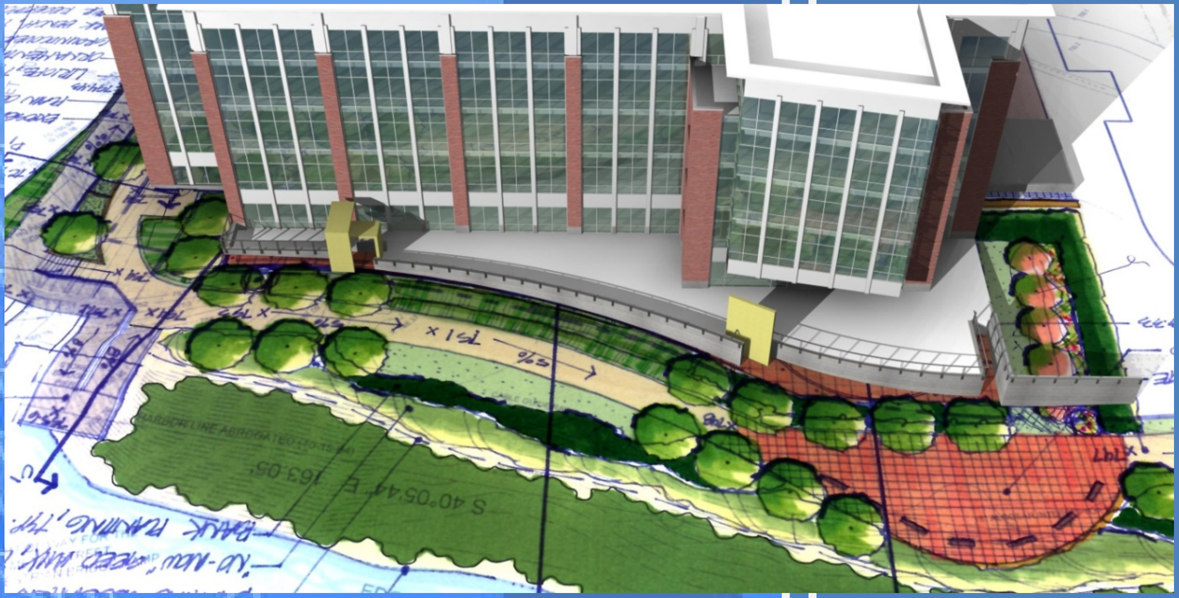
American Eagle Outfitters: Quantum III



Sam Jannotti

Structural Option

The Pennsylvania StateUniversity

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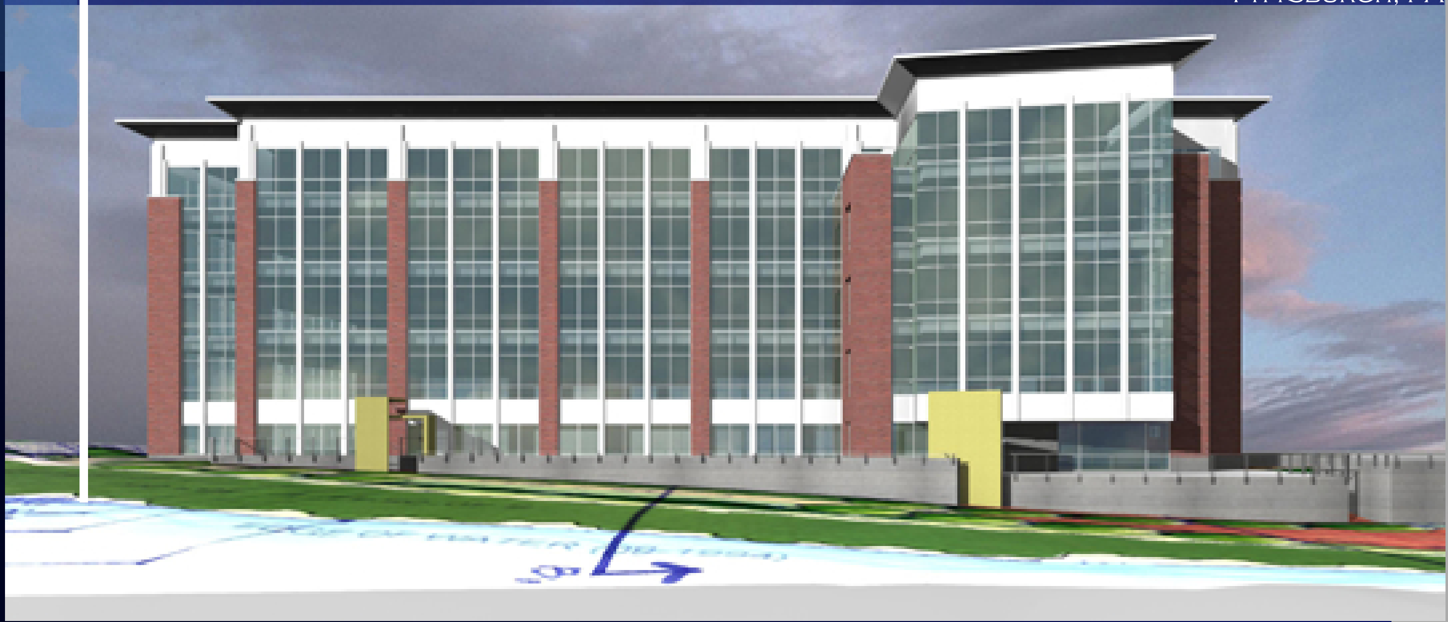
American Eagle Outfitters

