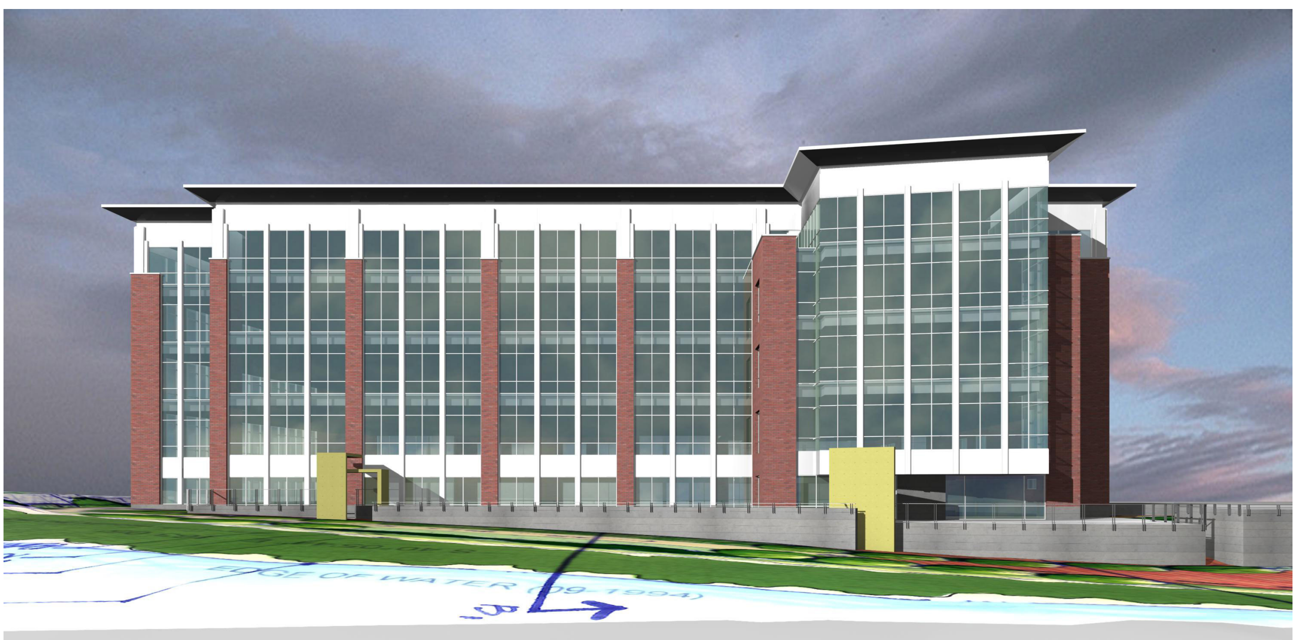
Technical Report I

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. Then, building loads and detailed spot checks of vital structural systems are explored. The analysis concludes with an appendix specifying all uncovered calculations and assumptions with provided spreadsheets and diagrams to further progress the reader's understanding of Quantum III. Finally, modeling reports, spreadsheet details and other calculations are available upon request.



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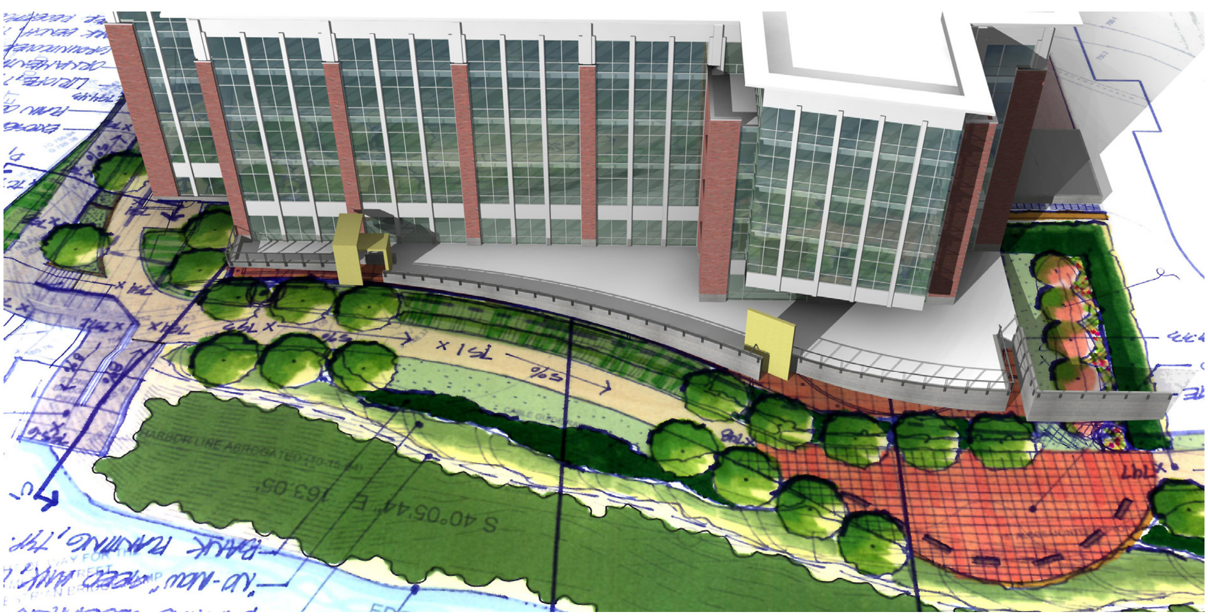
I. Introduction

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.

The structural system reflects the need for flexibility with 30'x30' bays and a superimposed 20 psf partition load over all office spaces. Although only a 50 psf live load is required for office areas, 80 psf was used to account for unpredictability of corridor locations on each floor. 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.

Following is an analysis to create a foundation from which to expand understanding of the existing structure of Quantum III. Lateral force resisting systems, gravity structure, economy, and flexibility are the basis of analysis, and are studied in detail throughout the subsequent pages.



II. Structural Systems

Foundations and Geotechnical Concerns

The foundation of Quantum III will be constructed on abandoned steel industry 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 considering possible loading. The fill located deeper in the subgrade has a higher bearing capacity than the aforementioned soils so 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. The bedrock is located roughly 85 feet below the surface, with the water table resting at 730 ft—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 is composite lightweight concrete slab on 3" galvanized steel deck. Shear studs are 4" long and ¾" diameter in 2.5" lightweight concrete for a total slab and deck thickness of 5.5". The typical roof framing contains 3" roof deck save the mechanical unit area, where 2" deck with 3" lightweight concrete provides added support and isolates mechanical vibrations???. Typical girders are W24x55 with 28 studs with W18x35 infill beams with 16 studs spaced at 10' center to center. All exceptions are explained below.

