# Technical Report III

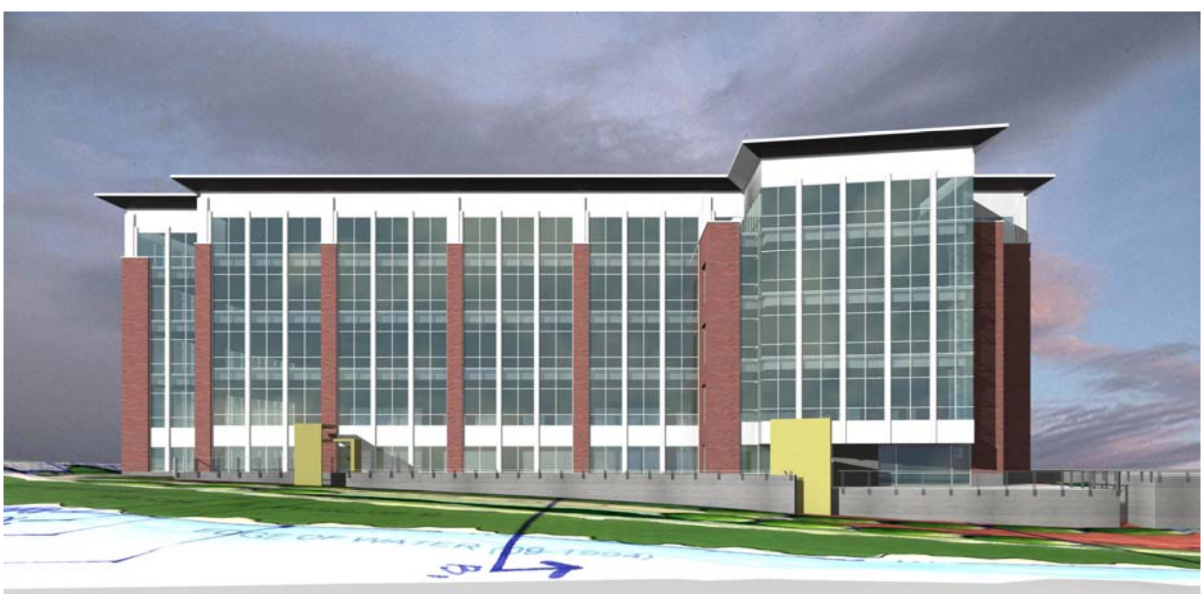
## Executive Summary

American Eagle Outfitters: Quantum III is a steel framed office building located in the South Side Works of Pittsburgh, Pennsylvania. This report analyzes the structure of this building and it's adequacy on the basis of currently accepted national codes, economy, and flexibility.

An introduction to the building and its structural systems is provided by outlining the anomalies in each of its aspects: foundations, separate floor framing, columns, and lateral load resisting systems. Next, codes used by Atlantic Engineering Services and those utilized in this analysis are described. Building material grades and strengths follow. An overview of floor framing and elevations of the five braced frames throughout the building give the reader a visual on which to build the concepts covered in this analysis. Gravity loads are then outlined.

Following, lateral load resisting systems are explored in detail for wind and seismic loading. Analysis criteria, methodology, and results are outlined. Story and frame shears are determined and presented in part in this section. Story drifts are then compared, with a conclusion of the adequacy of design methods and results previously presented. The report concludes with a series of appendices which show the subtle aspects of lateral design.

Overall, this report analyzes the detriments associated with computer modeling as a "black box". Not only must the user be aware of the structural systems and their design assumptions, but must also know how to implement them in a computer interface. As with this analysis, windscreens and composite action frame members hindered my ability to precisely model QIII. Minor differences resulted in story shears and drift, but were not significant enough to insinuate fundamental design errors. The errors described above are limited to the limits present in computer aided engineering software's. Details on framing analysis by hand and electronically are explored on the following pages.



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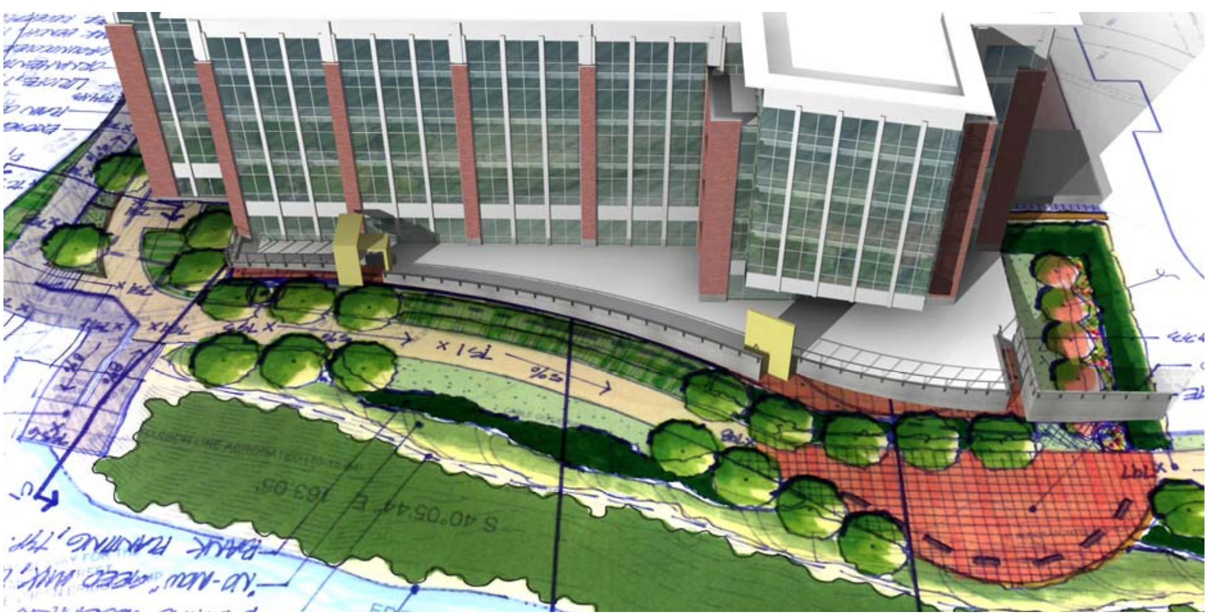
## I. Introduction

American Eagle Outfitters Quantum III: South Side Works is a genuine combination of structural design for flexibility and the blending of the architectural tastes of the developer, The Soffer Organization, with that of the existing South Side of Pittsburgh, PA. The building is 5 stories tall and contains loading, fire pump, and generator rooms on the first floor with the remainder of the first through the fifth floor having open plans for tenant fit-out. The roof holds a mechanical area surrounded by 12' tall windscreens for protection from the environment.

Open plans require a tradeoff between increased structural steel depths and beam span. The structural system of QIII reflects the need for flexibility with 30'x30' bays and a superimposed 20 psf partition load over all office spaces. The superimposed load is added onto the office live load with a supplemental 10 psf to account for the unpredictability of floor layouts.

Vertical trusses are placed at either the core of the building—the mechanical spaces, stairwells, and elevators; or the shell to limit interference with the open plan architecture. QIII is clad in curtain walls with interspersed brick façade. The overwhelming majority of the shell is composed of curtain walls including the entire north elevation shown below.

Following is an analysis to create a foundation from which to expand understanding of the existing lateral force resisting system of Quantum III. A combination of RAM Structural System, SAP2000, and hand calculations were used to analyze the lateral system. Computer strength design was verified with several hand calculations outlined in VI. Lateral Analysis on page 16.



## II. Structural Systems

### *Foundations and Geotechnical Concerns*

The foundation of Quantum III will be constructed on abandoned steel industry facility foundations with fills consisting of silty sand, cinder and slag. With the unpredictability of the subgrade to the deeper bedrock, and the Monongahela River directly adjacent to the building, shallow foundations cannot be used. The fill located deeper in the subgrade has a higher bearing capacity than the aforementioned soils. Therefore, Geo-Mechanics Inc. insisted on 16” diameter auger cast piles with an ultimate load capacity of 300 kips, and design load capacity of 120 kips. Bedrock is located roughly 85 feet below the surface. With the water table resting at 730 ft above sea level—slab on grade is proposed to be at 753’.

Since the building includes no plans for a basement, slab on grade connects with pile caps and grade beams to make up the foundation of QIII. Grade beams line the exterior of the building and connect pile caps where lateral frames are located. Interior gravity columns typically have four piles with a single, separate pile cap, while columns on the exterior wall tie in with grade beams and three- to four-pile configurations.

### *Floor Framing*

All floor framing and steel deck is composite. A lightweight concrete slab on 3” galvanized steel deck was incorporated. Shear studs are 4” long and ¾” diameter in 2.5” lightweight concrete topping. The total slab and deck thickness is 5.5”. Typical roof framing consists of 3” metal roof deck, except the mechanical unit area. 2” deck with 3” lightweight concrete provides added support and dampens mechanical vibrations here. Typical girders are W24x55 with 28 studs. Infill beams are W18x35’s spaced at 10’ center to center with 16 studs. Refer to Figures 2 and 3 for the floor framing layout. All exceptions are explained in Technical Report I, available online at Sam Jannotti’s CPEP website.

### *Columns*

American Eagle Outfitters: Quantum III has a wide range of column sizes, ranging from W10’s to W14’s. Gravity columns range from a W10x33 to a W12x72. Moment frame columns run from W14x74’s to W14x193’s. Floor to floor heights are typically 13’-8”. Column splices for both gravity and lateral resistance are on the third and fifth floors with all roof framing columns being less than one floor height high. Unbraced length is not an issue in Quantum III since columns are braced at each floor.

### *Lateral Load Resisting System*

Five vertical trusses are arranged throughout the building core and exterior. Three of the five trusses are forms of a Chevron truss, with one x braced frame and the last being a single strut truss. Only one truss is on the exterior and is an excellent display of structure—a curtain wall provides a view of it from the exterior of the building. The remaining four trusses are interior and border stairs, elevators, or mechanical shafts. One of the interior trusses is eccentric to avoid a conflict with stair access doors on the easternmost corner of the building. Refer to page 8 for diagrams of the five vertical trusses outlined above.

## III. Framing Plans and Elevations

### *Typical Floor Plan*

Quantum III is designed for flexibility to allow individual tenants to lay out each floor as they please. It utilizes 30’ by 30’ bays with a two ‘cores’ containing elevators, stairs, mechanical openings and bathrooms. Since the extent of the work of the firms stated (Atlantic Engineering Services, The Design Alliance Architects, etc.) was core and shell—the exact placement of partitions is not addressed in the architectural plans as seen in Figure 1.

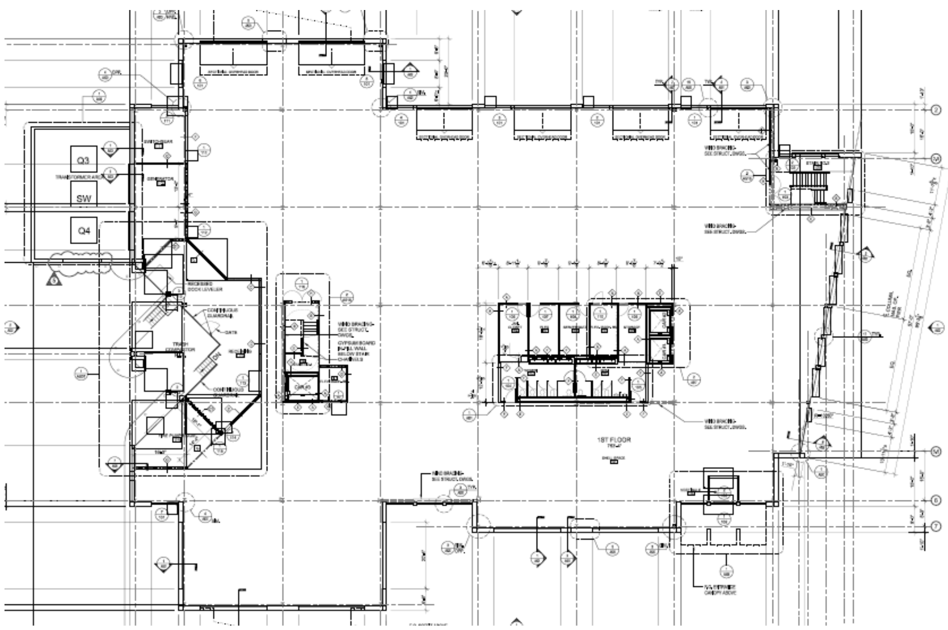


*Figure 1 – Typical Architectural Floor Plan*

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As you can see from the architectural plan, no partitions are even considered in this stage of the building development. To expand upon the structural system, typical bays for the second through fifth floors are shown below in Figure 2.