Quantum III: SouthSide Works

Pittsburgh, PA



The Project Team

Owner: American Eagle Outfitters
Architect: The Design Alliance Architects
Construction Manager/Developer: The Soffer Organization
Structural Engineer: Atlantic Engineering Services
MEP Engineer: Tower Engineering
Civil: The Gateway Engineers, Inc.
Landscape: Environmental Planning and Design

Structure

- Wide flange columns, beams, and girders with composite lightweight concrete on steel deck
- Typical bays are 30' on an open plan
- Bathrooms, mechanical spaces, and elevators/egress located in center of plan, also housing two vertical trusses to counteract lateral loads
- 60 ton auger cast piles and 3000 psi spread foundations

Architecture

- Transparency through curtain walls, mass shown through brick facade
- Composite aluminum panels and cornice unify building facades
- Open plan for future tenant fit-out
- Single vertical truss fully visible through curtain wall, demonstrating building structure

Building Statistics

Location: 19 Hot Metal Street, Pittsburgh, PA
Occupancy: Office
Size: 5 stories and 150,000 sq. ft.
Construction Dates: May 2007-October 2008
Cost: $16 million Building Shell and Core
Delivery Method: Design-Bid-Build

Lighting and Electrical

- 277/480 V, 3 phase, 4 wire system dropped down to a 208/120 V system
- Transformers present at each level in panel room
- At least two panels for each voltage level on each floor
- Only lighting included in contract is emergency and egress fluorescent tubes, exit signs, and loading areas with metal halide mounted on walkways and in trees for aesthetic purposes
- Each floor lighting to be furnished by tenant

Mechanical

- Two air handling units providing 120,000 CFM total
- 30% or 36,000 CFM outside air
- Heat recovery/enthalpy wheels operate at 64% efficiency for cooling and 77% efficiency for heating

Samuel M. P. Jannotti
Structural
http://www.engr.psu.edu/ae/thesis/portfolios/2008/smj167/

Executive Summary

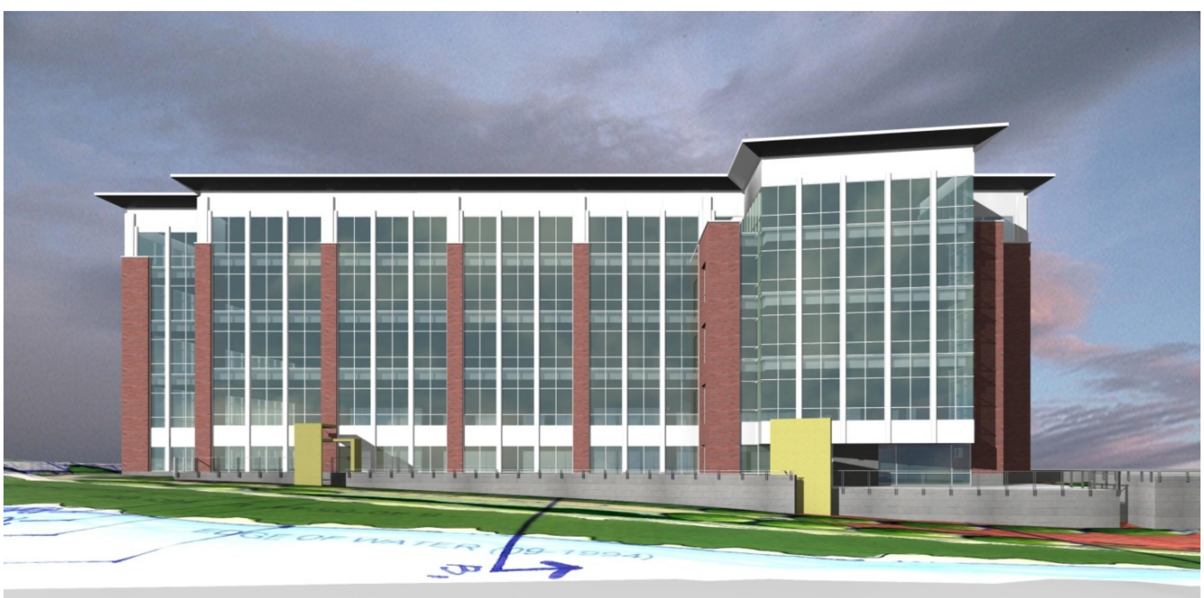
American Eagle Outfitters: Quantum III is a steel framed office building located in the South Side Works of Pittsburgh, Pennsylvania. Design changes were introduced including moving the building to Oakland, California and increasing it's profile by two stories. This report analyzes the structure of this building and it's adequacy on the basis of currently accepted national codes, economy, and flexibility.

Lateral systems were designed to withstand seismic category E design forces. This was achieved through numerous framing layout iterations and a preliminary beam, column, and brace design. Torsion, redundancy, and p-delta effects were all taken into consideration for design. The completed preliminary analysis was checked for story drift limitations for both wind and seismic forces to demonstrate the difference in Pittsburgh, Pennsylvania and Oakland, California requirements.

The redesign of building shell elements was completed as well. Window assemblies were analyzed for their mechanical and architectural properties. A double glazed window with a spectrally selective tint was chosen. Satisfying a wide range of aesthetic uses, it also provides a U-factor of 0.3, greatly reducing heating and cooling load losses for QIII. The building scale was changed from 67' to over 96' tall, possibly requiring rescaling of building elements. Additionally, shell elements were changed to better reflect the aura of Oakland, California.

