# Belmont Executive Center; Building A

## Ashburn, VA

**Technical Report 3**
**Nicholas L. Ziegler: Structural**
**Advisor: Professor M. Kevin Parfitt**
**December 1, 2009**

# Table of Contents

## Executive Summary

The purpose of Technical Report 3 is to analyze the existing lateral system in the Belmont Executive Center; Building A. The load path of lateral loads was first determined through the concept of relative stiffnesses. It was also determined how the load due to torsion was distributed. A 3-D ETABS model of the building was created, and two hand calculations verified that it was behaved correctly. Wind deflections were analyzed for the four load cases set forth by ASCE 7-05. Case 2 created the largest deflection in brace 1, which was less than the maximum allowable deflection of L/400. A manual hand calculation, considering direct load and load due to torsion, for case 2 was then completed and the computed loads were then applied to a 2-D SAP model. The resulting deflection was larger than that of the ETABS model, but was very close. Seismic deflections were then calculated using the model and were less than the max permissible deflection of .025*h. Brace 1 was then analyzed for overturning and member strength checks using the load combination 1.2D + 1.6W + L + 0.5(Lr or S or R), and 0.9D + 1.0E + L + 0.2S. It was determined that overturning was not an issue, and lowest column of the braced frame is capable of supporting the factored loads.

## Introduction

The Belmont Executive Center; Building A is located in the Belmont Executive Center, which will include office, retail, restaurant, daycare, and hotel spaces. Residents of the Dulles North area will be offered daily shopping, specialty shopping, and dining choices.

Building A is a 125,000 SF, 5-story office building designed to accommodate multiple tenants. The façade of the building is constructed primarily of brick on light gage metal studs. Vertical brick columns are spaced around the perimeter façade, some of which enclose structural columns, others which do not support any load. A large curtain wall system distinguishes the entrance of the building, and the corners of the building also have a curtain wall system. The structural system of the building is constructed of steel framing with light weight concrete on composite deck as the floor system. Lateral bracing is provided by four concentrically braced frames.

Each floor provides unobstructed open space on both sides of the core, and a floor to ceiling height of 9'. Lateral forces are resisted by three braced frames in the north-south direction of the building, and one braced frame in the east-west direction.

# Structural System

## Foundation System

The foundation system is made up of spread footings located at the base of the steel columns, and range from 19'-6" square to 10'-6" square, depending on the location. Larger footings are located in the center right part of the building, to support a mechanical room and the restrooms. Smaller foundations are located at the exterior columns. All larger foundations are shown in yellow in the Figure 1 below. The perimeter footings are connected by grade beams that support the masonry facade. A stepped grade beam is located just to the left of the entrance to allow a connection to the sanitary line. There is also a stepped grade beam on the right side of the building for the domestic water line and fire service line connection. The ground floor is a 5" thick concrete slab on grade reinforced with #3 rebar @ 15"o.c. running both directions. A 6" slab on grade is located to the right side of the building to support a 30 yard trash compactor, and is highlighted in purple in Figure 1. It is reinforced with #3 rebar @ 12"o.c. each way. The slabs are supported by 4" granular material, on top of compacted soil.

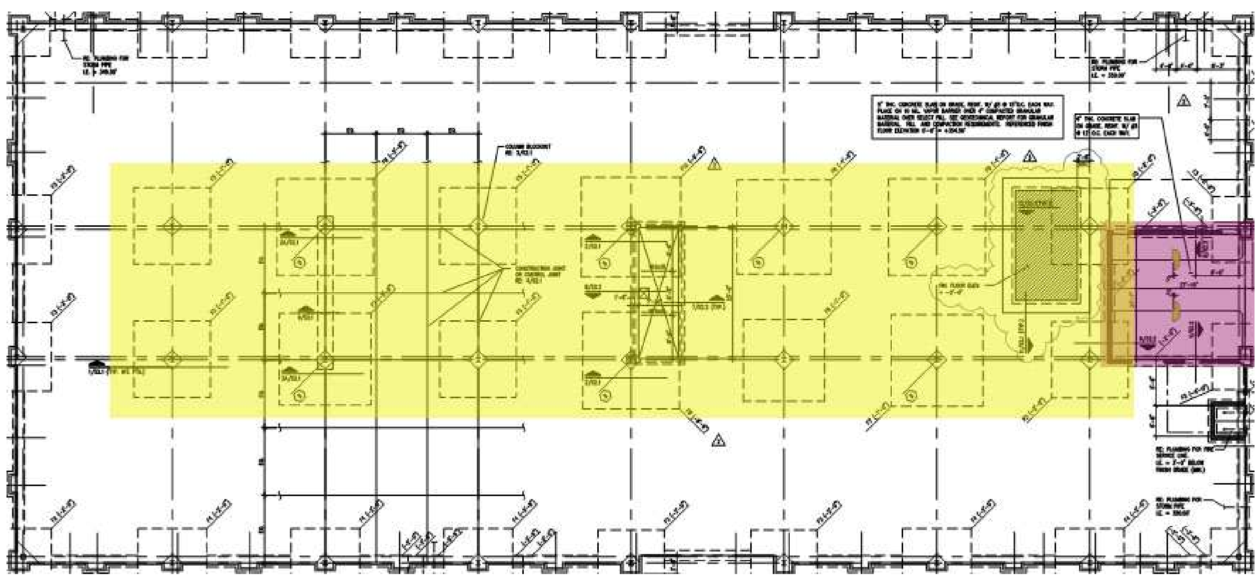


**Figure 1: Foundation Layout**

## Column System

The floor and roof system are supported by three column lines in the north-south direction and nine rows of columns in the east-west direction. Because the exterior column spacing is dictated by the architecture of the building, the columns on the left and right side of the building are offset from those in the interior. At the corners of the building they are offset by 1'-3" and the interior columns are offset by 7 ¼". This offset creates a slight skew in the beams spanning from the exterior to the interior. Figure 3 shows the column offset. Most of the columns are W shape steel beams, and a few are HSS columns. Hollow structural steel columns are located at the front left and right corners of the building. They are also used in the left rear and right rear corners, on floors three to five, and to provide intermediate bracing below the exterior terrace on the fifth floor. The typical bay sizes for each floor is 38'x 30' and 26'x30'. Figure 3 shows the typical column layout.

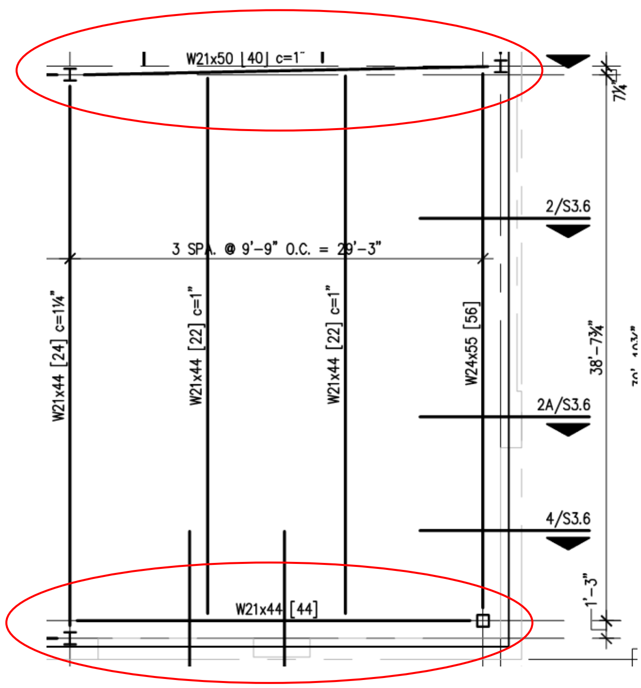



**Figure 2: Column Offset**

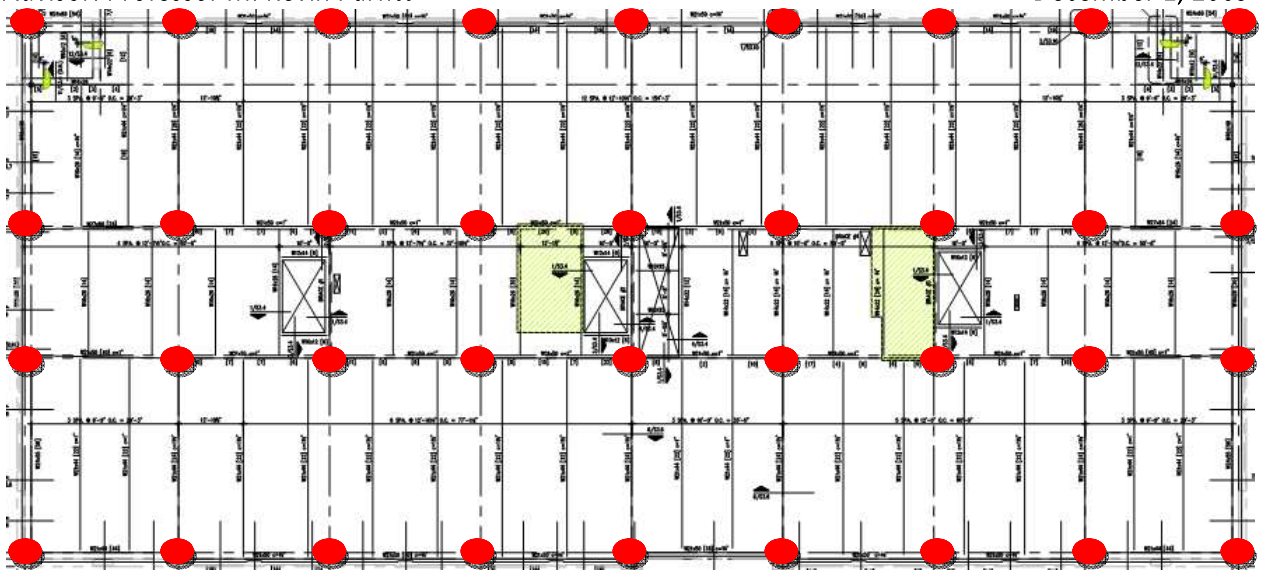


**Figure 3: Column Layout**

## Floor System

Floors 2-4 are constructed of 3-1/4" light weight concrete, on 3" composite metal deck. The deck is reinforced by 6 x6 - W1.4 x W1.4 welded wire fabric, and supported by W-shape steel beams. There are three bays in the north-south direction, and ten in the east-west direction of the building. For reference, the outer lying bays are highlighted in red, and the middle bay is highlighted in green, see Figure 4. Typically, there are W21x44 beams spaced 12'-10 ¼" to 9'-9", on floors 2 through 5, in the two outside bays.  In the middle bay the beams are typically W16x26. Between the elevators and stairwell three, the steel beams are W14x22. Composite action is provided shear studs, and most beams also have upward camber ranging from ¾" to 1" to compensate for service and live load deflections. W 21x50 girders support the load reactions from the beams.  On the second floor there is no framing at the main entrance, because this area is open to the ground floor. Floors 3-5 are framed similarly. On the fifth floor the exterior terrace floor is supported by W10x12 steel beams.