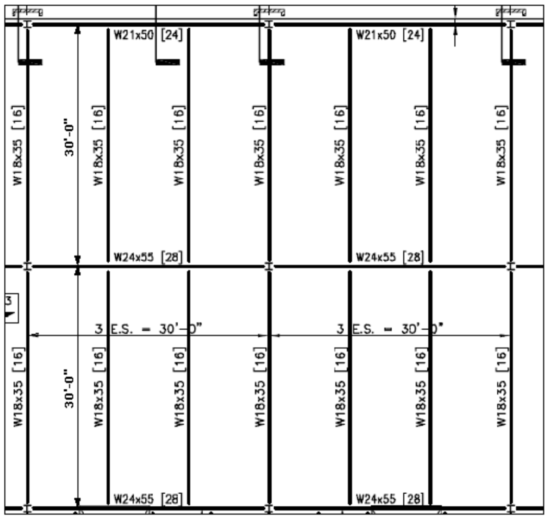


*Figure 2 – Typical Bay*

The W24x55 girders are 30' on center, with W18x35's at 10' on center. American Eagle Outfitters Quantum III has two bays to the north of the building cores as discussed earlier, and one set of bays to the south as seen in Figure 3.

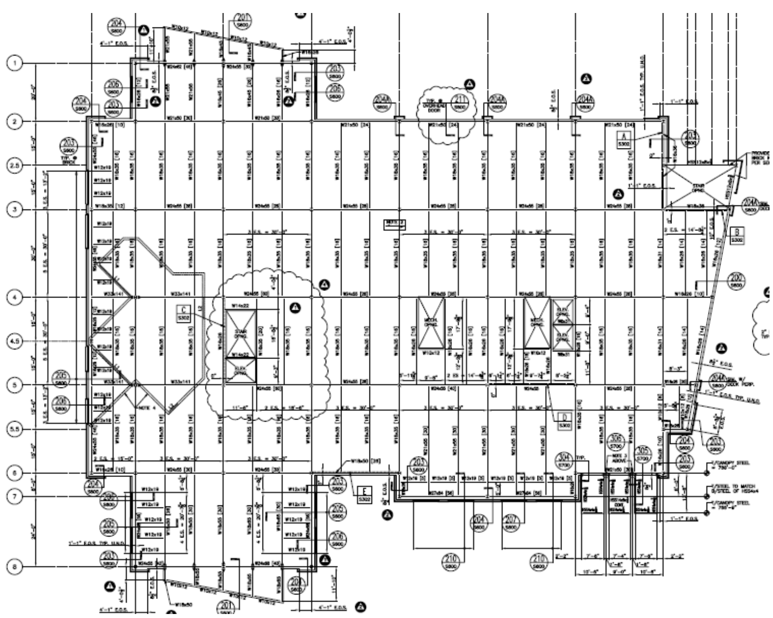

*Figure 3 – Typical Floor Framing*

***Lateral Load Resisting Elements***

As stated earlier there are five vertical trusses arranged throughout the shell and core of American Eagle Outfitters Quantum III. As shown in Figure 4, their placement was based on resisting interference with the open plan. Also, on the next page are elevations of the vertical trusses in Figures 5 and 6.

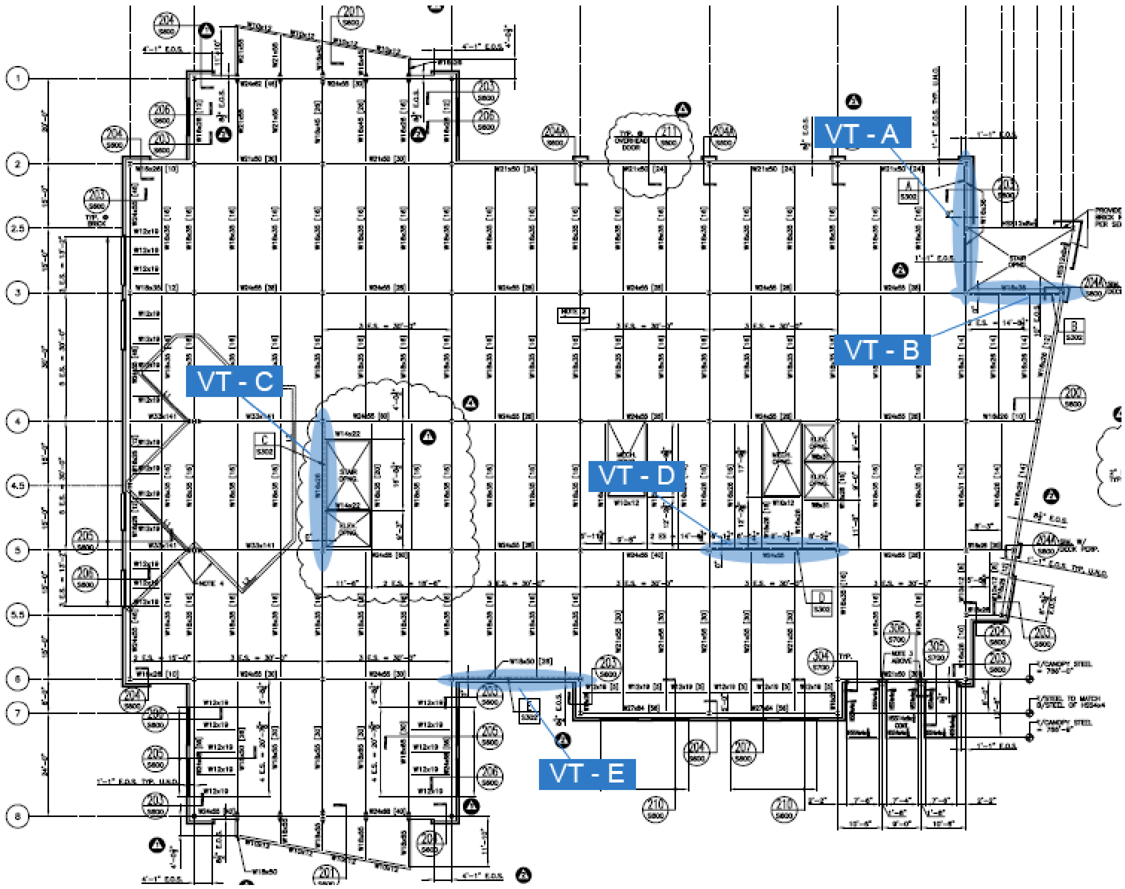


*Figure 4 – Vertical Truss Locations*

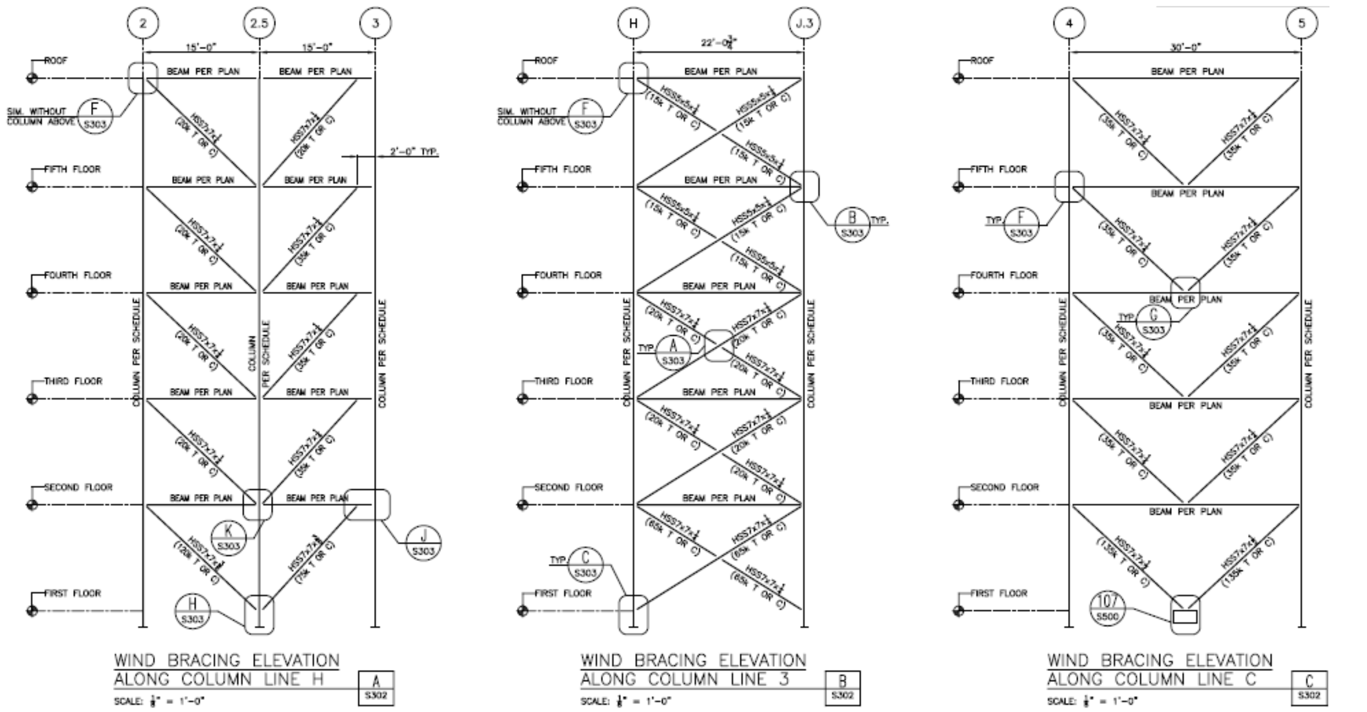


*Figure 5 – Vertical Trusses A, B and C (VT-A, B, C)*

Vertical truss (VT) A is a single strut truss, VT-B is an x-braced frame, and VT-C is a Chevron truss. VT-A contains an eccentricity to avoid an architectural conflict with stair access doors. All three of the above trusses are located on the interior of the building around stairs, elevators, or mechanical shafts.

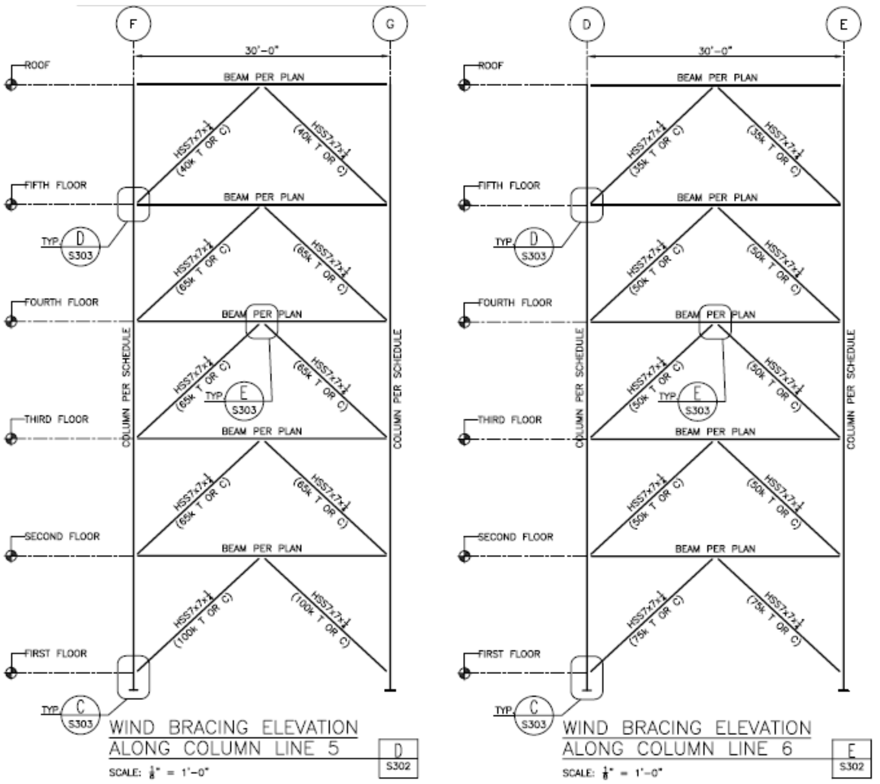


*Figure 6 – Vertical Trusses D and E (VT-D, E)*

As shown above, VT-D and E are inverted Chevron trusses. VT-E is the only truss situated on an exterior wall of the building as described earlier.