Mechanical system design was performed for the existing and proposed Quantum buildings. They were compared based on their overall efficiency and heat loss through curtain wall systems. The added two floors greatly increased heating and cooling loads, so efficiency was calculated based on relative percentages.

The following report describes the considerations and details that composed the studies outlined above.



Acknowledgements

Completing this senior thesis could not have happened if it weren't for the help and patience given by all these family members, friends, design professionals, and firms:

To my family:
You taught me how to appreciate life and the people around me. Thank you for your continuing support and always keeping an open ear.

To my friends:
Thanks to Steve Reichwein for dealing with my one thousand questions per week, Jason Sambolt for his help with Trane TRACE 700 and mechanical design, and Gary Newman for making me step back and breathe once in a while. To the best time of our life, five years and counting!

Atlantic Engineering Services:
Tim Jones
Andy Verrengia
John Schneider
Chris Kim

American Eagle Outfitters

The Design Alliance Architects

The Soffer Organization

Tower Engineering

The Gateway Engineers, Inc.

Environmental Planning and Design

Finally, To the AE Faculty:
Kevin Parfitt
Andres Lepage
Ali Memari
Robert Holland
The entire AE Faculty and Staff

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1. Introduction

The Architectural Engineering Senior Thesis is the culmination of five years of foundation course work, resulting in a presentation and report that outline all facets of engineering study. The year long class involves analyzing the existing building, proposing a design change, and evaluating the new design. The class is conducive to gaining invaluable experience in typical engineering practices as a stepping stone to entering the industry.

Beginning in the fall semester, students analyze the building critically—from gravity loads to lateral force resisting systems and even seismic design details. Students build on their knowledge of the building through three technical reports that each focus on a separate aspect of architectural engineering. By the end of the fall semester, a significant foundation is placed, allowing each student to branch off into a depth study consistent with their focus in the Architectural Engineering curriculum.

The spring semester is composed of following a task schedule to achieve a design worthy of an engineer in training. It is highly dependent on the student's ability of to meet self-set deadlines throughout the semester. The requirement is an in-depth study reflecting knowledge the student obtained their respective focus. On top of this, they must demonstrate their wide spectrum of architectural engineering knowledge through two "breadth" studies. These result in a capstone final report and presentation to a faculty jury.

This report represents my five years of study in the Architectural Engineering curriculum. It is, without doubt, the capstone of my hard work within the program and represents my ability to learn engineering design methods both in class and independently. In addition, this report contains the results of a full year of study on American Eagle Outfitters: Quantum III. The report is divided into depth and breadth sections with appendices relating to each for ease of reference.

The primary goal of this report is to obtain a preliminary lateral frame design in Oakland, California. This was assessed based on effectiveness, constructability, and economy. Breadth areas were architecture and mechanical engineering. Wall assemblies were also considered and related to both breadths.

All materials submitted as part of the final report and senior thesis are available online at: http://www.engr.psu.edu/ae/thesis/portfolios/2008/smj167/. The report and all materials posted online and presented in this report are for educational purposes only and represent Sam Jannotti's personal views and design work. These materials in no way reflect American Eagle Outfitter's corporate or mercantile plans and were presented for the sole purpose of education.

2. Building Background

2.1 General Information

Quantum III is a product of the continuing expansion of American Eagle Outfitters Corporate Headquarters in the South Side of Pittsburgh, Pennsylvania. It is a genuine combination of structural design for flexibility and the blending of architectural tastes of the existing South Side of Pittsburgh with that of the developer, The Soffer Organization. At one end of Hot Metal Bridge, and bordering the Monongahela River lies Quantum III. The existing office building is five stories tall and contains loading, fire pump, and generator rooms on the first floor. The second through fifth stories have open plans for tenant fit-out.

Figure 1 – Location of AEO: QIII

Atlantic Engineering Services took QIII as a design-bid-build, core and shell project. The shell involves the building exterior and enclosures while the core contains layouts for elevators, stairs, mechanical shafts, telecommunications and bathrooms. They designed the steel framing system and strategically placed lateral force resisting systems to cause minimal interference with the open layout.

Quantum III is optimized for flexibility with 150,000 gross square feet of open layout. Floor to floor height for levels 2 through 5 is 13’-8” with the top and bottom story supplying extra space for added mechanical ductwork. Project construction is scheduled for May 2007 through October 2008 and total cost is estimated at $16 million.

2.2 Architectural Overview and History

American Eagle Quantum III will expand the corporate office and retail space provided by American Eagle Outfitters in the Pittsburgh, PA area while broadening the spectrum of services offered in South Side Works.

South Side Works formerly was the home of 40,000 immigrants who would walk to neighboring steel mills for work, but the collapse of the industry in the 1970's cleared the area. Since then, the local Bingham Street Church has been converted to studio residential spaces and the Jones and Laughlin Steel Mill has been converted to a retail and dining plaza. Fine cuisine and upscale retailers to top-end living units now occupy the 34-acre site of the mill. See Figure 2.

Figure 2 – View of South Side Works

2.3 Building Envelope Architecture

Quantum III will reflect the existing mood in South Side works with an envelope that emphasizes mass through brick façade while providing transparency through aluminum and glass curtain walls. The building is set atop a solid concrete retaining wall, and the large yellow colored mass in the forefront of the renderings is a "branding wall" featuring a larger than life American Eagle Outfitters logo. Due to cost issues, the branding wall has since been removed from the project.

