Zenith Tower - Design Report

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Zenith Tower
20 W 34th St, New York NY 10001

Table of Contents

1. About Elite Engineering ................................................................. 5
2. Project Outline ................................................................................. 5
   2.1. Project Location ......................................................................... 5
   2.2. Project Purpose and Scope ......................................................... 6
3. Architectural Design ....................................................................... 8
   3.1. Architectural Layout Description ............................................... 8
       3.1.1. Sub-Cellar and Cellar ......................................................... 8
       3.1.2. First Floor/Lobby ............................................................... 8
       3.1.3. Mezzanine ....................................................................... 10
       3.1.4. Typical Floor .................................................................. 10
       3.1.5. Terraced Floors ................................................................. 10
       3.1.6. Green Roof ..................................................................... 11
       3.1.7. Curtain Wall System ......................................................... 12
4. Codes and Manuals ........................................................................ 13
5. Load Calculations ........................................................................... 13
   5.1. Dead Loads (DL) ................................................................. 13
   5.2. Live Loads (LL) ................................................................. 14
   5.3. Superimposed Dead Loads (SDL) .......................................... 14
   5.4. Snow Loads with Drift (S) ..................................................... 15
   5.5. Wind Loads (W) ............................................................... 16
   5.6. Seismic Loads (E) ............................................................ 18
6. Global Analysis .............................................................................. 22
   6.1. Modal Response .................................................................. 22
   6.2. Seismic Force Resistance System (SFRS) ............................... 26
   6.3. Wind Load Response ........................................................... 29
7. Structural System ........................................................................... 30
   7.1. Gravity Force Resisting System ............................................. 30
   7.2. Lateral Force Resisting System .............................................. 31
8. Structural Components .................................................................... 31
   Composite Floor System .............................................................. 33
   8.1. Composite Beams .................................................................. 34
   8.2. Gravity Column ................................................................... 35
   8.3. Foundation ........................................................................... 35
Table of Tables
Table 1: Occupancy Loads for Typical Floor ................................................................. 15
Table 2: Snow Load Initial Parameters ......................................................................... 15
Table 3: Slow Load and Drift Values ............................................................................. 16
Table 4: Wind Load Parameters .................................................................................... 18
Table 5: Seismic Design Parameters ........................................................................... 19
Table 6: Calculated Values for Equivalent Lateral Force Analysis ......................... 20
Table 7: Seismic Design Coefficients .......................................................................... 26
Table 8: Seismic Base Reactions .................................................................................. 28
Table 9: Loading Schedule .......................................................................................... 40
1. **About Elite Engineering**

   Elite Engineering is a global structural solutions firm whose core pillars include excellence, innovation, sustainability, inclusivity, and above all else: the success of our clients. We thrive to have projects that are on time, on budget, and have our Elite quality assurance. Elite Engineering was established in 2019 and has quickly risen in the ranks in our field. Our focus on sustainability and innovation has kept us ahead of curve and, more importantly, pushing the boundaries of what a structural solution looks like.

2. **Project Outline**

   The purpose of this report is to outline the design strategy behind the construction of Zenith Tower.

2.1. **Project Location**

   The location of Zenith Tower is to be erected at 20 W 34th St, New York NY 10001 which is located in Midtown of Manhattan. The cross streets for this location are 34th St between 5th and 6th Ave. The approximate coordinates for this project are 40.7484°N, 73.9857°W. A site map for this location can be found in Figure 1 below. This location not only creates the opportunity to add to the iconic New York City skyline but promises significant interest by businesses as this area is close to other major businesses as well as residential buildings.
2.2. **Project Purpose and Scope**

The purpose of Zenith Tower will be to provide commercial and office spaces to its tenants as well as including certain amenities such as a restaurant, dance/event space, and cafes. As sustainability is a focus among us at Elite Engineering, green roofs will also be implemented throughout the design in order to offset some of the carbon footprint created by the everyday uses of the building.

In terms of existing conditions for this building, they are as follows: a subsurface investigation revealed the top 25 ft of soil are made up of well graded sand (SW) with an allowed bearing capacity of 12,000 psf and a percolation rate of 4 in/hr. The sand layer is underlain by a layer of fractured rock that has an allowable bearing capacity of 24,00 psf.

In terms of the structural scope of the building, the building will be designed as a structural steel building with a primarily glass curtain wall making up the façade. Particulars and specific selections for members and systems will be discussed further in
the report. The primary footprint of the building is 150’ x 160’ or 24,000 sqft. The building is made up of a total of 22 floors including the bulkhead. Throughout the course of 4 setbacks or terraces, the roof perimeter is 100’ x 100’ or 10,000 sqft. All setbacks and terraces will be accessed by the public and with utilize a green roof system. As mentioned previously, the majority of floors will be designated as office space. Two cellars beneath the building will contain over 60 parking spots including ADA accessible spots.

Additional amenities in the building include café and restaurant spaces on the lobby and mezzanine floors, as well as an event space on the 13th floor. Additional specifics in terms of floor plans can be found further in the report as well as in the drawing set.

The deliverables for this project will consist of architectural and structural. The architectural plans to be submitted will contain floor plans with architectural details, provided amenities, and means of egress. All of these shall be in accordance with the International Building Code. The structural deliverables will consist of framing plans, foundation plans, sections and details of connections, details of the structural system in terms of gravity and lateral systems, a column schedule, sample hand calculations and the analysis done on the building done with computer aided structural analysis. For all member selections and analysis done, logic and/or calculations shall be shown in order to defend the choice. The final set of deliverables for Zenith Tower will consist of a cost estimate as well as theorized construction schedule with an understood estimate of risk.
3. Architectural Design

The following explains the architectural selections and design for Zenith Tower in order for a better understanding of the building layout.

3.1. Architectural Layout Description

The overall concept for Zenith Tower is a modern space which utilizes the site as well as creating a sleek silhouette. Features include pocketed doors, balconies on several floors at different sides of the building, flexible floor plans for tenants, and green space. The design of the building is meant to reflect what is being seen in much of modern architecture throughout New York today such as glass facades, asymmetry, and buildings which taper as they rise in elevation. By creating flexible floor plans as well as access to green space in office floors, tenants can ensure a layout that fits their needs as well as optimizing productivity and raising work environments.

3.1.1. Sub-Cellar and Cellar

Zenith Tower contains two sub-grade floors which will contain 70 parking spots including ADA accessible spaces. Access to the parking will be via a ramp on the street level on the east side of the building. The ramps to both parking areas will be two-way to allow for circulation, with a counterclockwise run down the ramp. The slab used on the sub-cellar floor shall be non-structural. A core which runs the full elevation of the building shall start in the sub-cellar. The core shall include four passenger elevators, a service elevator, two staircases for emergency egress, an electrical closet, and a janitor’s closet. Two additional storage spaces within the core on the sub-cellar and cellar level, transform to men’s and women’s bathrooms on floors above.