The mechanical equipment in the penthouse is supported by a typical concrete floor, constructed of lightweight concrete on composite metal deck. This is the only concrete slab on the roof level. W16x26 beams span across the bay to support the floor.

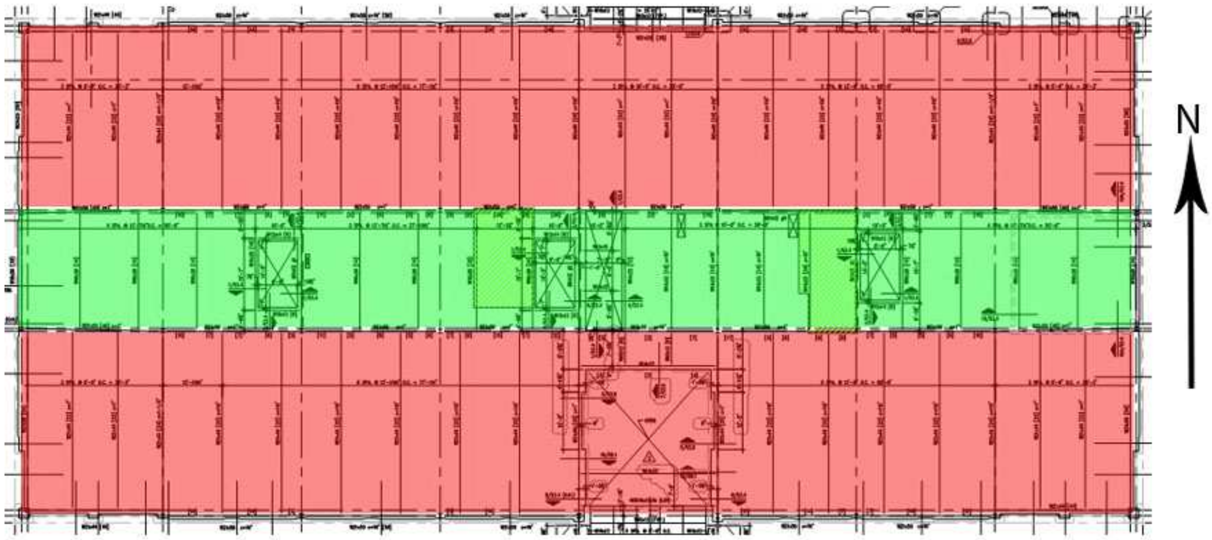



**Figure 4: Typical Beam Size and Spacing**

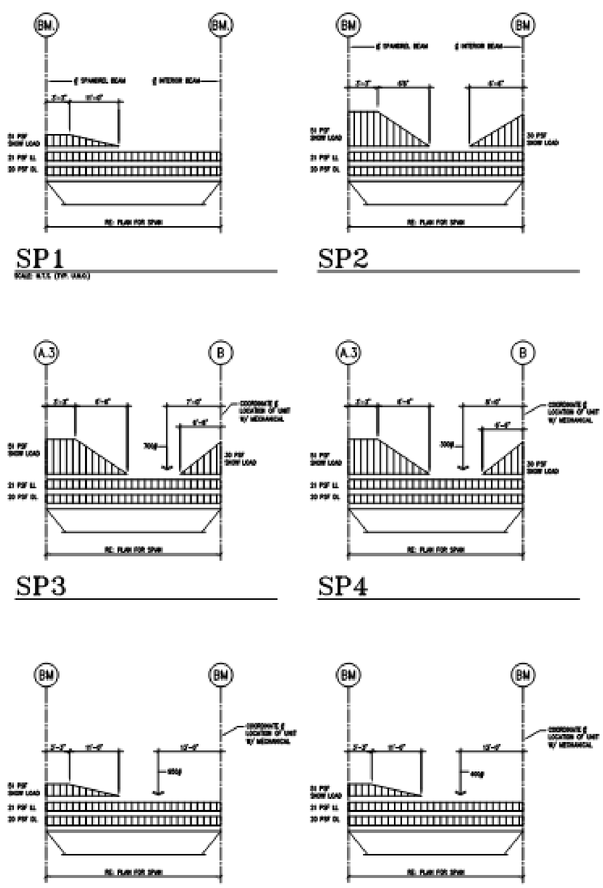



**Figure 5: Special Loading Conditions**

## Roof System

The roof system is supported by K-series joist, spanning across the three bays in the north-south direction. All the joists in the outside two bays are spaced at 6'-0" on center. Joists in the front and rear bays were designed for specifically by the joist manufacturer for snow drifting, because this can be a critical load failure for open web joists. All joists that were specially designed are denoted by SP, and there are 6 different loading conditions. Each loading condition is shown in Figure 5. Three rows of bracing are provided in the rear bay, to prevent lateral torsional buckling. Regular K series joists ranging from 22K5 to 18K3 support the roof in the middle bay. The penthouse roof is supported by 20K3 spaced at 6'-0", with 3 rows of bridging.

The standing seam metal roof screen that shields the penthouse from view is supported by a combination of K Series joists and W shape beams. At roughly every 30' W shaped steel beams are angled at 45 degrees, and are supported by steel posts. Between the beams, four K series joists run parallel to the building perimeter. L 2 x 2 x1/8" angle provides bracing at 6', between the joists. Figure 6 shows the angled beams, highlighted in yellow, and the joists can be seen spanning between them. Figure 7 shows a typical cross section of the roof screen.