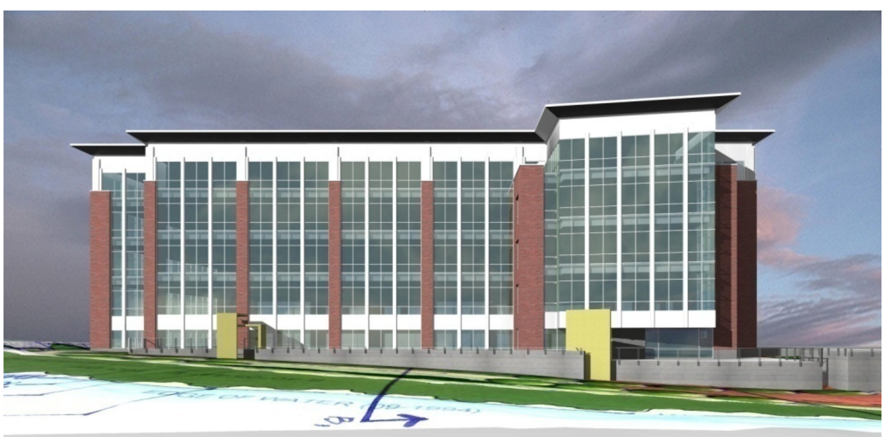


Figure 3 – North Perspective with Branding Wall

Vertical columns of façade brick backed by an airspace and 6” to 8” light gauge steel studs segment and add mass to a façade dominated by aluminum and glass on the south elevation. The north elevation includes this envelope but presents more brick with frequent large bay windows. Riverfront terrace, with the featured “Branding Wall” lines the north elevation as well. The east and west elevations are progressively clad with increasing amounts of brick façade, and the west elevation features the service entrances. Facades are all tied together with composite aluminum panel walls and a similar cornice.

The roofing system consists of fully adhered EPDM single ply membrane on rigid insulation; backed with 3”, 20 gauge, galvanized steel roof deck throughout. The deck has at least 3 continuous spans, and the rigid insulation is added to allow a ¼” per foot slope to drain water while providing an R-value of 30. The membrane is wrapped around the inside and top of the parapet to prevent leakage throughout the structure and wall systems.

2.4 Building Plan Architecture

American Eagle Outfitters South Side Works features an open plan featuring only those partitions required in the core of the building: where elevators, stairs, bathrooms, storage, and lateral resisting frames are present. The remainder of the plan is dotted with steel columns.

2.5 Zoning

B-2 new construction classified as B (business) in Pittsburgh County, Pennsylvania.

2.6 Structural Systems

The structural system for American Eagle Quantum III is primarily composed of wide-flange steel columns and composite beams. The typical floor is 3” composite light weight reinforced topping slab on 2” 20 gauge steel deck. Girders are typically W24x55 with W18x35 infill beams spaced at 10’ on center. The roof is constructed of W16’s with W12 infill beams with a portion of composite slab to support the mechanical units. A windscreen surrounds the mechanical equipment to counteract wind forces and hide it from sight of pedestrians below. Connections are mainly simple shear connections. Columns are typically W10’s and W12’s placed on a 30’x30’ grid.

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.

2.7 Mechanical Systems

QIII has two 35,000 pound rooftop air handling units providing a total 120,000 CFM. Heat recovery wheels are installed and operate at 64% efficiency for cooling and 77% efficiency for heating. The system is designed to use 36,000 CFM, or 30%, outside air. The boiler room is located on the fifth floor, simplifying HVAC system layout by placing the units and boiler room close vertically and horizontally. Hot water is supplied via two pumps operating at 66% efficiency, pumping 250 gpm. There are typically two VAV boxes per floor, regulating air flow vertically throughout the building.

2.8 Construction and Management

The delivery method is design-bid-build, with The Soffer Organization managing and developing the land. American Eagle Outfitters Quantum III went out to bid December 2006, and bids were selected based on economy, constructability, and quality. Groundbreaking occurred in May 2007 and the building envelope and core construction is scheduled to be completed in October 2008.

The contractor is responsible for the demolition of existing steel mill foundations, estimated at +/- 40’ thick, with their location to be field verified. The majority of the site is covered by the proposed building, with roads on two sides and the Monongahela River on another—construction will therefore be tight. Storage of materials and the construction process will require thinking outside of the box to limit interference with Pittsburgh area traffic and congestion.

2.9 Electrical Systems

American Eagle Outfitters Quantum III has 277/480 V incoming power in a 3 phase 4 wire system including a 150 kVA transformer, two 277/480 V panelboards, and four 208/120 V panelboards on the first floor. There is a separate panel for low voltage lighting as well. Floors 2 and 5 have four panels of each voltage while floors 3 and 4 have similar layouts, but only have two 277/480 V panels. Finally, power is transferred between floors via 2000A vertical bus systems.

2.10 Lighting Systems

Lighting fixtures will be provided only in stairs, emergency egress areas, and the receiving and storage facilities. Four foot fluorescent fixtures will be pendant mounted in receiving and storage, and fixtures are ceiling mounted in stair wells. Metal halide is provided for the terrace area, building façade, and aesthetically mounted in trees. Fluorescent bulbs must have a minimum of 80 color rendering index (CRI) while metal halide lamps must achieve a CRI of 70.

The curtain wall façade will provide natural light throughout the interior of Quantum III while allowing for spectacular views of the Pittsburgh skyline and historical bridges. Building tenants must supply all other lighting and electrical components to suit individual needs.

