\
S_{M1} = F_v S_1 \\
S_{DS} = \frac{2}{3} S_{MS} \\
S_{D1} = \frac{2}{3} S_{M1}
\]

ASCE 7-10, Eq. 11.4-1

ASCE 7-10, Eq. 11.4-2

ASCE 7-10, Eq. 11.4-3

ASCE 7-10, Eq. 11.4-4

After the parameters are calculated, the following formulas are used to determine the base shear \((V)\) and earthquake load effect \((E)\). From ASCE 7-10, Table 11.6-1 we see that for \(0.167g \leq S_{DS} < 0.33g\) and \(0.067 g \leq S_{D1} < 0.133g\). For the Risk category II, we can determine that the building belongs to Seismic Design Category B, But for the academic purposes, it will be designed for Seismic Design Category C.

\[
C_s = \frac{S_{DS}}{T_e} \quad \text{ASCE 7-10, Eq. 12.8-2}
\]

\[
C_{vX} = \frac{W_i h_i^b}{\sum_i W_i h_i^b} = 0.044 \quad \text{ASCE 7-10, Eq. 12.8-12}
\]

\[
V_b = C_e W \quad \text{ASCE 7-10, Eq. 12.8.3}
\]

The value of seismic response coefficient was chosen based on the limits given in the ASCE 7-10 Section 12.8-3 and 12.8-5:

\[
C_s = \frac{S_{D1}}{T\left(\frac{S_{DS}}{T_e}\right)} \quad T_a < T, \quad C_s = 0.044 S_{DS} I_e \geq 0.01.
\]

The base shear obtained during the calculations was \(V=414.13\) kip, using the seismic importance coefficient of 0.0204. Table 11 lists the calculated values for the equivalent load analyses. The height of the building was taken from the ground level \((h=220\) ft). Underground levels where not included in the calculations.
Table 11: Calculated Values for Equivalent Lateral Force Analyses

<table>
<thead>
<tr>
<th>Description</th>
<th>Variable</th>
<th>Value</th>
<th>Units</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height</td>
<td>$S_{MS}$</td>
<td>0.441</td>
<td>g</td>
<td>ASCE 7-10 Eq. 11.4-1</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>$S_{M1}$</td>
<td>0.175</td>
<td>g</td>
<td>ASCE 7-10 Eq. 11.4-2</td>
</tr>
<tr>
<td>Mapped long-period transition</td>
<td>$S_{DS}$</td>
<td>0.294</td>
<td>g</td>
<td>ASCE 7-10 Eq. 11.4-3</td>
</tr>
<tr>
<td>Approximate Fundamental period</td>
<td>$S_{D1}$</td>
<td>0.117</td>
<td>g</td>
<td>ASCE 7-10 Eq. 11.4-4</td>
</tr>
<tr>
<td>Seismic Response coefficient</td>
<td>$C_s$</td>
<td>0.0204</td>
<td></td>
<td>ASCE 7-10 Eq. 12.8-3</td>
</tr>
<tr>
<td>Base Shear</td>
<td>$V_b$</td>
<td>414.13</td>
<td>kips</td>
<td>ASCE 7-10 Eq. 12.8-3</td>
</tr>
<tr>
<td>Vertical Seismic Load Effect</td>
<td>$E_v$</td>
<td>1191.3</td>
<td>kips</td>
<td>ASCE 7-10 Eq. 12.8-4</td>
</tr>
<tr>
<td>Horizontal Seismic Load Effect</td>
<td>$E_h$</td>
<td>414.1</td>
<td>kips</td>
<td>ASCE 7-10 Eq. 12.4-3</td>
</tr>
<tr>
<td>Vertical Seismic Load Effect (with over strength)</td>
<td>$E_{m,h}$</td>
<td>1242.4</td>
<td>kips</td>
<td>ASCE 7-10 Eq. 12.4-7</td>
</tr>
</tbody>
</table>

The hand calculated values compared to ETABS values were very similar. Table 12 shows the percentage errors calculated for ETABS values and hand calculations.

Table 12: Comparison of obtained values for seismic analyses

<table>
<thead>
<tr>
<th>Results comparison</th>
<th>ETABS</th>
<th>Hand calculations</th>
<th>% error</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_s$</td>
<td>0.02045</td>
<td>0.02045</td>
<td>0.00</td>
</tr>
<tr>
<td>Effective Weight (kip)</td>
<td>19934.785</td>
<td>20252.65</td>
<td>1.57</td>
</tr>
<tr>
<td>Base Shear (kip)</td>
<td>407.668</td>
<td>414.13</td>
<td>1.56</td>
</tr>
</tbody>
</table>

*Figure 7 shows the lateral forces distributed on each floor. The results were obtained from hand calculations. Which is shown in Appendix C: Seismic Load Calculations in the attached Excel file.*
Figure 7: Equivalent Lateral forces distributed on each story
4.0 Structural System

The system that will be able to withstand the loads, and safely transfer it down to the foundation is referred to as a structural system. There are two main types of system: Gravity Force Resisting System (GFRS) and Lateral Force Resisting System (GFRS).

4.1 Gravity Force Resisting System (GFRS)

The structural system of the proposed building is divided into a Gravity Force Resisting System (GFRS) and a Lateral Force Resisting System (LFRS) where certain loads will transfer to one or more structural components. For this building structure, the GFRS is comprised of all structural members, primarily:

- Beams (i.e. ordinary, transfer, and filler beams)
- Trusses
- Columns
- Composite metal decks
- Foundation

The load path of the GFRS commences at the composite deck, which resists gravity loads from expected occupancies, as well as the self-weight of structural and non-structural components. These loads are then transferred into the beams and girders, which then transfer the load into columns. Afterwards, the load propagates through the columns to the foundation. From the foundation, the gravity loads are dispersed into stable bedrock under the surface of the Earth.

4.2 Lateral Force Resisting System (LFRS)

The Lateral Force Resisting System (LFRS) refers to the structural components resisting lateral loads, primarily from wind and seismic loads. For 1161 1st Avenue, the LFRS is comprised of and inverted-V chevron braced frames, moment frames, and diaphragm action provided by the composite deck. The braced frames have been oriented such that they resist the lateral loads applied in the x-direction, and the moment frames have been oriented such that they resist the lateral loads applied in the y-direction. An inverted-V chevron braced frame was used because of its high efficiency and economic value, particularly because of its high stiffness. However, the chevron
brace is limited architecturally because it lacks spaces for openings because of the obstruction presented by the braces. Consequently, the chevron braces were oriented such that they did not conflict with any of the openings in the core. Then, since the other direction had architectural limitations (i.e. openings), moment frames were used to take the lateral loads, because they could accommodate for openings. However, using moment frames also implied a reduction in stiffness. This was accounted for by adding more moment frames to best mitigate lateral loads.

The lateral load path commences at the point of application between the load and the building structure. If the lateral load is directly applied to a structural member that is part of the LFRS, then the load will be directly transferred from the lateral force resisting system down towards the foundation. If the lateral load is not directly applied to a structural member that is part of the LFRS, then the load is transferred through the diaphragm action of the slab to the nearest and stiffest structural member of the LFRS. Lateral load transfer through diaphragm action also occurs during the out-of-plane irregularities, where the lateral load needs to be transferred from one structural component that is part of the LFRS, to the nearest and stiffest structural members. Then, all later loads eventually transfer down to the foundation, ending either in footings or in concrete piers that end at wall footings. The LFRS for this building structure is shown in Figure 8.

Figure 8: LFRS Decomposition
5.0 Global Analysis and Performance

The analysis of the global performance describes how building behaves when it is exposed to the different kinds of loading. It checks the strength of the structure globally by observing the parameters of the building’s modal parameters, interstory drift, P-∆ effect, and panel zone deformations.

5.1 Modal Response

An important characteristic of a building structure is the modal response of the building, which captures the dynamic performance of the structure. It provides the resonant shapes of the building, which would be indicative of the response of the building during seismic events, particularly when the frequency of the earthquake matches the natural frequency of the building. Ultimately, in the event of lateral loads, the deformed shape of the structure can be interpreted to be a combination of various modal responses of the building. The first three modal shapes of the building structure were obtained using ETABS, and are illustrated in Figure 9, Figure 10 and Figure 11, respectively. From the obtained modal responses, it can be observed that the moment frame is stiffer than the braced frame because natural period is larger for the first mode shape, which pertains to loads applied in the x-direction.
Figure 9: Modal Shape #1 ($T=4.027$ s)

Figure 10: Modal Shape #2 ($T=3.498$ s)
5.2 Seismic Force Resistance System (SFRS)

1161 1st Avenue falls under the seismic design category C, as determined in the sample calculations of the seismic load in Appendix C: Seismic Load Calculations. A steel system not specifically detailed for seismic resistance: Seismic Force Resisting System (SFRS) H, was used to design 1161 1st Avenue. The associated seismic design coefficients are tabulated in Table 13.

Table 13: Seismic Design Coefficients

<table>
<thead>
<tr>
<th>SFRS</th>
<th>R</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

Analysis of the SFRS was done on ETABS and manually verified based on Chapter 12 of the ASCE 7-10, to account for horizontal and vertical irregularities in the x-direction and y-direction. The story shear, story overturning moment and story drift are shown in Figure 12, Figure 13 and Figure 14, respectively. The sample verification of story shears, and the base shear are shown in Appendix C: Seismic Load Calculations, following the equivalent lateral force procedure.
Figure 12: Seismic Story Shear

Figure 13: Seismic Story Overturning Moment
The base shear and base overturning moment determined from Figure 12 and Figure 13, respectively, and are tabulated in Table 14.

Table 14: Seismic Base Reactions

<table>
<thead>
<tr>
<th>Reaction</th>
<th>Units</th>
<th>X-Direction</th>
<th>Y-Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Shear ((V_B))</td>
<td>[kips]</td>
<td>267</td>
<td>275</td>
</tr>
<tr>
<td>Overturning Moment ((M_B))</td>
<td>[kip-ft]</td>
<td>39,494</td>
<td>38,675</td>
</tr>
</tbody>
</table>

Along with the calculation of the base shear and overturning moments, we also noticed three irregularities within our seismic force resisting system. According to Chapter 12 of ASCE 7-10, our building has a torsional irregularity, extreme torsional irregularity, and Out-of-Plane Offset irregularity. By definition, a building will have torsional irregularity when the maximum story drift at one end of the structure is 1.2 times more than the average of the story drifts at the two ends of the structure. Similarly, a building can have extreme torsional irregularities when the maximum story drift at one end of the structure is 1.4 times the average of the story drifts. Lastly, the out-of-plan irregularity is defined as a discontinuity within the lateral force-resistance path, which occurs in our building because of the setbacks.
The following table below shows the analysis approach that was taken in order to analyze the braced frames in the x-direction and the moment frames in the y-direction:

*Table 15: Irregularity Checks for Braced Frames*

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
<th>Found in Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Torsional Irregularity $(\delta_{average}/\delta_{max}) = 2.868 &gt; 1.2$</td>
<td>Yes</td>
</tr>
<tr>
<td>1b</td>
<td>Extreme Torsional Irregularity $(\delta_{average}/\delta_{max}) = 2.868 &gt; 1.4$</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>Reentrant Corner Irregularity</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>Diaphragm Discontinuity Irregularity</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>Out-of-Plane Offset Irregularity</td>
<td>No</td>
</tr>
<tr>
<td>5</td>
<td>Nonparallel System Irregularity</td>
<td>No</td>
</tr>
</tbody>
</table>

Using the ASCE 7-10 Minimum Design Loads for Buildings and Other Structures, the following requirements were implemented and checked for in our design to account for these irregularities:

a) Structural Modeling (ASCE 7-10, Section 12.7.3)

b) Amplification of Accidental Torsional Moment (ASCE 7-10, Section 12.8.4.3)

c) Story Drift Limit (ASCE 7-10, Section 12.12.1)

d) Modeling (ASCE 7-10, Section 16.2.2)
<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
<th>Found in Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Torsional Irregularity (\frac{\delta_{\text{average}}}{\delta_{\text{max}}} = 2.326 &gt; 1.2)</td>
<td>Yes</td>
</tr>
<tr>
<td>1b</td>
<td>Extreme Torsional Irregularity (\frac{\delta_{\text{average}}}{\delta_{\text{max}}} = 2.326 &gt; 1.4)</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>Reentrant Corner Irregularity</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>Diaphragm Discontinuity Irregularity</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>Out-of-Plane Offset Irregularity</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>Nonparallel System Irregularity</td>
<td>No</td>
</tr>
</tbody>
</table>

*Table 16: Irregularity Checks for Moment Frames*

Using the ASCE 7-10 Minimum Design Loads for Buildings and other structures, the following requirements were implemented and checked for in our design to account for these irregularities:

a) Structural Modeling (ASCE 7-10, Section 12.7.3)
b) Amplification of Accidental Torsional Moment (ASCE 7-10, Section 12.8.4.3)
c) Story Drift Limit (ASCE 7-10, Section 12.12.1)
d) Modeling (ASCE 7-10, Section 16.2.2)
e) Elements Supporting Discontinuous Walls or Frames (ASCE 7-10, Section 12.3.3.3)

Concerning vertical irregularities that are also a part of the ASCE 7-10 seismic analysis, we found that there were no vertical irregularities within our building. This is mainly because there were no in-plane offset of our vertical seismic force-resisting element (Table 12.3-2). Additionally, there is no discontinuity in lateral strength where the story lateral strength is less than 65 percent of that in the story above.