First Floor

Since Quantum III does not have a basement level, the first floor is slab on grade. The northwest wall contains the receiving, generator, and fire pump rooms all with 6" concrete slab while the remaining space is 4" slab on grade. All slab on grade has construction joints at 15' on center. The receiving and loading areas are angled recesses to account for the limited clearance to the edge of the site and include a one truck bay and a trash collection/compaction bay.

Second Floor

Office space, with a rectilinear wall, overhangs the recessed loading docks and is framed with cantilevered W33x141's replacing the typical W24x55 girders. Interior infill beams are still W18x35's, but are at 7.5', with W12x19's at 6' center to center framing into the W18's. These distribute the weight of the cantilevered wall by transferring load onto beams cantilevered half the total overhang length. This greatly reduces the moment placed on the W33x141's.

The northeast and southwest walls feature a cantilevered angle in plan that complicates the façade and requires cantilevered infill beams. W10x12's provide the exterior wall support. Finally, HSS 4x4's frame an entrance canopy on the south corner of the building.

Third, Fourth, and Fifth Floor

Floors three and four have the same exact framing plan, and continue the cantilevered angle up the building plan. The fifth floor differs in minute details, offering sporadic reinforcing and framing for the roof level.

Roof

The roof framing is separated into two portions: the typical height roof with W16x26 infill beams and W21x44 girders and the mechanical space with similar infill beams and W24x55 girders. The mechanical area occupies the center portion of Quantum III and is 2" lower to maintain the same elevation for the entire roof. This may help reduce mechanical vibrations as well. Infill beams in this area are spaced closer to distribute the load of two 36,000 pound mechanical units. Surrounding the mechanical units is a windscreen, framed into typical infill beams stiffened with W12x14s placed on top the roof slab. The W12's distribute the load to the two typical infill beams on either side to limit torsion and provide extra moment resistance.

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; while moment frame columns run from W14x74's to W14x193's. 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 (13'-8") 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.

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, 13th 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.

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

IV. 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.

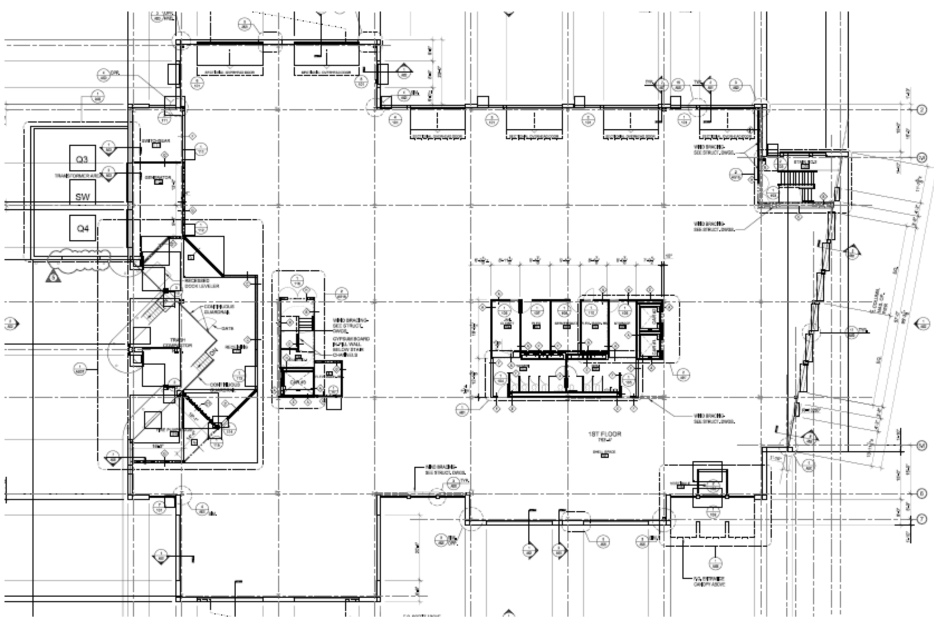


Figure 1 – Typical Architectural Floor Plan

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.

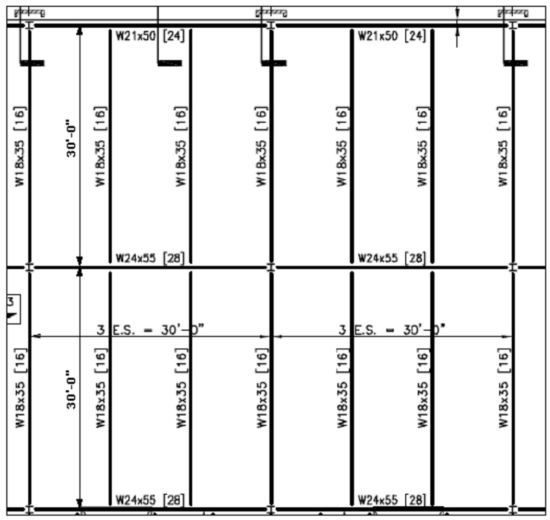


Figure 2 – Typical Bay

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.

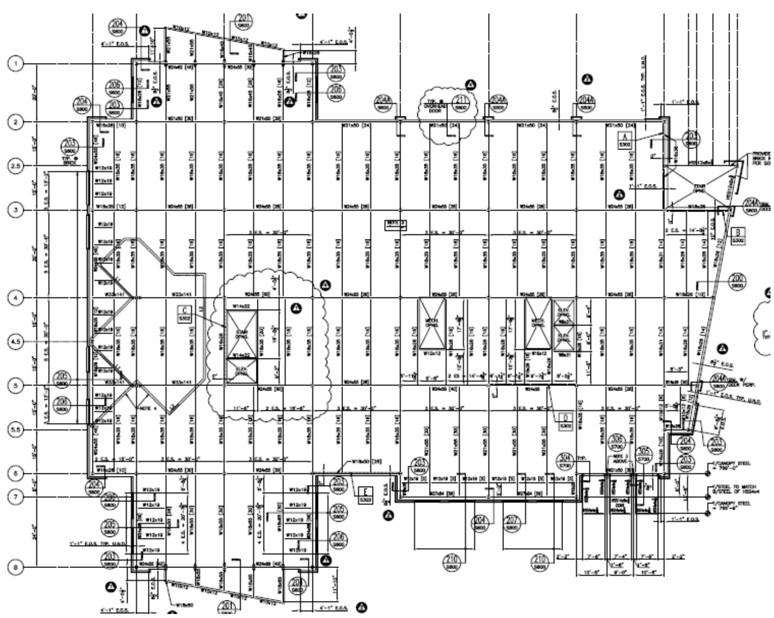


Figure 3 – Typical Floor Framing

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.