2.11 Fire Protection

All exit passageways, storage rooms over 100 square feet, and elevator shafts are rated for 2 hours, while stairwells are rated for 1 hour. A smoke control system is proposed though not required by code. The structural frame and other floor and roof construction require no specific fire protection—therefore no special protection is provided.

Two fire pumps supply water to the two sprinkler zones, with sprinklers located 12’ on center—spacing is lowered where NFPA has special wall spacing requirements. Also, standpipes are located in each of the two stairwells of American Eagle Outfitters: Quantum III. One stairwell is located on the exterior wall towards the east corner of the building, and the other is an interior stairwell on the north half of QIII.

2.12 Transportation

There are three entrances/exits on the first floor with two exits on each floor above. Loading and unloading areas are provided on the north sides of the building. The loading docks are angled roughly 45 degrees to allow a semi trailer and trash collection to fit on the northeast side of the site, given the tight edge clearance of the building on all sides. The northwest side contains a separate entrance and overhead partitioned doors in each bay, resulting in six separate loading areas.

Three elevators are provided. The first is a cargo elevator provided by the interior stair, while the remaining two border the core bathrooms and mechanical shafts. These two elevators are open to future tenant use.

2.13 Communications

Two way communication between the building tenants/operators and fire agencies is provided with each individual tenant installing personal communication needs. Service and data rooms are provided with their own VAV boxes on each floor and are aligned vertically for easy installation of multiple floor systems.

2.14 Project Team

- Owner: American Eagle Outfitters
    - http://www.ae.com/web/index.jsp
- Architect: The Design Alliance Architects
    - http://www.tda-architects.com/
- Construction Manager/Developer: The Soffer Organization
    - http://www.sofferorganization.com/
- Structural Engineer: Atlantic Engineering Services
    - http://www.aespj.com/index.html
- MEP Engineer: Tower Engineering
    - http://www.tei-usa.com/
- Civil: The Gateway Engineers, Inc.
    - http://www.gatewayengineers.com/
- Landscape: Environmental Planning and Design

3. Structural Depth

3.1 Existing Structural Systems

3.1.1 Geotechnical and Foundation 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. Foundations are 3000 psi concrete with 5000 psi, 16" end bearing 60 ton auger-cast piles. Reinforced concrete grade beams aid in counteracting lateral load uplift underneath the six vertical trusses as well as provide stability around the perimeter of American Eagle Outfitters Quantum III. Foundation stability is a pressing issue given the Monongahela River is but 45' away.

Figure 4 – Ongoing QIII Construction by Monongahela River

3.1.2 Floor Framing

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 5 – Typical Architectural Floor Plan.

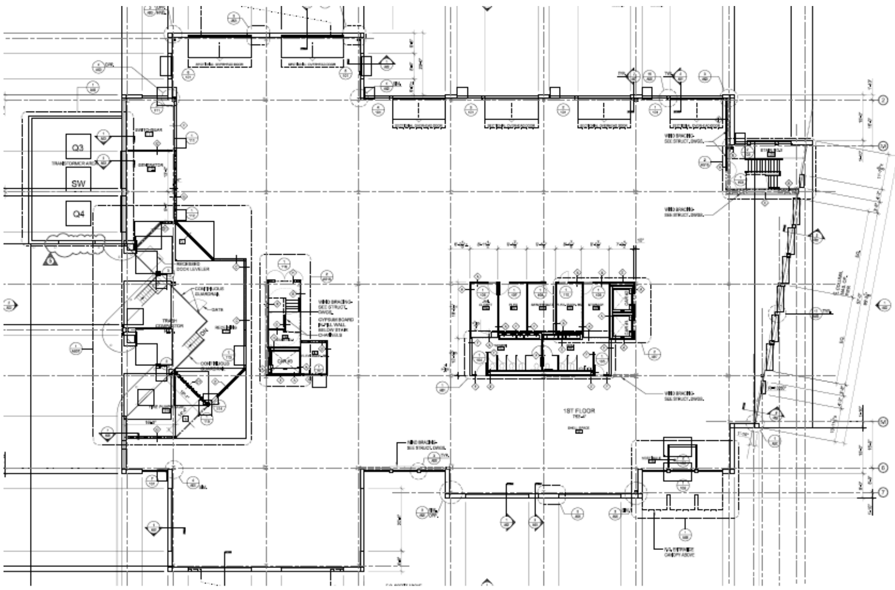


Figure 5 – Typical Architectural Floor Plan

Figure 6 – Typical Floor System Construction

As you can see from the architectural plan, partition placement is not 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 7.

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 Figure 7 and Figure 8 for the floor framing layout. 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 8 – Typical Floor Framing.