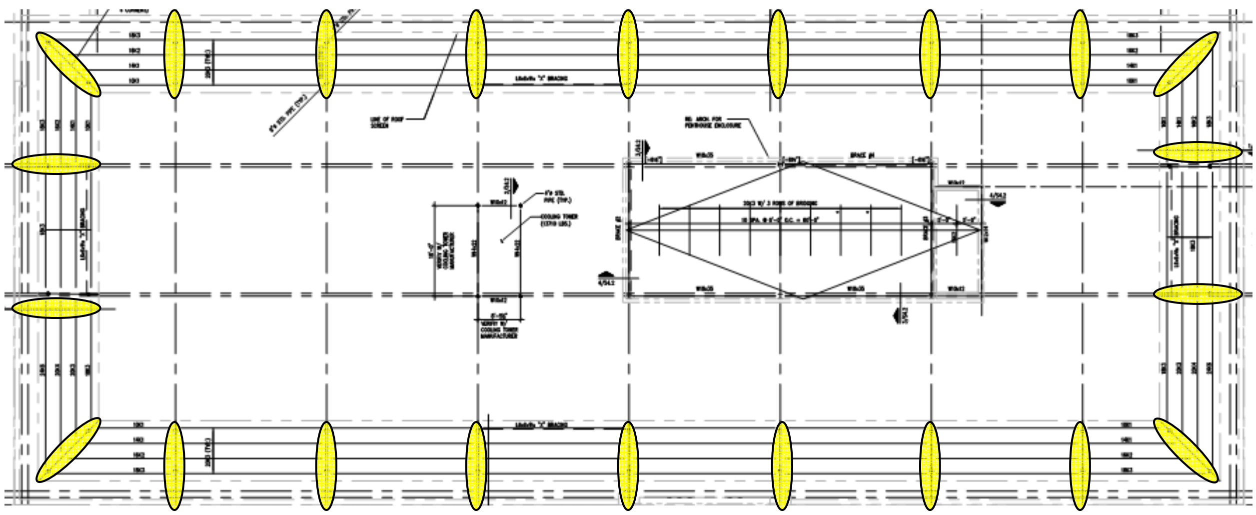


**Figure 6: Angled W Shape Beams**

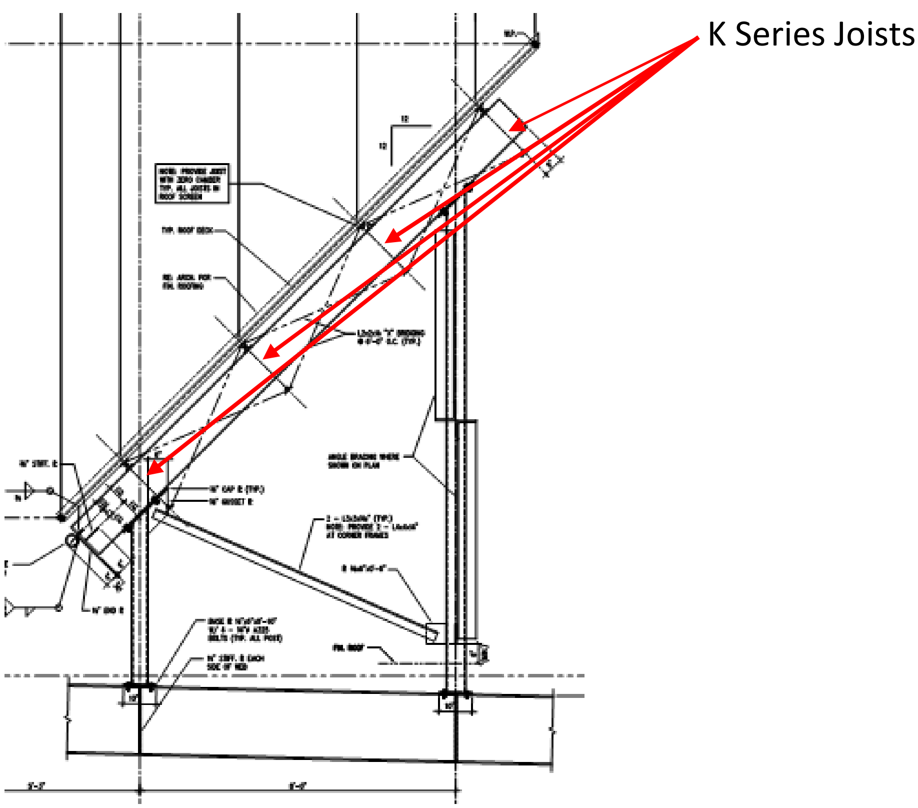



**Figure 7: Roof Screen Support**

## Lateral System

Lateral loads on the building are supported by four concentrically braced frames. Three of the frames are located in the north-south direction to support higher wind loads from the broad side of the building, and one frame is located in the east-west direction. The three frames in the north-south direction are located on the column lines, adjacent to stairwell one and two. The other is located to the left of stairwell three. In the east-west direction the frame is located between columns B6 and B7. All frames are braced with hollow structural steel ranging in size 8 x 8 x ¼ at the first floor to 4 x 4 x ¼ on the fifth floor. Figure 8 shows the elevations of each braced frame, and Figure 9 shows the location of each frame.

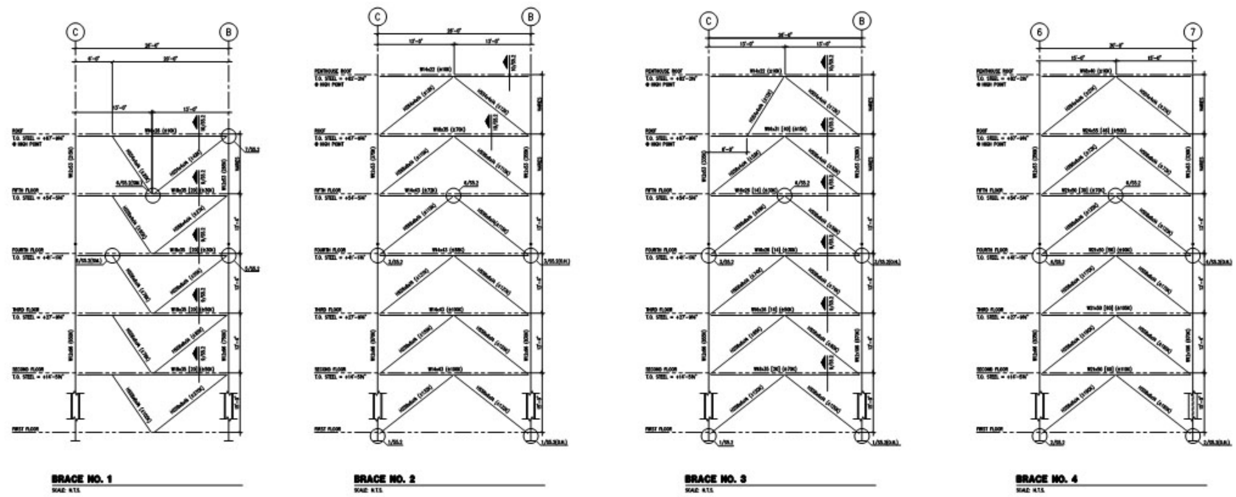



**Figure 8: Braced Frame Elevations**

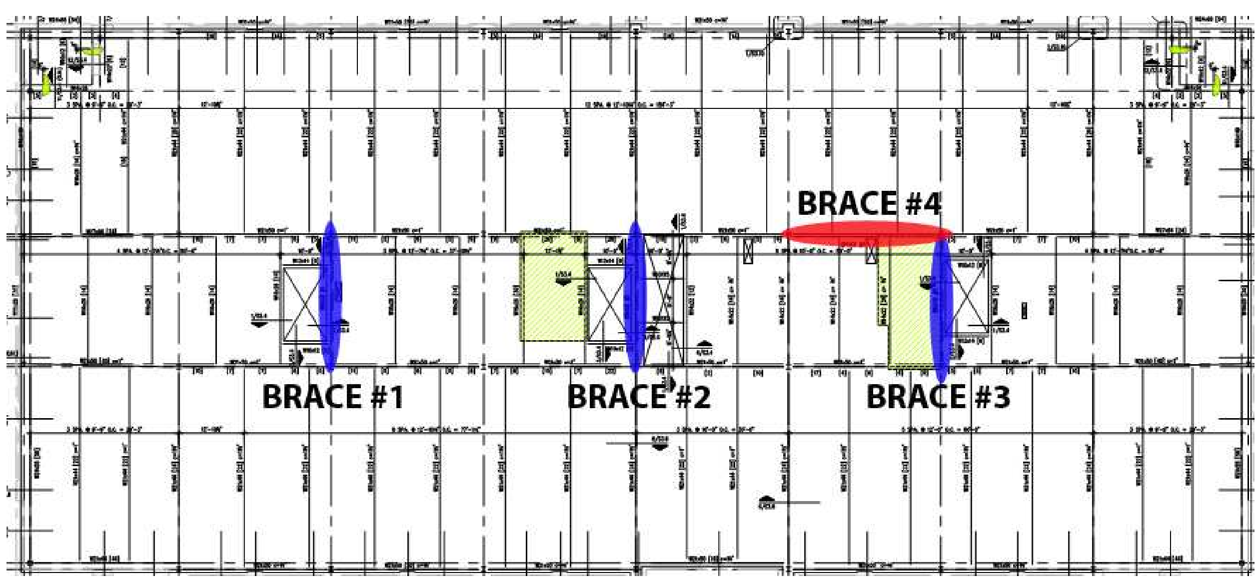



**Figure 9: Location of Braced Frames**

# Materials

Concrete – All concrete shall have natural sand fine aggregate, and Type I Portland Cement conforming to ASTM C150. Concrete in the footings, pilasters, and slabs on grade shall be prepared with normal weight course aggregates conforming to ASTM C33. The concrete in the composite slabs shall have lightweight course aggregates conforming to ASTM C330, and a maximum unit weight of 115 pounds per cubic foot.

Compressive Strength

| | |
|---|---|
| Footings | 3000 psi |
| Pilasters | 3000 psi |
| Slabs on Grade | 4000 psi |
| Composite Slabs | 3500 psi |

Reinforcing Bars – Must conform to ASTM A615, grade 60.

Welded Wire Fabric – Must conform to ASTM A185.

Roof Deck – All Type B deck shall be 22 gage cold formed steel conforming to ASTM A653 SQ Grade 33. The deck shall be 1 – ½ inches deep and have a minimum section modulus of 0.186 inches cubed per foot of width.

Composite Steel Deck - Composite steel deck shall be 18 gage minimum cold-formed steel conforming to ASTM A611, Grade D and shall have a phosphatized and painted lower surface and a phosphatized only top surface. The deck shall be 3 inches deep and shall have a minimum section modulus of 0.803 inches cubed per foot of width.

Structural Steel

W Shapes – Shall conform to ASTM A992
Other Steel Shapes, Plates, Angles and Channels – Shall conform to ASTM A36
Steel Pipe – Shall conform to ASTM A53, Grade B
Steel Tubing – Shall conform to ASTM A500, Grade B
Anchor Bolts – Shall conform to ASTM F1554, Grade 36
Bolts – Shall meet or exceed the requirements of ASTM A325, Type N, X, or F

Concrete Masonry

Concrete masonry shall have a minimum compressive strength of 1500 PSI on the net cross sectional area at 28 days

Masonry Units – Shall be grade N, Type I light weight or medium weight hollow concrete units meeting fire rating requirements and conforming to the requirements of ASTM C90

Mortar – shall conform to the requirements of ASTM C270, type M or S

Grout – shall conform to ASTM C476

# Codes

Building Code
Virginia USBC (IBC 2000)

Structural Steel
AISC Specification for Structural Steel Buildings
AISC Code of Standard Practice for Steel Buildings and Bridges
*Exception of paragraph 4.2.1 – Deletion of the following sentence: "This approval constitutes the owner's acceptance of all responsibility for the design adequacy of any connections designed by the fabricator as part of his preparation of these shop drawings."
AISC Manual of Steel Construction – Allowable Stress Design, $9^{th}$ Addition
Steel Joist Institute Standard Specifications for Open Web Steel Joists
AISI Specification for the Design of Cold-Formed Steel Structural Members

Concrete
ACI Details and Detailing of Concrete Reinforcement, ACI 315
ACI Detailing Manual, ACI SP-66
ACI Manual of Engineering and Placing Drawings for Reinforced Concrete Structures, ACI 315R
CRSI Manual of Standard Practice

Concrete Masonry
ACI Building Code Requirements for Concrete Masonry Construction, ACI 530
ACI Specifications for Masonry Structures, ACI 530.1

Design Loads
International Building Code 2000
American Society of Civil Engineers (ASCE), ASC- 7

## Load Combinations

Two load combinations were considered for this report. They are listed below.

1.2D + 1.6W + L + 0.5(Lr or S or R)
0.9D + 1.0E + L + 0.2S

The first was used to determine if the compressive member in one of the braces had adequate strength to support the load due to gravity and wind, and the second was used to determine if overturning would control foundation design.