### 5.3 Wind Load Response

In accordance with section C.1.2 of Appendix C in ASCE 7-10, pertaining to serviceability considerations, the lateral deflections or drift of the structure due to wind loads will not impair the serviceability of the structure. The main requirement for wind loads is that the cladding is strong enough to handle the expected demand, and then transfer the load to the LFRS. Nevertheless, the
wind load is a substantial lateral load resisted by the structure. The cumulative story shear due to the wind loads is shown in Figure 15, and the overturning moment due to the wind load is shown in Figure 16.

Figure 15: Wind Load Story Shear

Figure 16: Wind Load Story Overturning Moment
The base shear and base overturning moment determined from Figure 15 and Figure 16, respectively, and are tabulated in Table 17.

**Table 17: Wind Base Reactions**

<table>
<thead>
<tr>
<th>Reaction</th>
<th>Units</th>
<th>X-Direction</th>
<th>Y-Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Shear ($V_B$)</td>
<td>[kips]</td>
<td>1,354</td>
<td>1,560</td>
</tr>
<tr>
<td>Overturning Moment ($M_B$)</td>
<td>[kip-ft]</td>
<td>149,000</td>
<td>177,000</td>
</tr>
</tbody>
</table>

**6.0 Structural Components**

The individual components of the structural system such as: composite floor system, beams and columns, contribute their different parts to structural stability. The framing system mainly consists of steel columns aligned in a grid system. The decision was made to create a uniform grid system in order to simplify design process. The uniformity throughout the building allows for a faster design process and is more economical, as well as efficient. Another consideration that we used was to stay in a range of optimal bay sizes, which starts from 25 ft up to 45 ft spans. This range allows the most efficient use of steel framing systems. The grid system of choice is 30 ft by 30 ft bays, uniformly throughout the building. Since the footprint of the building is 150 ft by 150 ft, 30 ft was the perfect factor for the 150 ft giving uniform bays. Also, it was optimal for us to have the bay divided into three (3) equal spans in order to arrange the filler beams (see Figure 17). Most of the deck sizes work perfect with 10 ft spacing between the filler beams.

However, two (2) underground levels and two (2) setback levels are composed of multiple bay sizes, from which the biggest bay is 40’x30’ and is located on the setback levels. The grid system and bay sizes for the two underground levels were chosen so that it would allow an optimal layout of the parking spaces. The typical bay (30’x30’) consists of four (4) girders supported by four columns at each corner, creating square frame. In between two horizontal girders we have two filler beams separated 10 feet from each other.
6.1 Composite Floor System

The floor system of choice is a composite floor system, consisting of 2 inches tall, gage 19 steel deck and 3 ¼ inches thick light weight concrete layer, for a total of 5 ¼ -inches-thick floor deck (See Figure 18). The 3 ¼ -inches light weight concrete layer is a requirement for a two-hour fire rating.

The choice of the number of filler beams was based on two important considerations: the first was to comply with the span length specified in the catalog of the composite steel deck manufacturer and the second was to make sure that the actual service loads on the deck would not exceed the allowable loads provided from the deck manufacturer\(^1\). The table for light weight

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http://www.ecs.umass.edu/cee434/handouts/CMCDeckCatalog.pdf
concrete on page 52-53 of the catalog notes that the desired maximum un-shored clear length before concrete is cured and the composite action of the deck is achieved, is 10 ft., and it allows the deck to carry 310 psf unfactored uniform live service loads.

The structural beams are supporting the composite steel deck, more importantly along with the deck they form a composite floor system. The beams are connected to the composite deck with the shear studs, and they all together act one structural piece. Shear studs help to increase the stiffness of the composite floor system as well as its strength. Please see Appendix E: Deck Hand Calculations.

6.2 Composite Beams

Composite beams were designed using ETABS 2016. The ETABS design and manual verification of the beam design were achieved in accordance to the NYCBC 2014, AISC 318-14, AISC Design Guide 3, and ASCE 7-10. The design output obtained from ETABS reflected the design section assigned to the beam, shear stud quantity for non-uniform spacing, and the camber of the beam. Each beam on a drawing has assigned specific depth of camber, which is based on governing deflection depth. The AISC Design Guide 3 suggests that we use camber size of 80% of the governing deflection. Minimum cambering was restricted to 3/4".

Beams and girders were used to transfer gravity loads to the columns, which then directs the loads down to the foundation, and is then dispersed into the bedrock. Beams were modeled to have shear connection, which is reflected by assigning moment end releases on both ends of the beam, and a torsional end release on one end of the beam. Moreover, they were designed for pre-composite and composite action, reflected through the design load combinations, which represent construction and strength loading scenarios. Moreover, serviceability of the beams was also considered through a deflection load combination. Sample calculations for the design of a girder in the building are attached in Appendix G: Girder Hand calculations:. The beam framing plan for each unique floor is displayed in drawings S-103 to S-111. A summary of the beams section used in the building are shown in Table 18.
Table 18: Beam Design Summary

<table>
<thead>
<tr>
<th>Section</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>W 33 x 130</td>
<td>1</td>
</tr>
<tr>
<td>W 30 x 124</td>
<td>1</td>
</tr>
<tr>
<td>W 30 x 108</td>
<td>6</td>
</tr>
<tr>
<td>W 27 x 94</td>
<td>2</td>
</tr>
<tr>
<td>W 27 x 84</td>
<td>48</td>
</tr>
<tr>
<td>W 24 x 76</td>
<td>4</td>
</tr>
<tr>
<td>W 24 x 68</td>
<td>1</td>
</tr>
<tr>
<td>W 24 x 55</td>
<td>3</td>
</tr>
<tr>
<td>W 21 x 62</td>
<td>7</td>
</tr>
<tr>
<td>W 21 x 55</td>
<td>133</td>
</tr>
<tr>
<td>W 21 x 44</td>
<td>88</td>
</tr>
<tr>
<td>W 18 x 50</td>
<td>6</td>
</tr>
<tr>
<td>W 18 x 40</td>
<td>25</td>
</tr>
<tr>
<td>W 18 x 35</td>
<td>149</td>
</tr>
<tr>
<td>W 16 x 31</td>
<td>43</td>
</tr>
<tr>
<td>W 16 x 26</td>
<td>743</td>
</tr>
<tr>
<td>W 14 x 30</td>
<td>23</td>
</tr>
<tr>
<td>W 14 x 26</td>
<td>2</td>
</tr>
<tr>
<td>W 14 x 22</td>
<td>29</td>
</tr>
<tr>
<td>W 12 x 22</td>
<td>16</td>
</tr>
<tr>
<td>W 12 x 19</td>
<td>60</td>
</tr>
<tr>
<td>W 12 x 16</td>
<td>24</td>
</tr>
<tr>
<td>W 12 x 14</td>
<td>13</td>
</tr>
<tr>
<td>W 10 x 15</td>
<td>2</td>
</tr>
<tr>
<td>W 10 x 12</td>
<td>265</td>
</tr>
</tbody>
</table>
6.3 Gravity Column

Gravity columns were designed to resist the loads such as self-weight, superimposed dead load, live loads and snow loads. The design was completed in ETABS software. The attached set of drawings shows exact dimensions of the columns and their reaction in the column schedule (S-500 and S-501). Gravity columns were only designed for axial forces. Smallest gravity columns obtained were W10x33 (total of 48), whereas the biggest gravity column is W14x311.

6.4 Foundation

Foundation system is comprised of isolated spread footing for interior columns to transfer gravity load to foundation, a foundation wall, and a wall footing to transfer the loads from the wall into the foundation. Section 6.4 outlines the design approach of the structural components of the foundation.

6.4.1 Isolated Footing

Appendix L: Isolated Footing Design shows the calculations for each footing. There was no software used in the design of the isolated footings. They are all manually calculated. The largest size of the footings are located in the core of the building. They are 12 ft square with 29’-3” deep. The reason for the oversized footings is that it is located right underneath the columns that are part of the braced frame. Since braced frame resists the lateral loads such as wind and seismic, it creates tension in the column base and causes uplift. In order to overcome uplift the footings needed to me resized to bigger dimensions. The smaller footings are located underneath the edge columns of the footings. The footing sizes and Footing schedule is shown in the drawing sets, on the foundations plan (S-100).

6.4.2 Buttresses

Buttresses were designed as concrete columns. The slenderness of the column was disregarded since the buttress is restrained the foundation wall. Buttresses where designed only for the axial load, coming from the Steel columns rested on top of them. The biggest dimension of buttress obtained was 24”x24”, majority of the columns were 18”x18”. Appendix K: Buttress Design shows the design method for the columns and the assumptions.
6.4.3 Foundation Wall

Foundation walls were designed to support the loads from the surrounding soil pressure. Though the walls are resisting axial loads due to surcharge loads, they are relatively minor in comparison to the soil pressure. Consequently, other loads being taken by the wall such as surcharge loads were neglected. These walls were designed to be reinforced concrete walls with 4 ksi concrete and 60 ksi steel. The wall design was based on the Allowable Stress Design (ASD), since the walls are resisting geotechnical loads.

The walls were analyzed using SAP 2000, as shown in Appendix I: Foundation Wall Design. Then, the SAP analysis results provided the moment distribution over a unit foot of the wall. From the SAP results, the longitudinal reinforcement (i.e. #6 @ 10 E.F.) and wall thickness (i.e. 12 inches) were designed using spColumn. Assuming that the soil distribution is uniform around the perimeter of the foundation wall, the unit foot reinforcement was applied throughout the entire wall. The design results from spColumn are shown in figures below.
Figure 19: Foundation Wall Design (Parameters)

MATERIAL:
\* \* \* \* \* \* \\
\* fc = 4 ksi \\
\*Ec = 360 ksi \\
\* ec = 3.4 ksi \\
\* beta1 = 0.85 \\
\* fy = 60 ksi \\
\* Es = 29000 ksi

SECTION:
\* \* \* \* \* \* \\
\* Ag = 144 in^2 \\
\* Ix = 1728 in^4 \\
\* Iy = 1728 in^4 \\
\* xo = 0 in \\
\* yo = 0 in

REINFORCEMENT:
\* \* \* \* \* \* \\
\* 4 #6 bars @ 1.222% \\
\* As = 1.76 in^2 \\
\* Confinement: Tied \\
\* Clear Cover = 1.00 in \\
\* Min Clear Spacing = 0.50 in

SLENDERNESS:
\* \* \* \* \* \* \\
\* N/A
Figure 20: Foundation Wall Design (P-M Diagram)

6.5 Connections

Please see typical drawings for reference of typical connections used in the design.
6.6 Summary

Table 19 lists the total number of building components used in the structural system. The steel framing per gross area of the building is 6481.4 US tons over 247,980 ft².

*Table 19: Structural Building Components*

<table>
<thead>
<tr>
<th>Element Type</th>
<th>Material</th>
<th>Total Weight (kip)</th>
<th># Pieces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>A992Fy50</td>
<td>1125.2349</td>
<td>516</td>
</tr>
<tr>
<td>Beam</td>
<td>A992Fy50</td>
<td>1675.2949</td>
<td>1700</td>
</tr>
<tr>
<td>Brace</td>
<td>A992Fy50</td>
<td>154.255</td>
<td>112</td>
</tr>
<tr>
<td>Floor</td>
<td>3000Psi</td>
<td>11322.7083</td>
<td>N.A.</td>
</tr>
<tr>
<td>Metal Deck</td>
<td>N.A.</td>
<td>10008</td>
<td>N.A.</td>
</tr>
<tr>
<td>Building area (gross)</td>
<td>N.A.</td>
<td>247980.1</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
Appendix A: Dead-Load Sample Calculations

Steel Frame

1. Typical Bay:

Bay Size: Bay Base: 30 ft
Bay Width: 30 ft
Bay Area: 900 ft²

Girders: W16x77
Girder Self-Weight (plf): 77

Beams: W12x35
Beam Self-Weight (plf): 35

\[
\text{Total Weight} = 77 \frac{\text{lbs}}{\text{ft}} \times 30 \text{ ft} \times 2 + 35 \frac{\text{lbs}}{\text{ft}} \times 30 \text{ ft} \times 2
\]

\[
\text{Total Weight} = 6,720 \text{ lbs}
\]

\[
\text{Area Load} = \frac{\text{Total Weight}}{\text{Bay Area}} = \frac{6,720 \text{ lbs}}{900 \text{ ft}^2} = 7.47 \text{ psf}
\]
2. Elevator Core Bay:

Bay Size: Bay Base: 30 ft
Bay Width: 30 ft

Girders: W16x77
Girder Self-Weight (plf): 77
Beams: W12x35 and W8x10
Beam Self-Weight (plf): 35 (W12x35) and 10 (W8x10)

\[
Total \ Weight = (4 * 10 \ plf \times 5.5) + (2 * 10 \times 10 \ plf) + (3 * 35 \ plf \times 30 \ ft) + (2 * 30 \ ft \times 77 \ plf)
\]

\[
Total \ Weight = 8,190 \ lbs
\]

\[
Area \ Load = \frac{Total \ Weight}{Bay \ Area} = \frac{8,190 \ lbs}{900 \ ft^2} = 9.10 \ psf
\]

3. Stair Core Bay
Bay Size:  
Bay Base: 30 ft  
Bay Width: 30 ft

Girders:  
W16x77

Girder Self-Weight (plf):  
77

Beams:  
W12x35 and W8x10

Beam Self-Weight (plf):  
35 (W12x35) and 10 (W8x10)

Total Weight = (10 plf * 8.62) + (10 * 10 plf) + (20.5 * 35 plf) + (30 ft * 35 plf)  
+ (20 * 10 plf) + (2 * 30 ft * 77 plf) + (77 plf * 20)  

Total Weight = 8,313.7 lbs

Area Load = \[
\frac{Total \ Weight}{Bay \ Area} = \frac{8,313.7 \ lbs}{900 \ ft^2} = 9.24 \ \text{psf}
\]
Composite Deck

1. Steel Deck

   Bay Size
   Bay Size
   Bay Base: 30 ft.
   Bay Width: 30 ft.
   Bay Area: 900 ft²

   Area Load (psf)
   2.05 (CMC steel deck catalog)

2. Concrete Deck

   Bay Size
   Bay Size
   Bay Base: 30 ft.
   Bay Width: 30 ft.
   Bay Area: 900 ft²

   Deck Size
   Avg. Height: 3.625 in
   Volume: 271.875 ft³

   Density (pcf)
   115

\[
\text{Weight of Concrete Deck} = \text{Volume} \times \text{Density} = 271.875 \text{ft}^3 \times 115 \frac{\text{lb}}{\text{ft}^3}
\]

\[
\text{Weight of Concrete Deck} = 31,265.625 \text{ lbs}
\]

\[
\text{Area Load (psf)} = \frac{\text{Weight of Concrete Deck}}{\text{Bay Area}} = \frac{31,265.625 \text{ lbs}}{900 \text{ ft}^2} = 34.7395 \text{ psf}
\]

\[
\text{Total Area Load} = 2.05 \text{ psf} + 34.74 \text{ psf} = 36.79 \text{ psf}
\]
Appendix B: Snow Loads and Snow Drift Sample Calculations

Risk Category: II

\( C_e = \text{Exposure factor} = 1.0 \)  
\( C_t = \text{Thermal factor} = 1.0 \)  
\( I_s = \text{Importance factor} = 1 \)  
\( p_g = \text{Ground snow load} = 25 \text{ psf} \)  
\( h_0 = \text{Obstruction Height} = 12 \text{ ft} \)  
\( h_p = \text{Parapet height} = 4 \text{ ft} \)  
\( L_u = \text{Length of High Roof} = 30 \text{ ft} \)  
\( L_L = \text{Length of Low Roof} = 20 \text{ ft} \)

\( p_f = \text{Flat roof snow load} \)
\( (p_f)_{\text{min}} = 20 I_s = 20 \text{ psf} \)

\( p_f = 0.7 C_e C_t I_s p_g \)  
\( p_f = 0.7(1)(1)(1)(25) = 17.5 \text{ psf} \)

\( p_f = \max \left( (p_f)_{\text{min}}, p_f \right) = 20.0 \text{ psf} \)

\( C_s = \text{Roof Slope Factor} = 1.0 \)  
\( p_s = \text{Sloped Roof Snow Load} \)

\( p_s = C_s p_f \)  
\( p_s = 1.0(20.0) = 20.0 \text{ psf} \)

\( \gamma = \text{Snow Density} \)

(NYCBC 2014 Table 1604.5)  
(NYCBC 2014 Table 1608.3.1)  
(NYCBC 2014 Table 1608.3.2)  
(NYCBC 2014 Table 1604.5.2)  
(NYCBC 2014 Sec. 1608.2)

(ASCE 7-10 Equation 7.3-1)  
(ASCE 7-10 Figure 7-2a)  
(ASCE 7-10 Equation 7.4-1)
\[ \gamma = \min(0.13p_g + 14, 30 \text{pcf}) \]
\[ \gamma = 0.13(25) + 14 = 17.3 \text{pcf} \]  

\[ h_b = \text{Height of balanced snow load} \]
\[ h_b = \frac{p_s}{\gamma} \]
\[ h_b = \frac{20.0}{17.3} = 1.16 \text{ ft} \]
\[ h_c = \text{Clear Parapet Height} = h_p - h_b \]
\[ h_c = 4 - 1.16 = 2.84 \text{ ft} \]  

\[ h_d = \text{Snow Drift Height} \]

**Leeward (L):**
\[ h_d = 0.43^{\frac{3}{4}}L_u^{\frac{1}{4}} \sqrt{p_g + 10} - 1.5 \]  
\[ h_d = 0.43^{\frac{3}{4}}\sqrt[4]{30^{\frac{1}{4}}25} + 10 - 1.5 = 1.75 \text{ ft} \]  

**Windward (W):**
\[ h_d = 0.75 \left(0.43^{\frac{3}{4}}L_u^{\frac{1}{4}} \sqrt{p_g + 10} - 1.5\right) \]
\[ h_d = 0.75(0.43^{\frac{3}{4}}20^{\frac{1}{4}}25 + 10 - 1.5) = 1.00 \text{ ft} \]
\[ (h_d)_{use} = \max((h_d)_L, (h_d)_W) = 1.75 \text{ ft} \]

\[ p_d = \text{Max. Intensity of Drift Surcharge Load} \]
\[ p_d = h_d \gamma \]
\[ p_d = (1.75)(17.3) = 30.2 \text{ psf} \]
\[ p_{rs} = \text{Rain on Snow Surchage Load} \]

*Since* \( p_g > 20 \text{ psf} \) \( \rightarrow p_{rs} = 0 \text{ psf} \) *(negligible)* (ASCE 7-10 Sec. 7.10)

\[ w = \text{Snow drift width} \]

\[ h_d < h_c \rightarrow w = 4h_d = 4(1.75) = 7.00 \text{ ft} \] (ASCE 7-10 Sec. 7.7.1)
Appendix C: Seismic Load Calculations

Initial Data

Building Height = h = 220 ft, (Not counting underground levels)

Seismic force resisting system = H  

Response Modification factor = R = 3  

Overstrength Factor = \( \Omega_0 = 3 \)  

Deflection Amplification Factor = \( C_d = 3 \)  

Seismic Importance Factor = \( I_e = 1 \)

\[ S_s = 0.281 \, g \]  

\[ S_1 = 0.073 \, g \]  

\[ T_6 = 6 \]  

\[ F_a = 1.57 \]  

\[ F_v = 2.40 \]  

\[ T_a = 1.9 \, s \]  

Calculations:

1) Seismic parameters calculations:

\[ S_{MS} = F_a S_s = 1.57 \times 0.281 \, g = 0.441 \, g \]  

\[ S_{M1} = F_v S_1 = 2.4 \times 0.073 \, g = 0.175 \, g \]  

\[ S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.441 \, g = 0.294 \, g \]  

\[ S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.175 \, g = 0.117 \, g \]  

2) Effective seismic weight: (Sample calculation for the story weight for Level 16)

Self-Weight:

\[ \text{Area excluding openings} = 439.58 \, ft^2 \]

\[ \text{Deck self weight} = 36.1 \, psf \]

\[ \text{Framing self weight} = 5 \, psf \]
Weight due to self weight = 439.48 ft²(36.1 psf + 5 psf) = 183.32 lb

Weight due to loading:

<table>
<thead>
<tr>
<th>Occupancy</th>
<th>Area</th>
<th>SDL (psf)</th>
<th>LL (psf)</th>
<th>Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L16 Stairs</td>
<td>351.25</td>
<td>35</td>
<td>0</td>
<td>12.29375</td>
</tr>
<tr>
<td>IT room</td>
<td>113</td>
<td>50</td>
<td>0</td>
<td>5.65</td>
</tr>
<tr>
<td>Lobby</td>
<td>419.5</td>
<td>35</td>
<td>0</td>
<td>14.6825</td>
</tr>
<tr>
<td>Restroom</td>
<td>378.6</td>
<td>35</td>
<td>0</td>
<td>13.251</td>
</tr>
<tr>
<td>Jan closet</td>
<td>54</td>
<td>35</td>
<td>32</td>
<td>3.618</td>
</tr>
<tr>
<td>Office</td>
<td>3391.46</td>
<td>35</td>
<td>0</td>
<td>118.7011</td>
</tr>
</tbody>
</table>

For Detailed calculations see Attached Excel Sheet: CE4822_S18_Groupr01_SeismicLoadCalc

Total story weight = 168.2 + 183.32 = 351.5 lb

Effective seismic weight = $W = 20,252.65$ lb

$$C_s = \frac{S_{DS}}{R} = \frac{0.294}{3} = 0.098$$

$$C_s = \frac{S_{D1}}{T(\frac{R}{T_e})} \quad T_a < T \quad \text{ASCE 7-10, Sec. 12.8.1.1}$$

$$C_s = \frac{S_{D1}}{T(\frac{R}{T_e})} = \frac{0.117}{1.9 \left(\frac{3}{1.00}\right)} = 0.0204$$

$$C_s = 0.044S_{DS}l_e \geq 0.01 = 0.044(0.294)(1.00) < 0.01 = 0.013 < 0.01$$

$$C_s = 0.0204$$

$$V_b = C_s W \quad \text{ASCE 7-10, Sec. 12.8.3}$$

$$V_b = 0.0204(20,252.65 \text{ lb}) = 414.13 \text{ kip}$$

$$V_b = C_s W$$

$$F_x = C_vxV \quad \text{ASCE 7-10, Sec. 12.8-11}$$

$$C_vx = \frac{w_k h_k^w}{\sum_{i=1}^n w_i h_i^k} = 0.044 \quad \text{ASCE 7-10, Sec. 12.8-12}$$

$$F_{16} = C_{16}V = 0.044 \times 414.13 \text{ kip} = 18.18 \text{ kip}$$
The calculations yield following distribution of lateral loads throughout the stories:

<table>
<thead>
<tr>
<th>Story</th>
<th>Weight (kips)</th>
<th>Story height (ft)</th>
<th>Vertical distribution factor $C_{vx}$</th>
<th>$V_{by}$ (kip)</th>
<th>$V_{bx}$ (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulkhead</td>
<td>177.22</td>
<td>12</td>
<td>0.026</td>
<td>10.899</td>
<td>10.899</td>
</tr>
<tr>
<td>L17 (Roof)</td>
<td>444.64</td>
<td>12</td>
<td>0.061</td>
<td>25.139</td>
<td>25.139</td>
</tr>
<tr>
<td>L16</td>
<td>351.52</td>
<td>12</td>
<td>0.044</td>
<td>18.179</td>
<td>18.179</td>
</tr>
<tr>
<td>L15</td>
<td>1236.58</td>
<td>12</td>
<td>0.140</td>
<td>58.170</td>
<td>58.170</td>
</tr>
<tr>
<td>L14</td>
<td>1039.90</td>
<td>12</td>
<td>0.107</td>
<td>44.211</td>
<td>44.211</td>
</tr>
<tr>
<td>L13</td>
<td>1039.90</td>
<td>12</td>
<td>0.096</td>
<td>39.666</td>
<td>39.666</td>
</tr>
<tr>
<td>L12</td>
<td>1039.90</td>
<td>12</td>
<td>0.085</td>
<td>35.288</td>
<td>35.288</td>
</tr>
<tr>
<td>L11</td>
<td>1039.90</td>
<td>12</td>
<td>0.075</td>
<td>31.085</td>
<td>31.085</td>
</tr>
<tr>
<td>L10</td>
<td>1039.90</td>
<td>12</td>
<td>0.065</td>
<td>27.063</td>
<td>27.063</td>
</tr>
<tr>
<td>L9</td>
<td>1039.90</td>
<td>12</td>
<td>0.056</td>
<td>23.231</td>
<td>23.231</td>
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<td>1039.90</td>
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<td>0.047</td>
<td>19.599</td>
<td>19.599</td>
</tr>
<tr>
<td>L7</td>
<td>1664.81</td>
<td>12</td>
<td>0.063</td>
<td>25.902</td>
<td>25.902</td>
</tr>
<tr>
<td>L6</td>
<td>1524.54</td>
<td>12</td>
<td>0.046</td>
<td>19.037</td>
<td>19.037</td>
</tr>
<tr>
<td>L5</td>
<td>1524.54</td>
<td>12</td>
<td>0.036</td>
<td>14.712</td>
<td>14.712</td>
</tr>
<tr>
<td>L4</td>
<td>1524.54</td>
<td>12</td>
<td>0.026</td>
<td>10.774</td>
<td>10.774</td>
</tr>
<tr>
<td>L3</td>
<td>1524.54</td>
<td>12</td>
<td>0.018</td>
<td>7.269</td>
<td>7.269</td>
</tr>
<tr>
<td>L2</td>
<td>1398.27</td>
<td>12</td>
<td>0.009</td>
<td>3.905</td>
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</tr>
<tr>
<td>GF</td>
<td>1602.15</td>
<td>28</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Totals</td>
<td>20252.65</td>
<td>1.00</td>
<td>414.130</td>
<td>414.130</td>
<td></td>
</tr>
</tbody>
</table>