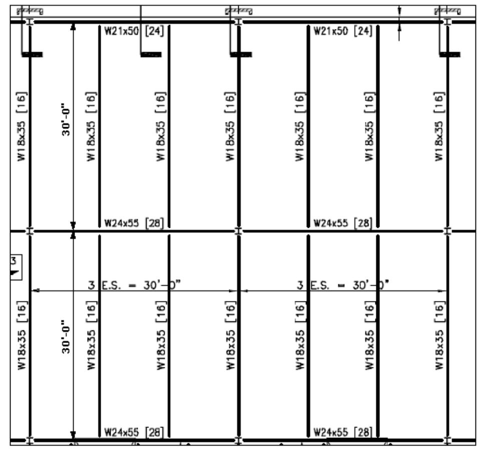


Figure 7 – Typical Bay

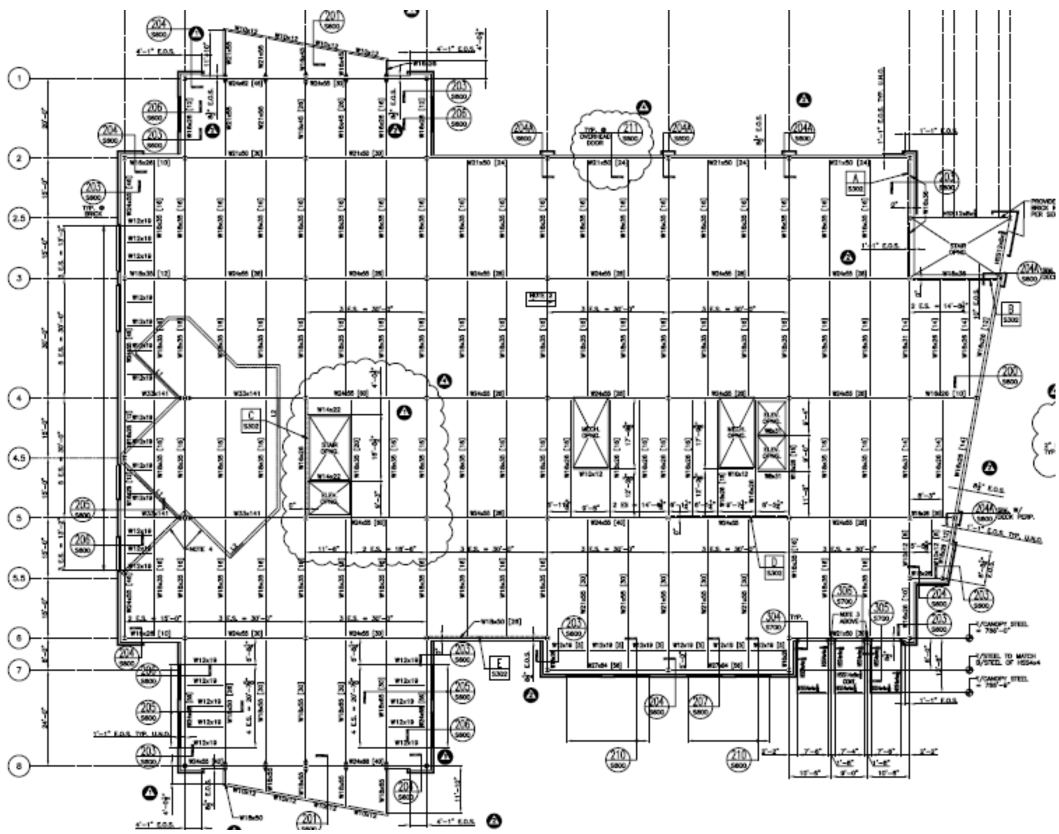


Figure 8 – Typical Floor Framing

3.1.3 Gravity System Columns

Typical columns in AEO: QIII consist of W10's and W12's. Splices are typically located four feet above the top of slab. The fifth floor contains additional columns bearing on transfer beams to support davit pedestals. Columns are placed on a 30' by 30' grid typically.

3.1.4 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 9, their placement was based on resisting interference with the open plan. Also, on the next page are elevations of the vertical trusses in Figure 10 and Figure 12.

Figure 9 – Vertical Truss Locations

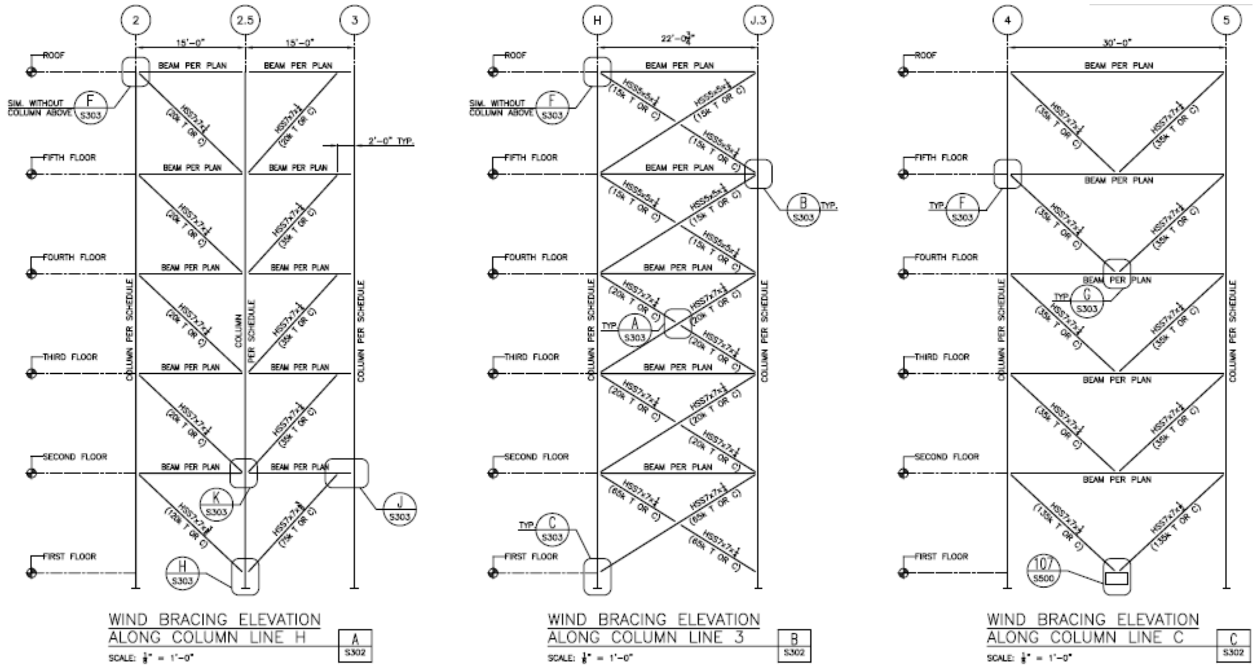


Figure 10 – Vertical Trusses A, B and C (VT-A, B, C)

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. Braces are HSS7x7's with lateral frame columns ranging from W14x82's to W14x193's. A standard inverted V-truss brace connection is detailed below.