***3D Model Images***

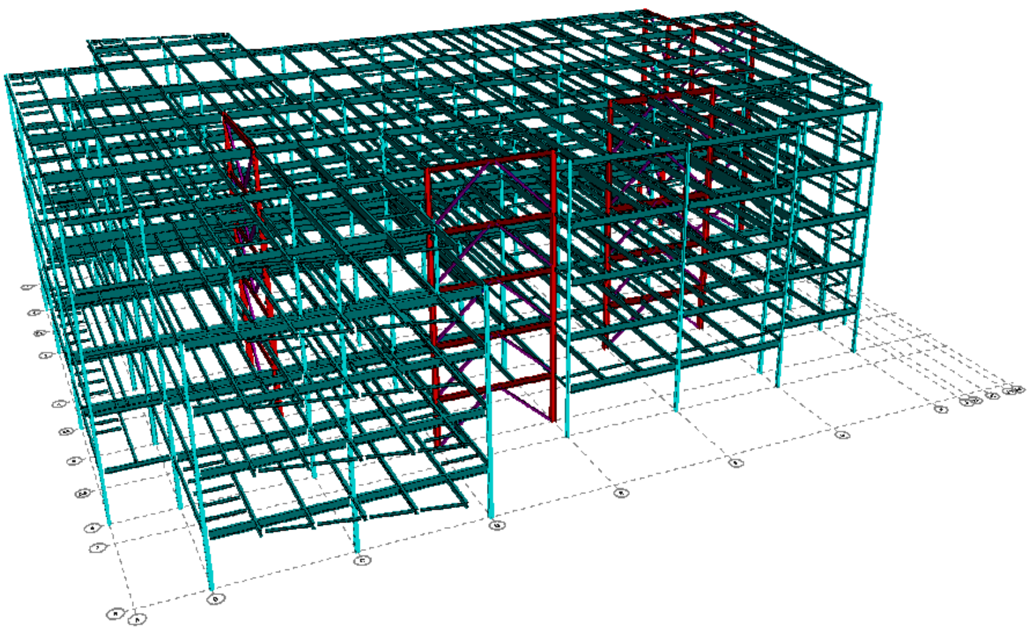

*Figure 7 – 3D View from West Building Corner*

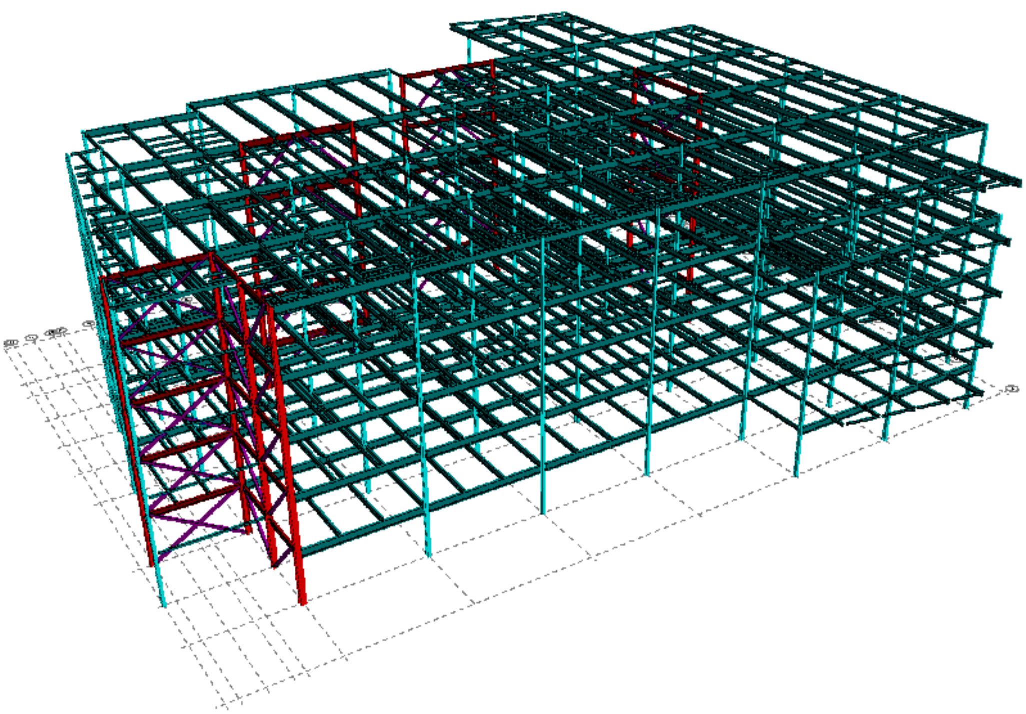

*Figure 8 – 3D View from East Building Corner*

## III. Codes and Material Properties

### *Codes and Referenced Standards*

American Eagle Outfitters Quantum III uses the 2003 International Building Code (IBC) as amended by the City of Pittsburgh Building Department. The 2003 IBC references ASCE 7 – 02 and ACI 318-02. All analysis and design was performed by Atlantic Engineering Services using Allowable Stress Design (ASD) as opposed to Load and Resistance Factor Design (LRFD), which is used throughout this technical report. These design methods are prescribed in the AISC Steel Construction Manual, $13^{th}$ edition, as used for this report.

Codes used for this analysis are IBC 2006 without any Pittsburgh amendments, ASCE 7 – 05 and ACI 318 – 05.

### *Load Cases and Combinations*

Below are the load cases considered for Quantum III. Wind and seismic loads were applied in multiple directions to determine the most severe combination. Snow loads were not included in this analysis. Hand calculations for wind loads focused on the north and south elevations, the axis' where VT-A and VT-C act.

1. 1.4(D)
2. 1.2(D) + 1.6(L) + 0.5($L_r$)
3. 1.2(D) + 1.6($L_r$) + (0.5L or 0.8W)
4. 1.2(D) + 1.6(W) + 0.5(L) + 0.5($L_r$)
5. 1.2(D) + 1.0E + 0.5L
6. 0.9(D) + (1.6W or 1.0E)

### *Material Properties*

**Concrete**

| | |
|---|---|
| Foundations | 3000 psi |
| Terrace Walls | 4000 psi |
| Interior Slabs | 4000 psi |
| Exterior Slabs | 4000 psi |
| Site Access Canopy Walls | 5000 psi |
| Auger Pile Grout | 5000 psi |
| Reinforcing Steel (Yld) | 60 ksi |
| Headed Concrete Anchors (Yld) ASTM A108 Grades 1015-1020 | 60 ksi |

**Steel**

*Structural Steel*

| | | |
|---|---|---|
| W Shapes | ASTM A992 | 50 ksi |
| M, S, HP Shapes | ASTM A572 Grade 50 | 50 ksi |
| Channels | ASTM A572 Grade 50 | 50 ksi |
| Steel Tubes (HSS Shapes) | ASTM A500 Grade B | 46 ksi |
| Steel Pipes (Round HSS) | ASTM A500 Grade B | 42 ksi |
| Angles | ASTM A36 | 36 ksi |
| Plates | ASTM A36 | 36 ksi |

*Galvanized Structural Steel*

| | | |
|---|---|---|
| Structural Shapes and Rods | ASTM A123 | Zinc coating, Strength of base |
| Bolts, Fasteners, and Hardware | ASTM A153 | Zinc coating, Strength of base |
| Metal Decking (Yield Strength) | | 33 ksi |
| Light Gage Studs, 12-16 Gage | ASTM A653 Grade D | 50 ksi |
| Light Gage Studs, 18-20 Gage | ASTM A653 Grade A | 33 ksi |

**Masonry**

| | | |
|---|---|---|
| Mortar (Prism Strength) | ASTM C270 | F'm = 2500 psi |
| Grout | ASTM C476 | F'c = 3000 psi |
| Masonry (Prism Strength, 28-day) | | F'm = 1500 psi |

## V. Gravity Loads

### *Live Loads*

The typical bay for the roof has the same dimensions as that for the typical floor, so all reduced live loads are based on the bays and spacing outlined in III. Framing Plans and Elevations, Figure 2, page 6.

| Location | Load (psf) | Description |
|---|---|---|
| Roof | 20<br>**18** | $A_t$ = 10' x 30' = 300 $ft^2$<br>$\therefore R_1 = 1.2 - 0.001A_t = 1.2 - 0.001 * (300\ ft^2) = 0.9$<br>F = 0, the roof pitch is small enough to be negligible<br>$\therefore R_2 = 1$<br>$\therefore L_r = R_1 * R_2 * L = 0.9 \times 1.0 * 20 =$ **18 psf** |
| Offices and corridors above the first floor | 80<br>**54.6**<br>**48.3** | Offices require only 50 psf but since the building is designed to be flexible for tenant fit out, the location of corridors is not currently known, and the conservative corridor load is applied over the entire plan<br>$K_{LL}$ = 4 : Interior Beams<br>$A_{t,\ beam}$ = 300 $ft^2$<br>$A_{t,\ girder}$ = 15 ft x 30 ft = 450 $ft^2$<br>$L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) =$<br>$= 80 \times \left(0.25 + \frac{15}{(4 \times 300\ ft^2)^{0.5}}\right) =$ **54.6 psf**<br>$L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) =$<br>$= 80 \times \left(0.25 + \frac{15}{(4 \times 450\ ft^2)^{0.5}}\right) =$ **48.3 psf** |
| Lobbies and first floor corridors | 100 | Irreducible per ASCE 7-05 Section 4.8.2 |
| Stairs | 100 | |

### *Dead Loads*

Unit weights and dead loads are taken from the AISC Steel Manual, 13th Edition. Wall weights are supplied in the structural documents of American Eagle Outfitters: Quantum III. Mechanical unit surface loads described in Figure 10 below are based on an AES design method: distribute two-thirds of the unit weight over one-third the area and the reciprocal distribution of the remaining weight. Of the four distributed loads, the most severe combination is applied to the structure. This assumes most of weight is focused in one section of the mechanical unit and insures QIII is designed for the worst case scenario. The 'opening' refers to the opening for mechanical ducts. Finally, all supporting calculations are available on page 29 in Appendix A.

| Dead Loads | | | | |
|---|---|---|---|---|
| Component | | Typical Floor | Roof | Mechanical Roof |
| Concrete Slab | Topping | 24 | | 28.8 |
| | Deck | 21.6 | | 14.4 |
| Metal Decking | | 2.5 | 2 | 1.5 |
| Flooring/Ceiling | | 3 | 4 | 3 |
| M/E/P | | 7 | 10 | 7 |
| Rigid Insulation | | | 9 | |
| Membrane | | | 2 | |
| Total Dead Load | | 58.1 | 27 | 54.7 |

*Figure 9 – Dead Loads*

| Mechanical Unit Surface Loads | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 2/3 Weight Over 1/3 Area | | | | 1/3 Weight Over 2/3 Area | | | |
| | With Opening | | No Opening | | With Opening | | No Opening | |
| Total Weight (lb) | Area ($ft^2$) | Surface Load | Area ($ft^2$) | Surface Load | Area ($ft^2$) | Surface Load | Area ($ft^2$) | Surface Load |
| 40000 | 122.5 | 217.69 | 225 | 118.52 | 272.5 | 48.93 | 450 | 29.63 |

*Figure 10 – Mechanical Unit Dead Loads*

### Wall Loads

Curtain Walls………………………………...20 psf (specified in AEO:QIII General Notes)
8" CMU, grout/rein. 24" cc……………...…..51 psf
Partitions……………………………………..20 psf (specified in AEO:QIII General Notes)

## VI. Lateral Analysis

Lateral load resisting elements were analyzed on the basis of relative stiffness. RAM Structural System and SAP2000 were each used to analyze aspects of American Eagle Outfitters: Quantum III. The composite concrete slab distributes load to each of the vertical trusses. All floor slabs were considered rigid. The roof contains a composite slab where the mechanical units are placed, but is surrounded by noncomposite roof deck. It was assumed that the composite system is rigid, and all roof weights were attached to this diaphragm. A combination of computer drafting and modeling programs were used to analyze the lateral systems and lateral load distribution.

### *SAP2000 Models and Hand Calculations*

SAP2000 was used to model each of the vertical trusses and determine relative stiffness. A unit load was applied at the roof level of each truss, and the inverse of deflection at each floor was taken as the frame's stiffness. The resultant stiffnesses were then used to calculate the center of rigidity using Microsoft Excel spreadsheets. Methodology is covered in detail for wind and seismic loadings on pages 18 and 24 respectfully. In depth calculations are in Appendices B and C on pages 30 and 38 respectfully.

Combining the SAP model with hand calculations disregarded the effects of a semi rigid diaphragm at the roof level. The recessed composite deck on the roof has negligible effects on relative rigidity. The analysis also assumed the center of rigidity of *each frame* is at its midpoint for the *story* center of rigidity calculation.

### *RAM Structural System Calculations*

RAM was utilized to obtain more accurate story weights, centers of rigidity, and torsional effects resulting in a detailed lateral analysis. Wind calculations are less accurate because RAM cannot model lateral members supported by gravity members. Therefore, the windscreen and roof access stair cannot distribute wind loads to the building structure accurately. The fact that they are at the top of the building significantly alters overturning moment values. RAM seismic capabilities provide a more accurate analysis since the center of rigidity for frames and stories, torsional effects, and building weights are modeled precisely.