# Gravity Loads

## Snow Load

Snow loads were calculated in accordance with ASCE 7-05 Chapter 7. As mentioned earlier, special snow drift conditions were considered for the K series joists supporting the roof. Snow drifting was considered against the parapet, and the penthouse. One calculation determined that a load of 49 – 50 pounds per square foot should be applied where drifting occurs. This matches the loading of the structural engineers, who calculated a load of 51 psf. The calculated ground snow load 21 PSF also matched the load listed in the structural notes. See Appendix A for calculations.

## Dead/Live Loads

| Live Loads | |
|---|---|
| **Area** | **Design Load** |
| Office Space | 100 |
| Permanent Corridors | 100 |
| Lobbies, Stairs, and Assembly Areas | 100 |
| Mechanical Space | 125 |
| Light Storage (Mechanical Rooms) | 125 |
| Roof | 30 |
| **Dead Loads** | |
| MEP | 5 |
| Ballasted Single Ply Roof | 11 |
| Finishes/Partitions | 20 |
| 3 1/4" Lightweight Concrete on 3" Metal Deck | 60 |

**Table 1: Design Gravity Loads**

# Lateral Loads

## Wind Loads

Wind loads on the building were calculated in accordance to ASCE 7-05, Chapter 6. Analytical method number two was used to determine wind loads in both the north-south direction and in the east-west direction. For the purpose of this report, I made a few assumptions to simplify the calculations. The assumptions made were to include the roof screen height into the total building height, and to assume the building horizontal projections to be rectangular. Wind effects on the building parapets and roof screen were not taken into consideration. A more detailed and accurate analysis will be performed in future technical reports.

All the variables and coefficients used in the calculations are located in the Appendix B. Table 2 summarizes the wind pressures in the North-South direction. As mentioned earlier the wind loads in this direction are higher than those in East-West direction because this is the broader side of the building. Table 3 summarizes the wind pressures in the East-West direction. It should be noted that internal pressures caused by openings in the building façade were not incorporated in the total wind pressure. An internal pressure of ±3PSF will be added when determining the worst possible loading for the design of the exterior walls.

| North-South Direction | | | | | | | |
|---|---|---|---|---|---|---|---|
| Story | Height (ft) | Wind Pressures: Windward Walls | Wind Pressures: Leeward Walls | Total (psf) | Story Force (kips) | Story Shear (kips) | Overturning Moment (ft-kips) |
| 1 | 15 | 6.89 | -6.64 | 13.52 | 24.7 | 357.1 | 371 |
| 2 | 28.33 | 8.26 | -6.64 | 14.90 | 49.0 | 332.3 | 1387 |
| 3 | 41.67 | 9.22 | -6.64 | 15.86 | 50.0 | 283.3 | 2084 |
| 4 | 55 | 9.98 | -6.64 | 16.62 | 52.8 | 233.3 | 2905 |
| 5 | 68.21 | 10.62 | -6.64 | 17.25 | 54.8 | 180.5 | 3740 |
| Roof | 84.5 | 11.29 | -6.64 | 17.92 | 125.7 | 125.7 | 10619 |
| | | | | Total | 357.1 | 1512 | 21107 |
| | | | | Internal Pressure = | | ±3PSF | |

**Table 2 wind pressures (North-South Direction)**

| East-West Direction | | | | | | | |
|---|---|---|---|---|---|---|---|
| Story | Height (ft) | Wind Pressures: Windward Walls | Wind Pressures: Leeward Walls | Total (psf) | Story Force (kips) | Story Shear (kips) | Overturning Moment (ft-kips) |
| 1 | 15 | 6.89 | -3.72 | 10.60 | 19.4 | 292.5 | 291 |
| 2 | 28.33 | 8.26 | -3.72 | 11.98 | 38.9 | 273.1 | 1102 |
| 3 | 41.67 | 9.22 | -3.72 | 12.94 | 40.5 | 234.3 | 1688 |
| 4 | 55 | 9.98 | -3.72 | 13.70 | 43.3 | 193.7 | 2383 |
| 5 | 68.21 | 10.62 | -3.72 | 14.33 | 45.4 | 150.4 | 3096 |
| Roof | 84.5 | 11.29 | -3.72 | 15.00 | 105.0 | 105.0 | 8875 |
| | | | | Total | 292.5 | 1249 | 17435 |
| | | | | Internal Pressure = | | ±3PSF | |

**Table 3 wind pressures (East-West Direction)**

**Wind Story Load Diagrams**

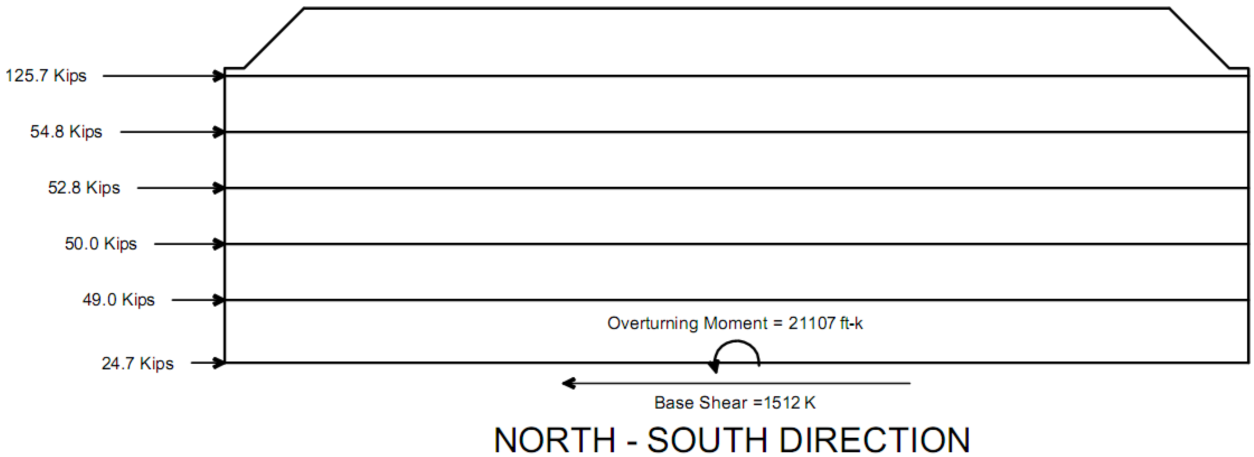



**Figure 10: Load Distribution along E-W Face**

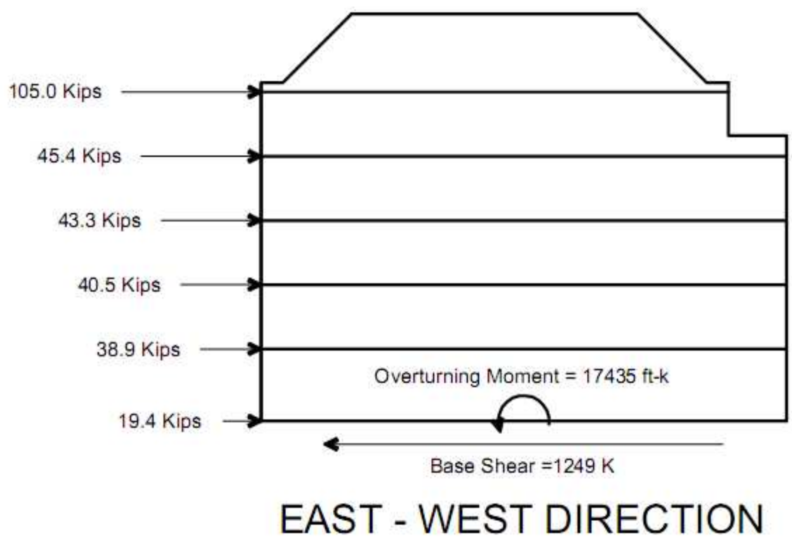



**Figure 11: Load Distribution along N-S Face**

The highest wind pressure calculated is 17.92 PSF. Because the roof screen was assumed to be another story, this value will be conservative. Overall the values obtained were reasonable for a five story office building. In the structural notes, the structural engineers listed a design value of 18.3 PSF. If the engineers used the highest value as a blanket load on the entire façade, it is within 10% of my calculated value.

## Seismic Loads

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Values for the short period response accelerations and the one second period response accelerations differed from those of the structural engineers. Both values I found were from the USGS computer program and the USGS seismic maps. A reason for the variance in the numbers could be caused by the fact that the structural engineers followed IBC 2000 when designing the building. Also, there could be a local provision in the Virginia area, which requires the response accelerations to be a certain value. Although research was done to find such a provision, none was discovered.