3) Seismic Load effect calculation:

$$E_v = 0.2S_{Ds}D = 0.2(0.294)20252.65 = 1191.31$$

$$E_h = \rho Q_E = 1.0(414.130)$$

$$E_{mh} = \Omega_0 Q_E = 3(414.130) = 1242 \text{ kips}$$

$$E_v = 0.2S_{Ds}D = 0.2(0.294)20252.65 = 1191.31$$

The Seismic load effect with and without the over strength is listed below as well:

<table>
<thead>
<tr>
<th>Without over strength</th>
<th>With over strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_h$ (kip)</td>
<td>$E_v$ (kip)</td>
</tr>
<tr>
<td>414.1297995</td>
<td>1191.314827</td>
</tr>
<tr>
<td>$E_{mh}$ (kip)</td>
<td>$E_v$ (kip)</td>
</tr>
<tr>
<td>1242.39</td>
<td>1191.314827</td>
</tr>
</tbody>
</table>
The following graph represents the equivalent lateral forces on the building:
Appendix D: Wind Load Calculations

\[ n_1 \approx \frac{75}{h} = \frac{75}{220} = 0.341 \text{ Hz} \]  
(ASCE 7-10 Eq. 26.9-4)

\[ n_1 \approx \frac{75}{220} = 0.341 \text{ Hz} \]  
(ASCE 7-10 Ch. 26.2)

\[ n_1 < 1 \text{ Hz} \rightarrow \text{ Building is flexible} \]

\[ I_Z = c \left( \frac{33}{z} \right)^{1/6} \]  
(ASCE 7-10 Eq. 26.9-7)

\[ I_Z = 0.30 \left( \frac{33}{0.6(220)} \right)^{1/6} = 0.238 \]

\[ L_Z = l \left( \frac{z}{33} \right)^{\epsilon} \]  
(ASCE 7-10 Eq. 26.9-9)

\[ L_Z = 320 \left( \frac{0.6(220)}{33} \right)^{1/3} = 508 \text{ ft} \]

\[ g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} \]  
(ASCE 7-10 Eq. 26.9-11)

\[ g_R = \sqrt{2\ln(3600(0.341))} + \frac{0.577}{\sqrt{2\ln(3600(0.341))}} = 3.92 \text{ Hz} \]

\[ \bar{V}_z = \bar{b} \left( \frac{z}{33} \right)^{\frac{88}{60}} \cdot V \]  
(ASCE 7-10 Eq. 26.9-16)

\[ \bar{V}_z = 0.45 \left( \frac{0.6(220)}{33} \right)^{0.25} \left( \frac{88}{60} \right)(124) = 115.7 \text{ fps} \]
\[ R_l = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta}), \quad (\eta > 0) \]

\[ \eta_h = \frac{4.6n_1h}{V_z} = \frac{4.6(0.341)(220)}{115.7} = 2.98 \]

\[ \eta_B = \frac{4.6n_1B}{V_z} = \frac{4.6(0.341)(150)}{115.7} = 2.03 \]

\[ \eta_L = \frac{15.4n_1L}{V_z} = \frac{15.4(0.341)(150)}{115.7} = 6.80 \]

\[ R_h = \frac{1}{2.98} - \frac{1}{2(2.98)^2} (1 - e^{-2(2.98)}) = 0.279 \]

\[ R_B = \frac{1}{2.03} - \frac{1}{2(2.03)^2} (1 - e^{-2(2.03)}) = 0.373 \]

\[ R_L = \frac{1}{6.80} - \frac{1}{2(6.80)^2} (1 - e^{-2(6.80)}) = 0.136 \]

\[ N_l = \frac{n_1L_z}{V_z} \]

\[ N_1 = \frac{(0.341)(508)}{115.7} = 1.50 \]

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

\[ R_n = \frac{7.47(1.50)}{(1 + 10.3(1.50))^{5/3}} = 0.105 \]

\[ R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47R_L)} \]

(ASCE 7-10 Eq. 26.9-15a)

(ASCE 7-10 Eq. 26.9-14)

(ASCE 7-10 Eq. 26.9-13)

(ASCE 7-10 Eq. 26.9-12)
\[ R = \sqrt{\frac{1}{0.01} (0.105)(0.279)(0.373)(0.53 + 0.47(1.36))} = 0.571 \]

\[ Q = \sqrt{\frac{1}{1 + 0.63 \left( \frac{B + h}{L_Z} \right)^{0.63}}} \]

\[ Q = \sqrt{\frac{1}{1 + 0.63 \left( \frac{150 + 220}{508} \right)^{0.63}}} = 0.812 \]

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

\[ G_f = 0.925 \left( 1 + 1.7(0.238)\sqrt{(3.4)^2(0.828)^2 + (3.92)^2(0.571)^2} \right) \]

\[ G_f = 0.89 \]

\[ \frac{K_z}{K_{zg}} = 2.01 \left( \frac{z}{z_g} \right)^{2/\alpha} \]

\[ K_Z = 2.01 \left( \frac{52}{900} \right)^{2/9.5} = 1.10 \]

\[ K_{z(h)} = 2.01 \left( \frac{220}{900} \right)^{2/7.0} = 1.49 \]
\[ q_z = 0.00256 K_z K_{zt} K_d V^2 \]

\[ q_z = 0.00256(1.10)(1.0)(0.85)(124)^2 = 36.9 \text{ psf} \]

\[ q_h = 0.00256(1.49)(1.0)(0.85)(124)^2 = 50.0 \text{ psf} \]

\[ q_l = 50.0(0.18) = 8.99 \text{ psf} \]

\[ p_w = q G_f C_p - q_l (G C_{pi}) \]

\textit{Note: Internal Pressure (}q_l\textit{) cancels out on both sides}

\[ p_w = 36.9(0.89)(0.80) + 50.0(0.89)(0.5) = 48.4 \text{ psf} \]

\[ F_x = p_w A_T = \frac{48.4(12)(150)}{1000} = 87.1 \text{ kips} \]

\[ F_{ETABS} = 87.3 \text{ kips} \]

\[ \text{Percent Error} = \frac{|F_x - F_{ETABS}|}{F_{ETABS}} \times 100\% \]

\[ \text{Percent Error} = \frac{|87.1 - 87.3|}{87.3} \times 100\% = 0.20\% \]
Wind Load: Y-Direction

- ASCE 7-10
- ETABS
Appendix E: Deck Hand Calculations

Requirements and Initial Data
- Bay size = 30 ft x 30 ft
- 2 hour fire rating requiring 3.25 in concrete slab
- 2 in high steel deck
- Total depth of the slab = 5.25 in

Values obtained from CMC deck catalog pg. 52:

Light Weight Concrete: $\rho_c = 115 \, pcg$

$f'_c = 3 \, ksi$ and $F_y = 40 \, ksi$

19 gage steel

Unshored span = 10 ft

Filler Beams: Use 3@ 10 ft

Diameter of stud anchors: $d_{sa} = \frac{3}{4} \, in$

Dead Loads

Pre Composite:

\[ Slab = 50 \, psf \]  

CMC Catalog, Sec. 3.1D, Pg.70

Beam Self Weight = 5psi (Assumption)

Composite

Office Loads = 30 psf

Live Loads

Pre-Construction:

\[ Construction = 30 \, psf \]  

CMC Catalog, Sec. 2.4A, Pg.67

Composite:

\[ Office \, live \, loads = 50 \, psf \]  

ASCE 7-10, Table 4-1
Pre-Composite Design:

Checks for Composite Deck requirements

✓ Concrete Strength

$$3 \text{ ksi} \leq f'_c \leq 10 \text{ ksi}$$

$$f'_c = 3 \text{ ksi} \text{ OK!}$$

✓ Rib Height ($h_r$) and width ($w_r$)

$$h_r = 2 \text{ in} < 3 \text{ in} \text{ OK!}$$

$$w_r = 6 \text{ in} > 2 \text{ in} \text{ OK!}$$

✓ Slab thickness = 3\(\frac{1}{4}\) in $\geq$ 2 in above steel deck **OK for Fire Rating**!

✓ Stud length

$$l_{sa} \geq 4d_{sa} \quad \text{→} \quad l_{sa} \geq 3 \text{ in}$$

✓ Concrete cover

$$\text{Min. Concrete Cover} = \frac{1}{2} \text{ in} \quad \text{→} \quad l_{sa} \leq 5\frac{1}{4} - \frac{1}{2} = 4.75 \text{ in}$$

$$3 \text{ in} \leq l_{sa} \leq 4.75 \text{ in} \quad \text{→} \quad \text{Choose } l_{sa} = 4.5 \text{ in}$$

$$L_{sa} = 4.5 \text{ in} > h_r + 1.5 = 3.5 \text{ OK!}$$
Appendix F: Composite Beam Hand Calculations

Composite filler beams between Grid section BC1 and BC2 on L-03 (See Drawing sets-S-104)

Requirements and Initial Data

- Steel A992
  \[
  \begin{align*}
  F_y &= 50 \text{ ksi} \\
  F_u &= 65 \text{ ksi}
  \end{align*}
  \]
  AISC 360-10, Table 2.4

- Beam length \( L = 30 \text{ ft} \)
- Slab thickness \( t_{slab} = 5.25 \text{ in} \)

Dead Loads

Pre Composite:

\[
Slab = 50 \text{ psf} \quad \text{CMC Catalog, Sec. 3.1D, Pg.70}
\]

\[
\text{Beam Self Weight} = 5\text{psi (Assumption)}
\]

Composite

\[
Office Loads = 30 \text{ psf}
\]

Live Loads

Pre-Construction:

\[
\text{Construction} = 30 \text{ psf} \quad \text{CMC Catalog, Sec. 2.4A, Pg.67}
\]

Composite:

\[
Office \text{ live loads} = 50 \text{ psf} \quad \text{ASCE 7-10, Table 4-1}
\]

Pre-Composite Design:

- Minimum flange thickness \( t_f \) based on anchor diameter \( d_{sa} = \frac{3}{4} \text{ in} \)

\[
d_{sa} \leq 2.5 \ t_f \quad \rightarrow \quad t_f \geq \frac{\frac{3}{4} \text{ in}}{2.5} = 0.30 \text{ in}
\]

AISC 360-10, I8.1

\[
\text{Tributary Area} = A_T = 30 \text{ ft} \times 10 \text{ ft} = 300 \text{ ft}^2
\]

\[
w_{LL} = 10 \text{ ft} \times (30 \text{ psf}) = 0.30 \frac{\text{kip}}{\text{ft}}
\]

\[
w_{DL} = 10 \text{ ft} \times (50 \text{ psf} + 5 \text{ psf}) = 0.55 \frac{\text{kip}}{\text{ft}}
\]

From AISC 360-10, Eq. 2-3b, ultimate uniform load can be calculated as:
\[ w_{DL} = 10 \text{ ft} \times (50 \text{ psf} + 5 \text{ psf}) = 0.55 \frac{\text{kip}}{\text{ft}} \]

\[ w_u = 1.4 \times (0.55) = 0.77 \frac{\text{kip}}{\text{ft}} \]

\[ w_u = 1.2 w_{DL} + 1.6 w_{LL} = 1.2 \left(0.55 \frac{\text{kip}}{\text{ft}}\right) + 1.6 \left(0.30 \frac{\text{kip}}{\text{ft}}\right) = 1.14 \frac{\text{kip}}{\text{ft}} \quad \text{Governs!} \]

\[ M_u = \frac{w_u L^2}{8} = \frac{1.14 \frac{\text{kip}}{\text{ft}} \times (30 \text{ ft})^2}{8} = 128.25 \text{ kip. ft} \quad \text{AISC 360-10, Table 3-23 (Case 1)} \]

Maximum deflection occurs at the center of the beam.

\[ Z_x = \frac{M_u}{\phi_b F_y} = \frac{128.25 \text{ kip. ft} \times \frac{12\text{ in}}{\text{ft}}}{0.9(50 \text{ ksi})} = 34.2 \text{ in}^3 \]

Enter AISC 360-10, Table 3-2 with \( Z_x = 34.2 \text{ in}^3 \)

\[ W16\times26 \quad Z_x = 44.2 \text{ in}^3 \quad t_f = 0.345 \geq 0.30 \text{ in} \]

\[ W14\times26 \quad Z_x = 40.2 \text{ in}^3 \quad t_f = 0.42 \geq 0.30 \text{ in} \]

**Try W16x26**

\[ I_x = 301 \text{ in}^4 \]

\[ A = 7.68 \text{ in}^2 \]