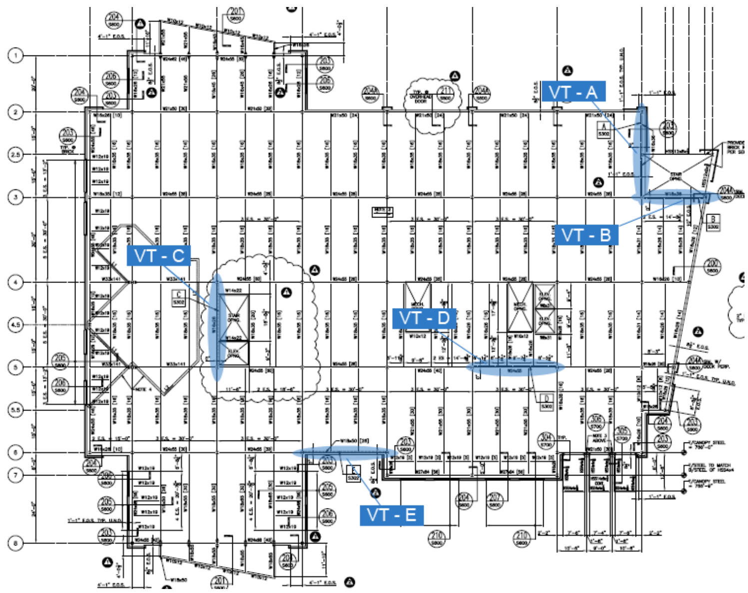


Figure 4 – Vertical Truss Locations

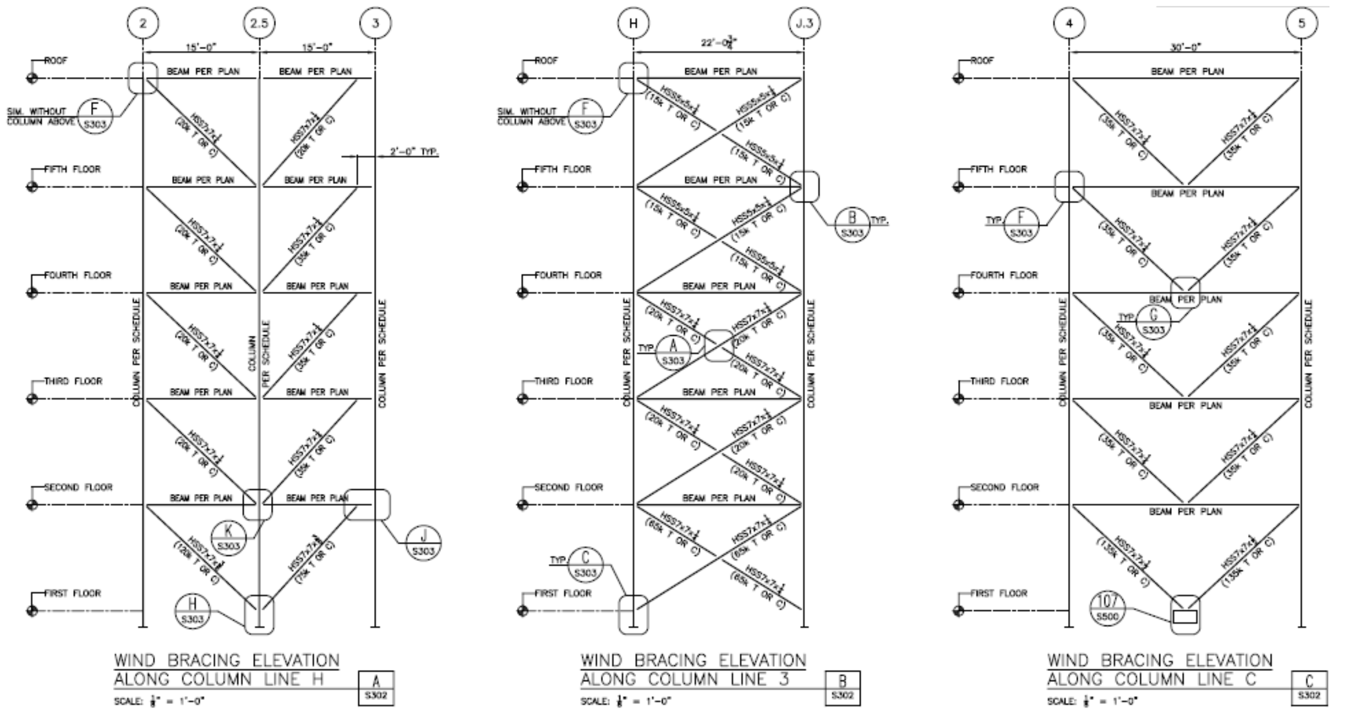


Figure 5 – Vertical Trusses A, B and C (VT-A, B, C)

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.

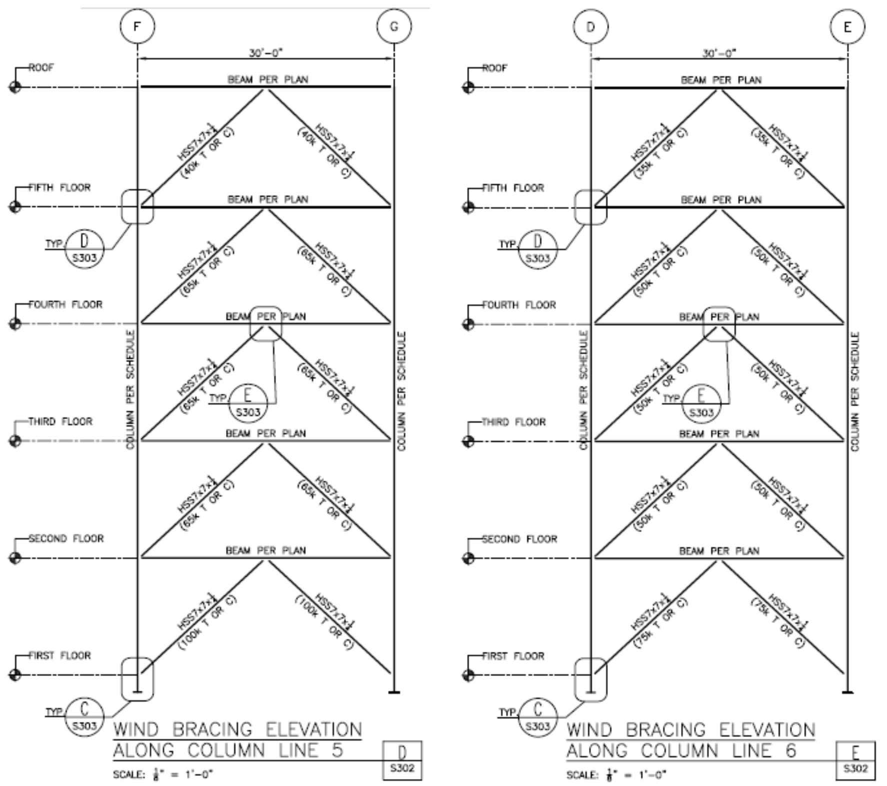


Figure 6 – Vertical Trusses D and E (VT-D, E)

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.