Because of the building height, soil class, and response accelerations the building fell into seismic design category A. Buildings in this category need only to be designed in accordance to section 11.7 of ASCE 7-05. Table 4 shows the equivalent force on each floor. The base shear created by the seismic load is much less than the base shear created by the wind loads.

| Seismic Loads | | | | |
|---|---|---|---|---|
| **Floor** | **$h_x$ (ft)** | **Weight (kips)** | **$F_x = 0.01\ w_x$ (kips)** | **$M_x$ (ft-kips)** |
| 2 | 15 | 2279 | 23 | 342 |
| 3 | 28.33 | 2256 | 23 | 639 |
| 4 | 41.67 | 2256 | 23 | 940 |
| 5 | 55 | 2042 | 20 | 1123 |
| Roof | 68.21 | 682 | 7 | 465 |
| | | **Total** | 95 | 3509 |

**Table 4: Equivalent Seismic Load**

# Load Distribution

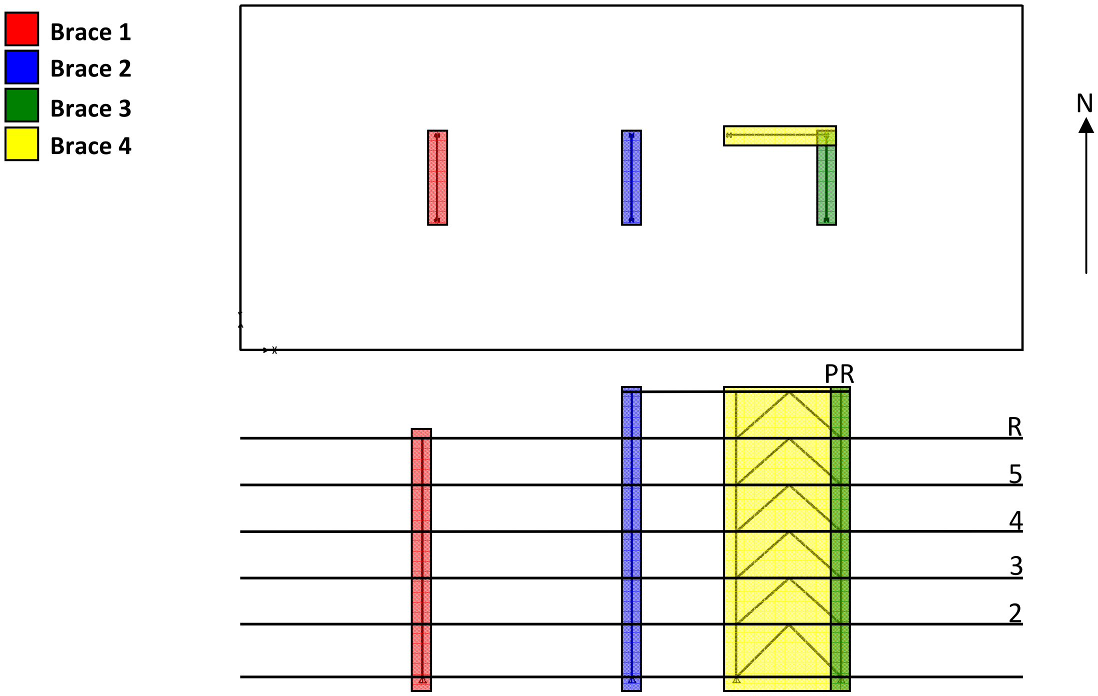



**Figure 12: Brace Frame Locations**

Loads applied to the exterior walls are transferred to the floors, which are assumed to be rigid diaphragms, and then to the braced frames. The percentage of total force applied to each frame is known as direct force or shear. In the north-south direction the total load is distributed between braces 1-3. Brace 4 resists all lateral loads applied in the east-west direction of the building, since it is the only one this direction; there is no distribution to other frames. Therefore, only load distribution in the north-south direction needs to be considered.

The percentage of the total load resisted by each frame is a function of each frame's relative stiffness. Two separate stiffness calculations had to be made; the stiffness of the three frames at the roof level, and the stiffness of braces 2 and 3 at the penthouse level. The reason for this is because on all floors below the roof, the loads are distributed between three of the frames, but above the roof only braces 2 and 3 resist loads. For simplification purposes it was assumed that the stiffness factors remain constant for all floors below the roof, although this not true. Figure 12 shows how only one brace lies in the east-west direction and the elevation shows how brace 1 does not extend to the penthouse roof. Each frame is designated by the color coordination shown in the key.

An approximation of each frame stiffness was determined by applying an arbitrary load to the desired level, and the deflection due to this load was recorded. The k-value was then calculated by dividing the load by the displacement. The percentage of load transferred to each frame was then calculated by dividing each individual frame's stiffness factor by the sum of all the other k-values at the desired level.

| Load Distribution | | | | | |
|---|---|---|---|---|---|
| Brace No. | Load Applied (kips) | Level Applied | Disp. (in.) | k-value | % Load |
| 1 | 1 | Roof | 0.01365 | 73 | 29 |
| 2 | 1 | Roof | 0.01059 | 94 | 37 |
| 3 | 1 | Roof | 0.01123 | 89 | 35 |
| 2 | 1 | Penthouse | 0.0159 | 63 | 52 |
| 3 | 1 | Penthouse | 0.01702 | 59 | 48 |
| 4 | - | - | - | - | 100 |

**Table 5: Load Distribution**

Displacements were found using 2-D models, created in SAP. Table 5 summarizes the load distribution between the frames. On the lower floors, brace 1 resists the least amount load, which is expected because the diagonal members in the brace on the left side do not frame into the beam/column connection.

For all the levels braces 2 and 3 resist similar amounts of load, with brace 2 resisting a slightly larger load. This would be due to varying member sizes between the two frames. Highlighted in blue is the distribution on the lower floors, and the load distribution at the penthouse roof is highlighted in green.

## Torsion

Lateral loads applied to the building create torsion because of an eccentricity caused by a difference in the location of the center of rigidity and the center of pressure for wind loads, or the difference between the center of rigidity and the center of mass for seismic loads. All frames, regardless of their orientation, will have an influence on the distribution of the torsional force to each frame. A conceptual idea of torsion, due to wind in the north-south direction on Building A, was reviewed by hand calculations of the center of rigidity.

The x-component of the center of rigidity was calculated using the formula $CRX = \frac{\sum k_y * x}{\sum k_y}$, where x is the distance from the lower left corner of the building to each frame in the y-direction, (braces 1-3). The y-component of the center of rigidity lies along the same column line brace 4 is located on, because there are no other frames in the x-direction to influence it up or down. Center of rigidity calculations are summarized in Table 6.

| Center of Rigidity | | |
|---|---|---|
| | x | y |
| Floors 2-R | 124 | 66.1 |
| Penthouse | 150 | 66.1 |

**Table 6: Center of Rigidity**

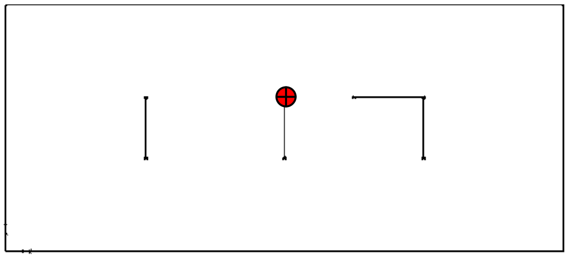


**Figure 14: CR, 2-R**

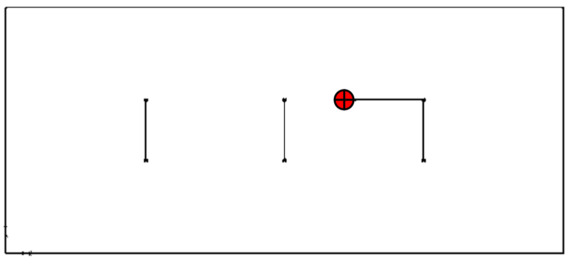


**Figure 13: CR, PR**

Figure 14 shows the center of rigidity for floors 2-R, and Figure 13 shows the location of the center of rigidity for the penthouse roof. The location of each center of rigidity is verified by the percentage of load resisted by each frame. For the lower floors the center of rigidity should be located slightly to the right of brace 2, because brace 3 is stiffer than brace 1 but more flexible than brace 2. At the penthouse the center of rigidity is slightly to the left of the centerline between brace 2 and 3 which corresponds to the fact that brace 2 resists slightly more load than brace 3.The forces on the frames due to torsion are calculated using the formula, $F_{it} = \frac{e_x * P_y * d_i * k_i}{\sum k_j * d^2}$, where d is the distance from each frame to the center of rigidity. The denominator is the sum of every frame multiplied by its distance to the center of rigidity squared, because as mentioned before all frames will influence the distribution of the torsional load. The numerator consists of the load, eccentricity, and individual k and d values for each frame. Brace 4 lies along the y-component of the center of rigidity, and therefore no torsional force will exist in the brace, and it will have no influence on the distribution of the force due to torsion. For this part of the report these forces were not calculated, the formulas were presented for a conceptual understanding of torsional forces. A hand calculation including both direct shear and torsional shear is completed later to verify the ETABS output.

With a wind load in the north direction the force due to torsion will add to the direct force in braces 1 and 2, and subtract in brace 3. Figure 15 shows how the torsion created by the wind in the north-south direction exerts torsional forces in each brace. The direct force on each frame is shown in red, and the torsional force is shown in black.