3.1.2. First Floor/Lobby

The first floor of Zenith Tower continues with footprint of the sub-grade floors below with the addition of 20’ x 30’ pockets on the east and west sides of the building which create pockets for revolving doors on either side. Space on the north and south of the building are reserved for retail space and may be outfitted with kitchens should a café or restaurant obtain the space. Once entering the lobby
for tenants of Zenith Tower, patrons will be greeted by reception desks as well as common lobby outfitting’s. Before entering the entrance to either staircase or elevator, members will pass through security in the form of security e-gates. A general lobby floor plan for Zenith Tower can be found in Figure 2 below. The bottom L shaped retail space has been configured specifically to house some type of restaurant or café which needs the use of a kitchen, therefore in that area additional occupancy loads have been added in order to take into account this additional machinery, as will be discussed later on, however should this space not be rented as such a use, it may be outfitted or retail or any other public use.

Figure 2: Lobby Floor Plan
3.1.3. **Mezzanine**

The retail space to be seen at the north and south of building continues up to a partial mezzanine. These separate spaces would be outfitted with stairs not seen on the architectural plans. This mezzanine space will allow for additional retail or seating space by these tenants as well as creating a double height space for the internal lobby. This will help achieve a more grandiose entrance from those who work and enjoy this space.

3.1.4. **Typical Floor**

As terraces set back floor places several times throughout the building, there is not one typical floor plan throughout the building. However, as each tenant shall outfit the floor to their needs, each floor remains open for their own additions of walls or internal partitions. Column locations remain constant throughout the building to ensure continuity throughout the design as well as allowing each tenant to create their own space without compromising on structural integrity. Elements seen within the core remain constant throughout each of the floors above as seen in the lobby.

3.1.5. **Terraced Floors**

Floors 6, 10, 13, and 18 contain setbacks or terraces. The terrace on the 6th floor is on the west side of the building and is 20’ wide, for the 10th it’s on the east side and is 20’ wide, for the 13th it’s on the north side and 30’ wide, and the 18th floor contains a 10’ setback on all sides. All these terraces will contain the green roof system to be discussed next in this report. There will be several access points for the tenants throughout the floor and each terrace will contain a 4’ brick partition to ensure their safety. The 13th floor and its terrace shall be reserved to be rented out to be used as an event space or dance hall and therefore shall not be rented out long term by a tenant for other purposes.
3.1.6. **Green Roof**

As mentioned previously a green roof system shall be implemented throughout the building to aid in sustainability and carbon footprint of the building. The green roof system chosen for Zenith Tower is the LiveRoof STANDARD SYSTEM. The detail for this system can be found in *Figure 3* below as well as in the detail section of the drawing set. This system was chosen for its flexibility in use, its low additional load, and its previous successful use in similar projects. This green roof system shall be implemented within each terrace, as well as on the roof of the building which shall only be accessed for maintenance or use of bulkhead utilities. This system shall not only raise the enjoyment of the space for the tenant but shall also prove an effective method to deal with excess rainwater that may collect on the roof.

![Figure 3: Green Roof System](image-url)
3.1.7. **Curtain Wall System**

The curtain wall system chosen for Zenith Tower is one by KAWNEER Company. A company with offers several curtain wall solutions as well as keeping in mind sustainability, and thermal efficacy. The system chosen is the 1600 Wall System 4 Curtain Wall which is 6 inches deep with a 1 1/4" infill. This system was chosen for a variety of reasons. One which is this system is specifically applicable for mid-rise steel frame building such as this. As well, KAWNEER has previously ensured that their designed adhere to the NYCBC, allowing for no modifications being needed to occur for this system to work. As well, thermal performance technology within the system such as thermal breaks and peptide technology makes this system work well. The sustainability in materials used as well as performance for building efficiency in terms of heating and cooling makes this a good choice. Finally, it was determined that the cladding strength of the system can sustain the external forces this system will feel including lateral and wind.
4. **Codes and Manuals**

The design of Zenith Tower is governed by the local building code of New York City, the most current having been published in 2022 (NYCBC 2022). All of what is referenced throughout this report in terms of supplemental codes are done so out of accordance to NYCBC. All additional codes have been referenced within the NYCBC or the code referenced is more conservative. These codes include ASCE 7-16, AISC 360-16, and egress requirements seen in NYCBC. References shall be made within calculations as well as throughout this report when selections are explained. All references shall be found at the end of this report.

5. **Load Calculations**

Throughout this design, several different types of loads were considered. The overarching types of loads were gravity loads and lateral loads. These types are then further broken down into the types that are seen below in the further subsections of this section. This section of the report will quantify and explain the implication of each of these types of loads on the design of Zenith Tower.

5.1. **Dead Loads (DL)**

Dead loads represent self-weight of the structural system used throughout the building. This may contain several different elements such as the self-weight of beams, columns, composite metal decks, and the weight of the building. Based on the members chosen and the use of the different floors, dead loads may differ. This may be based on the addition of stairs, elevators, or the size of the members. A full table of the for the dead load of each floor can be found in *Appendix A*.

In terms of the composite metal deck chosen for Zenith Tower, the NYCBC was consulted for requirements in terms of thickness requirements for fire as well as load. For this project the VULCRAFT 2VLI deck was chosen with 3 ¼” of light weight concrete (LWC) with a 2” deck and 18” deck gauge. This ensures a 2-hour fire rating as required.
5.2. Live Loads (LL)

Live loads throughout this project will largely be based on the floors occupants as well was movable furniture including the final architectural fit out or arrangement that each tenant makes. The live loads within this building were done in accordance with ASCE 7-16, as referenced by NYCBC. It was determined in line with the use of the building as well as expecting certain addition equipment on some floors. Occupancy loads which tabulate the expected live loads, in psf, can be found in the section below which contains indications where live load reduction has been allowed within this building in accordance with NYCBC. These values were found within ASCE 7-16.

Throughout the designation and implementation of live loads throughout Zenith Towers it was determined that uniform loads in bays/areas of typical floors governed over the same calculation using concentrated loads, an analysis indicated by NYCBC Ch. 16. A sample calculation of this analysis can be found in Appendix A. For this reason, all values seen throughout the design for live loads, are uniform loads in psf unless otherwise noted.