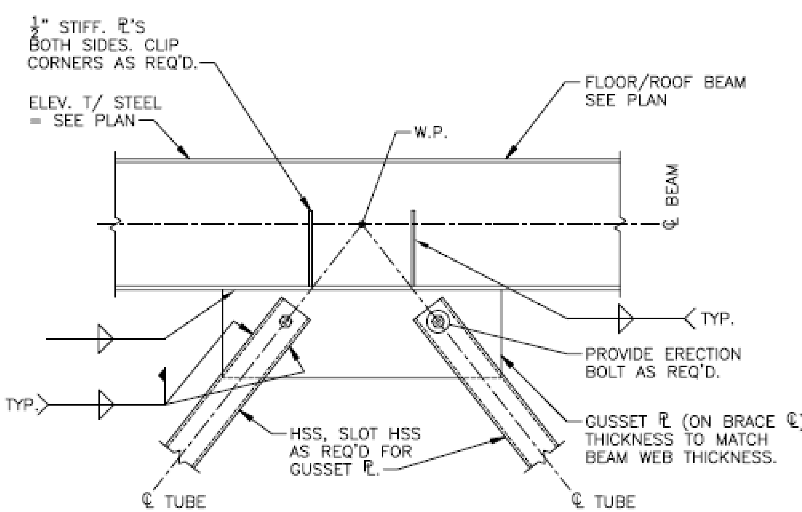


Figure 11 – Brace Connection Detail

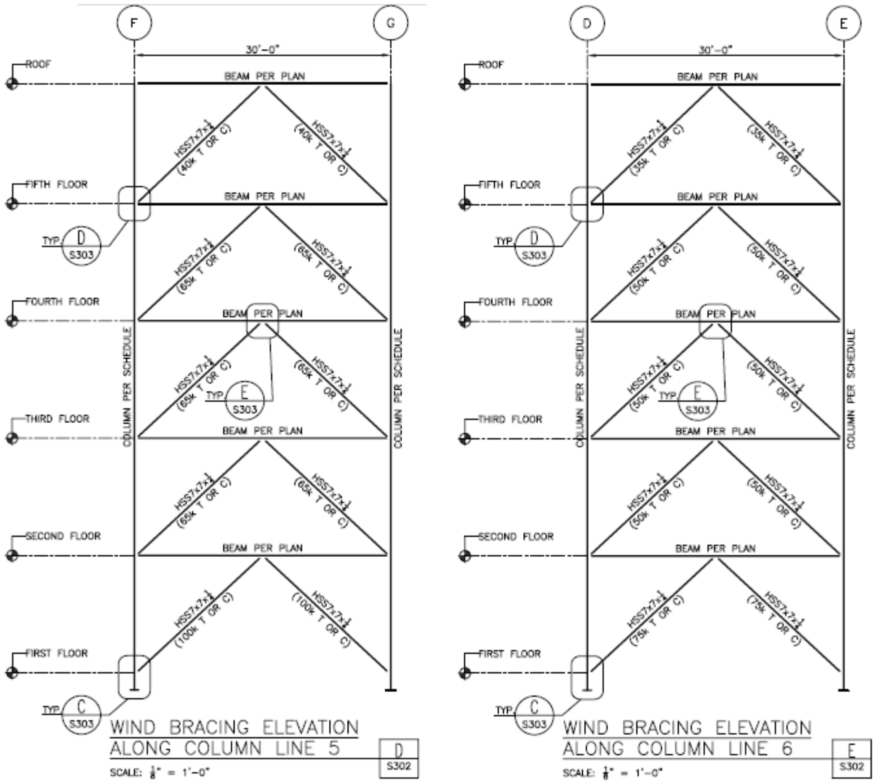


Figure 12 – Vertical Trusses D and E (VT-D, E)