V. Building 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 IV. Framing Plans and Elevations, Figure 2, page 9.

Location	Load (psf)	Description
Roof	20 **18**	A_t = 10' x 30' = 300 ft^2 $\therefore R_1 = 1.2 - 0.001A_t = 1.2 - 0.001 * (300\ ft^2) = 0.9$ F = 0, the roof pitch is small enough to be negligible $\therefore R_2 = 1$ $\therefore L_r = R_1 * R_2 * L = 0.9 \times 1.0 * 20 =$ **18 psf**
Offices and corridors above the first floor	80 **54.6** **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 K_{LL} = 4 : Interior Beams $A_{t,\ beam}$ = 300 ft^2 $A_{t,\ girder}$ = 15 ft x 30 ft = 450 ft^2 $L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) =$ $= 80 \times \left(0.25 + \frac{15}{(4 \times 300\ ft^2)^{0.5}}\right) =$ **54.6 psf** $L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) =$ $= 80 \times \left(0.25 + \frac{15}{(4 \times 450\ ft^2)^{0.5}}\right) =$ **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 a combination of the AISC Steel Manual, 13th Edition and the PCI Design Handbook, page 11-2 as attached in the appendix of this technical report. Wall weights are supplied in the structural documents of American Eagle Outfitters: Quantum III. Finally, all supporting calculations are available on page 25.

Component	Typical Floor	Roof	Mechanical Roof
Concrete Slab	52.7	-	47.9
Metal Decking	2.5	2.5	2.5
Flooring	2	-	-
Ceiling	3	3	3
M/E/L	5	10	10
Insulation	-	9	-
Membrane	-	2	2
Total Dead	65	27	65

Wall Loads

Curtain Walls…………………………………...20 psf
8" CMU, grout/rein. 24" cc……………...…..51 psf
Partitions……………………………………..20 psf

Snow Loads

American Eagle Outfitters: Quantum III can be subjected to minor snow drifts which can cause possible overloading of the roof framing. The total number of snow drift cases is dependant on which way the wind is blowing: from the North, South, East or West. Drifts are the largest in magnitude when North-South winds occur, and can cause the total height of snow to be 3.36 feet, adding a surcharge snow load of 39.2 psf for a total snow load of 60.2 psf. Backup calculations, snow drift diagrams and a chart of snow loading outcomes is available in the Appendix, page 25.

Base Ground Snow Load (p_g)	30 psf
Importance Factor (I)	1.0
Thermal Factor (Ct)	1.0
Snow Exposure Factor (Ce)	1.0
Flat Roof Snow Load	21 psf

Wind Forces

A comparison of wind pressures acting on the main wind force resisting system is described below. Atlantic Engineering Services was conservative in the fact they took the total height (h) including the parapet, in order to determine the acting wind pressures. Although this is only a difference in 4', it resulted in the velocity pressure, q_h, to be roughly 5% larger. 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 29.

Assumptions		**Differing Assumptions by AES**
Building Height (h)	68.33'	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 Load Summary:

MWFRS Design Pressures						
Walls						
	Wind Direction		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
	Wind Direction	Height (feet)	Pressures (lb/ft^2)			
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

Windward (continued)	Wind Direction	Height (feet)		Pressures (lb/ft^2)		
	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

MWFRS Design Pressures

Roof

	Wind Direction	Distance From Windward Wall (feet)								
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 =	-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

Parapet

	GC_{pn}	K_p	q_p				
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				
Windward	1.5	1.21	21.30	P =	31.95	±	3.71
Leeward	-1	1.21	21.30	P =	-21.30	±	3.71

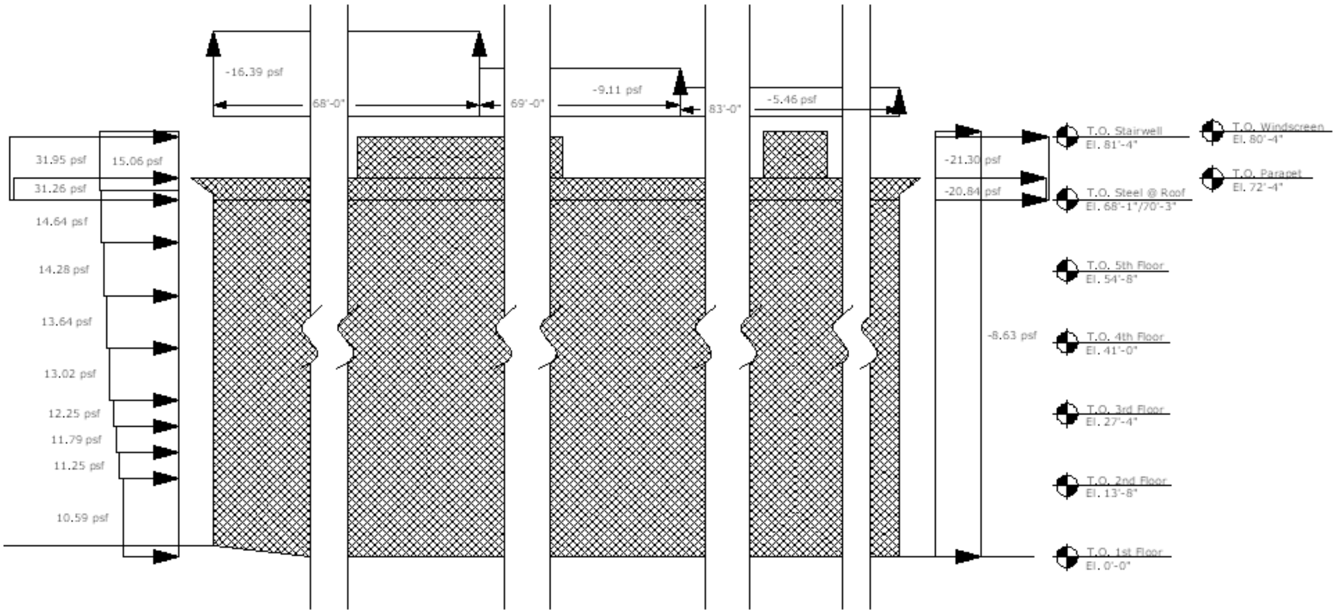


Figure 7 – North Elevation: East-West Wind Pressures

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.