5.3. Superimposed Dead Loads (SDL)

The implication of superimposed dead loads within this context are additional dead loads that add onto the structural system but are not directly apart of it. In a building such as Zenith Tower, this may include drop ceilings, HVAC, duct work, any additional applied membranes, lighting fixtures, storage or IT space, sprinkler systems etc. In the case of this project the green roof is also considered to by superimposed as it is not directly structural material. The load values for these elements were found using ASCE 7-16 and the tabulated value for the occupancy loads for a typical floor can be found in Table 1 below. Appendix A contains a full table of all occupancy loads within the building. With different floor layouts containing various combinations of these elements, the total SDL is as based on the respective elements from that floor plan, as can be seen on the architectural drawings.
Table 1: Occupancy Loads for Typical Floor

<table>
<thead>
<tr>
<th>FL.</th>
<th>OCC.</th>
<th>LL RED</th>
<th>SLAB</th>
<th>CLNG</th>
<th>FIN.</th>
<th>PART.</th>
<th>RFG &amp; INSUL.</th>
<th>TOT</th>
<th>LL</th>
<th>TOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>TYP.</td>
<td>OFFICE</td>
<td>41</td>
<td>5</td>
<td>4</td>
<td>20</td>
<td></td>
<td></td>
<td>70</td>
<td>50</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>RESTR.</td>
<td>X</td>
<td>41</td>
<td>5</td>
<td>23</td>
<td>12</td>
<td></td>
<td>81</td>
<td>50</td>
<td>131</td>
</tr>
<tr>
<td></td>
<td>ELEC.</td>
<td>X</td>
<td>41</td>
<td>5</td>
<td>32</td>
<td></td>
<td></td>
<td>78</td>
<td>75</td>
<td>153</td>
</tr>
<tr>
<td></td>
<td>STAIRS</td>
<td>10</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td>100</td>
<td>115</td>
</tr>
</tbody>
</table>

5.4. Snow Loads with Drift (S)

Snow loads throughout this design were done in accordance with NYCBC and ASCE 7-16, as directed by NYCBC, in order to determine the load and geometry created by snowfall and its drift throughout the design of Zenith Tower. Table 4 below shows the original parameters set by these aforementioned codes in order to further calculate loads and drifts. These values are based on location, elevation of the building, and other preexisting conditions. Table 5 below tabulates the calculated values for the snow load such as the flat snow load, the height of drift and other calculated values based on the parameters found in Table 4. Sample calculations for the snow load and drifts can be found in Appendix B. Figure 4 below shows a diagram of the snow loads and drifts on the roof level of Zenith Tower.

Table 2: Snow Load Initial Parameters

<table>
<thead>
<tr>
<th>Var.</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category</td>
<td>III</td>
<td>ASCE 7-16 Table 1.5-1</td>
</tr>
<tr>
<td>Exposure Factor</td>
<td>$C_e$</td>
<td>ASCE 7-16 Table 7.3-1</td>
</tr>
<tr>
<td>Thermal Factor</td>
<td>$C_t$</td>
<td>ASCE 7-16 Table 7.3-2</td>
</tr>
<tr>
<td>Roof Slope Factor</td>
<td>$C_s$</td>
<td>ASCE 7-16 Table 7.4-2</td>
</tr>
<tr>
<td>Importance Factor</td>
<td>$I_s$</td>
<td>ASCE 7-16 Table 1.5-2</td>
</tr>
<tr>
<td>Ground Snow Load</td>
<td>$p_g$</td>
<td>NYCBD Sec. 1608.2</td>
</tr>
</tbody>
</table>
Table 3: Slow Load and Drift Values

<table>
<thead>
<tr>
<th>Var.</th>
<th>Unit</th>
<th>Leeward</th>
<th>Windward</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Roof Load</td>
<td>$p_f$</td>
<td>psf</td>
<td>23.1</td>
</tr>
<tr>
<td>Balanced Load Height</td>
<td>$h_p$</td>
<td>ft</td>
<td>1.34</td>
</tr>
<tr>
<td>Height of Drift</td>
<td>$h_d$</td>
<td>ft</td>
<td>3.84</td>
</tr>
<tr>
<td>Height of Drift</td>
<td>$h_{duse}$</td>
<td>ft</td>
<td>3.84</td>
</tr>
<tr>
<td>Max. Intensity of Drift Surcharge</td>
<td>$p_{dlong}$</td>
<td>psf</td>
<td>79</td>
</tr>
</tbody>
</table>

Figure 4: Snow Load Diagram

5.5. **Wind Loads ($W$)**

Wind loads for Zenith Tower were calculated in accordance with ASCE 7-16 in addition to NYCBC 2022. Chapter 26 and 27 of ASCE 7-16 were implemented in order to define the general parameters needed to calculate the wind loads. These parameters were also then used within ETABS to simulate the wind force and wind load response within the building which will be discussed later in the report. Sample calculations for the wind load can be found in Appendix D below. Table 4 below tabulates the wind parameters found for this project based on location, elevation, and structural system.
In order to determine the exposure category for the building accurately, the exposure in each cardinal direction was considered in accordance with ASCE and NYCBC. Figure 5 below shows a radius of half a mile or approximately 2600 ft in all direction from the site of the building. As it can be seen, while the radius in all directions lands within Manhattan, an understanding of the topography of this area will show that the building to the West and North of the site have a mean elevation significantly higher than those to the East and South, taking these considerations into account as well as the definitions of exposure categories listed in the NYCBC, it was determined that exposure category B would be sufficient for the wind calculation. One such reason for classifying this site as B rather than C is that this building does not lie within 2,600 ft of the shoreline, as can be seen in the image, which would designate the NE and SE cardinal quarters as exposure C based on section 1609.4 of the NYCBC.

Figure 5: 0.5 mi Radius from Site for Exposure Category Determination
Table 4: Wind Load Parameters

<table>
<thead>
<tr>
<th>Var.</th>
<th>Value</th>
<th>Unit</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic Wind Speed</td>
<td>V</td>
<td>117  mph</td>
<td>NYCBC Table 1609.3</td>
</tr>
<tr>
<td>Building Risk Category</td>
<td>II</td>
<td></td>
<td>NYCBC Table 1604.5</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>B</td>
<td></td>
<td>NYCBC Fig. 1604.3 (1)</td>
</tr>
<tr>
<td>Mean Roof Height</td>
<td>h</td>
<td>264  ft</td>
<td></td>
</tr>
<tr>
<td>Gust Effect Factor</td>
<td>$G_f$</td>
<td>0.9969</td>
<td>ASCE 7-16 Sec 26.11 -&gt; 26.11.5</td>
</tr>
<tr>
<td>Wind Directionality Factor</td>
<td>$K_d$</td>
<td>0.85</td>
<td>ASCE 7-16 Table 26.6-1</td>
</tr>
<tr>
<td>Wind Topo Factor</td>
<td>$K_{zt}$</td>
<td>1</td>
<td>ASCE 7-16 Sec. 26.8.2</td>
</tr>
<tr>
<td>Grad. Height</td>
<td>$z_g$</td>
<td>900  ft</td>
<td>ASCE 7-16 Table 26.11-1</td>
</tr>
<tr>
<td>Internal Pressure Coeff</td>
<td>$G_{C_{pi}}$</td>
<td>0.18</td>
<td>ASCE 7-16 Table 2613-1</td>
</tr>
</tbody>
</table>