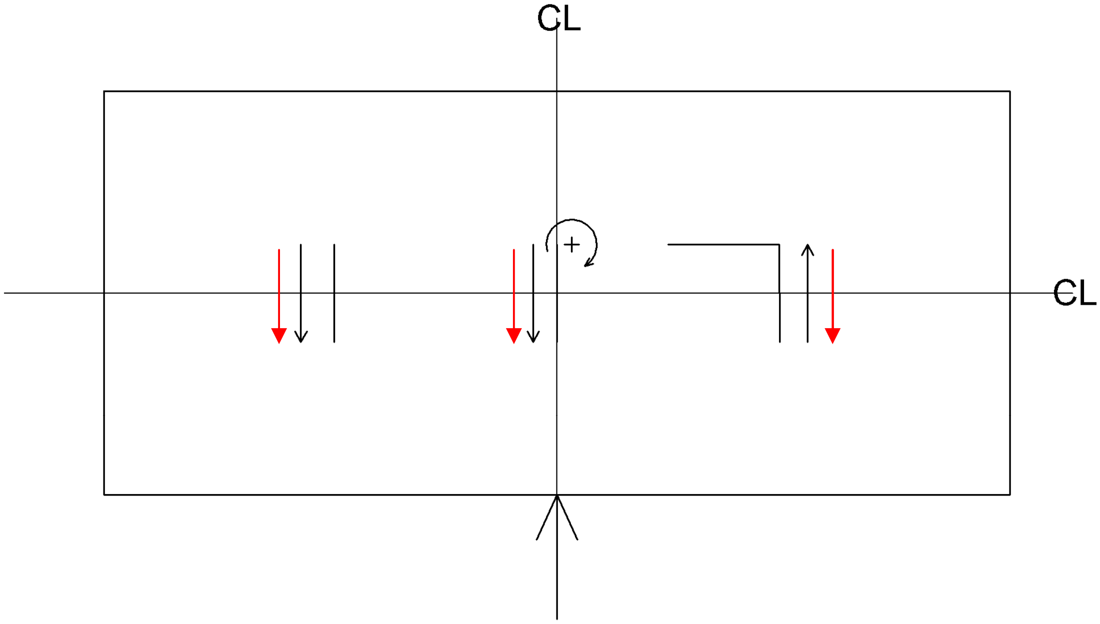



**Figure 15: Force Due to Torsion**

## ETABS Model

In order to perform an accurate lateral analysis of the Belmont Executive Building Center; Building A, a 3-D model of the building was created using the computer program ETABS. Because only the lateral system was being analyzed, no gravity members were added. The four braced frames are the only members present. All floors were modeled as rigid diaphragms, and assigned an appropriate mass. For floors 2-5 and the penthouse floor, the mass included the weight of the steel framing, concrete on metal deck, MEP, partitions, and finishes. The roof mass included the weight of the roof joists, MEP, and the weight of the ballasted roof. Because floor openings would influence the center of mass, the large atrium opening on the 2$^{nd}$ floor was modeled. Figure 16 is a picture of the 3-D model; the atrium opening can be seen in white. Stairwell and elevator openings were not modeled because they are symmetric about the centerline of the building in both the x and y directions. Therefore they would have no influence on the center of mass.

The foundation system of the building consists of square spread footings. Because they are capable of resisting very little to no moment, the base connections were set as pin connections. Another key modeling assumption was to release the diagonal members so they could not carry any moment. It should also be noted that no direct loads were applied to the penthouse level because all load is assumed to be blocked by the roof screen. All wind forces encountering the horizontal projection of the roof screen are transferred to the roof level.

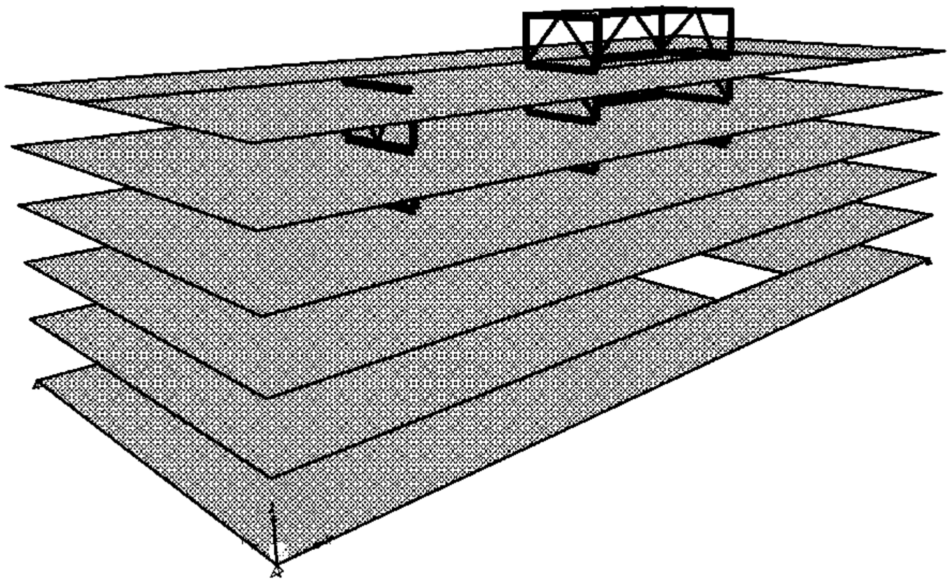


Figure 16: ETABS Model

To verify that the building model was correct, hand calculations of the center of rigidity were compared to the output from ETABS. An accurate location of the center of rigidity was found by determining the stiffness of each frame on all levels. The corresponding x and y coordinates were then determined through the same method as mentioned before. Table 7 summarizes the hand calculations and the ETABS output.

| Center of Rigidity Comparison | | | | | | |
|---|---|---|---|---|---|---|
| | Hand Calculations | | ETABS Output | | % Diff. | |
| Floor | x | y | x | y | x | y |
| 2 | 129 | 66.1 | 124 | 66 | 4 | 0 |
| 3 | 125 | 66.1 | 130 | 67 | 4 | 1 |
| 4 | 123 | 66.1 | 132 | 68 | 7 | 3 |
| 5 | 123 | 66.1 | 134 | 70 | 9 | 6 |
| R | 124 | 66.1 | 136 | 72 | 10 | 9 |
| PR | 150 | 66.1 | 139 | 72 | 7 | 9 |

Table 7: CR Comparison

The largest discrepancy between the two calculations is the location of the y-component of the center of rigidity. Although only brace 4 is in the x-direction, the center of rigidity lies above brace 4, not in line with it. This exists because brace 3 and 4 share a column, and in order to balance the twisting caused by the lateral loads, the center of rigidity must lie above brace 4. The principle can be compared to a c-channel, which has a shear center located outside of the section to balance the moments created by the shear forces in the web and flanges when the member is in bending. Figure 17 shows the location of the shear center for a channel highlighted in red. Further investigation into this discrepancy will be conducted later. For the purpose of this report it will be assumed the model is correct. Compared to the hand calculations the center of rigidities did not differ by more than 10%. These minor discrepancies could be caused by out of plane bending in the other braces.

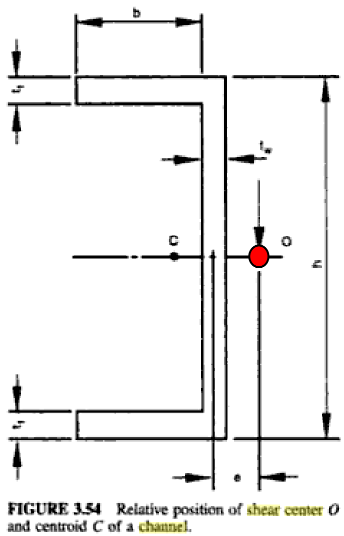



**Figure 17: C-Channel CS Location**

With the model verified all lateral analysis was completed with the aid of ETABS.

## Wind Analysis Results

Four separate load cases in ASCE 7-05 were considered when determining the total displacement caused by wind loads. Figure 18 shows each load case. For cases 2 and 4 the additional eccentricity was subtracted from the center of pressure in both the x and y direction to create a larger torsional moment. It is expected that either case 2 or 4 will control because the frames are located at the center of the building. Each case was analyzed in ETABS, and the total displacement of each brace was then recorded. The results of the analysis are shown in Table 8. Case 2 was in fact the controlling case, and the largest displacement of 1.99" was measured for brace 1.

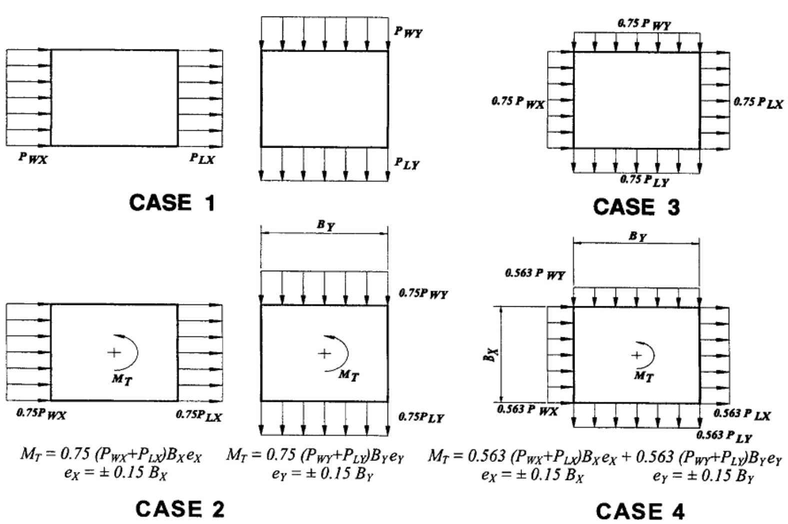



**Figure 18: Wind Cases**

This is less than the allowable limit of L/400 or 2.05”.

| Total Displacement: ETABS Output | | | | | | |
|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 3 | Case 4 | Height (ft.) | Allowable Displacement L/400 |
| **Brace 1** | 1.04 | 1.99 | 0.88 | 1.26 | 68.33 | 2.05 |
| **Brace 2** | 0.80 | 1.00 | 0.79 | 0.72 | 81.67 | 2.45 |
| **Brace 3** | 0.45 | -0.18 | 0.61 | 0.07 | 81.67 | 2.45 |
| **Brace 4** | 0.66 | 0.51 | 0.55 | 0.36 | 81.67 | 2.45 |

**Table 8: Total Displacement**

A hand calculation was then performed to verify the ETABS output. Both direct shear and shear due to torsion were considered. Only the wind load in the north direction was considered, because the direct shear in this direction would be much greater than the force due to torsion created by the wind load in the east-west direction. For simplification of the distribution of loads, the k-value was considered to be constant on every floor, and stiffness values found at the roof level was used to determine the load distribution. Figure 15 how the wind in case 2 is distributed to the braces. The only difference between case 2 and the figure is the presence of a larger eccentricity. It can be seen that the torsional force will add to the direct force for braces 1 and 2, and will subtract from the direct force for brace 3. All calculations were carried out following the process described previously. Table 9 summarizes the calculations.

| Case 2 | | Force Due to Direct Load | | | | Force Due to Torsion | | | | Total Force | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Brace No. | | | | Brace No. | | | | Brace No. | | | |
| Floor | Py | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 |
| 2 | 38.6 | 11.01 | 14.17 | 13.42 | 0.00 | 13.22 | 2.11 | 12.12 | 0.00 | 24.23 | 16.29 | 1.30 | 0.00 |
| 3 | 38.7 | 11.04 | 14.21 | 13.45 | 0.00 | 13.26 | 2.12 | 12.15 | 0.00 | 24.29 | 16.33 | 1.30 | 0.00 |
| 4 | 40.6 | 11.58 | 14.91 | 14.11 | 0.00 | 13.91 | 2.22 | 12.75 | 0.00 | 25.48 | 17.13 | 1.37 | 0.00 |
| 5 | 42.1 | 12.01 | 15.46 | 14.64 | 0.00 | 14.42 | 2.30 | 13.22 | 0.00 | 26.43 | 17.76 | 1.42 | 0.00 |
| R | 75 | 21.39 | 27.54 | 26.07 | 0.00 | 25.69 | 4.11 | 23.55 | 0.00 | 47.08 | 31.64 | 2.53 | 0.00 |
| PR | 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

**Table 9: Hand Calculation**

The total forces were then applied to the 2-D model of brace 1 in SAP. A total deflection of 2.19 inches was calculated, which is very close to the ETABS output, and further verifies that the model is correct. A reason for the difference between the two deflections is most likely caused by the difference in the center of rigidity, and/or a more accurate calculation of the brace stiffness in ETABS.