***Center of Mass and Rigidity Comparisons***

| Center of Area | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Area ($ft^2$) | | | X (ft) | | | Y (ft) | | |
| Story | Hand | RAM | Percent Difference | Hand | RAM | Percent Difference | Hand | RAM | Percent Difference |
| Roof | 28836.9 | 28080.9 | 2.69 % | 98.92 | 100.43 | 1.50 % | 87.48 | 90.09 | 2.90 % |
| 5 | 30289.6 | 29483.6 | 2.73 % | 97.20 | 94.97 | 2.35 % | 86.87 | 88.66 | 2.02 % |
| 4 | 30631.9 | 29825.9 | 2.70 % | 96.80 | 95.10 | 1.79 % | 86.68 | 88.53 | 2.09 % |
| 3 | 30631.9 | 29825.9 | 2.70 % | 96.80 | 95.13 | 1.76 % | 86.68 | 88.53 | 2.09 % |
| 2 | 30631.9 | 29825.9 | 2.70 % | 96.80 | 95.13 | 1.76 % | 86.68 | 88.50 | 2.06 % |

*Figure 11 – Center of Area Comparison*

| Center of Rigidity | | | | | | |
|---|---|---|---|---|---|---|
| | X Direction | | | Y Direction | | |
| Story | SAP | RAM | Percent Difference | SAP | RAM | Percent Difference |
| Roof | 124.69 | 118.45 | 5.27 % | 68.87 | 72.23 | 4.65 % |
| 5 | 127.50 | 117.28 | 8.71 % | 67.25 | 72.69 | 7.49 % |
| 4 | 124.03 | 115.43 | 7.45 % | 72.54 | 74.24 | 2.30 % |
| 3 | 126.58 | 111.69 | 13.33 % | 73.16 | 74.71 | 2.07 % |
| 2 | 105.76 | 101.94 | 3.75 % | 72.37 | 75.42 | 4.05 % |

*Figure 12 – Center of Rigidity Comparison*

***Wind Criteria***

A comparison of wind pressures acting on the main wind force resisting system is described below. Since the lateral frames VT-A and VT-C rigidities were compared, lateral forces are only analyzed for the North or South face of the building. Also, an expanded version of the wind spreadsheet and calculations is available on page 31.

| **Assumptions** | | **Differing Assumptions by AES** |
|---|---|---|
| Building Height (h) | 68.67’ | 72.33’ |
| Basic Wind Speed (3 second gust) | 90 | |
| Exposure Category | C | |
| Enclosure Classification | Enclosed | |
| Building Category | II | |
| Importance Factor | 1.0 | |
| Internal Pressure Coefficient | ±0.18 | |
| Wind Directionality Factor (Kzt) | 0.85 | |
| Topographic Factor (Kd) | 1.0 | |
| Gust Effect Factor (G) | 0.84, 0.89 | |

***Wind Analysis***

Hand-calculated and modeling program frame shears were compared for the Y-direction of one floor of American Eagle Outfitters: Quantum III. Wind pressures were calculated in Excel, and drafted on an elevation in AutoCAD. The area they acted on was then determined graphically and the product with pressures were summed into story forces and shears. Excel was again utilized to distribute story shear to frames and compare results to RAM output. Windscreens nor roof access stairs could be modeled in RAM because the software does not allow lateral members to be supported by gravity members. For this reason story forces determined by hand were input into RAM. The flowchart in Figure 13 details the methodology of calculations. The following page begins to detail the wind pressures acting on the elevations of QIII.

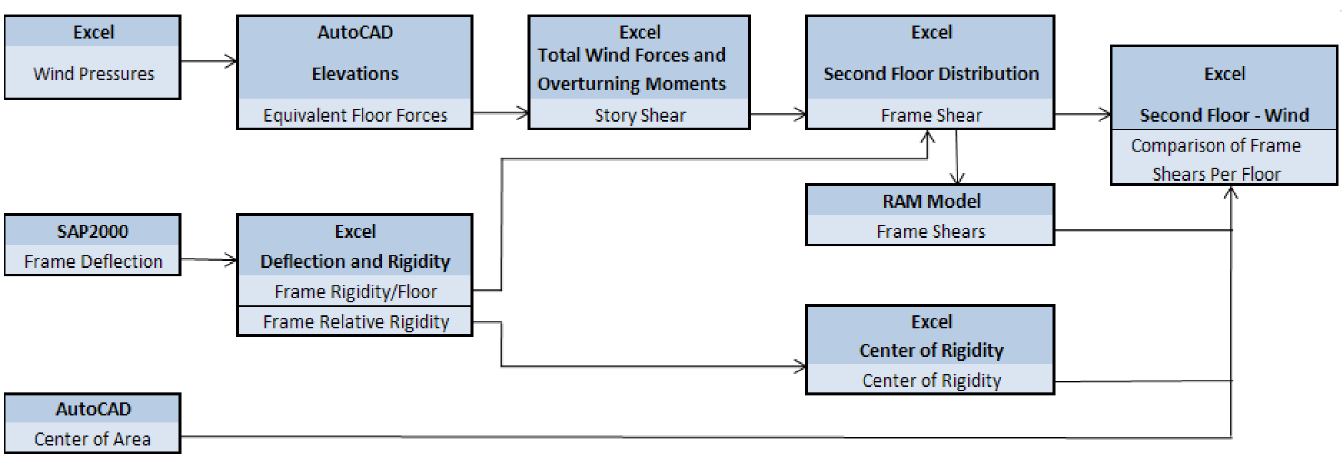


*Figure 13 – Calculation Methodology Flowchart*

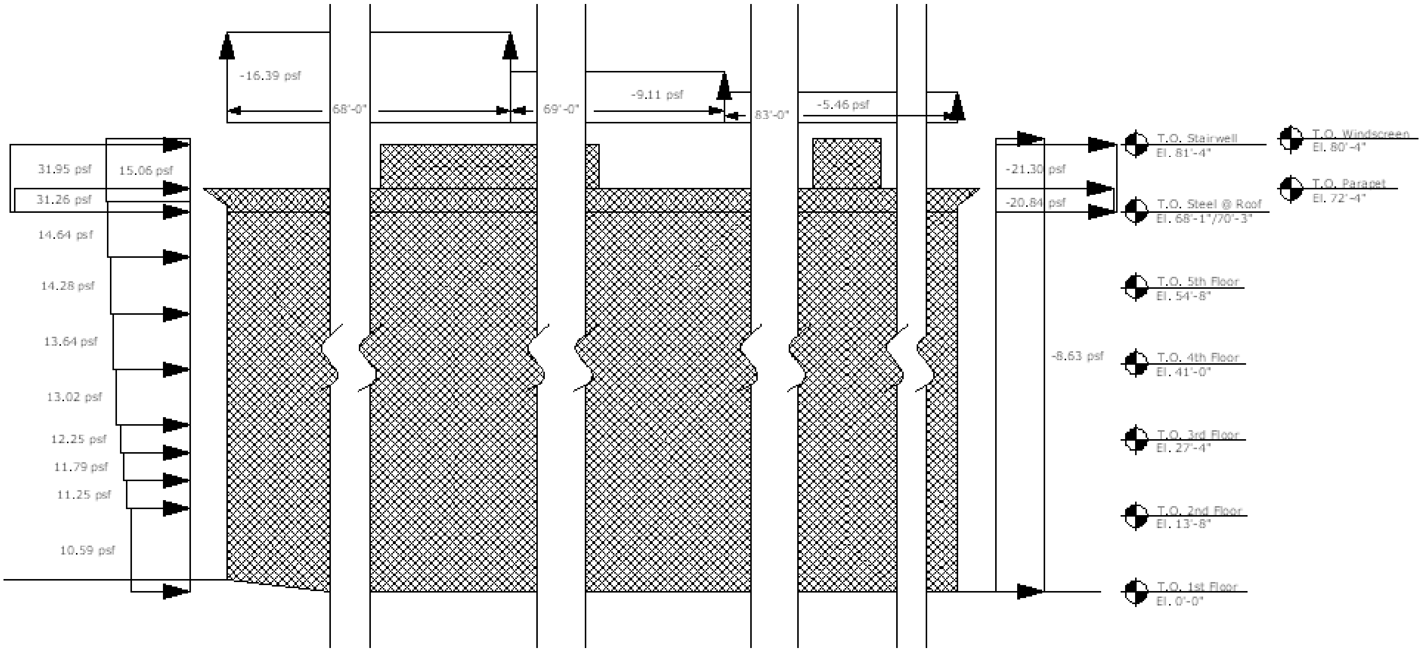


*Figure 14 – North Elevation: East-West Wind Pressures*

The wind pressure diagram above describes the magnitude of forces acting on each surface of American Eagle Outfitters: Quantum III. At the top of the building, three lateral pressures are shown overlapping. The largest magnitude pressure is that acting on the windscreen; this was modeled as a parapet since pressures can act on both sides of the structure. The smallest pressure, following the pattern of other gradually increasing ones up the elevation of the building is that acting on the roof access stair, shown as the right-most structure on the roof of the building. The last, slim and large magnitude force is that acting on the parapet. These can be seen on the East Elevation on the following page.

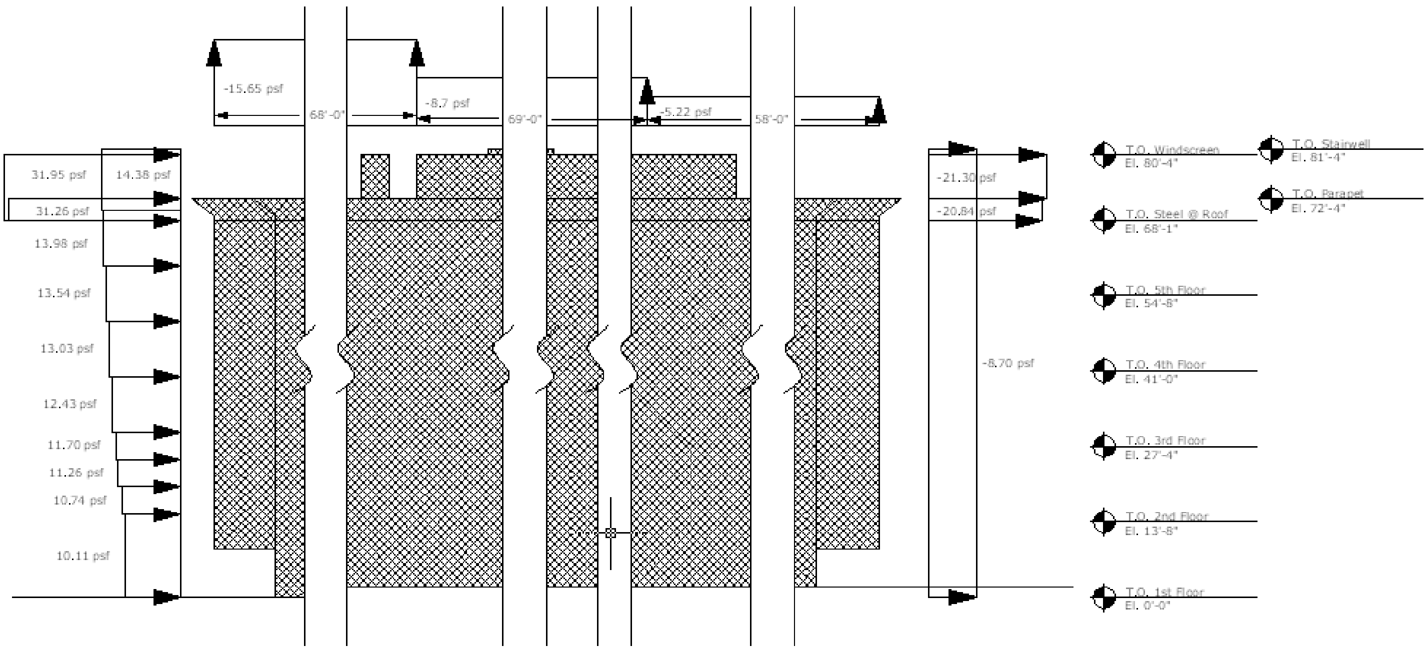


*Figure 15 – East Elevation: South-North Wind Pressures*

Below are the base shear, overturning moment results, and frame shear comparisons from my wind analysis. Since these are unfactored, and the load cases combining dead, live, wind and seismic give wind a 1.6 multiplier, wind will most definitely control the design of my vertical trusses. For determining these values, overturning moment was calculated from the equivalent forces of wind pressures acting on the north or south face of the building rather than the wind pressures themselves. Structures above the roof slab were assumed to transfer all wind load directly to the top floor lateral load. Again, spreadsheets on which these calculations were performed are in Appendix B, page 30.

| South East Elevation - SW to NE Wind | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Total Wind Forces and Overturning Moments | | | | | | | | | | | |
| Height Above Grade | | Wind Pressure (Windward) | Wind Pressure (Leeward) | Total Wind Pressure | Level | T.O.S. Height | Total Area per Level and Pressure | Force | Total Level Force F (k) | Total Story Shear (k) | Overturning Moment M (k-ft) |
| Min | Max | | | | | | | | | | |
| 0 | 6.84 | 10.11 | -8.70 | 18.81 | 1 | 0 | 1480 | 27.83 | 27.83 | **384.31** | 0.0 |
| 6.84 | 15 | 10.11 | -8.70 | 18.81 | 2 | 13.67 | 1795 | 33.76 | 57.65 | 326.67 | 788.0 |
| 15 | 20 | 10.74 | -8.70 | 19.44 | 2 | | 1100 | 21.38 | | | |
| 20 | 20.5 | 10.74 | -8.70 | 19.44 | 2 | | 129 | 2.51 | | | |
| 20.5 | 25 | 10.74 | -8.70 | 19.44 | 3 | 27.34 | 971 | 18.87 | 60.66 | 266.01 | 1658.4 |
| 25 | 30 | 11.70 | -8.70 | 20.39 | 3 | | 1100 | 22.43 | | | |
| 30 | 34.2 | 12.43 | -8.70 | 21.13 | 3 | | 916 | 19.35 | | | |
| 34.2 | 40 | 12.43 | -8.70 | 21.13 | 4 | 41 | 1264 | 26.70 | 72.06 | 193.95 | 2954.5 |
| 40 | 47.8 | 13.03 | -8.70 | 21.72 | 4 | | 1612 | 35.02 | | | |
| 47.8 | 50 | 13.03 | -8.70 | 21.72 | 4 | | 476 | 10.34 | | | |
| 50 | 60 | 13.54 | -8.70 | 22.23 | 5 | 54.68 | 2198 | 48.87 | 90.42 | 103.53 | 4944.0 |
| 60 | 61.5 | 13.98 | -8.70 | 22.68 | 5 | | 316 | 7.17 | | | |
| 61.5 | 68.7 | 13.98 | -8.70 | 22.68 | 5 | | 1516 | 34.38 | | | |
| 68.7 | 70 | 31.26 | -20.84 | 52.09 | Parapet | 70.08 | 368 | 19.17 | 46.52 | | 3260.1 |
| 70 | 72.3 | 31.26 | -20.84 | 52.09 | Parapet | | 525 | 27.35 | | | |
| 68.7 | 70 | 31.95 | -21.30 | 53.26 | Windscreen | 74.21 | 280 | 14.91 | 53.41 | | 3963.8 |
| 70 | 80 | 31.95 | -21.30 | 53.26 | Windscreen | | 723 | 38.50 | | | |
| 68.7 | 70 | 13.98 | -8.70 | 22.68 | Stair | 74.71 | 20 | 0.45 | 3.60 | | 268.8 |
| 70 | 80 | 14.38 | -8.70 | 23.08 | Stair | | 120 | 2.77 | | | |
| 80 | 81.7 | 14.74 | -8.70 | 23.44 | Stair | | 16 | 0.38 | | | |
| | | | | | | | | Totals | 384.31 | | 17837.6 |