5.6. Seismic Loads ($E$)

To determine the seismic effects of the building, the guidelines described in NYCBC Section 1613 and ASCE 7-16 chapters 11 and 12 were used. As seen in the section above as well as described by ASCE the building is considered a Risk Category II. As well, based on the soil conditions described for this site listed as: a top 25 ft of soil consist of well graded sand (SW) with an allowable bearing capacity of 12,000 psf, and a percolation rate of 4 in/hr. The sand later is underlain by a fractured rock layer with an allowable bearing capacity of 24,000 psf, it was determined that Site Class D would be used as data concerning the average field standard penetration resistance, average field standard penetration resistance for cohesionless soil layers, and average shear wave velocity are not known and thus, per ASCE, Site Class D is chosen. As well, this structure has not been tested nor analyzed for specifically detailed seismic resistance, therefore the seismic force resisting system chosen shall be H. Table 5 below shows initial values, in accordance with the codes, that are relevant in the design and analysis of the seismic system for Zenith Tower.
Table 5: Seismic Design Parameters

<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 Cat</td>
<td>II</td>
<td></td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
<td></td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Seismic Design Cat</td>
<td>C</td>
<td></td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Seismic Force Resisting System</td>
<td>R</td>
<td>H</td>
<td>ASCE 7-16</td>
<td></td>
</tr>
<tr>
<td>Response Modification Factor</td>
<td>( \Omega_o )</td>
<td>3</td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Over Strength Factor</td>
<td>( C_d )</td>
<td>3</td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Deflection Amplification Factor</td>
<td>( I_e )</td>
<td>1</td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>( F_e )</td>
<td>1.56</td>
<td></td>
<td>ASCE 7-16</td>
</tr>
<tr>
<td>Site Coeff.</td>
<td>( F_v )</td>
<td>2.4</td>
<td></td>
<td>NYCBC Sec. 16</td>
</tr>
<tr>
<td>Site Coeff.</td>
<td>( S_s )</td>
<td>0.296</td>
<td></td>
<td>NYCBC Sec. 16</td>
</tr>
<tr>
<td>Building Height</td>
<td>( S_1 )</td>
<td>0.061</td>
<td>g</td>
<td>NYCBC Sec. 16</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>( T_1 )</td>
<td>0.05</td>
<td>g</td>
<td>NYCBC Sec. 16</td>
</tr>
<tr>
<td>Mapped Long-Period Transition</td>
<td>( T_L )</td>
<td>6</td>
<td>s</td>
<td>NYCBC Sec. 16</td>
</tr>
<tr>
<td>Approx. Fund. Period</td>
<td>( T_a )</td>
<td>1.8</td>
<td>s</td>
<td>ETABS</td>
</tr>
<tr>
<td>Redundancy Factor</td>
<td>( \rho )</td>
<td>1</td>
<td></td>
<td>NYCBC Sec. 16</td>
</tr>
</tbody>
</table>

Several values within this table were calculated using equations laid out in their respective code such as the parameter for short periods \( S_{MS} \) and at 1 second \( S_{M1} \). These values then translate to the calculation of other parameters such as the spectral response acceleration parameter at short periods \( S_{DS} \) and the spectral response acceleration parameter at 1 second periods \( S_{D1} \).

Via the risk category of the building and the location, the building could have been designed using Seismic Design Category B however for academic purposes and to be conservative, Design Category C was used.

The following table contains the calculated values for the equivalent lateral force analysis done for seismic using the parameters laid out above. These values were determined in ETABS as well.
as verified via the referenced equations in ASCE. Hand calculations for these values can be found in Appendix C.

<table>
<thead>
<tr>
<th>Description</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height</td>
<td>$S_{MS}$</td>
<td>0.462</td>
<td>g</td>
<td>ASCE 7-16 Sec 11.4</td>
</tr>
<tr>
<td>Seismic Importance Factor</td>
<td>$S_{M1}$</td>
<td>0.146</td>
<td>g</td>
<td>ASCE 7-16 Sec 11.4</td>
</tr>
<tr>
<td>Mapped Long-Period Transition</td>
<td>$S_{DS}$</td>
<td>0.308</td>
<td>g</td>
<td>ASCE 7-16 Sec 11.4</td>
</tr>
<tr>
<td>Approximate Fundamental Period</td>
<td>$S_{D1}$</td>
<td>0.0976</td>
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<td>$E_{mh}$</td>
<td>2048</td>
<td>kips</td>
<td>ASCE 7-16 Sec 12.4</td>
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The figure below depicts the lateral load distribution to each story throughout the building. It was determined that the loads in the X and Y directions for seismic would be equivalent thus only one graph is shown.
Figure 6: Equivalent Lateral Loads Per Story
6. Global Analysis

This section will cover how the structure will behave in reaction to various loads and load combinations. Checks done for the global performance includes the strength of the structure, observation of the modal parameters, inter-story drift, P-Δ effect as well as panel zone deformations.

6.1. Modal Response

Modal response of the building describes its dynamic performance. This details the resonant shapes which encapsulate how the building would response in the result of a seismic event. This is particularly crucial when the seismic frequency and the building’s natural frequency align. This modal response can also encapsulate the response to lateral loads and depict lateral deformation. Thus, various combinations of these loads are shown to give a full picture of the modal response of the structure. The modal shapes seen below are the result of modal analysis done within ETABS. From these responses, details of the moment frame compared to the braced frame can be seen and analyzed.

The first three modal shapes for the structure can be seen in the figures below. In the respective loads of X, Y and then in a double curve. The moment frame has a higher overall stiffness than the braced frame as the displacement in the second modal shape higher. The scale bar at the bottom of each figure is in feet. These figures were obtained via ETABS. The period for the first shave is 9.2s, the second is 5.5s, and the third is 3.6s. These periods respectively reference the figures below.
Figure 7: Modal Shape #1 (UX)
Figure 8: Modal Shape #2 (UY)
Figure 9: Modal Shape #3
6.2. Seismic Force Resistance System (SFRS)

As previously discussed, the seismic design category for this building was determined to be C in a conservative sense. Thus, the seismic system within ETABS was designed with design category C in mind. As well this then meant, per NYCBC and ASCE 7-16, that the steel system was not to be specifically detailed for seismic resistance. Hence, Seismic Force Resisting System (SFRS) H, via ASCE, was used. The coefficients respective to this resisting system can be seen in the table below.