## Seismic Analysis Results

Seismic loads found in technical report 1 were placed into the ETABS model, and forces in both directions were analyzed separately. Seismic loads in the east-west direction were used to determine the total drift of brace 4, and for braces 1-3 the seismic load in the north-south direction was considered. All story drifts were limited to .025h. Because the seismic loads are much smaller in comparison to the wind loads, the resulting displacements were also much less. All displacements were acceptable, and are listed in Table 10.

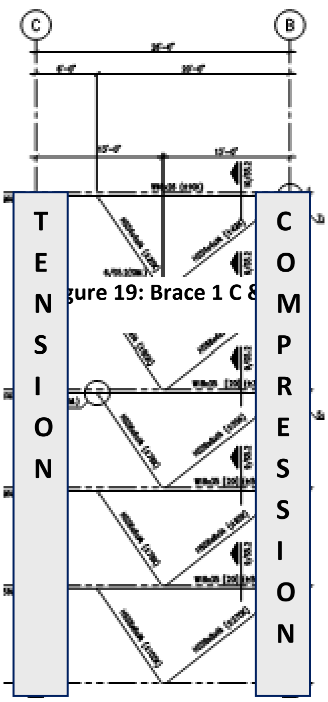
| Seismic Displacement: ETABS Output | | | |
|---|---|---|---|
| | | Height (ft.) | Allowable Displacement .025*h |
| **Brace 1** | 0.22 | 68.33 | 1.71 |
| **Brace 2** | 0.21 | 81.67 | 2.04 |
| **Brace 3** | 0.19 | 81.67 | 2.04 |
| **Brace 4** | 0.40 | 81.67 | 2.04 |

**Table 10: Seismic Displacement**

# Overturning/Spot Check

Only brace 1 was analyzed to see if overturning would be a controlling factor in the foundation design. To determine if the column in compression would fail, the load combination 1.2D + 1.6W + L was considered. The load combination 0.9D + 1.6W was considered when determining if the column in tension would fail. Different load cases were used because for the compression member the 1.2D + L combination creates an additional compressive force. Likewise the tensile member sees a greater tension force because there is less gravity load to counteract the upward force created by the wind. Figure 19 shows which members are in compression and which ones are in tension. ETABS was used to find the axial force due to the wind load, in the lowest columns of brace 1. Manual takedowns were performed to determine the amount of gravity load acting on both columns. The wind load was applied at the center of pressure of the building.

Table 11 summarizes the loads on the lowest compressive column. A total axial load of 841 kips was found to be exerted at the lowest floor. With an effective length of 13.33 ft the column is capable of supporting1050 kips, which is greater than the factored load. The member is adequate to resist the loads.




gure 19: Brace 1 C &

| Compression Column | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story | Tributary Area (ft^2) | Dead Load (psf) | Live Load (psf) | Influence area (ft^2) | Reduction Factor | Live Load (kips) | Dead Load (kips) | Load Combination | Load at Floor (kips) | Accumulated Load (kips) | 1.6W Load (kips) | Total Load (kips) |
| R | 992 | 33 | 20 | 3966 | 0.50 | 10 | 33 | 1.2D + 1.6W + L +0.5Lr | 44 | 44 | | |
| 5 | 992 | 85 | 100 | 7932 | 0.42 | 42 | 84 | 1.2D + 1.6W + L | 143 | 187 | | |
| 4 | 992 | 85 | 100 | 23796 | 0.40 | 40 | 84 | 1.2D + 1.6W + L | 141 | 328 | | |
| 3 | 992 | 85 | 100 | 95184 | 0.40 | 40 | 84 | 1.2D + 1.6W + L | 141 | 469 | | |
| 2 | 992 | 85 | 100 | 475920 | 0.40 | 40 | 84 | 1.2D + 1.6W + L | 141 | 610 | 231 | 841 |

**Table 11: Compressive Colum Check**

Table 12 shows both the gravity load and the wind load acting on the lower column. It can be seen that the gravity load is greater than the tension force created by the factored wind load. Therefore uplift caused by overturning was not an issue for foundation design.

| Tension Column | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Story | Tributary Area (ft^2) | Dead Load (psf) | Dead Load (psf) | Load Combination | Load at Floor (kips) | Accumulated Load (kips) | 1.6W Load (kips) | Total Load (kips) |
| R | 992 | 33 | 33 | 0.9D + 1.6W | 29 | 29 | | |
| 5 | 992 | 85 | 84 | 0.9D + 1.6W | 76 | 105 | | |
| 4 | 992 | 85 | 84 | 0.9D + 1.6W | 76 | 181 | | |
| 3 | 992 | 85 | 84 | 0.9D + 1.6W | 76 | 257 | | |
| 2 | 992 | 85 | 84 | 0.9D + 1.6W | 76 | 333 | -125 | 208 |

**Table 12: Overturning Check**

## Conclusion

An understanding of how the lateral loads are distributed and resisted by Building A helped verify that a 3-D ETABS model of the building was correct. Using the model it was found that Case 2 was the controlling wind combination. The maximum total displacement measured was for brace 1, and it was less than the maximum limit of L/400. A hand calculation of deflection produced results very similar to that of the ETABS program, and further verified that the building had been modeled correctly. Seismic drifts were calculated, and as expected were much less than those caused by wind. An analysis of brace 1 determined that the columns on the first floor were adequate to support the factored axial load. It was also determined that the amount of gravity load on the lower column was greater than that of the upward wind force. Therefore overturning/uplift would not affect the foundation.

# Appendix A

## Snow load calculations

NICK ZIEGLER | SNOW LOAD CALCULATIONS | (1)

FLAT ROOF SNOW LOAD

ROOF SLOPE = 1/4 in PER FOOT

$p_f = 0.7 \cdot C_e \cdot C_t \cdot p_g \cdot I$

THERMAL FACTOR: $C_t = 1.0$ (TABLE 7-3)

IMPORTANCE FACTOR: $I$

BUILDING CATEGORY = II → $I = 1.0$ (TABLE 7-4)

EXPOSURE FACTOR: $C_e$

TERRAIN CATEGORY = B
PARTIALLY EXPOSED

$C_e = 1.0$ (TABLE 7-2)

GROUND SNOW LOAD: $p_g$

$p_g = 30$ psf (FIGURE 7-1)

$p_f = 0.7(1.0)(1.0)(30)(1.0) = 21$ psf

DRIFT SNOW LOADS FOR CERTAIN JOISTS

2LKSP1 – DRIFT ACCUMULATES AGAINST PARAPET

$\gamma = 0.13(30) + 14 = 17.9$ lb/ft³

WINDWARD DRIFT

$h_c$ = HEIGHT OF PARAPET $= 4'-4\,3/8''$

$$h_d = 0.75\left[0.43(l_u)^{1/3}(p_g+10)^{1/4} - 1.5\right]$$
$$= 0.75\left[0.43(107)^{1/3}(30+10)^{1/4} - 1.5\right]$$
$$= 2.73 \text{ ft} < h_c \rightarrow w = 4(2.73) = 10.92 \text{ ft}$$

$$p_d = 17.9(2.73) = 48.867 = 49 \text{ lb/ft}$$

# Appendix B

**Wind Load Variables, Coefficients, Calculations**

| Variables | |
|---|---|
| **Basic Wind Speed** | V = 90MPH |
| **Wind Directionality Factor** | $K_d$ = 0.85 |
| **Importance Factor** | I = 1.0 |
| **Exposure Category** | B |
| **Guss Factor** | G=0.85 |
| **External Pressure Coefficient Windward Wall** | $C_p$=0.8 |
| **External Pressure Coefficient (N-S Leeward Wall)** | $C_p$= -0.5 |
| **External Pressure Coefficient (E-W Leeward Wall)** | $C_p$= -0.28 |
| **Internal Pressure Coefficient** | $GC_p$= ±0.18 |

**Table 13: Wind Variables**

| Velocity Pressure Coefficients: $K_{z,}$ $K_h$ | | | |
|---|---|---|---|
| **Story** | **Height** | **$K_z$** | **$q_z$** |
| 1 | 15 | 0.57 | 10.13 |
| 2 | 28.33 | 0.69 | 12.15 |
| 3 | 41.67 | 0.77 | 13.56 |
| 4 | 55 | 0.83 | 14.68 |
| 5 | 68.21 | 0.89 | 15.61 |
| Roof Screen | 84.5 | 0.94 | 16.60 |

**Table 14: Pressure Coefficients**

METHOD 2

BASIC WIND SPEED: $V = 90$ MPH
WIND DIRECTIONALITY FACTOR: $K_d = 0.95$
IMPORTANCE FACTOR: $I = 1.0$
EXPOSURE CATEGORY: B
TOPOGRAPHIC FACTOR: $K_{zt} = 1.0$
VELOCITY PRESSURE COEFFICIENTS: $K_h$