Adjust for new self-weight \( w_{sw}=26 \text{ lb/ft} \)

\[ w_{DL} = 10 \text{ ft} \times (50 \text{ psf}) + 26 \frac{\text{lb}}{\text{ft}} = 0.526 \text{ kip} \]

\[ w_u = 1.2 w_{DL} + 1.6 w_{LL} = 1.2 \left(0.526 \frac{\text{kip}}{\text{ft}}\right) + 1.6 \left(0.30 \frac{\text{kip}}{\text{ft}}\right) = 1.11 \frac{\text{kip}}{\text{ft}} \]

\[ M_u = \frac{1.114 \frac{\text{kip}}{\text{ft}} \times (30 \text{ ft})^2}{8} = 138.91 \text{ kip. ft} \]
Pre construction deflection \( \leq \frac{L}{360} \)

\[
\Delta_{nc} = \frac{5w_{DL}^4}{384Ei} \quad \text{AISC 360-10, Table 3-23 (Case 1)}
\]

\[
\Delta_{nc} = \frac{5 \left( 0.526 \frac{kip}{ft} \times \frac{1 ft}{12 in} \right) \left( 30 \text{ ft} \times 12 \text{ in} \right)^4}{384(29000 \text{ ksi})(301 \text{ in}^4)} = 1.1 \text{ in}
\]

\[
\Delta_{nc} = 0.7953 \text{ in} + 0.0061 \text{ in} = 0.801 \text{ in} = \frac{L}{327} < \frac{L}{360} \quad \text{Not Good!}
\]

Camber = 0.8 (1.1) = 0.88 \quad \text{AISC Design Guide 3. Chapter 5. Pg. 22}

use 1 in camber

Designing for Composite flexure:

- Live load reduction
  \( K_{LL} = 2 \) for interior beams
  \[ A_T = 30 \times 10 = 300 \text{ ft}^2 \]
  \( K_{LL}A_T = 600 \text{ ft} > 400 \text{ ft}^2 \) Reducible \quad \text{ASCE 7-10, Sec. 4.7.2}

\[ LL = LL_0 \left( 0.25 + \frac{15}{\sqrt{K_{LL}A_T}} \right) \quad \text{ASCE 7-10, Eq. 4.7-1} \]

\[ LL = 50 \left( 0.25 + \frac{15}{\sqrt{600}} \right) = 43.11 \text{ psf} \]

- Ultimate load calculations

\[ w_{DL} = 10 \text{ ft} \left( 50 + 2.6 + 30 \right) \text{ psf} = 0.826 \frac{kip}{ft} \]

\[ w_{LL} = 10 \text{ ft} \left( 43.11 \text{ psf} \right) = 0.436 \frac{kip}{ft} \]

\[ w_u = 1.2 w_{DL} + 1.6 w_{LL} = 1.2 \left( 0.826 \frac{kip}{ft} \right) + 1.6 \left( 0.436 \frac{kip}{ft} \right) = 1.68 \frac{kip}{ft} \]

\[ M_u = \frac{w_u L^2}{8} \quad \text{AISC 360-10, Table 3-23 (Case 1)} \]

\[ M_u = \frac{1.68 \frac{kip}{ft} \times (30 \text{ ft})^2}{8} = 189.0 \text{ kip. ft} \]
Effective width \( b = \min \left\{ \frac{30\text{ft}}{8} = 7.50\text{ft} \right. \text{ Governs } AISC\ 360-10, \text{ Sec. I3.1a} \)

\[ b = 7.50\text{ft} \]

\[ a_{\text{trial}} = \frac{\sum q_n}{0.85 f'_c b} \]

\[ \sum Q_n = 0.5 A_S F_Y \]

\[ a_{\text{trial}} = \frac{0.5 A_S F_Y}{0.85 f'_c b} = \frac{0.5 \times (7.68\text{ in}^2)(50\text{ ksi})}{0.85(3\text{ ksi}) \left( 7.50\text{ ft} \times 12 \frac{\text{in}}{\text{ft}} \right)} = 0.84\text{ in} \approx 1\text{ in} \]

\[ Y_2 = Y_{\text{concrete}} - \frac{a_{\text{trial}}}{2} \]

\[ Y_2 = 5.25 - \frac{1}{2} = 4.75\text{in} \]

Use \( \sum Q_n = 192\text{ kips} \) and \( Y_2 = 4.75\text{ in} \) and Enter \( AISC\ 360 - 10, \text{ Table 3.19} \)

\( \text{PNA is located in section 5} \rightarrow \text{BFL} \rightarrow \sum Q_n = 192\text{ kips} \rightarrow \phi_b M_n = 293\text{ kip-ft} \)

\[ \phi_b M_n = 293\text{ kip-ft} > M_u = 189\text{ kip} \quad \text{OK!} \]

Live Load Deflection:

\[ \Delta_{nc} = \frac{5 w_{DL} L^4}{384 E I} \quad \text{AISC 360-10, Table 3-23 (Case 1)} \]

\[ L_{LL} \leq \frac{L}{360} \quad \text{NYCBC 2014, Table 1604.3} \]

\[ I_{LB} = 705\text{ in}^4 \quad \text{AISC 360-10, Table 3-10} \]

\[ \Delta_{LL} = \frac{5 \left( \frac{0.43}{12} \text{ in} \right) (30 \times 12 \text{ in})^4}{384 \times 29000 \text{ ksi} \times 705\text{ in}^4} = 383 = \frac{L}{940} < \frac{L}{360} \quad \text{OK!} \]

Steel anchor strength:

\[ Q_n = 17.1 \quad \text{one anchor per rib} \quad \text{AISC 360-10, Table-3-21} \]
Number and spacing of anchors

- Estimate number of deck flutes
  \[ n_{fluts} = \frac{30 - 2}{2} + 1 = 15 \text{ fluts} \]

- Estimate number of studs
  \[ N_{anchor} = \frac{\sum Q}{Q_n} = \frac{192 \text{ kips}}{17.1 \text{ kips}} = 11.23 \approx 12 \text{ anchors} \]

- Estimate anchor spacing
  \[ \text{Max spacing}=8 t_{z,lab} = 8(5.25) = 42 \text{ in} > 12 \text{ in} \quad \text{OK!} \]

Available Shear strength:

\[ V_u = \frac{w_u l}{2} \quad \text{AISC 360-10, Table 3-23 (Case 1)} \]

\[ V_u = \frac{1.68 \text{ kip/ft} \times 30 \text{ ft}}{2} = 25.26 \text{ kips} \]

\[ \phi_v V_v = 106 \text{ kips} > 25.26 \quad \text{OK!} \quad \text{AISC 360-10, Table 3-2} \]

USE W16x26 with camber=1.00 in
Appendix G: Girder Hand calculations:

Designing Girder between columns B2 and C2 on L-03 (See Drawing sets-S-104)

Requirements and Initial Data

- Steel A992

\[
\begin{align*}
F_y &= 50 \text{ ksi} \\
F_u &= 65 \text{ ksi}
\end{align*}
\]

AISC 360-10, Table 2.4

- Beam length= \( L = 30 \text{ ft} \)
- Self-weight of filler beams = 26 psf
- Slab thickness = \( t_{slab} = 5.25 \text{ in} \)
- Space between filler beams = 10 ft

Dead Loads

Pre Composite:

\[
\text{Slab} = 50 \text{ psf} \quad \text{CMC Catalog, Sec. 3.1D, Pg.70}
\]

\[
\text{Beam Self Weight} = 5\text{psi} \quad \text{(Assumption)}
\]

Composite

\[
\text{Office Loads} = 30 \text{ psf}
\]

Live Loads

Pre-Construction:

\[
\text{Construction} = 30 \text{ psf} \quad \text{CMC Catalog, Sec. 2.4A, Pg.67}
\]

Composite:

\[
\text{Office live loads} = 50 \text{ psf} \quad \text{ASCE 7-10, Table 4-1}
\]

Pre-Composite Design:

- Minimum flange thickness (\( t_f \)) based on anchor diameter \( d_{sa} = \frac{3}{4} \text{ in} \)

\[
d_{sa} \leq 2.5 \ t_f \quad \rightarrow \quad t_f \geq \frac{\frac{3}{4} \text{in}}{2.5} = 0.30 \text{ in}
\]

AISC 360-10, I8.1

Tributary Area=\( A_T = 30 \text{ ft} \times 15 \text{ ft} = 450 \text{ ft}^2 \)

We have point loads (P) form beams and distributed loads (w) due to self-weight of girder and concrete deck
\[ P_{DL} = 2 \left( 15 \text{ ft} \times 10 \text{ ft} \times 50 \frac{\text{lb}}{\text{ft}^2} + 15 \text{ ft} \times 26 \frac{\text{lb}}{\text{ft}} \right) = 15.78 \text{ kip} \]

\[ P_{LL} = 2(15 \text{ ft} \times 10 \text{ ft} \times 30) = 9.0 \text{ kip} \]

\[ w_{DL} = 50 \frac{\text{lb}}{\text{ft}} = 0.05 \frac{\text{kip}}{\text{ft}} \]

\[ P_u = 1.4 \times P_{DL} = 1.4 (15.78 \text{ kip}) = 22.10 \]

\[ P_u = 1.2 \times P_{DL} + 1.6 \times P_{LL} = 1.2 (15.78 \text{ kip}) + 1.6(9.0 \text{ kip}) = 33.34 \text{ kip} \quad \text{Governs!} \]

\[ w_u = 1.4 \left( 0.050 \frac{\text{kip}}{\text{ft}} \right) = 0.070 \frac{\text{kip}}{\text{ft}} \]

\[ M_u = \frac{w_u l^2}{8} + P_u a \quad \text{AISC 360-10, Table 3-23 (Case 1, Case 9)} \]

\[ M_u = \frac{0.070 \frac{\text{kip}}{\text{ft}} \times (30 \text{ ft})^2}{8} + 22.10 \text{ kip} \times 10 \text{ ft} = 228.88 \text{ kip} \times \text{ft} \]

Maximum deflection occurs at the center of the beam.

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

\[ Z_x = \frac{M_u}{F_y \Phi_b} = \frac{228.88 \text{ kip} \times \text{ft} \left( \frac{12 \text{ in}}{\text{ft}} \right)}{0.9(50 \text{ ksi})} = 61.03 \text{ in}^3 \]

Enter AISC 360-10, Table 3-2

\[ \text{Try W21x55} \]

\[ t_f = 0.522 \geq 0.30 \text{ in} \quad \text{OK!} \]
From AISC 360-10, Table 3.2 we obtain following parameters:

\[ L_p = 6.11 \text{ ft} \]
\[ L_r = 17.4 \text{ ft} \]
\[ \phi_b M_{px} = 473 \text{ kip. ft} \]
\[ \phi_b M_{rx} = 289 \text{ kip. ft} \]
\[ \phi_b BF = 16.3 \text{ kips} \]

\[ L_p < L_b \leq L_r \rightarrow \text{Inelastic LTB} \]

\[ \phi_b M_n = C_b \left( \phi_b M_{px} - \phi_b BF(L_b - L_p) \right) \leq \phi_b M_{px} \]

For Center Part where maximum moment occurs \( C_b = 1 \)

\[ \phi_b M_n = 1.0 \left( 473 \text{ kip. ft} - 16.3 \text{ kip}(10 \text{ ft} - 6.11) \right) \leq 473 \text{ kip. ft} \]

\( 409.6 \text{ kip. ft} \leq 473 \text{ kip. ft} \quad \text{OK!} \)

\[ A = 16.2 \text{ in}^2 \]
\[ l_x = 1140 \text{ in}^4 \]
\[ Z_x = 126 \text{ in}^3 \]
\[ b_f = 8.22 \text{ in} \]
\[ d = 20.8 \text{ in} \]
\[ h/t_f = 50 \text{ in} \]

Pre construction deflection \( \leq \frac{L}{360} \)

\[ \Delta_{nc} = \frac{p_p L^3}{28E_I} + \frac{5w_p L^4}{384E_I} \]

AISC 360-10, Table 3-23 (Case 1, Case 9)

\[ \Delta_{nc} = \frac{(15.78 \text{ kip})(30 \text{ ft} \times 12 \text{ in/ft})^3}{28(29000 \text{ ksi})(1140 \text{ in}^4)} + \frac{5 \left( 0.055 \frac{\text{kip}}{\text{ft}} \times \frac{1 \text{ ft}}{12 \text{ in}} \right) \left( 30 \text{ ft} \times 12 \text{ in/ft} \right)^4}{384(29000 \text{ ksi})(1140 \text{ in}^4)} \]

\[ \Delta_{nc} = 0.7953 \text{ in} + 0.0061 \text{ in} = 0.801 \text{ in} = \frac{L}{450} < \frac{L}{360} \quad \text{OK!} \]

\[ Camber = 0.8 (0.801) = 0.64 \]

AISC Design Guide 3. Chapter 5. Pg. 22

use 0.75 camber
Designing for Composite flexure:

- Live load reduction

\[ K_{LL} = 2 \text{ for interior beams} \]

\[ A_T = 30 \times 10 = 300 \text{ ft}^2 \]  
(We are using tributary areas for beams, since they will transfer loads as point loads)

\[ K_{LL}A_T = 600 \text{ ft} > 400 \text{ ft}^2 \]  Reducible

\[ LL = L L_0 \left( 0.25 + \frac{15}{\sqrt{K_{LL}A_T}} \right) \]  
ASCE 7-10, Eq. 4.7-1

\[ LL = 50 \left( 0.25 + \frac{15}{\sqrt{600}} \right) = 43.12 \text{ psf} \]

\[ P_{DL} = 2 \left( 15 \text{ ft} \times 10 \text{ ft} \left( \frac{50 \text{ lb}}{\text{ft}^2} + 30 \frac{\text{lb}}{\text{ft}^2} \right) + 15 \text{ ft} \times 26 \frac{\text{lb}}{\text{ft}} \right) = 24.78 \text{ kip} \]

\[ P_{LL} = 2(15 \text{ ft} \times 10 \text{ ft} \times 43.12 \text{ psf}) = 1.3 \text{ kip} \]

\[ w_{DL} = 55 \frac{\text{lb}}{\text{ft}} = 0.055 \frac{\text{kip}}{\text{ft}} \]

(We included construction dead loads as a point loads transferred from the girders)

\[ w_{DL} = 50 \frac{\text{lb}}{\text{ft}} = 0.05 \frac{\text{kip}}{\text{ft}} \]

\[ P_u = 1.2 P_{DL} + 1.6 P_{LL} = 1.2(27.78 \text{ kip}) + 1.6(1.3 \text{ kip}) = 35.4 \text{ kip} \]

\[ w_u = 1.4 \left( 0.055 \frac{\text{kip}}{\text{ft}} \right) = 0.077 \frac{\text{kip}}{\text{ft}} \]

- Figure E2: Moment Diagram
\[ M_{u_2} = \frac{w_u l^2}{8} + P_u a \]

AISC 360-10, Table 3-23 (Case 1, Case 9)

\[
M_{u_2} = \frac{0.077 \frac{kip}{ft} \times (30 \text{ ft})^2}{8} + 35.4 \text{ kip} \times 10 \text{ ft} = 366.06 \text{ kip. ft}
\]

\[
M_{u_{1,3}} = \frac{w_u L^2}{2} + P_u x
\]

\[
M_{u_{1,3}} = \frac{0.077 \frac{kip}{ft} \times 10 \text{ ft} \times 20 \text{ ft}}{2} + 35.74 \text{ kip} \times 10 \text{ ft} = 365.1 \text{ kip. ft}
\]

Effective width \( b = \min \left\{ \frac{30 \text{ ft}}{8}, \frac{30 \text{ ft}}{2} \right\}\) = 7.50 ft

Governs

AISC 360-10, Sec. I3.1a

Available flexural strength:

\[
h/t_f = 50 \text{ in}
\]

\[
3.76 \sqrt{\frac{E}{F_Y}} = 3.76 \sqrt{\frac{39000 \text{ ksi}}{50 \text{ kip}}} = 90.6 \text{ in}
\]

50 in < 90.6

\[
\frac{h}{t_f} < 3.76 \sqrt{\frac{E}{F_Y}}
\]

AISC 360 - 10, Sec.I3.2a

\( \rightarrow M_n \) determined from plastic stress distribution on the composite section

for the imit state of yielding

AISC 360-10, Sec. I3.2a

All W sections meet above requirements

AISC 360-10, Sec. I3.2a

\[
a_{\text{trial}} = \frac{\Sigma q_n}{0.85 f'_c b}
\]

AISC 360-10, Eq. 3-7

\[
\Sigma Q_n = 0.5 A_s F_Y = 0.5 (16.2 \text{ in}^2)(50 \text{ ksi}) = 405 \text{ kips}
\]

AISC 360-10, Fig. 3-3c

\[
a_{\text{trial}} = \frac{0.5 A_s F_Y}{0.85 f'_c b} = \frac{405 \text{ kips}}{0.85(3\text{ ksi}) \left(7.50 \text{ ft} \times 12 \text{ in/ft}\right)} = 1.76 \text{ in}
\]

\[
Y2 = Y_{\text{concrete}} - \frac{a_{\text{trial}}}{2}
\]

AISC 360-10, Eq. 3-6
\[ Y_2 = 5.25 - \frac{1.76}{2} = 4.37 \]

Use \( \sum Q_n = 405 \text{ kips} \) and \( Y_2 = 4.37 \text{ in} \) and Enter AISC 360 – 10, Table 3.19

**PNA is located in section 4 \( \rightarrow \sum Q_n = 488 \text{ kips} \) \( \rightarrow \phi_b M_n = 783 \text{ kip} - \text{ ft} \)

\[ \phi_b M_n = 783 \text{ kip} \cdot \text{ ft} > M_u = 366.06 \text{ kip} \quad \textbf{OK!} \]

\[ a_{actual} = \frac{\sum Q_n}{0.85 f'_c b} = \frac{488 \text{ kips}}{0.85(3\text{ ksi})(7.50 \text{ ft} \times 12 \text{ in/ft})} = 2.12 \text{ in} \]

**Steel anchor strength:**

\[ Q_n = 0.5 A_{sa} \sqrt{f'_c E_c} \leq R_g R_p A_{sa} F_u \quad \text{AISC 360-10, Eq. I8-1} \]

\[ Q_n = 0.5 \pi d_{sa}^2 \sqrt{f'_c \rho_c} = 0.5 \pi \left(\frac{3}{4} \text{ in}\right)^2 \sqrt{(3 \text{ ksi})((115)^{1.5} \sqrt{3} \text{ ksi})} = 17.67 \text{ kips/anchor} \]

\( R_g = 1.0 \text{ one or two steel studs welded in a rib} \quad \text{AISC 360-10, Sec. I8.2a} \)

\( R_p = 0.75 \)

\[ Q_n = R_g R_p A_{sa} F_u = 1 \times 0.75 \times \pi \left(\frac{3}{4} \text{ in}\right)^2 \times 65 \text{ ksi} = 21.53 \text{ kips} \quad \rightarrow \text{ Governs} \]

**Number and spacing of anchors**

\[ N_{anchor} = \frac{\sum Q}{Q_n} = \frac{405 \text{ kips}}{17.67 \text{ kips}} = 22.9 = 23 \text{ anchors} \]

**Max spacing = \( \min \left\{ \frac{8t_{slab}}{36}, \frac{8(5.25)}{42} \right\} = 42 \text{ in} \quad \text{Governs!} \quad \text{AISC 360-10, Sec. I8.2d} \)**

Add 3 anchors in the middle section,

Total # of anchors=26

**Live Load Deflection:**

\[ \Delta_{LL} = \frac{P_{LL} L^3}{2BE_{LL}} \quad \text{AISC 360-10, Table 3-23 (case 9)} \]

\[ \Delta_{LL} \leq \frac{L}{360} \quad \text{NYCBC 2014, Table 1604.3} \]
$I_{LB} = \text{Nominal strength of steel anchors and is calculated between the point of maximum positive moment and the point of zero moment}$

$I_{LB} = I_s + A_s(Y_{ENA} - d_3)^2 + \left( \frac{\Sigma Q}{F_y} \right) (2d_3 + d_1 - Y_{ENA})^2$  

$\Sigma Q_n = 26 \text{ anchors} \times 17.56 \frac{kips}{\text{Anchor}} = 452.13 \text{ kips}$

$a = \frac{\Sigma Q_n}{0.85f_y' b} = \frac{452.13 \text{ kips}}{0.85(3 \text{ ksi})(7.50 \text{ ft} \times 12 \text{ in/ft})} = 1.96 \text{ in}$

$d_1 = t_{slab} - \frac{a}{2} = 4.27 \text{ in}$  

$d_2 = \frac{x}{2} = \frac{A_{sFy} - \Sigma Q_n}{2bf_y}$  

$d_3 = \frac{d}{2} = \frac{20.8}{2} = 10.4 \text{ in}$  

$Y_{ENA} = \frac{A_s d_3 + \left( \frac{\Sigma Q}{F_y} \right) (2d_3 + d_1)}{A_s + \left( \frac{\Sigma Q}{F_y} \right)}$  

$Y_{ENA} = \frac{(16.2 \text{ in}^2)(10.4 \text{ in}) + \left( \frac{452.13 \text{ kips}}{50 \text{ ksi}} \right)(2 \times 10.4 \text{ in} + 4.27 \text{ in})}{16.2 \text{ in}^2 + \left( \frac{452.13 \text{ kips}}{50 \text{ ksi}} \right)} = 15.66 \text{ in}$

$I_{LB} = 1140 \text{ in}^4 + 16.2 \text{ in}^2 (15.66 \text{ in} - 10.4 \text{ in})^2 + (9.04 \text{ in}^2)(20.8 \text{ in} + 4.27 - 15.66 \text{ in})^2$

$I_{LB} = 2025.70 \text{ in}^4$

$\Delta_{LL} = \frac{P_{LL}L^3}{28E I_{LB}} = \frac{13 \text{ kip} (30 \times 12 \text{ in})^3}{28 \times 29000 \text{ ksi} \times 2025.70 \text{ in}^4} = 0.368 = \frac{L}{978} \ll \frac{L}{360} \text{ OK!}$

**Available Shear strength:**

$V_u = \frac{wul}{2} + P_u$  

$V_u = \frac{0.077 \text{ kip/ft} \times 30 \text{ ft}}{2} + 35.4 \text{ kip} = 36.56 \text{ kips}$

$\phi_V V_o = 234 \text{ kips} > 35.56 \text{ OK!}$

**USE W21x55 with camber=0.75**
Appendix H: Transfer Beam Hand Calculations:
Designing Transfer Beam at L15 between D2 and C2 grid lines (See drawing Set, Sheet-S118)

Requirements and Initial Data
- Steel A992
\[
\begin{align*}
F_y &= 50 \text{ ksi} \\
F_u &= 65 \text{ ksi}
\end{align*}
\]
AISC 360-10, Table 2.4
- Beam length \( L = 30 \text{ ft} \)
- Beam size = W30x124
- Slab thickness = \( t_{\text{slab}} = 5.25 \text{ in} \)

Dead Loads
Pre Composite:
\[
\text{Slab} = 50 \text{ psf} \quad \text{CMC Catalog, Sec. 3.1D, Pg.70}
\]
\[
\text{Beam Self Weight} = 100 \text{ lb/ft (Assumption)}
\]
Composite
\[
\text{Office Loads} = 30 \text{ psf}
\]
\[
\text{Roof (green)} = 60 \text{ psf}
\]

Transferred point loads from the column above at center point:
\[
\text{Point load} = 69.08 \text{ kip} \quad \text{(From ETABS Values)}
\]

Live Loads
Pre-Construction:
\[
\text{Construction} = 30 \text{ psf} \quad \text{CMC Catalog, Sec. 2.4A, Pg.67}
\]
Composite:
\[
\text{Office live loads} = 50 \text{ psf} \quad \text{ASCE 7-10, Table 4-1}
\]
\[
\text{Roof (green)} = 100 \text{ psf}
\]
Transferred point loads from the column above at center point:
\[
\text{Point load} = 52.53 \text{ kips} \quad \text{(From ETABS Values)}
\]
Composite Design:

- Minimum flange thickness \( t_f \) based on anchor diameter \( d_{sa} = \frac{3}{4} \text{in} \)

\[
d_{sa} \leq 2.5 \ t_f \quad \rightarrow \quad t_f \geq \frac{\frac{3}{4} \text{in}}{2.5} = 0.30 \text{ in}
\]

AISC 360-10, I8.1

Tributary Area = \( A_T = 30 \text{ ft} \times 10 \text{ ft} = 300 \text{ ft}^2 \)

\[
P_{DL} = 86.68 \text{ kip}
\]

\[
P_{LL} = 45.8 \text{ kip}
\]

\[
w_{DL} = 10 \text{ ft} \times (30 \text{ psf} + 60 \text{ psf}) + 100 \text{ lb/ft} = 1 \frac{\text{kip}}{\text{ft}}
\]

\[
w_{LL} = 10 \text{ ft} \times (50 \text{ psf} + 100 \text{ psf}) = 1.5 \frac{\text{kip}}{\text{ft}}
\]

\[
P_u = 1.2 \ P_{DL} + 1.6P_{LL} = 1.2 \ (86.68 \text{ kip}) + 1.6(45.8 \text{ kip}) = 177.2 \text{ kip}
\]

\[
w_u = 1.2 \ w_{DL} + 1.6w_{LL} = 1.2 \ (1 \frac{\text{kip}}{\text{ft}}) + 1.6 \left( 1.5 \frac{\text{kip}}{\text{ft}} \right) = 3.6 \frac{\text{kip}}{\text{ft}}
\]

\[
M_u = \frac{w_u L^2}{8} + \frac{P_u L}{4}
\]

AISC 360-10, Table 3-23 (Case 1, Case 7)

\[
M_x = \frac{w_u x}{2} (L - x) + \frac{P_u x}{2}
\]

\[
C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 5M_B + 3M_{max}}
\]