Table 7: Seismic Design Coefficients

<table>
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<th>SFRS</th>
<th>R</th>
<th>Ω₀</th>
<th>C_d</th>
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<tbody>
<tr>
<td>H</td>
<td>3</td>
<td>3</td>
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</tbody>
</table>

The figures below show the story shears in the controlling direction, the story overturning moment in the controlling direction, and the maximum story drift, respectively.

Figure 10: Seismic Story Shear
Figure 11: Seismic Overturning Moment
Analysis of this resisting system was done within ETABS and verified using chapter 12 in ASCE 7-16. This verification was done as there are horizontal and vertical irregularities in the x and y direction within the structure. This verification was done for the story shear, the story overturning moment, and the story drift. These values can be seen in the table below.

<table>
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<th>Reaction</th>
<th>Units</th>
<th>X Direction</th>
<th>Y Direction</th>
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<tr>
<td>Base Shear ($V_B$)</td>
<td>Kips</td>
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<td>783</td>
</tr>
<tr>
<td>Overturning Moment ($M_B$)</td>
<td>Kip-ft</td>
<td>160,000</td>
<td>138,904</td>
</tr>
</tbody>
</table>

It was determined that there were some vertical irregularities in this structure, a verification required by ASCE 7-16. This was mainly due to the fact that there is an in-plane offset of the vertical force resisting element within the structure. Namely, the fact that at the 18th floor, the braced frame within the building offsets by 10 feet. Because of this it was required that the building was checked for these irregularities and verified.
6.3. Wind Load Response

Wind load response for the structure was considered primarily to account for the strength of the cladding and external finishes of the building in order to verify that the elements are strong enough to handle the expected demand. As well, it was crucial to determine that the transfer loads from these could be handled by the lateral force resisting system. In the location of the site, the wind force is considered more severe a risk than that of the seismic, this the lateral load from wind is taken to be more substantial. Thus, the outputs by ETABS for the wind load response were verified for overturning moment as well as story shear.

The figures below respectively show the story shears and story overturning moments in the X (blue) and Y (red) directions.

Figure 13: Wind Load Story Shear
7. **Structural System**

In order to withstand the loads as well as maintain safety throughout the building, both a gravity force resisting system and a lateral force resisting system were implemented within Zenith Tower. These systems allow for loads and forces on the building to transfer them through the foundation and to the ground while maintain stability.

7.1. **Gravity Force Resisting System**

The gravity force resisting system throughout this building is primarily made up of structural components such as beams, columns, composite metal decks, and the foundation elements. In order for the force resisting system to work effectively it must follow a load path. In this case it is a path of the load starting on a composite deck, from here moving to beams to girders where the load transfers to columns. The load then travels down the column to the foundation where it dissipates.
7.2. **Lateral Force Resisting System**

The lateral force resisting system in this building refers to two systems which work together to dissipate lateral forces working on the building such as wind or seismic. In the case of Zenith Tower, these systems are a set of parallel braced K frames and perpendicular to this, a system of moment frames. In order to encounter the most stability with the least disruption from setbacks or terraces, the braced frame was placed between grid lines 2 and 3, and 5 and 6, at A and A.1, as seen in the drawing set.

---

8. **Structural Components**

Throughout the design of Zenith Tower several structural components are implemented in order to contribute to the strength and stability of the structure. Namely these elements are the composite floor system, composite beams, gravity columns, and elements of the foundation. All of these shall be further explained for their respective selections in the subsections to follow. The selection of office space for the tower allows for flexibility in terms of fit out for the client but it also allows for more uniformity in terms of the layout of beams and columns creating a more efficient and streamlined construction process. As the rises in elevations the perimeter of floors become smaller due to terraces, however by initially creating slightly smaller bays along the east and west side of building on lower floors, uniform bays are able to be maintained throughout construction. Apart from the 18th floor and above where the building is set back 10’ on each side, the majority of bays within the building are 30’ x 30’ or 30’ x 20’. In 30’ x 30’ bays, two internal beams were placed, evenly spaced. In 30’ x 20’ bays, one bisecting beam is added between girders. *Figure 4*
below showcases both typical bays and beam arrangements. The only variation in this typical beam spacing occurs within the core where beams were placed based on stair and elevator placement. Additionally, on floors 18 and above there are 20’ x 20’ bays, of which also contain one bisecting beam, as seen in Figure 5. Column placement throughout the building is constant from the sub-cellar to the 17th floor. At the 18th floor, due to the setback, additional columns are set in 10’ on each side, parallel to the previous perimeter columns. On the mezzanine, roof, and bulkhead floors, hangers or posts are utilized in order to support additional structures such as the bulkhead structures or stairs in the mezzanine shall also be designed and analyzed as structural columns.

Figure 15: 10th Floor Framing Plan
Composite Floor System

The floor system to be used on all floors on or above grade throughout the building shall be a composite floor system. As mentioned previously the selected composite floor system is the VULCRAFT 2VLI deck was chosen with 3 ¼” of light weight concrete (LWC) with a 2” deck and 18” deck gauge. This totals a 5 ¼” thick floor deck. This ensures a 2-hour fire rating as required by NYCBC and the strength and spacing in terms of gauge for the spans and loads found in this building. As seen in the VULCRAFT catalogue, an 18-gauge deck with a 10’ span, as is the spacing between internal beams throughout Zenith Tower, the superimposed allowable load is 339 psf. As seen in the previous loading section this exceeds the requirements Zenith Tower shall have in terms of load due to service loads, construction loads, SDL, DL, and LL. This gauge shall also allow for a uniform spacing and number of shear studs needed to create the composite system between the concrete, deck, and structural steel beams beneath it.
As previously explained in the live load section of this report, a similar analysis was done for the deck selection in order to determine that uniform load placements shall be conservative and govern over the use of concentrated load regardless of their placement on the deck using an area of 2.5 ft by 2.5 ft. This analysis and check are explained in section 1607 of the NYCBC. In order to create the maximum load effect on the structural members, the concentrated load was placed between intermediary beams between girders on a typical 30’x30’ bay. Even in this case, using the mathematical example shown for the live load example, the uniform load continued to govern. Therefore, uniform loads were used throughout this design of the deck and its verification.