- CASE 2
- EXPOSURE B
- $h = 84'-6"$

- INTERPOLATE VALUE FROM TABLE 6-3

$$K_h = \frac{(84.5 - 80)(0.96 - 0.93)}{(90 - 80)} + 0.93$$

$$= 0.9435$$

VELOCITY PRESSURE COEFFICIENTS: $K_z$

$K_z = 0.59 \quad z < 15\text{ft}$
$K_z = 2.01\,(z/z_g)^{2/\alpha}$
$z_g = 1200$
$\alpha = 7.0$

- CASE 2
- EXPOSURE B

| HEIGHT | $K_z$ | $q_z$ |
|---|---|---|
| 0'– 15' | 0.59 | 10.05 |
| 20' | 0.62 | 10.93 |
| 25' | 0.66 | 11.63 |
| 30' | 0.70 | 12.34 |
| 40' | 0.76 | 13.40 |
| 50' | 0.81 | 14.28 |
| 60' | 0.85 | 14.98 |
| 70' | 0.89 | 15.69 |
| 80' | 0.93 | 16.39 |
| 84.5' | 0.9435 | 16.63 |

* DIFFERENT HEIGHTS IN EXCEL FILE/TABLE CALCULATED BY THE SAME METHOD

VELOCITY PRESSURES @ h & z

$$q_h = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$$
$$= 0.00256(0.9435)(1.0)(0.85)(90)^2(1.0) = 16.63$$

- SAME FORMULA USED FOR $q_z$: SEE TABLE ABOVE FOR VALUES

GUST FACTOR:

$$n_1 = \frac{100}{H} = \frac{100}{84.5} = 1.18 \geq 1.0 \quad \therefore G = 0.85$$

NICK ZIEGLER WIND LOAD CALCULATIONS (2)

PRESSURE COEFFICIENTS: $C_p$

- NORTH - SOUTH DIRECTION

$B = 244'$
$L = 109'$

$L/B = \frac{109}{244} = 0.447$

WINDWARD WALL: $C_p = 0.8$
LEEWARD WALL: $C_p = -0.5$

- EAST - WEST DIRECTION

$B = 109'$
$L = 244$

$L/B = \frac{244}{109} = 2.24$

WINDWARD WALL: $C_p = 0.8$
LEEWARD WALL: $C_p = -0.29$ - INTERPOLATED

INTERNAL PRESSURE COEFFICIENTS: $GC_{pi}$

$GC_{pi} = \pm 0.18$

DETERMINE DESIGN WIND PRESSURES

- WINDWARD WALLS: $p_z = q_z G C_p - [q_h(GC_{pi})]$ NEGLECT FOR WINDWARD & LEEWARD PRESSURE
- LEEWARD WALLS: $p_h = q_h G C_p - [q_h (GC_{pi})]$

* SEE TABLE FOR VALUES

# Appendix C

## Seismic Load Calculations

## SEISMIC CALCULATIONS

SPECTRAL RESPONSE ACCELERATIONS:

$S_S = 0.158$ (USGS MAPS)
$S_1 = 0.051$

SITE CLASS OF SOIL: C

$S_{MS} = F_a \cdot S_S$ (EQ. 11.4-1)

$S_{M1} = F_v \cdot S_1$ (EQ. 11.4-2)

$F_a = 1.2$ (TABLE 11.4.1)
$F_v = 1.7$ (TABLE 11.4.2)

$S_{MS} = 1.2(0.158) = 0.1896$
$S_{M1} = 1.7(0.051) = 0.0867$

$S_{DS} = 2 \frac{S_{MS}}{3}$ (EQ. 11.4-3)

$S_{D1} = 2 \frac{S_{M1}}{3}$ (EQ. 11.4-4)

$S_{DS} = 2 \frac{0.1996}{3} = 0.1331$

$S_{D1} = 2 \frac{0.0867}{3} = 0.0578$

OCCUPANCY CATEGORY: II

$T_a = C_t \cdot h_n^x$ Eq. 12.8-7

$= (0.02)(82.6)^{0.75}$ * CONCENTRICALLY BRACED FRAME

$= 0.548$ $h_n = 82.6'$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.0578}{0.1331} = 0.434$

$0.8T_s = 0.8(0.434) = 0.3472 < T_a$

$T_a > 0.8T_s$

$S_s \leq 0.15$ & $S_1 \leq 0.04$

SITE DESIGN CATEGORY A

**Building Weight**

To find the weight of the steel structure a typical bay was analyzed and the average weight per square foot was determined. A load of 15 PSF was assumed for the exterior façade, large MEP equipment was added in, and the weight of the roof screen was included. All applicable dead loads were also applied.

| Average Weight Per Floor | |
|---|---|
| **Material** | **PSF** |
| Steel Framing | 8 |
| 3 1/4" LWC on 3" comp. metal deck | 60 |
| MEP | 5 |
| Finishes | 5 |
| Partitions | 15 |
| **Total** | 93 |

**Table 15: Average Weight Per Floor**

| Building Weight Per Floor | | | |
|---|---|---|---|
| **Load (PSF)** | **Floor** | **Floor Area (SF)** | **Weight (kips)** |
| 93 | 2 | 25707 | 2391 |
| 93 | 3 | 25435 | 2365 |
| 93 | 4 | 25435 | 2365 |
| 93 | 5 | 22850 | 2125 |
| 22 | Roof | 22850 | 503 |
| | | **Total** | 9749 |

**Table 16: Total Weight Per Floor**

| Weight of Mechanical Equipment | |
|---|---|
| **Equipment Name** | **Equipment Weight (lbs)** |
| Cooling Tower | 13710.0 |
| EF-1 | 500.0 |
| OAF-1 | 1300.0 |
| EF-2 | 500.0 |
| OAF-2 | 1300.0 |
| EF-3 | 500.0 |
| HP-1 | 396.0 |
| HP-2 | 396.0 |
| **Total (kips)** | 18.6 |

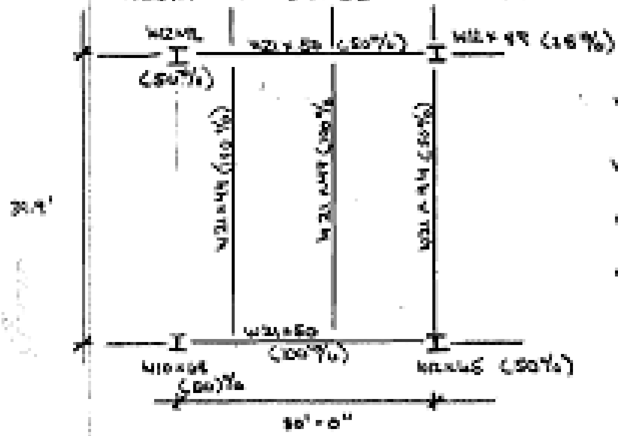
**Table 17: Weight of Large Mechanical Eq.**

| Total Building Weight (kips) | |
|---|---|
| Total Floor Weight | 9749 |
| Exterior Façade | 726 |
| Roof Screen | 15 |
| Large MEP Eq. | 18.6 |
| **Total** | 10509 |

**Table 18: Total Weight**

BUILDING SELF - WEIGHT 9)

WEIGHT OF STEEL IN TYPICAL BAY




TOTAL WEIGHT

W21x44: (39.9)(44)(2) + (39.9)(44)(0.50)
= 4389 lb
W21x50: (30)(50)(0.50) + (30)(50)
= 2250 lb
W10x68: (14)(68)(0.50)
= 476 lb
W12x65: (14)(65)(0.50)
= 455 lb
W12x87: (14)(87)(0.25)
= 305 lb
W12x96: (14)(96)(0.50)
= 672 lb

TOTAL WEIGHT: 8547 lb

TOTAL SF: 30(39.9) = 1197 ft²

AVERAGE lb/ft² = $\frac{8547}{1197}$ = 7.1 lb/ft²
≈ 8 lb/ft²

FLOOR AREA

2: 25,702 SF
3: 25,435 SF
4: 25,435 SF
5: 22,850 SF

DEAD LOADS

8 PSF - STEEL FRAMING
60 PSF - 3in COMP METAL DECK 5 1/4 in LWC
5 PSF - MEP
10 PSF - PARTITIONS
83 PSF

WEIGHT PER FLOOR

2: 2134 K
3: 2111 K
4: 2111 K
5: 1897 K
8253 K

EXTERIOR FACADE

* ASSUME 15 PSF FOR AVERAGE WEIGHT

GROSS WALL AREA = 48,379 SF

TOTAL WEIGHT = $\frac{48,379 (15)}{1000}$ = 726 K

ROOF

* ASSUME ROOF STEEL FRAMING ≈ 3/4 FLOOR STEEL FRAMING

6 PSF - STEEL
5 PSF - MEP
11 PSF - ROOFING
22 PSF

WEIGHT = $\frac{22(22850)}{1000}$ = 503 K

BUILDING SELF-WEIGHT (2)

ROOF SCREEN:

18 K3 - 6.6 lb/ft
16 K2 - 6.3 lb/ft
14 K1 - 5.2 lb/ft
10 K1 - 5.0 lb/ft

W14 x 22 @ 30', 7.5' LONG

$$\frac{7.5(22) = 165 \text{ lb}}{30} = 5.5 \text{ lb/ft}$$

$$\text{TOTAL} - \frac{29.6 \text{ lb/ft } (519 \text{ ft})}{1000} = 15.0^k$$

LARGE MEP EQ'

$18.6^k$

BUILDING WT

19.6
15.0
9235
726
503

$9515^k \approx 9600^k$

*I increased partitions load to 20PSF, and therefore the total building weight is listed as 1059 in Table 10.