*Figure 16 – Total Wind Forces and Overturning Moments*

**Second Floor - Wind**

| Axis | Center of Area | Center of Rigidity | Eccentricity (ft) |
|---|---|---|---|
| x | 96.8 | 105.76 | 8.96 |
| y | 86.68 | 77.63 | 9.05 |

| Story Shear | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Location | | Distance from | | Relative | | Rx*y2 | Ry*x2 | Direct Shear | Torsional Shear | Hn (kips) | RAM Load | RAM Hn | Percent Difference |
| | x (ft) | y (ft) | x (ft) | y (ft) | Rx | Ry | | | | | | | | |
| VT-A | 195 | 137 | 98.20 | 50.32 | 0 | 0.405 | 0.0 | 3905.5 | 132.3 | 6.729 | 139.05 | | 154.53 | 10.016 % |
| VT-C | 45 | 77 | -51.80 | -9.68 | 0 | 0.595 | 0.0 | 1596.5 | 194.3 | 7.660 | 202.01 | | 237.38 | 14.901 % |
| Totals | | | | | 0 | 1 | | 5502.0 | 326.7 | 14.389 | 341.06 | | 391.91 | **12.975 %** |

*Figure 17 - Frame Shears on Second Floor - Wind*

RAM frame shears and those determined from the methodologies described previously are believably different. A likely culprit is the varying effect of the flexible metal decking on VT-A, B, C, and E and the rigid composite slab on VT-D at the roof level. In RAM, all loads are filtered to the rigid diaphragm, and all lateral load resisting elements are theoretically connected to this diaphragm (if not physically). This could change the torsional load distribution throughout each story. As a result, the minute five percent difference in the second floor center of rigidity may be magnified to significantly alter frame shear. To further illustrate the vertical truss connections to the diaphragm, see Figure 18**.** The gray shaded portion illustrates location of composite slab. Another explanation for the discrepancies in wind shears could be from the windscreen model. Atlantic Engineering Services has modeled windscreens to be semi-permeable, allowing a certain percentage of the wind pressure pass through. In this respect, my model is conservative by assuming all pressure acting on the windscreen is distributed to the lateral members. Detailed calculations for wind shears are available in Appendix B, page 30.

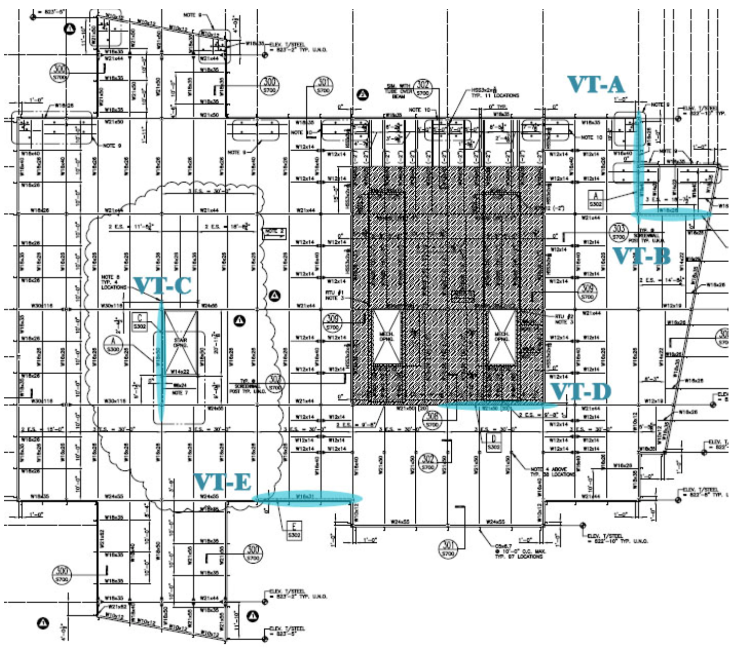


*Figure 18 - Roof Level Truss and Composite Slab Locations*

**Total Wind Base Shear** **(1.6)391.91 kips = 627.06 kip-ft**
**Total Overturning Moment** **(1.6)17837.6 kip-ft = 28540 kip-ft**

**CONTROLLING**

***Seismic Criteria***

Atlantic Engineering Services determined a Seismic Design Category of A for American Eagle Outfitters Quantum III, requiring equivalent lateral forces, $F_x$, to equal one percent of the total dead load assigned to or located at Level x. They arrived at this conclusion by obtaining different mapped spectral response accelerations of $S_S$ = 0.131 g and $S_1$ = 0.058 g**.** This carried throughout the entire seismic calculation, resulting in $S_{DS}$ = 0.1 g and $S_{D1}$ = 0.06 g—values small enough to qualify for a seismic design category of A. This can be attributed to differing latitude and longitude measurements. In this analysis, Google Earth was used to compute the latitude and longitude of QIII, which resulted in a seismic design category of B. The vertical truss analysis uses category B, and supporting calculations are on page 38.

Occupancy Category II
Seismic Use Group II
Importance Factor (I) 1.0
Latitude and Longitude……………………..40°25'32.71" N 79° 57'50.93" W
Mapped Spectral Response Accelerations
$S_s$ = 0.125 g
$S_1$ = 0.049 g
Site Class…………………………………....D
Site Class Factors
$F_a$ = 1.60
$F_v$ = 2.40
$S_{MS}$ ………………………………………… 0.20
$S_{M1}$ ………………………………………… 0.1176
$S_{DS}$ ………………………………………… 0.133
$S_{D1}$ ………………………………………… 0.0784
Seismic Design Category………………….. B
Braced Frames are a "Steel System Not Specifically Detailed for Seismic Resistance"
Response Modification Factor (R) ................3.0
Over-strength Factor ($W_o$) ............................3.0
Deflection Amplification Factor ($C_d$) ...........3.0
Seismic Response Coefficient ($C_t$) ................0.02
Period Coefficient ..........................................0.75
Seismic Coefficient (Cs) ................................0.0284
Building Period (T) ........................................0.921
k.....................................................................1.211

***Seismic Analysis***

Hand-calculated and modeling program frame shears were compared for one floor of American Eagle Outfitters: Quantum III. RAM Structural System generated floor weights and the building period (T) was compared to a hand calculated value. SAP2000 found frame deflections, which were used in Excel spreadsheets to find relative rigidities. Finally, each floor of the building was drafted in AutoCAD to determine the center of area. The flowchart below details the methodology of calculations. Spreadsheet names and programs are displayed in bold; the calculated value is in normal font below.

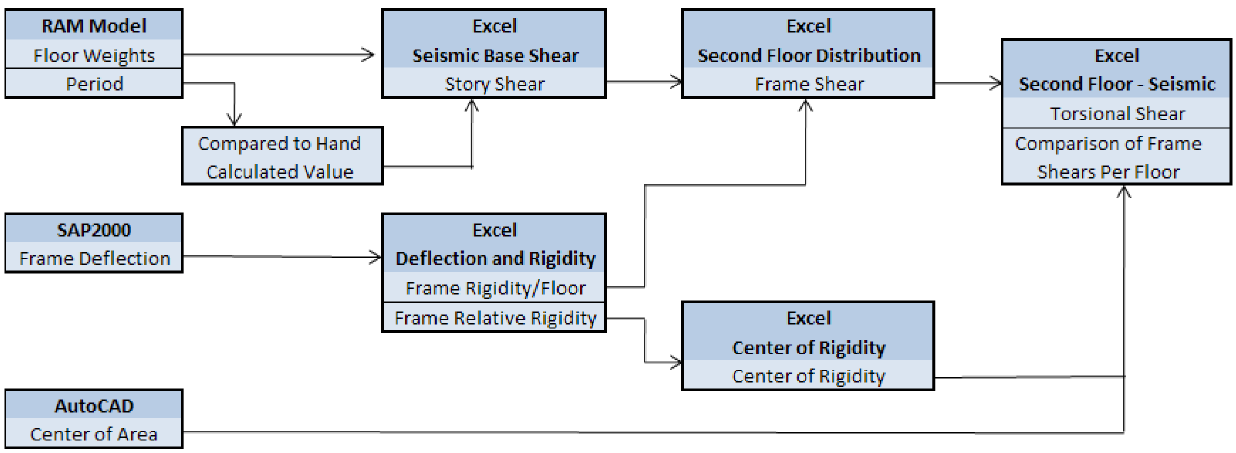


*Figure 19 – Seismic Calculation Methodology Flowchart*

RAM Structural System floor weights were more accurate than hand calculated values because the latter did not include exact steel section lengths, weights, or areas. The period RAM calculated was equivalent to that found manually (T=0.921s). Results and a comparison of frame shears on the second story are on the following page. Notice that both directions of seismic shear are considered, so the total Hn for both the manual and RAM methods are doubled.

| Second Floor - Seismic | | | |
|---|---|---|---|
| **Axis** | **Center of Area** | **Center of Rigidity** | **Eccentricity (ft)** |
| x | 96.8 | 105.76 | 8.96 |
| y | 86.68 | 77.63 | 9.05 |

| Story Shear | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Element** | **Location** | | **Distance from Center of Rigidity** | | **Relative Rigidity** | | **Rx*y2** | **Ry*x2** | **Direct Shear (kips)** | **Torsional Shear (kips)** | **Hn (kips)** | **RAM Load Case** | **RAM Hn (kips)** | **Percent Difference** | |
| | **x (ft)** | **y (ft)** | **x (ft)** | **y (ft)** | **Rx** | **Ry** | | | | | | | | | |
| VT-A | 195 | 137 | 98.20 | 50.32 | 0 | 0.405 | 0.0 | 3905.5 | 144.4 | 7.343 | 151.73 | E3 | 168.04 | 9.705 | % |
| VT-B | 206 | 122 | 109.24 | 35.32 | 0.361 | 0 | 450.3 | 0.0 | 128.6 | 2.076 | 130.65 | E2 | 142.27 | 8.167 | % |
| VT-C | 45 | 77 | -51.80 | -9.68 | 0 | 0.595 | 0.0 | 1596.5 | 212.1 | 8.358 | 220.43 | E3 | 266.73 | 17.358 | % |
| VT-D | 150 | 62 | 53.20 | -24.68 | 0.330 | 0 | 201.0 | 0.0 | 117.7 | 1.213 | 118.89 | E2 | 145.09 | 18.056 | % |
| VT-E | 90 | 32 | -6.80 | -54.68 | 0.309 | 0 | 923.9 | 0.0 | 110.2 | 2.357 | 112.56 | E2 | 136.52 | 17.547 | % |
| Totals | | | | | 1 | 1 | | 7077.3 | 712.9 | 21.348 | 734.27 | | 858.65 | **14.486** | **%** |

*Figure 20 – Frame Shears on Second Floor - Seismic*

Once again, RAM frame shears and those determined from the methodologies described previously are understandably different. The likely culprit is the same as that for the wind analysis—inconsistencies with the rigidity of the roof deck. Take note the difference for both the wind and seismic shears have similar error. This suggests the factor that increased the RAM results for both wind and seismic may be related. To further illustrate the vertical truss connections to the diaphragm, see Figure 22 in Wind Analysis. In depth calculations are on page 38.

**Total Seismic Base Shear** **380.21 kips**
**Total Overturning Moment** **18434.38 kip-ft**

## VII. Member Stresses

Since wind was controlling, story shears were input into RAM Structural System to determine member forces in VT-A and VT-C. Several braces failed as shown in Figure XXXX. This is due to the inadequacy of RAM for modeling composite lateral framing members. The majority of members passed though, indicating this minutely affected member loads.