8.1. Composite Beams

Composite beams for Zenith Tower were designed via the composite design module found in ETABS. The sections were designed using predetermined specifications from the NYCBC and ASCE in terms of seismic, wind, stiffness, moment, and torsion limits, based on the members location in the building as well as it’s respective loading. As previously mentioned, it was determined through analysis that using values from uniform loading of the members would control over the use of a concentrated load, as laid out in both aforementioned codes. The design output from ETABS was iterated several times in order to ensure that outputs from the analysis module and composite design module intersected at a member that was designed to the upmost efficiency while remaining in a reasonable depth for the use for the building. As well, the design module output the section shape, size, camber, shear studs required, as well as its depth. All these values could be limited in preferences previous to analysis and design. In this situation the camber limit was set to 1 1/4”.

In order to correctly analyze and design these members within ETABS, consideration was taken to assign moment releases on both ends of beams and with torsional releases on one end. This was done in order to ensure that beams were designed correctly based on their position in the building as well as ensuring that the load path for gravity members would translate correctly. All composite members were designed for pre-composite and
composite actions as seen from the load combinations within the ETABS model. The
beam framing plans can be seen within the drawing set and the annotated members
represented finalized analyzed and designed members output from ETABS and confirmed
by calculation. Typical sections seen within the building include W16X26, W18X35,
various W12X26, and W10X12. Other sized sections can be seen throughout the design
in scenarios where additional loading or needed additional support for a transfer beam
was required.

8.2. Gravity Column

Gravity columns were implemented throughout the design specifically to resist gravity
loads such as self-weight, SDL, LL, and snow loads. The analysis and design for these
members was done using ETABS in the irritative way mentioned in the past section.
Column dimensions, their reactions, and locations can be found in the column schedule
seen in the drawing set. For concrete columns used below grade, hand calculations were
used to determine size and reinforcement. Details for below grade footings and concrete
columns can be found under details in the drawing set.

8.3. Foundation

The foundation system of Zenith Tower is made up of spread footings for interior
columns, a foundation wall around the perimeter of the building, as well as the wall
footings and buttresses. Details of these can be found with similar details in the drawing
set.

8.3.1. Footing

The internal footings for Zenith Tower were designed using manual calculations
which can be found in Appendix L. The general sized for these internal footings
are 18” X 18”, details of which can be found in the drawing set. The
reinforcement for these is typically 18 #9 bars. For the design of all footings
reference the attached excel sheet.

8.3.2. Buttresses
Buttresses within this design were designed as concrete columns. The slenderness of these buttresses was disregarded as they pocket into the foundation wall. Specifically, these buttresses were designed for axial load only, specifically the lateral load coming from the steel columns sitting on them. The general dimensions for these buttresses were 14” x 14” with 8 #6 bars. Reference Appendix K and the excel sheet which shows the design method for these as well as the assumptions made.

8.3.3. **Base Plates**

The base plates for this design were done via excel as seen in the attached sheet as well. For the base plate located at C-1 it was determined to be 14” x 14”. These plates were designed for all locations where steel columns translate to concrete buttresses or foundation footings/walls. They were designed to uphold the axial load translated by the column above for efficient forces on the concrete element below.

8.3.4. **Foundation Wall**

The foundation walls in this building were modeled and designed to support the load from the building as well as the surrounding soil pressure. The majority of the loads that required resistance here were due to soil pressure as the majority of the axial loads were taken by the aforementioned elements. Appendix I shows the design method used for this wall design as well as their reinforcement. The thickness of the wall was found to be…

8.3.5. **Connections**

Please see typical drawings for the typical connections used throughout this design. As both moment frames and braced frames were used throughout Zenith Tower, connections for both of these scenarios have been included. As well shear and floor connections have been detailed. These typical details have been analyzed and designed as per the guidelines found in ASCE 7-16 and NYCBC.

9. **Conclusion**
Overall, the design and analysis of this building has found it to be within the range of acceptability based on the requirements laid out in the NYCBC. Overall Elite Engineering is confident in the design put forward by this proposal and feel the design, cost, and schedules take into account outside risks as well as being acceptable compared to current market values. Not only will this space integrate well into the surrounding areas, but the sustainability, green space, modernity, services, utilities, and overall look of the building are sure to make this an attractive space for several potential companies.

10. Appendix A: Dead Load Hand Calculation
Facade loads:
Curtain wall - 40 psf

Floor to floor height = 12 ft

\[ w = 40 \cdot 12 = 480 \text{ plf} \] facade load for exterior wall

Parapet load - 20 psf
Parapet height = 4.5 ft
\[ w = 20 \cdot 4.5 = 90 \text{ plf} \] parapet load

Masonry block wall:
grouted @ 16" = 66 psf
\[ w = 66 \cdot 12 = 792 \text{ plf} \]
grouted @ 8" at bulk head = 84 psf
\[ w = 84 \cdot 12 = 1008 \text{ plf} \]

Stair case loads:
LL = 100 psf
DL = 15 psf

\[ L = 14.5' \] span
\[ H = 12' \] height

\[ w = 4.25' \] stair width

Total distance = 16.25'
\[ R = v/L/2 = 100 \cdot (16.25)/2 = 812.5 \text{ plf} \]
\[ 15 \cdot (16.25)/2 = 217.875 \text{ plf} \]

Stair 1 on first floor:
additional 4.5' span
\[ R = v/L/2 = 100 \cdot (20.75)/2 = 1037.5 \text{ plf} \]
\[ 15 \cdot (20.75)/2 = 183.625 \text{ plf} \]

Stair 2 on first floor:
additional 5.5' span
\[ R = v/L/2 = 100 \cdot (21.75)/2 = 1087.5 \text{ plf} \]
\[ 15 \cdot (21.75)/2 = 163.125 \text{ plf} \]

Figure 17: Loads Calcs 1 of 2
Figure 18: Load Cales 2 of 2
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NOTES
1. LIVE LOADS HAVE BEEN REDUCED FOR THE INDICATED OCCUPANCIES.
2. FACADE, GLASS & MILLION = APP.