AISC 360-10, Eq. F1-1

\[
L_b = 15 \text{ ft}
\]
\[ M_A = \frac{3.6 \text{kip} \left( \frac{15 \text{ ft}}{4} \right)}{2} \left( 30 \text{ ft} - \frac{15 \text{ ft}}{4} \right) + \frac{177.2 \text{kip} \left( \frac{15 \text{ ft}}{4} \right)}{2} = 509.06 \text{ kip. ft} \]

\[ M_B = \frac{3.6 \text{kip} \left( \frac{15 \text{ ft}}{2} \right)}{2} \left( 30 \text{ ft} - \frac{15 \text{ ft}}{2} \right) + \frac{177.2 \text{kip} \left( \frac{15 \text{ ft}}{2} \right)}{2} = 967.5 \text{ kip. ft} \]

\[ M_C = \frac{3.6 \text{kip} \left( \frac{15 \text{ ft}}{4/3} \right)}{2} \left( 30 \text{ ft} - \frac{15 \text{ ft}}{4/3} \right) + \frac{177.2 \text{kip} \left( \frac{15 \text{ ft}}{4/3} \right)}{2} = 1375.31 \text{ kip. ft} \]

\[ M_u = M_{\text{max}} = \frac{3.6 \text{kip} \times (30 \text{ ft})^2}{8} + \frac{177.2 \text{kip} \times 30 \text{ ft}}{4} = 1732.5 \text{ kip. ft} \]

\[ C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 5M_B + 3M_{\text{max}}} = 1.46 \]

Enter Table 3.10 with \( L_b = 15 \text{ ft} \) and \( M_u = \frac{1732.5}{1.46} = 1186.64 \text{ kip. ft} \)

Try \( W30x124 \)

\[ t_f = 0.93 \geq 0.30 \text{ in} \quad \text{OK!} \]

From AISC 360-10, Table 3.2 we obtain following parameters:

\[ L_p = 7.88 \text{ ft} \]
\[ L_r = 23.2 \text{ ft} \]
\[ \phi_b M_{px} = 1530 \text{ kip. ft} \]
\[ \phi_b M_{rx} = 932 \text{ kip. ft} \]
\[ \phi_b BF = 39.0 \text{ kips} \]

\( L_p < L_b \leq L_r \rightarrow \text{Inelastic LTB} \)

AISC 360-10, Sec. F2.2b

\[ \phi_b M_n = C_b \left( \phi_b M_{px} - \phi_b BF(L_b - L_p) \right) \leq \phi_b M_{px} \]

AISC 360-10, Eq. 3.4a

For Center Part where maximum moment occurs \( C_b = 1 \)

AISC 360-10, Table 3.1

\[ \phi_b M_n = 1.46(1530 \text{ kip. ft} - 39 \text{ kip}(15 \text{ ft} - 7.88)) = 1828.39 \geq 1530 \text{ kip. ftz} \]

\[ \phi_b M_n = 1530 \text{ kip. ft} \]

From Table 1.1
May 3, 2018

1161 1st Avenue

\[ A = 38.8 \text{ in}^2 \]
\[ l_x = 5360 \text{ in}^4 \]
\[ Z_x = 408 \text{ in}^3 \]
\[ d = 30.2 \text{ in} \]
\[ h/t_w = 46.2 \text{ in} \]

Available flexural strength:

Effective width \( b = \min \left\{ \frac{30 \text{ ft}}{8} \cdot 2 = 7.50 \text{ ft} \quad \text{Governs} \quad \right\} \]
\[ b = 7.50 \text{ ft} \]
\[ h/t_f = 46.2 \text{ in} \]
\[ 3.76 \sqrt{\frac{E}{F_Y}} = 3.76 \sqrt{\frac{39000 \text{ ksi}}{50 \text{ kip}}} = 90.6 \text{ in} \]
\[ 46.2 \text{ in} < 90.6 \]

\[ \frac{h}{t_f} < 3.76 \sqrt{\frac{E}{F_Y}} \quad \text{AISC 360 - 10, Sec. I3.2a} \]

→ \( M_n \) determined from plastic stress distribution on the composite section

for the limit state of yielding \quad \text{AISC 360-10, Sec. I3.2a}

All W sections meet above requirements \quad \text{AISC 360-10, Sec. I3.2a}

\[ a_{trial} = \frac{\sum Q_n}{0.85 f'_c b} \quad \text{AISC 360-10, Eq. 3-7} \]

\[ \sum Q_n = 0.5 A_s F_Y = 0.5 (38.8 \text{ in}^2)(50 \text{ ksi}) = 970 \text{ kips} \quad \text{AISC 360-10, Fig. 3-3c} \]
\[ a_{trial} = \frac{0.5 A_s F_Y}{0.85 f'_c b} = \frac{970 \text{ kips}}{0.85(3 \text{ ksi})(7.50 \text{ ft} \times 12 \text{ in/ft})} = 4.23 \text{ in} \]

\[ Y2 = Y_{concrete} - \frac{a_{trial}}{2} \quad \text{AISC 360-10, Eq. 3-6} \]
\[ Y2 = 5.25 - \frac{4.23}{2} = 3.14 \]

Steel anchor strength:
\[ Q_n = 0.5 A_{sa} \sqrt{f_c'E_C} \leq R_g R_p A_{sa} F_u \]  
AISC 360-10, Eq. I8-1

\[ Q_n = 0.5 \pi d_{sa}^2 / 4 \sqrt{f_c'/\rho_c} = 0.5 \frac{\pi (3/4 \text{ in})^2}{4} \sqrt{(3 \text{ ksi}) ((115)^{1.5} \sqrt{3 \text{ ksi}})} = 17.67 \text{ kips/anchor} \]

\[ R_g = 1.0 \text{ one or two steel studs welded in a rib} \]  
AISC 360-10, Sec. I8.2a

\[ R_p = 0.75 \]

\[ Q_n = R_g R_p A_{sa} F_u = 1 \times 0.75 \times \frac{\pi (3/4 \text{ in})^2}{4} \times 65 \text{ ksi} = 21.53 \text{ kips} > 17.67 \text{ kips} \]

\[ Q_n = 17.67 \text{ kips} \]

Number and spacing of anchors

\[ N_{anchor} = \frac{\Sigma Q}{Q_n} = \frac{970 \text{ kips}}{17.67 \text{ kips}} = 56 \text{ anchors} \]

Max spacing = \[ \min \{ \frac{8t_{slab}}{36} = 8(5.25) = 42 \text{ in} \]  
\[ \text{Governs!} \]  
AISC 360-10, Sec. I8.2d

Live Load Deflection:

\[ \Delta_{LL} = \frac{P_{LL} L^3}{48 E I_{LB}} + \frac{5 w_{LL} L^4}{384 E I_{LB}} \]  
AISC 360-10, Table 3-23 (Case 1, case 7)

\[ \Delta_{LL} \leq \frac{L}{360} \]  
NYCBC 2014, Table 1604.3

\[ I_{LB} = \text{Nominal strength of steel anchors and is calculated between the point of maximum positive moment and the point of zero moment} \]

\[ I_{LB} = I_s + A_S (Y_{ENA} - d_3)^2 + \left( \frac{\Sigma Q}{F_y} \right) (2d_3 + d_1 - Y_{ENA})^2 \]  
AISC 360-10, Eq. C-I3-1

\[ \Sigma Q_n = 21 \text{ anchors} \times 21.53 \frac{\text{kips}}{\text{Anchor}} = 452.13 \text{ kips} \]

\[ a = \frac{\Sigma Q_n}{0.85 f_c' b} = \frac{970 \text{ kips}}{0.85(3 \text{ ksi})(7.50 \text{ ft} \times 12 \text{ in/ft})} = 4.23 \text{ in} \]

\[ d_1 = t_{slab} - \frac{a}{2} = 3.14 \text{ in} \]  
AISC 360-10, Fig. C-I3.3

\[ d_3 = \frac{d}{2} = \frac{30.2}{2} = 15.1 \text{ in} \]  
AISC 360-10, Sec. C-I3.2

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\[ Y_{ENA} = \frac{A_s d_3 + \left( \frac{V_0}{F_y} \right)(2d_3 + d_3)}{A_s + \left( \frac{V_0}{F_y} \right)} \]

AISC 360-10, Eq. C-I3-2

\[ Y_{ENA} = \frac{(38.8 \text{ in}^2)(15.1 \text{ in}) + \left( \frac{970 \text{ kips}}{50 \text{ ksi}} \right)(2 \times 15.1 \text{ in} + 3.14 \text{ in})}{38.8 \text{ in}^2 + \left( \frac{970 \text{ kips}}{50 \text{ ksi}} \right)} = 21.18 \text{ in} \]

\[ I_{LB} = 5360 \text{ in}^4 + 38.8 \text{ in}^2(21.18 \text{ in} - 15.1 \text{ in})^2 + (19.9 \text{ in}^2)(30.2 \text{ in} + 3.14 - 21.18 \text{ in})^2 \]

\[ I_{LB} = 5825.55 \text{ in}^4 \]

\[ \Delta_{LL} = \frac{P_{LL}L^3}{28EI_{LB}} = \frac{1.3 \text{ kip} (30 \times 12 \text{ in})^3}{28 \times 29000 \text{ ksi} \times 2025.70 \text{ in}^4} = 0.425 \approx \frac{L}{847.01} \ll \frac{L}{360} \quad \text{OK!} \]

\[ \Delta_{LL} = \frac{P_{LL}L^3}{48EI_{LB}} + \frac{5w_{LL}L^4}{384EI_{LB}} \]

\[ \Delta_{LL} = \frac{(45.8 \text{ kip})(30 \text{ ft} \times 12 \text{ in} \text{ ft}^{-1})^3}{48(29000 \text{ ksi})(5825.55 \text{ in}^4)} + \frac{5 \left(1.5 \text{ kip} \text{ ft}^{-1} \times 1 \text{ ft} \times 12 \text{ in} \text{ ft}^{-1} \right)(30 \text{ ft} \times 12 \text{ in} \text{ ft}^{-1})^4}{384(29000 \text{ ksi})(5825.55 \text{ in}^4)} = 0.425 \]

\[ \Delta_{LL} = 0.425 \approx \frac{L}{847.01} \ll \frac{L}{360} \quad \text{OK!} \]

Available Shear strength:

\[ V_u = \frac{w_{ul}}{2} + \frac{p_u}{2} \]

AISC 360-10, Table 3-23 (Case 1, Case 9)

\[ V_u = \frac{3.6 \text{ kip} \text{ ft}^{-1} \times 30 \text{ ft}}{2} + \frac{177.2 \text{ kip}}{2} = 142.6 \text{ kips} \]

\[ \phi_v V_v = 530 \text{ kips} > 142.6 \text{ kips} \quad \text{OK!} \]

AISC 360-10, Table 3-2

Use W30x124, no camber
Appendix I: Foundation Wall Design

Initial data:

Well graded sand = SW

Dry unit weight = $\gamma = 110$ pcf

Saturated unit weight = $\gamma_{sat} = 130$ pcf

Friction angle = $\varphi' = 38^\circ$

Water unit weight = $\gamma_w = 62.4$ pcf

Surcharge = $q = 250$ psf (ASCE 7-10, Table 4-1)

Find: Lateral Force Acting on the foundation wall

$$K_0 = 1 - \sin \varphi' = 1 - \sin(38^\circ) = 0.384$$

Vertical effective stress = $\sigma'_v = \gamma z$

Horizontal effective stress = $\sigma'_h = K_0 \sigma'_v$

@ $z = 12$ ft

$$\sigma'_v = q = 250 \text{ psf}$$

$$\sigma'_h = K_0 q = 0.384(250 \text{ psf}) = 96 \text{ psf}$$

$$u = 0$$

@ $z = 12$ ft

$$\sigma'_v = q + \gamma H_1 = 250 \text{ psf} + 110 \text{ pcf}(12 \text{ ft}) = 1570 \text{ psf}$$

$$\sigma'_h = K_0 \sigma'_v = 0.384(1570 \text{ psf}) = 602.88 \text{ psf}$$

$$u = 0$$

@ $z = 24$ ft

$$\sigma'_v = q + \gamma H_2 = 1570 \text{ psf} + 12 \text{ ft}(110\text{pcf}) = 2670 \text{ psf}$$

$$\sigma'_h = K_0 \sigma'_v = 0.384(2670 \text{ psf}) = 1025 \text{ psf}$$

$$u = 0$$

Using SAP 2000, we obtain following loading diagram which yields in the deflected shape shown in figure E3, and Shear and bending moment diagrams shown in Figure E4 and E5:
Figure I2: Loading Condition for foundation wall (From SAP 2000)

Figure I3: deflected shape of foundation wall (From SAP 2000)
Figure I4: Shear distribution of foundation wall (From SAP 2000)

Figure I5: Moment distribution of foundation wall (From SAP 2000)
SP-Column was used to design the foundation wall, the results obtained is shown below

**Figure I6. Foundation wall design**

<table>
<thead>
<tr>
<th>MATERIAL:</th>
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<tbody>
<tr>
<td>( f'c = 4 \text{ ksi} )</td>
</tr>
<tr>
<td>( Ec = 3605 \text{ ksi} )</td>
</tr>
<tr>
<td>( f_c = 3.4 \text{ ksi} )</td>
</tr>
<tr>
<td>( \beta_1 = 0.85 )</td>
</tr>
<tr>
<td>( f_y = 60 \text{ ksi} )</td>
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<tr>
<td>( E_s = 29000 \text{ ksi} )</td>
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<table>
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<th>SECTION:</th>
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<tbody>
<tr>
<td>( A_g = 144 \text{ in}^2 )</td>
</tr>
<tr>
<td>( b_x = 1728 \text{ in}^4 )</td>
</tr>
<tr>
<td>( b_y = 1728 \text{ in}^4 )</td>
</tr>
<tr>
<td>( x_0 = 0 \text{ in} )</td>
</tr>
<tr>
<td>( y_0 = 0 \text{ in} )</td>
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</tbody>
</table>