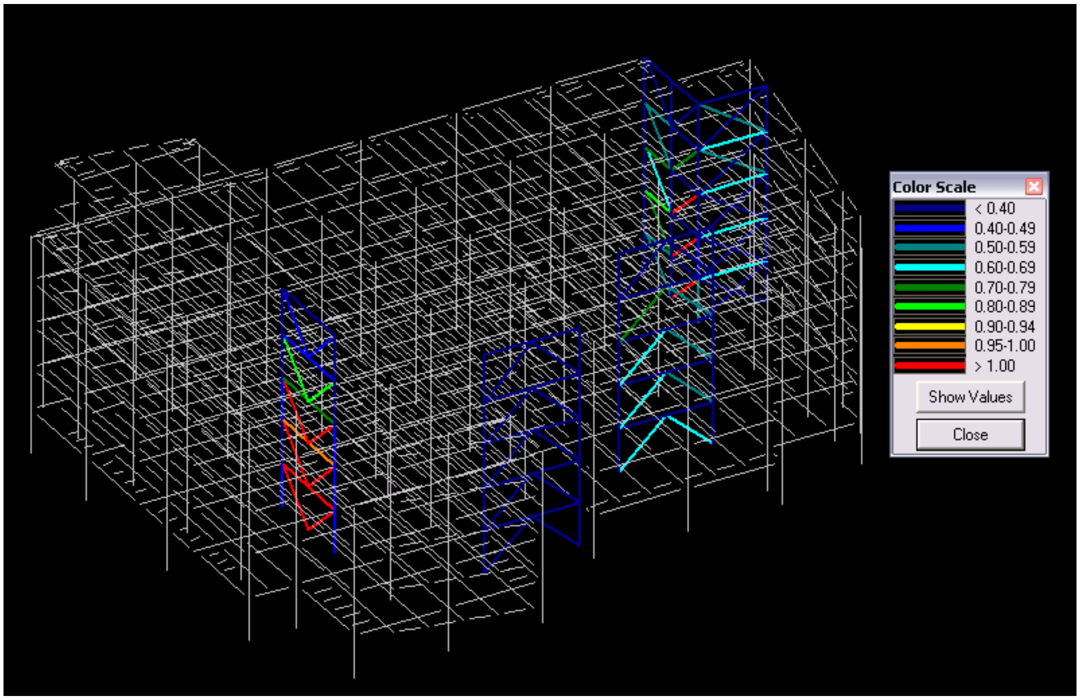


*Figure 21 - RAM Frame Member Stresses*

## VIII. Story Drift

Using RAM Structural System, the adequacy of the rigidity of braced frames for both wind and seismic shears were analyzed. Atlantic Engineering Services found a seismic design category of A while my calculations suggest B. Keep in mind seismic drift is conservative in this respect. The figures below outline story drift results.

| Wind Drift | | | | | |
|---|---|---|---|---|---|
| Level | Height | Actual Displacement RAM | Allowable Displacement L/400 | % | OK/NG |
| Roof | 68.670 | 0.543 | 2.060 | 26.4 | OK |
| 5th | 54.542 | 0.459 | 1.636 | 28.0 | OK |
| 4th | 40.875 | 0.357 | 1.226 | 29.1 | OK |
| 3rd | 27.208 | 0.243 | 0.816 | 29.8 | OK |
| 2nd | 13.542 | 0.114 | 0.406 | 28.1 | OK |

*Figure 22 - Wind Drift*

| Seismic Drift | | | | | |
|---|---|---|---|---|---|
| Level | Height | Actual Displacement RAM | Allowable Displacement 0.015*hx | % | OK/NG |
| Roof | 68.670 | 0.996 | 1.030 | 96.7 | OK |
| 5th | 54.542 | 0.855 | 0.818 | 104.5 | NG |
| 4th | 40.875 | 0.657 | 0.613 | 107.1 | NG |
| 3rd | 27.208 | 0.433 | 0.408 | 106.2 | NG |
| 2nd | 13.542 | 0.192 | 0.203 | 94.6 | OK |

*Figure 23 - Seismic Drift*

## IX. Conclusions

The lateral load resisting elements of American Eagle Outfitters: Quantum III were analyzed using a combination of computer models and hand calculations. Frame relative rigidities were determined using SAP2000 to model individual frames. Center of area and rigidity calculations combined Microsoft Excel spreadsheets with AutoCAD plans and elevations. The majority of the building was modeled in RAM to obtain comparison values.

The analysis methods used in this technical report demonstrated the significance of using RAM and other structural modeling programs as a "black box". It is vital that the engineer knows the significance of the data input to this software. Knowledge of the short comings of computer software was a key factor in this report. For instance, RAM does not allow the user to input lateral load resisting elements supported by gravity structures. These severely limit the program from analyzing windscreens or roof access stairs. As with QIII, the wind on the screen could significantly impact overturning moment, or the design of their supporting beams. The roof access stair contained all moment connections, with two roof-level columns tying into gravity beams—all of which could not be modeled. As a result, story shears as a result of the windscreen and stair were added as a "user-defined load case". In the field, these elements are analyzed separately then their reactions are input into RAM.

Some hand calculations did not coincide with computer modeled results. First, all story shears have an error of between 6-15 percent. It is here we discover another limitation to RAM. AEO has four levels of composite braced frame beams in VT-E. Composite action is not taken into account for RAM frame members. Not only do these cause imperfections in the center of rigidity calculation, but it affects the distribution of shears to each frame. Torsional affects can be amplified resulting in minute differences in everything from member loads to story drift. Second, this is demonstrated in the seismic story drifts of three levels to be unacceptable. Increased rigidity due to composite action would counteract this. Last, RAM cannot accurately model story drift as a result of wind analysis if windscreens significantly influence lateral systems.

Overall, this report has opened my eyes to the world of computer modeling. Not only must the program of choice contain options for all types of structural design, but must provide the user with a friendly interface where these options can be easily implemented. Debugging programs require a lengthy amount of time and knowledge of the results of each error. Finally, I am content with my knowledge of lateral load resisting elements and methods of analysis to make an informed decision on a structural system of my thesis proposal.

## Appendix A. Gravity Loads

### *Dead Loads*

#### 5½“ Composite Steel

2½“ LW Concrete Topping Slab = $\frac{115\text{ lb}}{\text{ft}^3} \times \frac{2.5\text{ in}}{12\text{ inches/ft}} =$ 24 psf + 2.5 psf deck

3” LW Composite Slab = 75% $\times \frac{115\text{ lb}}{\text{ft}^3} \times \frac{3\text{ in}}{12\text{ inches/ft}} =$ 21.6 psf

#### 5” Composite Steel

3" LW Concrete Composite Slab = $\frac{115\text{ lb}}{\text{ft}^3} \times \frac{3\text{ in}}{12\text{ inches/ft}} =$ 28.8 psf + 1.5 psf deck

2” LW Composite Slab = 75% $\times \frac{115\text{ lb}}{\text{ft}^3} \times \frac{2\text{ in}}{12\text{ inches/ft}} =$ 14.4 psf

#### 4” Noncomposite Steel

From United Steel Deck, Inc. Design Manual:

1½” 22-Gage Non-Composite Deck with 2.5” Topping = 29 psf
Reference available upon request

#### Roof System

6" Rigid Insulation = $\frac{1.5\text{ lb}}{\text{in-ft}^2} \times$ 6 in = 9 psf

Roof Deck and Insulation = 2 psf + 9 psf = 11 psf + 2 psf misc

#### Wall Systems

Curtain Walls = 20 psf x 13.67 ft = 275 plf
Partitions = 20 psf x 13.67 ft = 275 plf
8" Concrete Masonry Wall = 51 psf : based on 125 pcf unit with grout at 24” on center

## Appendix B. Wind Loads

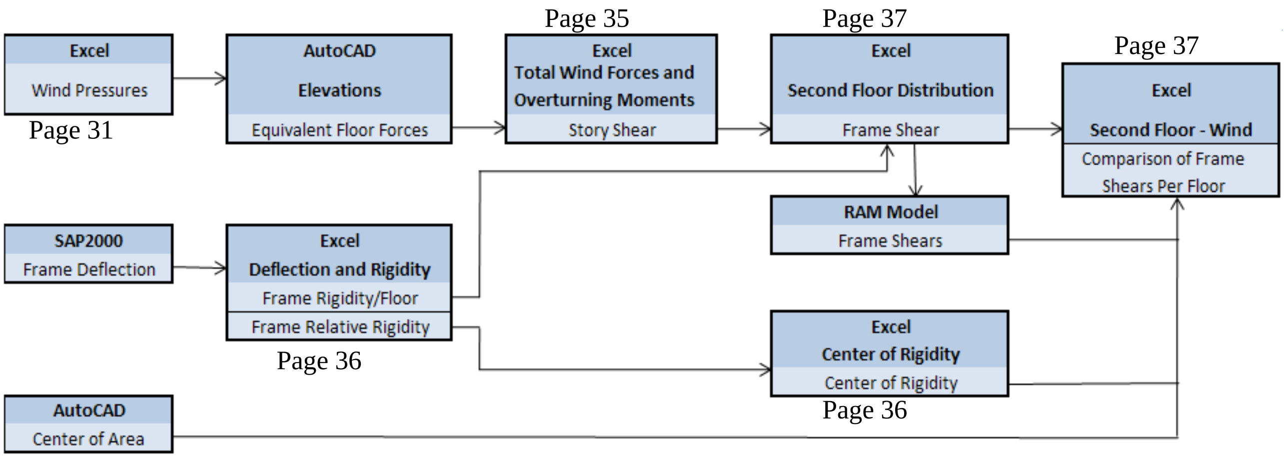


*Figure 24 – Wind Load Appendix Map*

***Wind Pressure Spreadsheets:***

| Locality Input | | | |
|---|---|---|---|
| Basic Wind Speed | V = | 90 | mph |
| Wind Directionality Factor | $K_d$ = | 0.85 | |
| Exposure | (B, C, or D) | C | |
| | | | |
| Enclosure | (E, PE, O) | E | |
| Building Category | | II | |
| Importance Factor | I = | 1 | |
| Mean Roof Height | h = | 68.33 | feet |
| Parapet Height | | 4 | feet |
| L (plan north-south) | | 194.33 | feet |
| L (plan east-west) | | 219.83 | feet |
| Rigid Structure? | Y/N | Y | |
| Roof Angle | $\theta$ = | 0 | |
| Topographic Factor | Kzt = | 1 | |

| Table 6-2 | | |
|---|---|---|
| Exposure | C | |
| $\alpha$ | 9.5 | |
| $z_g$ | 900 | feet |
| $\hat{a}$ | 0.11 | |
| $\hat{b}$ | 1 | |
| $\bar{\alpha}$ | 0.15 | |
| $\bar{b}$ | 0.65 | |
| c | 0.2 | |
| l | 500 | |
| $\bar{\epsilon}$ | 0.2 | |
| $z_{min}$ | 15 | feet |

| Gust Effect Factor | |
|---|---|
| $\bar{z}$ | 41.00 |
| $L_{\bar{z}}$ | 522 |
| Q (plan north-south) | 0.84 |
| Q (plan east-west) | 0.92 |
| $I_{\bar{z}}$ | 0.19 |
| $g_Q$ | 3.4 |
| $g_v$ | 3.4 |
| G (plan north-south) | 0.84 |
| G (plan east-west) | 0.88 |

| Internal Pressure Coefficients | |
|---|---|
| + GCpi | - GCpi |
| 0.18 | -0.18 |

***Wind Pressure Spreadsheets:***

| Wall Pressure Coefficients | | | | | |
|---|---|---|---|---|---|
| Surface | L/B | Cp | | Actual L/B | Cp |
| Windward Wall | All Values | 0.8 | | | 0.80 |
| Leeward Wall | 0-1 | -0.5 | North-South | 0.88 | -0.50 |
| | 2 | -0.3 | | | |
| | >=4 | -0.2 | | | |
| | 0-1 | -0.5 | East-West | 1.13 | -0.47 |
| | 2 | -0.3 | | | |
| | >=4 | -0.2 | | | |
| Side Wall | All Values | -0.7 | | | -0.70 |

| Roof Pressure Coefficients | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Wind Direction | h/L | Horizontal Distance from Windward Edge | Cp | Actual h/L | Actual Horizontal Distance (feet) | | Interpolate Between Cp | |
| North to South | <= 0.5 | 0 to h/2 | - 0.9, - 0.18 | 0.35 | <= | 34 | -0.90 | -0.18 |
| | | h/2 to h | - 0.9, - 0.18 | | 34 | 68 | -0.90 | -0.18 |
| | | h to 2h | - 0.5, - 0.18 | | 68 | 137 | -0.50 | -0.18 |
| | | > 2h | - 0.3, - 0.18 | | > | 137 | -0.30 | -0.18 |
| East to West | <= 0.5 | 0 to h/2 | - 0.9, - 0.18 | 0.31 | <= | 34 | -0.90 | -0.18 |
| | | h/2 to h | - 0.9, - 0.18 | | 34 | 68 | -0.90 | -0.18 |
| | | h to 2h | - 0.5, - 0.18 | | 68 | 137 | -0.50 | -0.18 |
| | | > 2h | - 0.3, - 0.18 | | > | 137 | -0.30 | -0.18 |

| Table 6-3 | | | | | |
|---|---|---|---|---|---|
| Height Above Ground Level, z | Exposure C Case 1 & 2 | $K_z$ | $K_h$ | $q_h$ | $q_z$ |
| 0-15 | 0.85 | 0.85 | 1.17 | 20.59 | 14.96 |
| 20 | 0.90 | 0.90 | 1.17 | 20.59 | 15.90 |
| 25 | 0.94 | 0.95 | 1.17 | 20.59 | 16.66 |
| 30 | 0.98 | 0.98 | 1.17 | 20.59 | 17.31 |
| 40 | 1.04 | 1.04 | 1.17 | 20.59 | 18.39 |
| 50 | 1.09 | 1.09 | 1.17 | 20.59 | 19.28 |
| 60 | 1.13 | 1.14 | 1.17 | 20.59 | 20.03 |
| 70 | 1.17 | 1.17 | 1.17 | 20.59 | 20.69 |
| 80 | 1.21 | 1.21 | 1.17 | 20.59 | 21.28 |
| 90 | 1.24 | 1.24 | 1.17 | 20.59 | 21.82 |
| 100 | 1.26 | 1.27 | 1.17 | 20.59 | 22.31 |