ON GRADE

40
Zenith Tower 20 W 34th St, New York NY 10001

11. Appendix B: Snow Loads and Snow Drift Calculations

Figure 19: Snow Loads 1 of 4
Drift height = \( \frac{3}{4} \) \( h_c + \frac{3}{4} [0.43^2 \sqrt{20 \cdot \sqrt{35} - 1.5}] (1.1)^{0.5} = 1.05' \) 

\( h_c \) Design Drift height = 1.05'

\[ 1.05(17.25) + 21.175 = 39.34 \text{ psf} \]

\[ w = 4 (h_c) = 4(1.05) = 4.21' < \theta_{hc} \quad \text{(w = 4.21')} \]

\[ h_c = \frac{3}{4} [0.43^2 \sqrt{20 \cdot \sqrt{35} - 1.5}] (1.1)^{0.5} = 2.21' < \theta_{hc} \]

\[ 2.21 (17.25) + 21.175 = 59.3 \text{ psf} \]

\[ w = 4 (h_c) = 4(2.21) = 8.84' \quad \text{(w = 8.84')} \]

Straight drift governs in south direction

High to low drift governs in north direction

\[ h_c = 10.772' \]

Drift = \( \frac{3}{4} [0.43^2 \sqrt{20 \cdot \sqrt{35} - 1.5}] (1.1)^{0.5} = 1.85' < \theta_{hc} \]

\[ 1.85 (17.25) + 21.175 = 53.1 \text{ psf} \]

\[ w = 4 (h_c) = 4(1.85) = 7.4' < \theta_{hc} = 8.5' \quad \text{(w = 7.4')} \]

\[ h_c = \frac{3}{4} [0.43^2 \sqrt{20 \cdot \sqrt{35} - 1.5}] (1.1)^{0.5} = 1.84' < \theta_{hc} \]

\[ 1.84 (17.25) + 21.175 = 52.83 \text{ psf} \]

\[ w = 4 (h_c) = 4(1.84) = 7.34' < \theta_{hc} \quad \text{(w = 7.34')} \]
Bulkhead 2 8 3

High to low in North South Directions

\[ h_c = 10.772', h_d = \left[ 0.43 \times \sqrt{20} \times \sqrt{35} - 1.5 \right](1.1)^{1/4} = 2.15'h_c \]

\[ 2.15'(17.25) + 21.125 = 58.2\text{ psf} \]

\[ w = 4'h_d = 4'(2.15) = 8.59'c h_c \quad w = 8.59' \]

\[ l = 9' \rightarrow u = 0 \quad l_k = 20' \]

\[ h_d = \left[ 0.43 \times \sqrt{20} \times \sqrt{35} - 1.5 \right](1.1)^{1/4} = 1.4'h_c \]

\[ 1.4'(7.25) + 21.125 = 55.39\text{ psf} \]

\[ h_d = \frac{4.75}{l_u}l_u = \frac{4.75}{9} = \frac{41.75}{9} = 4.62'h_c \quad w = 5.62' \]

Vertical Drift governs in East-West Direction

\[ h_c = 10.772' \]

\[ h_d = \frac{3.4}{l_u} \left[ 0.43 \times \sqrt{35} \times \sqrt{35} - 1.5 \right](1.1)^{1/4} = 1.32'h_c \]

\[ 1.32'(17.25) + 21.125 = 44.9184\text{ psf} \]

\[ w = 4'h_c = 4'(1.32) = 5.5'h_c \quad w = 5.5' \]

\[ h_d = \frac{3.4}{l_u} \left[ 0.43 \times \sqrt{65} \times \sqrt{35} - 1.5 \right](1.1)^{1/4} = 2.04'h_c \]

\[ 2.04'(17.25) + 21.125 = 56.56\text{ psf} \]

\[ w = 4'(2.04) = 8.16' \quad w = 8.16' \]

Figure 21: Snow Loads 3 of 4


Figure 22: Snow Loads 4 of 4
12. Appendix C: Seismic Load Calculations

Figure 23: Seismic Cals 1 of 1
Parameters via Codes:

Basic Wind Speed ($V$) = 17 mph  (NYCDB 160.9.3)
Building Risk Cat. = II  (NYCDB 160.9.5)
Exposure Cat. = B  (NYCDB 160.4.3)

based on 2600 ft radius from site
as well as Quarterly analysis based on Cardinal directions. Overall conservative value
was B.

Mean Roof Height ($h$) = 264 ft

Wind Directionality Factor ($K_d$) = 0.93  (ASCE 7-16, 26.6.1)
Wind Topographic Factor ($K_t$) = 1  (ASCE 7-16, 26.8-2)
Grade entered Height ($h_g$) = 900 ft  (ASCE 7-16, T 26.11-1)
Internal pressure coeff (C_{ip}) = 0.18  (ASCE 7-16, T 26.13-1)
Guest Effect Factor ($C_g$) = 0.99  (ASCE 7-16, 26.11)

Calculated next page.

not taken as $0.85 < 0.9 < 1.0$

*: Building Flexible

Figure 24: Wind Calc 1 of 4
\[ n = \frac{35}{2}, \quad h = 264, \quad \frac{35}{264} = 0.284 \text{ rad} \]  
\[ (\text{eqn. 26.11-4}) \]

Building: Flexible

\[ T_2 = C \left( \frac{32}{2} \right)^{1/2} = 0.2 \left( \frac{32}{2} \right)^{1/2} = 0.154 \]  
\[ (\text{eqn 26.11-7}) \]

\[ C = 0.2, \quad \frac{32}{2} = 0.6(264) = 158.4 \]  
\[ (T. 26.11-1) \]

\[ g_0 = g_v = 3.4 \]  
\[ (\text{sec. 26.11.5}) \]

\[ L_2 = \frac{1}{1 + 0.63} \left( \frac{158.4}{33} \right)^{0.63} = 140.63 \left( \frac{65.4}{3} \right)^{0.63} = 0.825 \]  
\[ (\text{eqn 26.11-1}) \]

\[ B = 160 \]  
\[ (\text{sec. 26.3}) \]

\[ g_0 = \sqrt{2 \ln(3600)} + \sqrt{2 \ln(3600)} = 3.88 \]  
\[ (\text{eqn. 26.11-11}) \]

\[ R = \frac{1}{1 + 0.53 + 0.4} = 0.57 \]  
\[ (\text{eqn 26.11-12}) \]

\[ R_n = \frac{3.03 M}{1 + 0.53 + 0.11} = 0.11 \]  
\[ (\text{eqn 26.11-13}) \]

\[ M = \frac{h^2}{2} = 0.224 \left( \frac{65.4}{3} \right) \]  
\[ (\text{eqn 26.11-14}) \]

\[ V = 0.65 \left( \frac{35}{33} \right)^{0.5} \]  
\[ (T. 26.11-15) \]

\[ \bar{h} = 0.65, \quad \bar{r} = 0.5 \]  
\[ (T. 26.11-1) \]
Figure 26: Wind Calc 3 of 4
Floor Sample