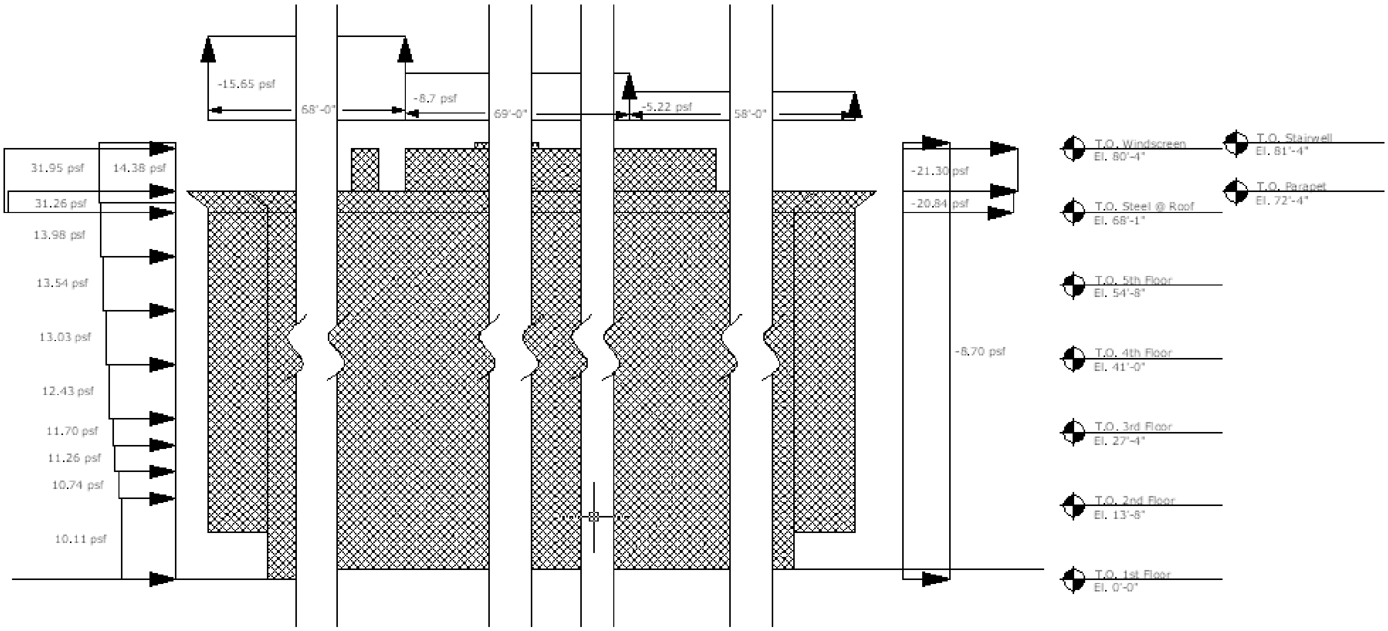


Figure 8 – East Elevation: South-North Wind Pressures

Below are the base shear and overturning moment results 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 truss. 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 top 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 the Detailed Calculations and Results section, page 25.

Base Shear (V)..412.14 k **Controls**
Overturning Moment (M)...........................178454 k-ft

Seismic Loads

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 34.

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

Base Shear (V) ....285.704 k
Overturning Moment (M) ....14016 k-ft

VI. Framing Spot Checks

Typical Bay

The following calculations determine the adequacy of the aforementioned loads as they pertain to the actual loading used by Atlantic Engineering Services. A comparison of results from analysis and the beam sizes given is outlined below as well. The typical bay is shown again to emphasize the spacing and member sizes.

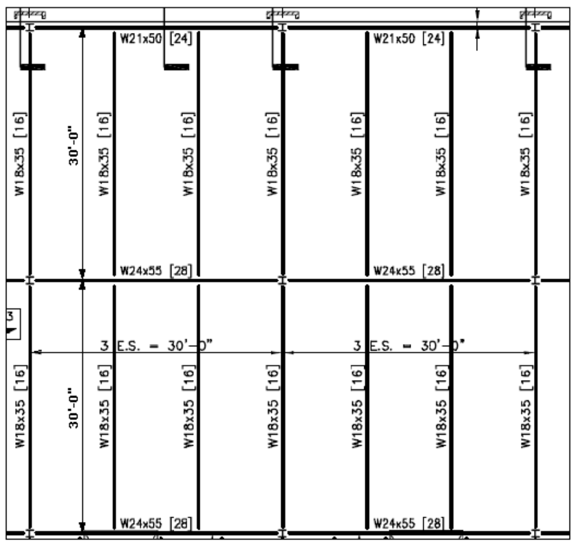


Figure 9 – Typical Bay

Material Properties and Loadings

Dead Load = 65 psf Live Load = 80 psf, **54.6 psf, 48.3 psf**	2.5" LW Concrete Slab 3" 20-Gage Steel Deck
Beams: W18x35, A_s = 10.3 in^2, d = 17.7 in	f'$_c$ = 4000 psi
Girders: W24x55, A_s = 16.2 in^2, d = 23.6 in	3/4" Diameter, 4" Long Studs
f_y = 50 psi	Proposed Fire Rating: 0 hrs

Typical Composite Beam Check:

Determine Beam Forces:

$$w_u = \frac{10 \text{ ft x } (1.2 \text{ x } 65 \text{ psf} + 1.6 \text{ x } 80 \text{ psf})}{1000 \text{ lbs}} = 2.06 \text{ k/ft}$$

$M_u = \frac{wl^2}{8} = \frac{2.06 \text{ k/ft} \times 30^2}{8} = 232 \text{ k-ft}$

$V_u = \frac{wl}{2} = \frac{2.06 \text{ k/ft} \times 30}{2} = 30.9 \text{ k}$

Find Plastic Neutral Axis Location:

$b_{eff} =$ spacing = 10 ft

$b_{eff} =$ 0.25 x span = 0.25 x 30 ft = 7.5 ft **minimum controls**

$P_c = b_{eff} \times d_{slab} \times f'_c * 0.85 = 7.5 \text{ ft} \times 12 \text{ in/ft} \times 5.5 \text{ in} \times 4 \text{ ksi} \times 0.85 = 1683 \text{ k}$

$P_t = A_s \times F_y = 10.3 \text{ in}^2 \times 50 \text{ ksi} = 515 \text{ k}$

∴ Plastic Neutral Axis is in concrete. Since concrete cannot act in tension, assume full composite action, or the axis to be at the top of the flange

Calculate Nominal Moment Capacity:

$\sum Q_n =$ 515 k : for full composite action

$a = \frac{P_t}{0.85 \times f'_c \times b} = \frac{515 \text{ k}}{0.85 \times 4 \text{ ksi} \times 7.5 \text{ ft} \times 12 \text{ in/ft}} = 1.683 \text{ in}$

$Y2 = d_{slab} - a/2 = 5.5 \text{ in} - (1.683 \text{ in})/2 = 4.66 \text{ in}$

$\varnothing M_n =$ 535 k-ft > $\varnothing M_n$ > 515 k-ft >> 232 k-ft OK ☑

Check Deflection:

$I_{LB} = 1430 \text{ in}^4$ (conservative)

$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5 \times 2.06 \text{ k/ft} \times (30 \text{ ft})^4 \times 1728}{384 \times 29{,}000 \text{ ksi} \times 1430 \text{ in}^4} = 0.905 \text{ in} = \frac{l}{398}$ OK ☑

Typical Composite Girder Check:

Determine Girder Forces:

$P = \frac{wl}{2} = \frac{1.654 \text{ k/ft} \times 30}{2} = 24.81 \text{ k}$

Point loads from beams are at 1/3 points along girder

$M_u = P \times a = 24.81 \text{ k} \times 10 \text{ ft} = 248.1 \text{ k-ft}$

$V_u = P = 24.81 \text{ k}$

Find Plastic Neutral Axis Location:

b_{eff} = spacing = 30 ft

b_{eff} = 0.5 x span = 0.25 x 30 ft = 7.5 ft **minimum controls**

P_c = b_{eff} x d_{slab} x f'_c x 0.85 = 7.5 ft x 12 in/ft x 5.5 in x 4 ksi x 0.85 = 1683 k

P_t = A_s x F_y = 50 ksi x 16.2 in^2 = 810 k

∴ Plastic Neutral Axis is in concrete. Since concrete cannot act in tension, assume full composite action, or the axis to be at the top of the flange

Calculate Nominal Moment Capacity:

$\sum Q_n$ = 810 k : for full composite action

$$a = \frac{P_t}{0.85 \times f'_c \times b} = \frac{810\text{ k}}{0.85 \times 4\text{ ksi} \times 7.5\text{ ft} \times 12\text{ in/ft}} = 2.65\text{ in}$$

$Y2$ = d_{slab} - a/2 = 5.5 in - (2.65 in)/2 = 4.175 in

$\varnothing M_n$ = 989 k-ft > $\varnothing M_n$ > 959 k-ft >> 248 k-ft OK ☑

Check Deflection:

I_{LB} = 3370 in^4 (conservative)

$$\Delta_{max} = \frac{0.036Pl^3}{EI} = \frac{0.036 \times 24.81\text{ k} \times (30\text{ ft})^3 \times 1728}{29{,}000\text{ ksi} \times 3370\text{ in}^4} = 0.426\text{ in} = \frac{l}{845}$$ OK ☑

In any engineering field, it is essential to design for economy. As shown above, the beams and girders are over-designed by at least a factor of 2:1. The beam design was controlled by deflection issues, but the girder seems to be significantly over designed for both issues. The obvious answer for this discrepancy is that the typical girders are designed for the worst case scenario. Economy has two sides: one in economy of materials and the other in economy of work. For an engineer to go in and design each individual beam would drive the engineering cost through the roof; but on the other hand, over designing beams means more money goes into materials. It is evident that the engineer found the middle ground between these two extremes. In other words, the beams and girders are over designed for this particular loading, but more severe loading may be present elsewhere in the building, for which the typical bay was considered.

Other possible factors for this over-design can be flexibility for the future tenant fit-out, the presence of axial loads transferring lateral building loads to the braced frame resisting systems, or to drive down vibrations throughout the structure.

Vertical Truss VT-C

As described earlier, relative rigidities are calculated for VT-A and VT-C, therefore one of these braced frames was chosen for a lateral load spot check. Relative rigidities were established on a floor-to-floor and overall building height basis. Unit loads were placed at each floor level on the north face of the truss, and deflection of the southernmost node were noted and compared to calculate rigidity. Based on these floor to floor relative rigidities, wind loads were distributed to truss VT-C and analyzed.

RAM Steel was used to evaluate the frame as a single element with hand calculated loads placed upon it.

Beam Loadings:

First Through Fifth Floor

w_u = 1.2 x 5 ft x 65 psf + 1.6 x 5 ft x 80 psf = 1.03 k/ft

P_u = 1.6 x (100 plf x 12 ft)/2 = 0.96 k : From W14x22 framing into beam interior

Roof

w_u = 1.2 x 5 ft x 27 psf + 1.6 x 5 ft x 20 psf = 0.322 k/ft

P_u = 1.6 x (1 klf x 12 ft)/2 + 1.2 x 27 psf x 4.5 ft x 12 ft/2 + 1.6 x 20 psf x 4.5 ft x 12 ft/2 = 1.75 k
: From W14x22 framing into beam interior

Column Loadings:

VT-C Columns									
Level	Area	Live Load	Reduced Live Load	Dead Load	P_u,d	P_u,l	P_u	P_u Total	P_u Factored Total
	sf	psf	psf	psf	k	k	k	k	k
2	788	80	41.37	65	51.22	32.60	83.82	83.82	113.63
3	788	80	41.37	65	51.22	32.60	83.82	167.65	227.26
4	788	80	41.37	65	51.22	32.60	83.82	251.47	340.89
5	788	80	41.37	65	51.22	32.60	83.82	335.29	454.51
Roof	788	20	12.00	27	21.28	9.46	30.73	366.02	495.17
						Total		366.02	495.17

Braced Frame Relative Rigidities													
	Frame A						Frame C						
	Total Deflection	Floor Deflection	Total Deflection Ratio	Floor Deflection Ratio	Total Relative Rigidity	Floor Relative Rigidity	Total Deflection	Floor Deflection	Total Deflection Ratio	Floor Deflection Ratio	Total Relative Rigidity	Floor Relative Rigidity	Loading (k)
Roof	0.03829	0.00481	0.59	0.56	0.41	0.44	0.02617	0.00374	0.41	0.44	0.59	0.56	1
Floor 5	0.03348	0.00707	0.60	0.58	0.40	0.42	0.02243	0.00506	0.40	0.42	0.60	0.58	1
Floor 4	0.02641	0.00905	0.60	0.58	0.40	0.42	0.01737	0.00651	0.40	0.42	0.60	0.58	1
Floor 3	0.01736	0.01031	0.62	0.59	0.38	0.41	0.01086	0.00729	0.38	0.41	0.62	0.59	1
Floor 2	0.00705	0.00705	0.66	0.66	0.34	0.34	0.00357	0.00357	0.34	0.34	0.66	0.66	1

The wind forces shown at the right are applied at each floor level up the elevation of the building. After applying all point, distributed, and wind loads on the frame, it was found that the bracing exceeded the designed strength as indicated on the structural drawings. The top eight braces were each designed for 20 kips in tension or compression. When referencing AISC, 13th Edition, it is found that an HSS7x7x1/4 yields in tension at 255 kips, and fails in compression at 168 kips. The largest force found through this analysis is 130 kips tension or compression, proving the member fine in tension, but minutely under sized for compression loading.

Distribution of Lateral Forces to VT-C Based on Relative Rigidity			
Level	Floor Relative Rigidity	Total Floor Wind Force F (k)	Wind Force Acting on VT-C (k)
2	0.56	57.65	**32.43**
3	0.58	60.66	**35.36**
4	0.58	72.06	**41.91**
5	0.59	90.42	**52.97**
Roof	0.66	103.53	**68.73**
		Total	**231.39**

A possible explanation for this difference is the possibility of torsional effects on the lateral bracing and the building itself. As witnessed from Figure 4 – Vertical Truss Locations, wind frames are by no means symmetrical throughout the building. With this in mind, dividing of wind loads was based on relative rigidity, which may not be an accurate model. This analysis will have to be checked to develop more accurate modeling of the wind bracing system.