***Wind Pressure Spreadsheets:***

MWFRS Design Pressures

| Walls | Wind Direction | Height (feet) | Pressures ($lb/ft^2$) | | | |
|---|---|---|---|---|---|---|
| Leeward | North/South | | P = | -8.70 | ± | 3.71 |
| | East/West | | P = | -8.63 | ± | 3.71 |
| Side | | | P = | -12.75 | ± | 3.71 |
| Windward | North-South | 0-15 | P = | 10.11 | ± | 3.71 |
| | | 20 | P = | 10.74 | ± | 3.71 |
| | | 25 | P = | 11.26 | ± | 3.71 |
| | | 30 | P = | 11.70 | ± | 3.71 |
| | | 40 | P = | 12.43 | ± | 3.71 |
| | | 50 | P = | 13.03 | ± | 3.71 |
| | | 60 | P = | 13.54 | ± | 3.71 |
| | | 70 | P = | 13.98 | ± | 3.71 |
| | | 80 | P = | 14.38 | ± | 3.71 |
| | | 90 | P = | 14.74 | ± | 3.71 |
| | | 100 | P = | 15.07 | ± | 3.71 |
| | East-West | 0-15 | P = | 10.59 | ± | 3.71 |
| | | 20 | P = | 11.25 | ± | 3.71 |
| | | 25 | P = | 11.79 | ± | 3.71 |
| | | 30 | P = | 12.25 | ± | 3.71 |
| | | 40 | P = | 13.02 | ± | 3.71 |
| | | 50 | P = | 13.64 | ± | 3.71 |
| | | 60 | P = | 14.18 | ± | 3.71 |
| | | 70 | P = | 14.64 | ± | 3.71 |
| | | 80 | P = | 15.06 | ± | 3.71 |
| | | 90 | P = | 15.44 | ± | 3.71 |
| | | 100 | P = | 15.78 | ± | 3.71 |

***Wind Pressure Spreadsheets:***

| MWFRS Design Pressures | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Roof | | | | | | | | | | |
| | Wind Direction | Distance From Windward Wall (feet) | Pressures (lb/ft$^2$) | | | | | | | |
| Windward | North-South | 0 to 34 | P = | -15.65 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | 34 to 68 | P = | -15.65 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | 68 to 137 | P = | -8.70 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | over 137 | P = | -5.22 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | East-West | 0 to 34 | P = | -16.39 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | 34 to 68 | P = | -16.39 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | 68 to 137 | P = | -9.11 | ± | 3.71 | or | -0.67 | ± | 3.71 |
| | | over 137 | P = | -5.46 | ± | 3.71 | or | -0.67 | ± | 3.71 |

| Parapet | | | | | | | |
|---|---|---|---|---|---|---|---|
| | $GC_{pn}$ | $K_p$ | $q_p$ | Pressures (lb/ft2) | | | |
| Windward | 1.5 | 1.18 | 20.84 | P = | 31.26 | ± | 3.71 |
| Leeward | -1 | 1.18 | 20.84 | P = | -20.84 | ± | 3.71 |

| Windscreen | | | | | | | |
|---|---|---|---|---|---|---|---|
| height = | 12 | feet | | | | | |
| | $GC_{pn}$ | $K_w$ | $q_w$ | Pressures (lb/ft2) | | | |
| Windward | 1.5 | 1.21 | 21.30 | P = | 31.95 | ± | 3.71 |
| Leeward | -1 | 1.21 | 21.30 | P = | -21.30 | ± | 3.71 |

# Technical Report III

| South East Elevation - SW to NE Wind | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Total Wind Forces and Overturning Moments** | | | | | | | | | | | |
| **Height Above Grade** | | **Wind Pressure (Windward)** | **Wind Pressure (Leeward)** | **Total Wind Pressure** | **Level** | **T.O.S. Height** | **Total Area per Level and Pressure** | **Force** | **Total Level Force F (k)** | **Total Story Shear (k)** | **Overturning Moment M (k-ft)** |
| Min | Max | | | | | | | | | | |
| 0 | 6.84 | 10.11 | -8.70 | 18.81 | 1 | 0 | 1480 | 27.83 | 27.83 | **384.31** | 0.0 |
| 6.84 | 15 | 10.11 | -8.70 | 18.81 | 2 | 13.67 | 1795 | 33.76 | 57.65 | 326.67 | 788.0 |
| 15 | 20 | 10.74 | -8.70 | 19.44 | 2 | | 1100 | 21.38 | | | |
| 20 | 20.5 | 10.74 | -8.70 | 19.44 | 2 | | 129 | 2.51 | | | |
| 20.5 | 25 | 10.74 | -8.70 | 19.44 | 3 | 27.34 | 971 | 18.87 | 60.66 | 266.01 | 1658.4 |
| 25 | 30 | 11.70 | -8.70 | 20.39 | 3 | | 1100 | 22.43 | | | |
| 30 | 34.2 | 12.43 | -8.70 | 21.13 | 3 | | 916 | 19.35 | | | |
| 34.2 | 40 | 12.43 | -8.70 | 21.13 | 4 | 41 | 1264 | 26.70 | 72.06 | 193.95 | 2954.5 |
| 40 | 47.8 | 13.03 | -8.70 | 21.72 | 4 | | 1612 | 35.02 | | | |
| 47.8 | 50 | 13.03 | -8.70 | 21.72 | 4 | | 476 | 10.34 | | | |
| 50 | 60 | 13.54 | -8.70 | 22.23 | 5 | 54.68 | 2198 | 48.87 | 90.42 | 103.53 | 4944.0 |
| 60 | 61.5 | 13.98 | -8.70 | 22.68 | 5 | | 316 | 7.17 | | | |
| 61.5 | 68.7 | 13.98 | -8.70 | 22.68 | 5 | | 1516 | 34.38 | | | |
| 68.7 | 70 | 31.26 | -20.84 | 52.09 | Parapet | 70.08 | 368 | 19.17 | 46.52 | | 3260.1 |
| 70 | 72.3 | 31.26 | -20.84 | 52.09 | Parapet | | 525 | 27.35 | | | |
| 68.7 | 70 | 31.95 | -21.30 | 53.26 | Windscreen | 74.21 | 280 | 14.91 | 53.41 | | 3963.8 |
| 70 | 80 | 31.95 | -21.30 | 53.26 | Windscreen | | 723 | 38.50 | | | |
| 68.7 | 70 | 13.98 | -8.70 | 22.68 | Stair | 74.71 | 20 | 0.45 | 3.60 | | 268.8 |
| 70 | 80 | 14.38 | -8.70 | 23.08 | Stair | | 120 | 2.77 | | | |
| 80 | 81.7 | 14.74 | -8.70 | 23.44 | Stair | | 16 | 0.38 | | | |
| | | | | | | | | **Totals** | **384.31** | | **17837.6** |

*Figure 26 – Total Wind Forces and Overturning Moments*

| Braced Frame Rigidities | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | | Deflection and Rigidity | | | | | | | | | | | | | | | | | | | |
| | | | Frame A Strong Axis-Y | | | | Frame B Strong Axis-X | | | | Frame C Strong Axis-Y | | | | Frame D Strong Axis-X | | | | Frame E Strong Axis-X | | | |
| Story | Total (ft) | Per Floor (ft) | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness |
| Roof | 69.1 | 14.6 | 0.222 | 0.018 | 55.56 | 0.531 | 0.226 | 0.022 | 46.30 | 0.314 | 0.232 | 0.02 | 49.02 | 0.469 | 0.238 | 0.019 | 52.08 | 0.353 | 0.248 | 0.02 | 49.02 | 0.333 |
| 5 | 54.6 | 13.7 | 0.204 | 0.032 | 30.86 | 0.550 | 0.204 | 0.04 | 25.25 | 0.294 | 0.211 | 0.04 | 25.25 | 0.450 | 0.218 | 0.032 | 30.86 | 0.359 | 0.228 | 0.034 | 29.76 | 0.347 |
| 4 | 40.9 | 13.7 | 0.172 | 0.053 | 18.94 | 0.527 | 0.164 | 0.043 | 23.15 | 0.366 | 0.172 | 0.059 | 17.01 | 0.473 | 0.186 | 0.049 | 20.33 | 0.321 | 0.194 | 0.05 | 19.84 | 0.313 |
| 3 | 27.2 | 13.7 | 0.119 | 0.062 | 16.03 | 0.544 | 0.121 | 0.056 | 17.73 | 0.373 | 0.113 | 0.074 | 13.44 | 0.456 | 0.137 | 0.066 | 15.15 | 0.319 | 0.144 | 0.068 | 14.62 | 0.308 |
| 2 | 13.5 | 13.5 | 0.056 | 0.056 | 17.73 | 0.405 | 0.065 | 0.065 | 15.43 | 0.361 | 0.038 | 0.038 | 26.04 | 0.595 | 0.071 | 0.071 | 14.12 | 0.330 | 0.076 | 0.076 | 13.23 | 0.309 |

Figure 27 - Deflection and Rigidity

| Center of Rigidity | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | X Direction | | | | | | | | | Y Direction | | | | | | | | | | | |
| | Frame A | | | Frame C | | | Center of Rigidity | | | Frame B | | | Frame D | | | Frame E | | | Center of Rigidity | | |
| Story | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Total R Per Floor | Total R*x Per Floor | Location (ft) | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Total R Per Floor | Total R*x Per Floor | Location (ft) |
| Roof | 55.56 | 195 | 10833 | 49.02 | 45 | 2206 | 104.58 | 13039 | **124.69** | 46.30 | 30 | 1389 | 52.08 | 90 | 4687 | 49.02 | 120 | 5882 | 147.40 | 11959 | **81.13** |
| 5 | 30.86 | 195 | 6019 | 25.25 | 45 | 1136 | 56.12 | 7155 | **127.50** | 25.25 | 30 | 758 | 30.86 | 90 | 2778 | 29.76 | 120 | 3571 | 85.88 | 7107 | **82.75** |
| 4 | 18.94 | 195 | 3693 | 17.01 | 45 | 765 | 35.95 | 4458 | **124.03** | 23.15 | 30 | 694 | 20.33 | 90 | 1829 | 19.84 | 120 | 2381 | 63.31 | 4905 | **77.46** |
| 3 | 16.03 | 195 | 3125 | 13.44 | 45 | 605 | 29.47 | 3730 | **126.58** | 17.73 | 30 | 532 | 15.15 | 90 | 1364 | 14.62 | 120 | 1754 | 47.50 | 3650 | **76.84** |
| 2 | 17.73 | 195 | 3457 | 26.04 | 45 | 1172 | 43.77 | 4629 | **105.76** | 15.43 | 30 | 463 | 14.12 | 90 | 1271 | 13.23 | 120 | 1587 | 42.78 | 3321 | **77.63** |

Figure 28 - Center of Rigidity

| Second Floor | | | | |
|---|---|---|---|---|
| Distribution of Wind Shears to Frames | | | | |
| Frame | Relative X | Relative Y | Story Shear (k) | |
| VT-A | | 0.405 | 326.67 | 132.32 |
| VT-B | 0.361 | | 326.67 | 117.83 |
| VT-C | | 0.595 | 326.67 | 194.35 |
| VT-D | 0.330 | | 326.67 | 107.84 |
| VT-E | 0.309 | | 326.67 | 101 |

*Figure 29 - Wind Frame Shear Distribution*

**Second Floor - Wind**

| Axis | Center of Area | Center of Rigidity | Eccentricity (ft) |
|---|---|---|---|
| x | 96.8 | 105.76 | 8.96 |
| y | 86.68 | 77.63 | 9.05 |

| Story Shear | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Location x (ft) | Location y (ft) | Distance from x (ft) | Distance from y (ft) | Relative Rx | Relative Ry | Rx*y2 | Ry*x2 | Direct Shear | Torsional Shear | Hn (kips) | RAM Load | RAM Hn | Percent Difference |
| VT-A | 195 | 137 | 98.20 | 50.32 | 0 | 0.405 | 0.0 | 3905.5 | 132.3 | 6.729 | 139.05 | | 154.53 | 10.016 % |
| VT-C | 45 | 77 | -51.80 | -9.68 | 0 | 0.595 | 0.0 | 1596.5 | 194.3 | 7.660 | 202.01 | | 237.38 | 14.901 % |
| Totals | | | | | 0 | 1 | | 5502.0 | 326.7 | 14.389 | 341.06 | | 391.91 | **12.975 %** |