Floor 5

\[ a_2 = 2.01 \left( \frac{7}{3} \right)^{2/3} \left( 15 \leq z \leq 20 \right) \]

\[ a_2 = 2.01 \left( \frac{30}{600} \right)^{2/3} = 1.11 \]

\[ k_2(1) = 2.01 \left( \frac{264}{900} \right)^{2/3} = 1.42 \]

\[ q_2 = 0.00256 \cdot k_2 \cdot k_2(1) \cdot \frac{1}{V^2} \]

\[ q_2 = 0.00256 \cdot (1.11) \cdot (0.85) \cdot (119)^2 = 33.9 \]

\[ q_2(1) = 0.00256 \cdot (1.42) \cdot (0.85) \cdot (119)^2 = 42.17 \]

\[ p = q \cdot C_p - q \cdot (C_p - 1) \]

\[ p = 33.9 \cdot (0.99) \cdot (0.85) - 42.17 \cdot (0.15) = 19.41 \]

Figure 27: Wind Calc 4 of 4
14. Appendix E: Deck Hand Calculations

Figure 28: Deck Hand Calcs 1 of 1
15. Appendix F: Composite Beam Hand Calculations

Figure 29: Composite Beam Hand Calc 1 of 2
Title: Composite Beam  
By: Charlotte Anderson  
Sheet 2 of 2

Self weight = 26 lb/ft for W16 x 26

\[ \begin{align*}
W_c &= 12 \times (50) + 24 \times 0.525 \text{ k}\ \\
W_c &= 12 \times 6 + 1.6 \times 6 = 12 (0.525) + 1.6 (0.3) = 11.1 \text{ k}
\end{align*} \]

\[ M_c = \frac{11 (30)}{2} = 165.75 \text{ k-ft} \]

Reconstruction deflection = \( \frac{1}{360} \)

\[ \Delta_{cc} = \frac{5}{360} \times \frac{5}{360} = \frac{5 (526) \times (30 \times 12)}{360} (360) = 0.11'' \]

\[ \therefore \text{ use 1.25'' Camber (conservative)} \]

Composite Torsion:

\[ \tau = \frac{2}{\pi} \left( \frac{r}{R} \right) \left( \frac{2}{3} \right) \]

\[ \begin{align*}
I &= 2 \left( \frac{r}{R} \right) \\
 &= 2 \left( \frac{30 (0.25)}{12000} \right) = 43.11 \text{ in}^4
\end{align*} \]

\[ u_{cc} = \frac{10 (50 + 24 + 3)}{210} = 0.825'' \text{ k}\]

\[ u_{cc} = \frac{10 (43.11)}{210} = 0.436'' \text{ k}\]

\[ u_{cc} = 1.2 (0.825) + 1.6 (2.45) = 1.68'' \text{ k}\]

\[ M_c = \frac{1.67 (30)}{2} = 129.75 \text{ k} \]

Excess width (b) = \( \frac{3}{2} \left( \frac{r}{R} \right) = 7.5 \text{ in} \quad \text{govern} \quad b = 7.5\] in

\[ d_h = 2.93 \text{ in} \quad \therefore d_h = 187 \text{ k} \quad \text{ok} \]

# Studs: \( n = \frac{w}{w} = \frac{192}{12.1} = 16.2 \rightarrow 12 \text{ studs} \]

Waterproofing: 8 x 4 x 6 = 48'' > 12'' \quad \checkmark

Shear strength \( V_s = \frac{w}{2} = \frac{1.67 (30)}{2} = 25.26 \text{ k}\)

\[ p = \frac{10.6 (25) > 25.26 \text{ k}}{} \quad \therefore \text{ use W16 x 26 (C=1.25)} \]

Figure 30: Composite Beam Hand Calc 2 of 2
16. **Appendix G: Girder Hand Calculations**

Zenith Tower
20 W 34th St, New York NY 10001

Figure 31: Girder Hand Calc 1 of 2
Girder Hand Cales

\[
\frac{h}{w} = 56'' < 3.96 \sqrt[3]{\frac{71800}{50}} = 90.5'' \\
\]

\[
\begin{align*}
&Q_1 = 445 k \text{ lb} \\
&Q_2 = 4139 k \text{ lb} \\
&K_{ISC} = 380 k \text{ lb} \\
&V_{n} = 78.3 k \text{ lb} > V_n = 586.04 k \text{ lb} \\
&\text{Shear Strength: } V_n = \frac{W_1}{2} + \frac{W_2}{2} = 400.6 k \text{ lb} \\
&\phi W = 234 k \text{ lb} > 35.5 k \text{ lb} \\
&\text{use } A24.55
\end{align*}
\]
Transfer Beam Hand Calc  Charlotte Adelson  Sheet 1 of 2

Verify Design by ETABS for W30 x 124

Material: A992 Steel  f_y = 50 ksi; f_t = 65 ksi
Beam length: L = 30’
Section: W30 x 124

DL: pro: 5/6 = 50 psf
Composite: traffic = 30 psf
\( f_{\text{coll}} = 50 \text{ psf} \) (approx.)

Point load = 67.4 kips (from column above)

LL: pro: Construction = 30 psf
Composite: traffic = 50 psf
\( f_{\text{coll}} = 100 \text{ psf} \)
Point load = 52.5 kips

Verifying W30 x 124
\( f_e = 0.73 > 0.3 \checkmark \)

\[ \frac{L}{r} = 23.2’’ \quad L/r = 23.2’’ \quad L/r = 11 \]

\( f_b = 1500 \text{ kips} \)
\( f_b = 1500 \text{ kips} \)
\( f_b = 3.9 \text{ kips} \)

\( P_{\text{Mn}} = 1.46 (f_b M_p) = 1.46 (1500 \times 1500) = 1828.52 > 1530 \text{ kips} \)

\( P_{\text{Mn}} = 1530 \text{ kips} \)

\[ A = 3.8 \text{ in.}^2 \quad I_x = 5300 \text{ in.}^4 \quad E = 400000 \text{ psi} \quad d = 322’’ \quad f_w = 48 \text{ psi} \]

\[ b = \left\{ \begin{array}{ll}
\frac{3b}{2} & = 7.5 \\
\frac{b}{2} & = 30
\end{array} \right. \]

\[ b = 30 \text{ in.} \quad 30 \text{ in.} \]

\[ f = 46.2’’ < 90.6’’ \quad \text{All U-pass} \]
18. **Appendix I: Foundation Wall Design**

   Please see attached excel for design calculations.

19. **Appendix J: Base Plate Design**

   Please see attached excel for design calculations.

20. **Appendix K: Buttress Design**

   Please see attached excel for design calculations.

21. **Appendix L: Isolated Footing Design**

   Please see attached excel for design calculations.
Figure 35: Meshing Reinforcement for Concrete on Metal Deck 1 of 1