<table>
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<th>REINFORCEMENT:</th>
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<tbody>
<tr>
<td>( 4 #5 \text{ bars @ 1.22%} )</td>
</tr>
<tr>
<td>( A_0 = 1.76 \text{ in}^2 )</td>
</tr>
<tr>
<td>Confinement: Tied</td>
</tr>
<tr>
<td>Clear Cover = 1.00 in</td>
</tr>
<tr>
<td>Min Clear Spacing = 8.50 in</td>
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<table>
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<th>SLENDERNESS:</th>
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<tbody>
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</tbody>
</table>

12 × 12 in
1.22% reinf.
Appendix J: Base Plate Design

Base Plates are designed using Excel spreadsheet. A sample calculation table is provided in this appendix along with the initial values. Please see:

\[ CE4822\_S18\_Group01\_BasePlateCalculations \]

Initial data

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<td>grout</td>
<td>( f_c ) (ksi)</td>
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</tr>
<tr>
<td>( F_u )</td>
<td>58</td>
<td>ksi</td>
</tr>
<tr>
<td>( \phi_c )</td>
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</tr>
<tr>
<td>Hole</td>
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<td>in</td>
</tr>
</tbody>
</table>

<table>
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<td>ksi</td>
</tr>
<tr>
<td>( F_u )</td>
<td>65</td>
<td>ksi</td>
</tr>
</tbody>
</table>

From Design ASCI design Guide 1:

\( \text{Case III: } A_1 < A_2 < 4A_1 \)

1. Calculate the required axial compressive strength, \( P_c(\text{LRFD}) \) or \( P_c(\text{ASD}) \).

2. Calculate the approximate base plate area based on the assumption of Case III.

\[
A_{b\text{(approx)}} = \frac{P_c}{2\phi_c 0.85 f'_c} \quad (\text{LRFD})
\]

Limit States: \( \phi_c P_p > P_u \)
Initial data | Initial calculations
---|---

<table>
<thead>
<tr>
<th>Column Dimensions</th>
<th>$P_u$ (kips)</th>
<th>$A_{1\text{req}}$ (in²)</th>
<th>$N$ (in)</th>
<th>$B$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>d (in)= 16.7</td>
<td>1005</td>
<td>227.38</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>$b_1$ (in)= 16.1</td>
<td>$\sqrt{A_{1\text{req}}}$</td>
<td>Calculate using delta</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$t_f$ (in)= 2.07</td>
<td>15.08</td>
<td>N (in)</td>
<td>B (in)</td>
<td></td>
</tr>
<tr>
<td>$t_w$ (in)= 1.29</td>
<td></td>
<td>16.5714776</td>
<td>12.63197587</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bearing strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_1$ (in²)</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>324</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Required thickness</th>
<th>$t_{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X$</td>
<td>$m$ (in)</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>0.702</td>
<td>1.068</td>
</tr>
<tr>
<td>4.099</td>
<td></td>
</tr>
</tbody>
</table>

**Chosen Plate**

PL 2x18x18
Appendix K: Buttress Design

Base plates are design using Excel spreadsheet. Please see attached file:

CE4822_S18_Group01_ButtressCaluclations

Initial Data

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete grade</td>
<td>4000 psi</td>
<td></td>
</tr>
<tr>
<td>Steel Grade</td>
<td>60000 psi</td>
<td></td>
</tr>
<tr>
<td>Φ</td>
<td>0.65</td>
<td>tied columns</td>
</tr>
<tr>
<td>Concrete cover</td>
<td>1.5 in</td>
<td></td>
</tr>
</tbody>
</table>

Limit states From ACI 318-14: \( \varphi_c P_n > P_u \)

Formulas used:

\[
0.01(A_g-A_{st}) < \rho < 0.08(A_g-A_{st}) \quad \text{ACI 318-14 Sec. 10.6.1.2.}
\]

\[
P_0 = 0.85f_c'(A_g - A_{st}) + f_y A_{st} \quad \text{ACI 318-14 Eq. 22.4.2.2}
\]

\[
P_{n,\text{max}} = 0.80P_0 \quad \text{ACI 318-14 Table 22.4.2.1}
\]

<table>
<thead>
<tr>
<th>Note:</th>
<th>All columns are square columns</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Equal reinforcement on all sides</td>
</tr>
<tr>
<td></td>
<td>The Notation A1 represents the gridline intersections, where the buttress is located</td>
</tr>
<tr>
<td>Butress</td>
<td>$\Phi P_n$ (kip)</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------</td>
</tr>
<tr>
<td>A1</td>
<td>405</td>
</tr>
<tr>
<td>A2</td>
<td>797</td>
</tr>
<tr>
<td>A3</td>
<td>357</td>
</tr>
<tr>
<td>A4</td>
<td>345</td>
</tr>
<tr>
<td>A5</td>
<td>899</td>
</tr>
<tr>
<td>A6</td>
<td>406</td>
</tr>
<tr>
<td>F1</td>
<td>789</td>
</tr>
<tr>
<td>F2</td>
<td>2041</td>
</tr>
<tr>
<td>F3</td>
<td>724</td>
</tr>
<tr>
<td>F4</td>
<td>722</td>
</tr>
<tr>
<td>F5</td>
<td>1960</td>
</tr>
<tr>
<td>F6</td>
<td>811</td>
</tr>
<tr>
<td>B1</td>
<td>684</td>
</tr>
<tr>
<td>C1</td>
<td>381</td>
</tr>
<tr>
<td>D1</td>
<td>786</td>
</tr>
<tr>
<td>E1</td>
<td>1324</td>
</tr>
<tr>
<td>B6</td>
<td>684</td>
</tr>
<tr>
<td>C6</td>
<td>391</td>
</tr>
<tr>
<td>D6</td>
<td>808</td>
</tr>
<tr>
<td>E6</td>
<td>1338</td>
</tr>
</tbody>
</table>
Appendix L: Isolated Footing Design

Base Plates are design using Excel spreadsheet. Sample calculation table is provided in this appendix along with the initial values. Please see attached file:

CE4822_S18_Group01_IsolatedFootingCaluclations

Initial data:

<table>
<thead>
<tr>
<th>Concrete details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Bearing Capacity=</td>
<td>24000</td>
</tr>
<tr>
<td>Concrete grade=</td>
<td>4000</td>
</tr>
<tr>
<td>Soil Density=</td>
<td>110</td>
</tr>
<tr>
<td>Concrete density=</td>
<td>150</td>
</tr>
<tr>
<td>Slab=</td>
<td>1</td>
</tr>
<tr>
<td>Concrete cover=</td>
<td>3</td>
</tr>
<tr>
<td>λ=</td>
<td>1</td>
</tr>
<tr>
<td>φ=</td>
<td>0.75</td>
</tr>
<tr>
<td>β=</td>
<td>0.85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reinforcement details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>j=</td>
<td>0.95</td>
</tr>
<tr>
<td>φ=</td>
<td>0.9</td>
</tr>
<tr>
<td>$f_{y}$=</td>
<td>60</td>
</tr>
</tbody>
</table>

Assumptions:

<table>
<thead>
<tr>
<th>columns at the center of the footing</th>
<th>$\alpha_s=40$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of footing assumption</td>
<td>1 to 2 times the late width</td>
</tr>
</tbody>
</table>
Equations used (design was guided By Reinforced Concrete design, & the d. James K. Wight):

\[
q_n = P_{alwa} - \left( \frac{W_{footing}}{f_t^2} + \frac{W_{soil}}{f_t^2} + \frac{W_{slab}}{f_t^2} + P_u \right)
\]

\[
A_{REQ} = \frac{P_u}{q_n}
\]

Checks for two way shear

\[
V_u = q_u(A - (b_{column} + d)^2)
\]

\[
b_0 = 4(c_1 + d)
\]

\[
v_u = \frac{V_u}{b_0 \times d}
\]

\[
v_c = \min\left\{ \frac{4\sqrt{f_c'}}{(2 + \frac{\alpha}{b})\lambda \sqrt{f_c'}} \left(2 + \frac{\alpha_{sd}}{b_0}\right) \lambda \sqrt{f_c'} \right\}
\]

ACI 318-14 Sec. 22.6.4.1

\[
\phi v_c > v_u \quad \text{OK}
\]

Checks for one way shear

\[
\phi V_c = \phi 2\lambda \sqrt{f_c'} b_w d > V_u \quad \text{OK!}
\]

Design the Flexural reinforcement

Assume: \( j = 0.95 \) & \( \varphi = 0.90 \)

\[
A_{s,min} = \max\left\{ \frac{0.0018f_y}{f_y}A_{g}, \frac{0.0014A_{g}}{0.85f_c'b} \right\}
\]

ACI 318-14 Sec. 8.6.1.1

\[
a = \frac{A_s f_y}{0.85 f_c' b}
\]

\[
\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) > M_u \quad \text{OK!}
\]
<table>
<thead>
<tr>
<th>Base plate width (in)</th>
<th>Base plate length (in)</th>
<th>$\alpha_s$</th>
<th>$P$ (kips)</th>
<th>$P_u$ (kips)</th>
<th>Depth (in)</th>
<th>$q_u$ (kips/ft)</th>
<th>$A_{rcr}$ (m$^2$)</th>
<th>$A_{sncr}$ (m$^2$)</th>
<th>approximate square</th>
<th>Square (ft$^2$)</th>
<th>$q_u$ (kips)</th>
<th>$\phi M_s$ (kip-ft)</th>
<th>$\phi V_s$ (kips)</th>
<th>$V_s$ (kips)</th>
<th>$V_v$ (psf)</th>
<th>$\phi V_v$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>18</td>
<td>40</td>
<td>590.343</td>
<td>1005.01</td>
<td>27</td>
<td>23.55</td>
<td>25.06</td>
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<td>6.00</td>
<td>27.91</td>
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<td>679.11</td>
<td>164.00</td>
<td>180.04</td>
<td>189.74</td>
<td>55.83</td>
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<tr>
<td>33</td>
<td>22</td>
<td>40</td>
<td>759.46</td>
<td>1655.00</td>
<td>33</td>
<td>23.48</td>
<td>32.35</td>
<td>5.69</td>
<td>7.00</td>
<td>33.80</td>
<td>29.00</td>
<td>1045.56</td>
<td>204.00</td>
<td>176.73</td>
<td>189.74</td>
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<tr>
<td>33</td>
<td>22</td>
<td>40</td>
<td>759.46</td>
<td>1655.00</td>
<td>33</td>
<td>23.48</td>
<td>32.35</td>
<td>5.69</td>
<td>7.00</td>
<td>33.80</td>
<td>29.00</td>
<td>1044.93</td>
<td>204.00</td>
<td>176.63</td>
<td>189.74</td>
<td>39.40</td>
</tr>
<tr>
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<td>18</td>
<td>40</td>
<td>593.343</td>
<td>1023.03</td>
<td>27</td>
<td>23.55</td>
<td>25.19</td>
<td>5.02</td>
<td>6.00</td>
<td>28.42</td>
<td>23.00</td>
<td>691.27</td>
<td>164.00</td>
<td>183.26</td>
<td>189.74</td>
<td>56.83</td>
</tr>
<tr>
<td>27</td>
<td>18</td>
<td>40</td>
<td>593.343</td>
<td>1023.03</td>
<td>27</td>
<td>23.55</td>
<td>25.19</td>
<td>5.02</td>
<td>6.00</td>
<td>28.42</td>
<td>23.00</td>
<td>691.27</td>
<td>164.00</td>
<td>183.26</td>
<td>189.74</td>
<td>56.83</td>
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<tr>
<td>27</td>
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<td>40</td>
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<td>1023.03</td>
<td>27</td>
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<td>25.19</td>
<td>5.02</td>
<td>6.00</td>
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<td>23.00</td>
<td>691.27</td>
<td>164.00</td>
<td>183.26</td>
<td>189.74</td>
<td>56.83</td>
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<tr>
<td>27</td>
<td>18</td>
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<td>6.00</td>
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<td>164.00</td>
<td>183.26</td>
<td>189.74</td>
<td>56.83</td>
</tr>
<tr>
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<td>18</td>
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<td>593.343</td>
<td>1023.03</td>
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<td>23.55</td>
<td>25.19</td>
<td>5.02</td>
<td>6.00</td>
<td>28.42</td>
<td>23.00</td>
<td>691.27</td>
<td>164.00</td>
<td>183.26</td>
<td>189.74</td>
<td>56.83</td>
</tr>
</tbody>
</table>
Works Cited


4. Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary on Building Code Requirements for structural Concrete (ACI 318R-14), American Concrete Institute, 2014.