*Figure 30 - Frame Shears on Second Floor - Wind*

## Appendix C. Seismic Loads

**Calculation of $S_{DS}$ and $S_{D1}$**

Occupancy Category.....................................II
Seismic Use Group II
Importance Factor (I) 1.0
Latitude and Longitude…………………….. 40°25'32.71" N 79°57'50.93" W
Mapped Spectral Response Accelerations

$S_s = 0.125$ g
$S_1 = 0.049$ g

Site Class……………………………………....D
Site Class Factors

$F_a = 1.60$
$F_v = 2.40$

$S_{MS} = F_a \times S_s = 1.60 \times 0.125 = 0.20$
$S_{M1} = F_v \times S_1 = 2.40 \times 0.049 = 0.1176$
$S_{DS} = 2/3 \times S_{MS} = 2/3 \times 0.20 =$ **0.133**
$S_{D1} = 2/3 \times S_{M1} = 2/3 \times 0.1176 =$ **0.0784**
Seismic Design Category………………….. A or B: **B controls**

**Finding Response Modification Factor (R)**

Braced Frames are a "Steel System Not Specifically Detailed for Seismic Resistance"
Response Modification Factor (R) ................3.0
Over-strength Factor ($W_o$) ............................3.0
Deflection Amplification Factor ($C_d$) ...........3.0

**Determination of T**

4/5 Braced Frames are not eccentric so it is conservative to use "All Other Structural Systems" for $C_t$ and x
Seismic Response Coefficient ($C_t$) ................0.02
Period Coefficient (x) ....................................0.75
$h_n = 81.33$ ft (max height)
$T_a = 0.1N = 0.1 \times 5 = 0.5$ : This is a very rough estimate
$T_a = C_t h_n^x = 0.02 \times (81.33 \text{ ft})^{0.75} =$ **0.542** : This is a better approximation and is conservative
$C_u = 1.7$ : $S_{D1} <= 0.1$
$T = C_u \times T_a = 1.7 \times 0.542 =$ **0.921**

**Calculation of $C_s$**

$$C_s = \frac{S_{DS}}{(R / I)} = \frac{0.133}{(3 / 1)} = 0.0443$$

Upper Bound

$$C_s <= \frac{S_{D1}}{T \times (R / I)} = \frac{0.0784}{0.921 \times (3 / 1)} = \mathbf{0.0284}$$

Lower Bound

$$C_s >= \quad 0.01$$

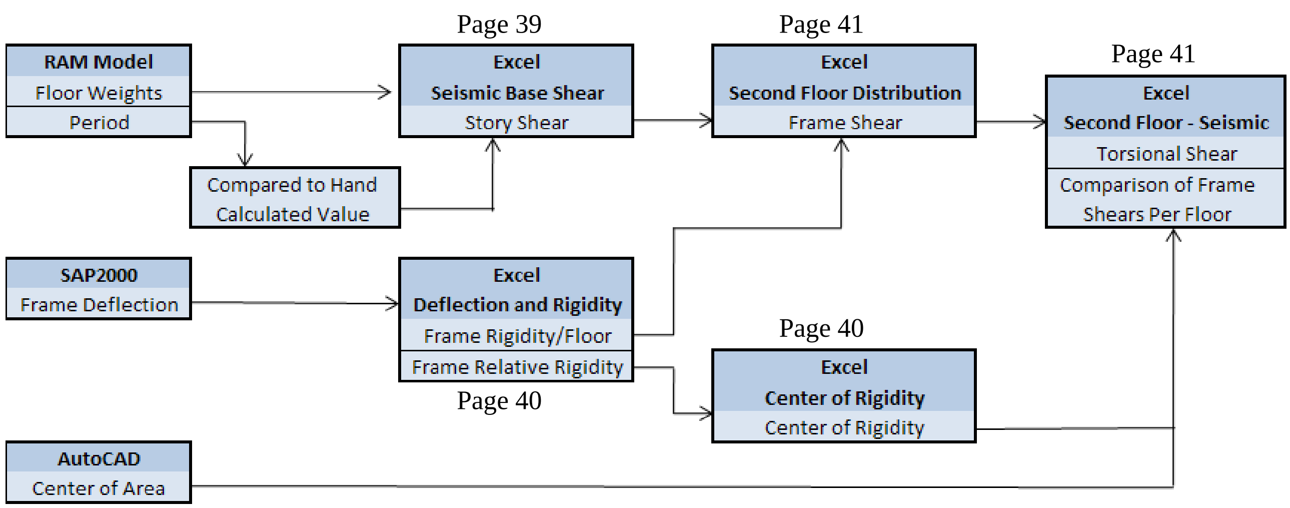


*Figure 31 – Seismic Load Appendix Map*

**Seismic Base Shear**

| Level | $h_x$ | $h_x^k$ | W | $W * h_x^k$ | $C_{vx}$ | F | V | M | ∑ M |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 68.67 | 167.266 | 1440 | 240828.9 | 0.211 | 80.40 | | 5521.363 | 5521.363 |
| 5 | 56.68 | 132.595 | 2980 | 395159.3 | 0.347 | 131.93 | 80.40 | 7477.779 | 12999.14 |
| 4 | 41.00 | 89.593 | 2986 | 267534.1 | 0.235 | 89.32 | 212.33 | 3662.129 | 16661.27 |
| 3 | 27.34 | 54.859 | 2992 | 164158.8 | 0.144 | 54.81 | 301.65 | 1498.419 | 18159.69 |
| 2 | 13.67 | 23.705 | 3001 | 71142.36 | 0.062 | 23.75 | 356.46 | 324.6888 | 18484.38 |
| 1 | 0.00 | 0.000 | 0 | 0 | 0.000 | 0.00 | 380.21 | 0 | 18484.38 |
| | | Totals | 13399.6 | 1138824 | 1 | 380.21 | | 18484.38 | |

| | Cs | W (kips) | | Total Force | |
|---|---|---|---|---|---|
| $V = C_s * W =$ | 0.028375 | 13399.6 | = | 380.21304 | k |

| T | k |
|---|---|
| 0.50 | 1 |
| 0.92 | 1.2105 |
| 2.50 | 2 |

| | Lower Bound | Exact | Upper Bound | Use |
|---|---|---|---|---|
| Cs = | 0.01 | 0.044333 | 0.028375 | 0.028375 |

*Figure 32 - Seismic Base Shear*

| Braced Frame Rigidities | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | | Deflection and Rigidity | | | | | | | | | | | | | | | | | | | |
| | | | Frame A Strong Axis-Y | | | | Frame B Strong Axis-X | | | | Frame C Strong Axis-Y | | | | Frame D Strong Axis-X | | | | Frame E Strong Axis-X | | | |
| Story | Total (ft) | Per Floor (ft) | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness | Total (in) | Per Floor (in) | Stiffness | Relative Stiffness |
| Roof | 69.1 | 14.6 | 0.222 | 0.018 | 55.56 | 0.531 | 0.226 | 0.022 | 46.30 | 0.314 | 0.232 | 0.02 | 49.02 | 0.469 | 0.238 | 0.019 | 52.08 | 0.353 | 0.248 | 0.02 | 49.02 | 0.333 |
| 5 | 54.6 | 13.7 | 0.204 | 0.032 | 30.86 | 0.550 | 0.204 | 0.04 | 25.25 | 0.294 | 0.211 | 0.04 | 25.25 | 0.450 | 0.218 | 0.032 | 30.86 | 0.359 | 0.228 | 0.034 | 29.76 | 0.347 |
| 4 | 40.9 | 13.7 | 0.172 | 0.053 | 18.94 | 0.527 | 0.164 | 0.043 | 23.15 | 0.366 | 0.172 | 0.059 | 17.01 | 0.473 | 0.186 | 0.049 | 20.33 | 0.321 | 0.194 | 0.05 | 19.84 | 0.313 |
| 3 | 27.2 | 13.7 | 0.119 | 0.062 | 16.03 | 0.544 | 0.121 | 0.056 | 17.73 | 0.373 | 0.113 | 0.074 | 13.44 | 0.456 | 0.137 | 0.066 | 15.15 | 0.319 | 0.144 | 0.068 | 14.62 | 0.308 |
| 2 | 13.5 | 13.5 | 0.056 | 0.056 | 17.73 | 0.405 | 0.065 | 0.065 | 15.43 | 0.361 | 0.038 | 0.038 | 26.04 | 0.595 | 0.071 | 0.071 | 14.12 | 0.330 | 0.076 | 0.076 | 13.23 | 0.309 |

Figure 33 - Deflection and Rigidity

| Center of Rigidity | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | X Direction | | | | | | | | | Y Direction | | | | | | | | | | | |
| | Frame A | | | Frame C | | | Center of Rigidity | | | Frame B | | | Frame D | | | Frame E | | | Center of Rigidity | | |
| Story | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Total R Per Floor | Total R*x Per Floor | Location (ft) | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Rigidity | Location (ft) | R*x | Total R Per Floor | Total R*x Per Floor | Location (ft) |
| Roof | 55.56 | 195 | 10833 | 49.02 | 45 | 2206 | 104.58 | 13039 | **124.69** | 46.30 | 30 | 1389 | 52.08 | 90 | 4687 | 49.02 | 120 | 5882 | 147.40 | 11959 | **81.13** |
| 5 | 30.86 | 195 | 6019 | 25.25 | 45 | 1136 | 56.12 | 7155 | **127.50** | 25.25 | 30 | 758 | 30.86 | 90 | 2778 | 29.76 | 120 | 3571 | 85.88 | 7107 | **82.75** |
| 4 | 18.94 | 195 | 3693 | 17.01 | 45 | 765 | 35.95 | 4458 | **124.03** | 23.15 | 30 | 694 | 20.33 | 90 | 1829 | 19.84 | 120 | 2381 | 63.31 | 4905 | **77.46** |
| 3 | 16.03 | 195 | 3125 | 13.44 | 45 | 605 | 29.47 | 3730 | **126.58** | 17.73 | 30 | 532 | 15.15 | 90 | 1364 | 14.62 | 120 | 1754 | 47.50 | 3650 | **76.84** |
| 2 | 17.73 | 195 | 3457 | 26.04 | 45 | 1172 | 43.77 | 4629 | **105.76** | 15.43 | 30 | 463 | 14.12 | 90 | 1271 | 13.23 | 120 | 1587 | 42.78 | 3321 | **77.63** |

Figure 34 - Center of Rigidity

| Second Floor | | | | |
|---|---|---|---|---|
| Distribution of Seismic Shears to Frames | | | | |
| Frame | Relative Rigidity | | Story Shear (k) | |
| | X | Y | | |
| VT-A | | 0.405 | 356.46 | 144.39 |
| VT-B | 0.361 | | 356.46 | 128.58 |
| VT-C | | 0.595 | 356.46 | 212.07 |
| VT-D | 0.330 | | 356.46 | 117.68 |
| VT-E | 0.309 | | 356.46 | 110.21 |

*Figure 35 - Seismic Frame Shear Distribution*

**Second Floor - Seismic**

| Axis | Center of Area | Center of Rigidity | Eccentricity (ft) |
|---|---|---|---|
| x | 96.8 | 105.76 | 8.96 |
| y | 86.68 | 77.63 | 9.05 |

| Story Shear | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Element | Location | | Distance from Center of Rigidity | | Relative Rigidity | | Rx*y2 | Ry*x2 | Direct Shear (kips) | Torsional Shear (kips) | Hn (kips) | RAM Load Case | RAM Hn (kips) | Percent Difference |
| | x (ft) | y (ft) | x (ft) | y (ft) | Rx | Ry | | | | | | | | |
| VT-A | 195 | 137 | 98.20 | 50.32 | 0 | 0.405 | 0.0 | 3905.5 | 144.4 | 7.343 | 151.73 | E3 | 168.04 | 9.705 % |
| VT-B | 206 | 122 | 109.24 | 35.32 | 0.361 | 0 | 450.3 | 0.0 | 128.6 | 2.076 | 130.65 | E2 | 142.27 | 8.167 % |
| VT-C | 45 | 77 | -51.80 | -9.68 | 0 | 0.595 | 0.0 | 1596.5 | 212.1 | 8.358 | 220.43 | E3 | 266.73 | 17.358 % |
| VT-D | 150 | 62 | 53.20 | -24.68 | 0.330 | 0 | 201.0 | 0.0 | 117.7 | 1.213 | 118.89 | E2 | 145.09 | 18.056 % |
| VT-E | 90 | 32 | -6.80 | -54.68 | 0.309 | 0 | 923.9 | 0.0 | 110.2 | 2.357 | 112.56 | E2 | 136.52 | 17.547 % |
| Totals | | | | | 1 | 1 | | 7077.3 | 712.9 | 21.348 | 734.27 | | 858.65 | **14.486 %** |

*Figure 36 - Frame Shears on Second Floor - Seismic*