As for the columns, the maximum axial load seen is 356 kips by the W14x176, where base plates were designed for a maximum 420 kips. This is reasonably close to the design value.

VII. Detailed Calculations and Results

Dead Loads

5½" LW Concrete Composite Slab = $\frac{115 \text{ lb}}{\text{ft}^3} \times \frac{5.5 \text{ in}}{12 \text{ inches/ft}}$ = 52.7 psf + 2.5 psf deck

5" LW Concrete Composite Slab = $\frac{115 \text{ lb}}{\text{ft}^3} \times \frac{5 \text{ in}}{12 \text{ inches/ft}}$ = 47.9 psf + 2.5 psf deck

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

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

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

Grout at 24 in on center

Snow Loads

The largest snow drift load calculated is for the inner portion of the windscreen. This was used for the example Snow Drift Calculation. The table on the following page shows the snow drifts calculated for each direction at all possible locations. The directions shown in gray are demonstrated in the diagrams following.

Input		
Ground Snow Load (p_g)	**30**	psf
Exposure Factor (C_e)	**1.0**	
Thermal Factor (C_t)	**1.0**	
Importance Factor (I)	**1.0**	
Roof Height Difference	**12.0**	ft
Upper Roof Length ($l_{u,u}$)	**0.0**	ft
Lower Roof Length ($l_{u.l}$)	**94**	ft
Windward or Leeward (W/L)	**W**	

Output		
Flat Roof Snow Load (p_f)	21	psf
Minimum Flat Roof Snow Load	20	psf
Snow Density (γ)	17.9	pcf
Balanced Snow Height (h_b)	1.17	ft
h_c	10.83	ft
Preliminary h_d	2.19	ft
h_d	**2.19**	**ft**
Drift Width (W)	**8.75**	**ft**
Maximum Drift Surcharge (p_d)	39.16	psf
Total Snow Load (p_g + p_d)	60.16	psf
Snow Drift Gradient	4.48	pcf

View	Wind Direction			h_d (ft)	W (ft)	h_b (ft)
North Elevation	East to West	Stair	Windward	1.45	5.79	1.17
	Left to Right		Leeward	1.38	5.54	1.17
		Windscreen, Outer	Windward	1.38	5.54	1.17
			Leeward	1.45	5.79	1.17
		Windscreen, Inner	Windward	1.84	7.37	1.17
			Leeward	1.84	7.37	1.17
		Parapet	Windward	1.38	5.54	1.17
			Leeward	1.38	5.54	1.17
North Elevation	West to East	Stair	Windward	1.38	5.54	1.17
	Right to Left		Leeward	1.45	5.79	1.17
		Windscreen, Outer	Windward	1.45	5.79	1.17
			Leeward	1.38	5.54	1.17
		Windscreen, Inner	Windward	1.84	7.37	1.17
			Leeward	1.84	7.37	1.17
		Parapet	Windward	1.38	5.54	1.17
			Leeward	1.38	5.54	1.17
East Elevation	South to North	Stair	Windward	2.01	8.04	1.17
	Left to Right		Leeward	2.12	8.48	1.17
		Windscreen, Outer	Windward	1.43	5.71	1.17
			Leeward	1.02	4.08	1.17
		Windscreen, Inner	Windward	2.19	8.75	1.17
			Leeward	2.19	8.75	1.17
		Parapet	Windward	2.12	8.48	1.17
			Leeward	2.01	8.04	1.17

East Elevation	North to South	Stair	Windward	2.12	8.48	1.17
	Right to Left		Leeward	2.01	8.04	1.17
		Windscreen, Outer	Windward	1.02	4.08	1.17
			Leeward	1.43	5.71	1.17
		Windscreen, Inner	Windward	2.19	8.75	1.17
			Leeward	2.19	8.75	1.17
		Parapet	Windward	2.01	8.04	1.17
			Leeward	2.12	8.48	1.17

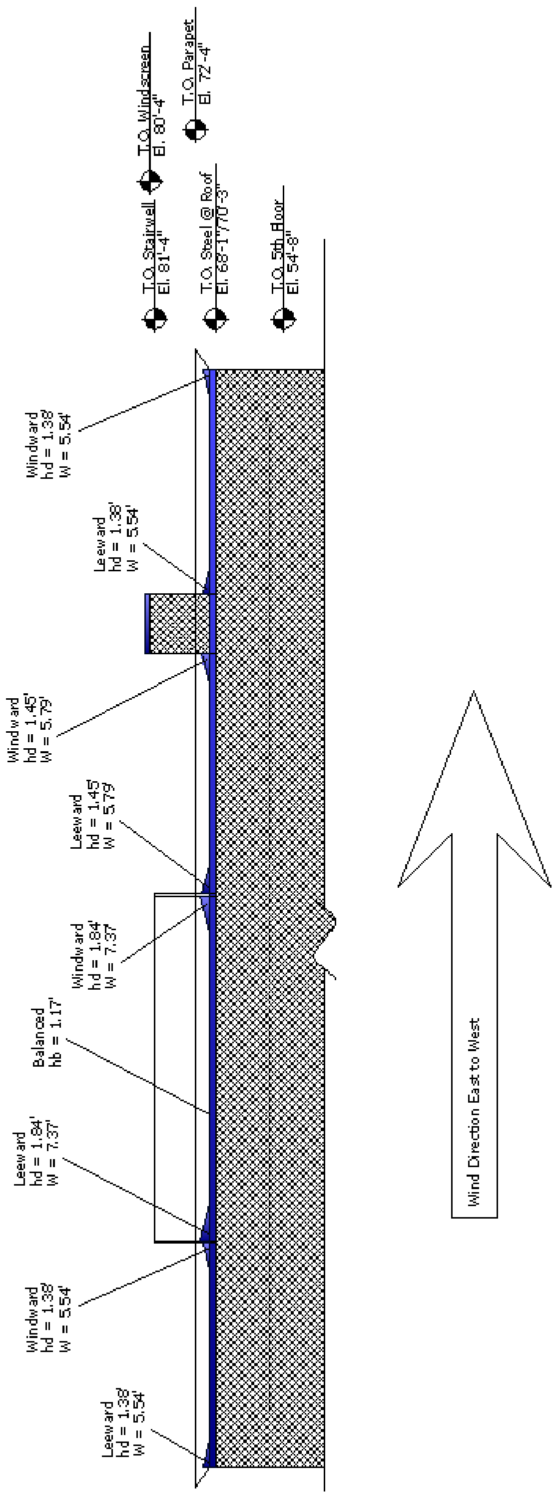


Figure 10 – North Elevation

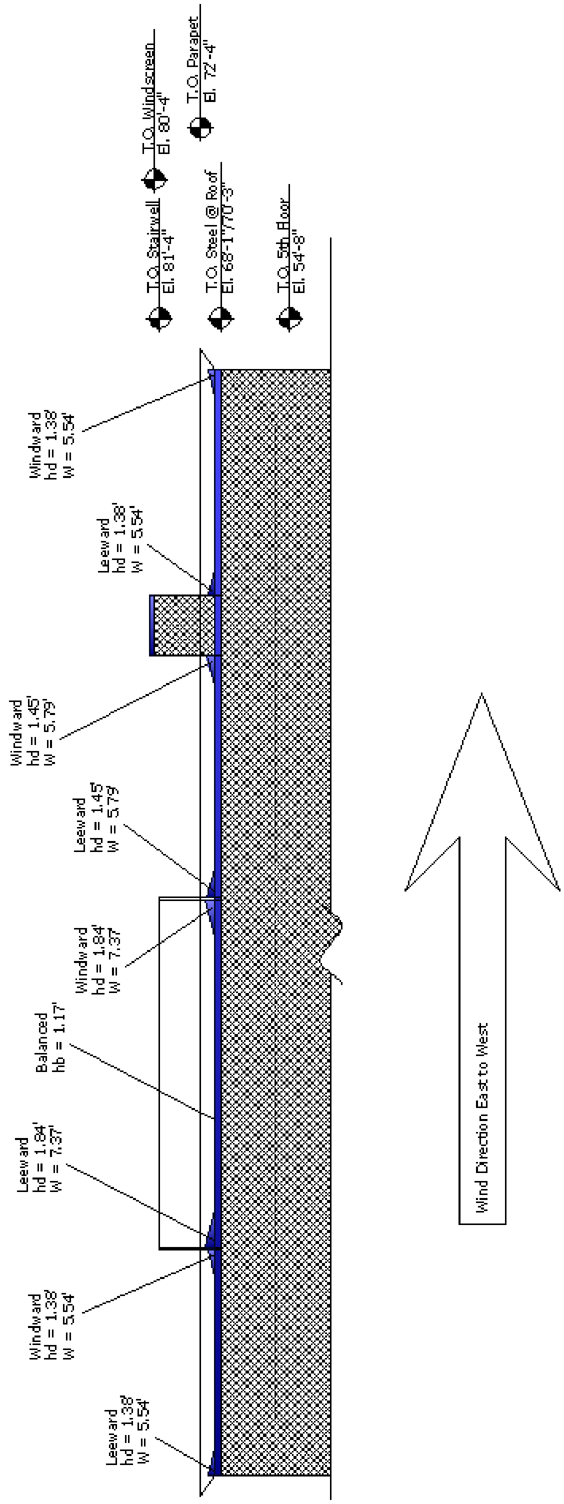


Figure 11 – East Elevation

Wind Pressure Spreadsheets:

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	θ =	0	
Topographic Factor	Kzt =	1	

Table 6-2

Exposure	C	
α	9.5	
z_g	900	feet
a^	0.11	
b^	1	
$\underline{\alpha}$	0.15	
$\underline{b}$	0.65	
c	0.2	
l	500	
$\underline{\epsilon}$	0.2	
z_{min}	15	feet

Gust Effect Factor

$\underline{z}$	41.00
L_z	522
Q (plan north-south)	0.84
Q (plan east-west)	0.92
I_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			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
	Wind Direction	Height (feet)		Pressures (lb/ft^2)		
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)								
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				
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				
Windward	1.5	1.21	21.30	P =	31.95	±	3.71
Leeward	-1	1.21	21.30	P =	-21.30	±	3.71

Total Wind Forces and Overturning Moments

Height Above Grade Min	Height Above Grade Max	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)	Overturning Moment M (k-ft)
0	6.835	10.11	-8.70	18.81	1	0	1480	27.83	27.83	0.0
6.835	15	10.11	-8.70	18.81	2	13.67	1795	33.76	57.65	788.0
15	20	10.74	-8.70	19.44	2		1100	21.38		
20	20.51	10.74	-8.70	19.44	2		129	2.51		
20.51	25	10.74	-8.70	19.44	3	27.34	971	18.87	60.66	1658.4
25	30	11.70	-8.70	20.39	3		1100	22.43		
30	34.18	12.43	-8.70	21.13	3		916	19.35		
34.18	40	12.43	-8.70	21.13	4	41	1264	26.70	72.06	2954.5
40	47.84	13.03	-8.70	21.72	4		1612	35.02		
47.84	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	4944.0
60	61.52	13.98	-8.70	22.68	5		316	7.17		
61.52	68.67	13.98	-8.70	22.68	5		1516	34.38		
68.67	70	31.26	-20.84	52.09	Parapet	68.67	368	19.17	46.52	3194.4
70	72.33	31.26	-20.84	52.09	Parapet		525	27.35		
68.67	70	31.95	-21.30	53.26	Windscreen	68.67	280	14.91	53.41	3668.0
70	80	31.95	-21.30	53.26	Windscreen		723	38.50		
68.67	70	13.98	-8.70	22.68	Stair	68.67	20	0.45	3.60	247.1
70	80	14.38	-8.70	23.08	Stair		120	2.77		
80	81.67	14.74	-8.70	23.44	Stair		16	0.38		
								Totals	412.14	17454.3

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

$$C_s >= 0.01$$

Floor Weights:

Floor Weights

	Curtain Wall (lf)	Load (psf)	Height (ft)	CMU Wall (lf)	Load (psf)	Height (ft)	Partitions (lf)	Load (psf)	Height (ft)	Typical Floor Area (sf)	Load (psf)	Roof Area (sf)	Load (psf)	Mechanical Roof Area (sf)	Load (psf)	Equipment Load (lbs)	# of Units	Total Weight (k)
Roof	794	20	10.83				60	20	13			25040	27	4140	65	35000	2	1203
5th Floor	823	20	13.67				572	20	9	28292	65							2167
4th Floor	823	20	13.67				514	20	9	29422	65							2230
3rd Floor	823	20	13.67				514	20	9	29422	65							2230
2nd Floor	823	20	13.67				514	20	9	29422	65							2230
																Total		10060 k

Seismic Base Shear

Level	h_x	h_x^k	W	$W * h_x^k$	C_{vx}	F	M	Σ M
1	0.00	0.000	0	0	0.000	0.00	0	0
2	13.67	23.736	2230	52931.28	0.061	17.48	238.9522	238.9522
3	27.34	54.949	2230	122534.9	0.142	40.47	1106.339	1345.292
4	41.00	89.760	2230	200160.3	0.231	66.10	2710.141	4055.433
5	56.68	132.863	2167	287906.9	0.333	95.08	5389.048	9444.481
Roof	68.67	167.620	1203	201606.3	0.233	66.58	4571.947	14016.43
			Totals	**865139.6**	**1**	**285.70**	**14016.43**	

$V = C_s * W =$	0.0284 * 10060	=	**285.704**	k

T	k
0.50	1
0.92	**1.2105**
2.50	2