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

3.1.5 3-D Model Images

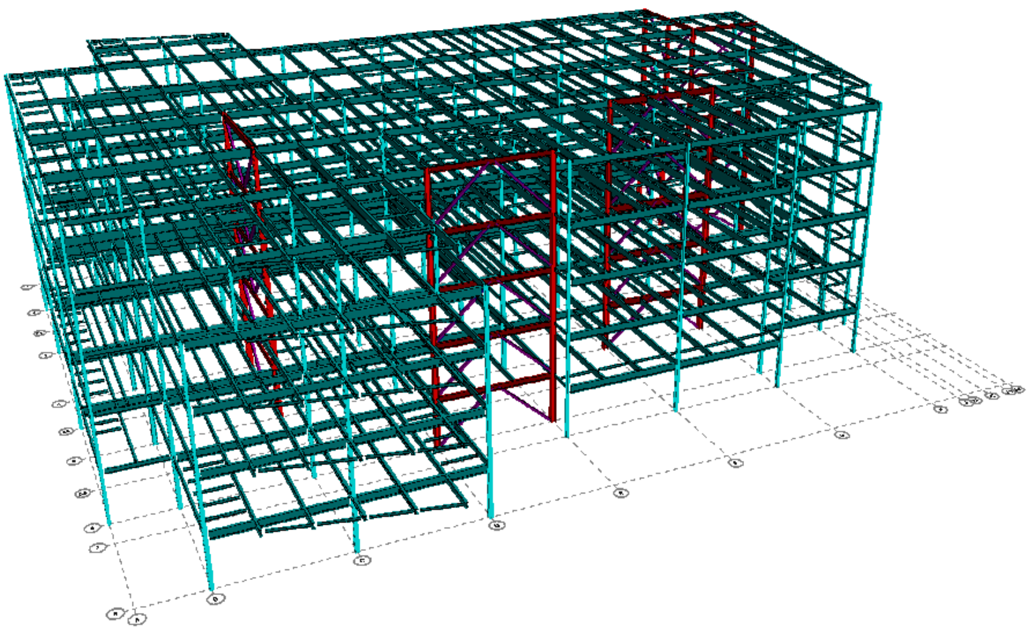


Figure 13 – 3D View from West Building Corner

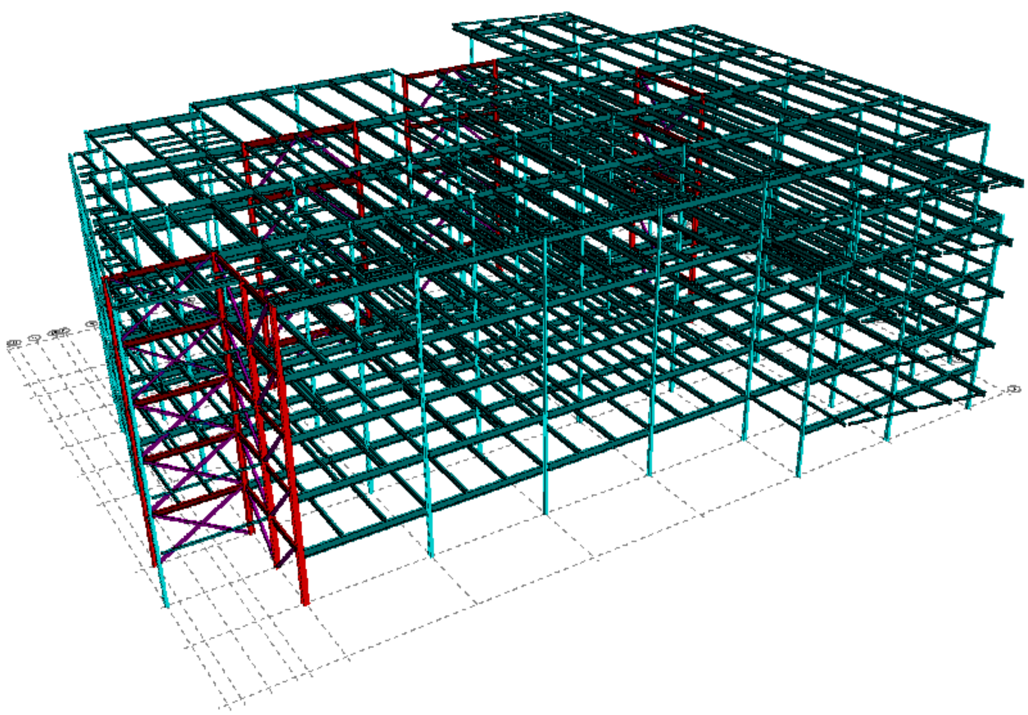


Figure 14 – 3D View from East Building Corner

3.2 Codes and Material Properties

3.2.1 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. Also, California State amendments and Oakland City amendments were analyzed. Upon inspection no amendments directly affected the following analysis.

3.2.2 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

3.3 Existing System Loads and Criteria

3.3.1 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.

$1.4(D)$
$1.2(D) + 1.6(L) + 0.5(L_r)$
$1.2(D) + 1.6(L_r) + (0.5L \text{ or } 0.8W)$
$1.2(D) + 1.6(W) + 0.5(L) + 0.5(L_r)$
$1.2(D) + 1.0E + 0.5L$
$0.9(D) + (1.6W \text{ or } 1.0E)$

3.3.2 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 16 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 in Appendix A.

Dead Loads			
Component	Typical Floor	Roof	Mechanical Roof
Concrete Slab	38		38
Metal Decking		2	
Flooring/Ceiling	3	4	3
M/E/P	7	10	7
Rigid Insulation		9	
Membrane		2	
Total Dead Load	48	27	48

Figure 15 – 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 16 – Mechanical Unit Surface Loads

3.3.3 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)

3.3.4 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 3.1.2 Floor Framing.

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	

3.3.5 Existing Building Wind Criteria

A comparison of wind pressures acting on the main wind force resisting system in Pittsburgh, Pennsylvania is described below. Since the seismic forces in southwestern PA are minimal, wind shears control the design of the lateral force resisting systems. The wind criteria determined for Oakland, California are presented in Appendix B.1.

Assumptions

Building Height (h)	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

3.3.6 Existing Building 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.

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

3.4 Basis for Structural Redesign

Evidence of American Eagle Outfitters current expansion is apparent in Pittsburgh, Pennsylvania. In the past few years, AEO has had two corporate expansions, of which Quantum III is the last installment. Michael Sandretto did a study on Quantum II just last year in AE 481W and 482. The fast turnout of additional corporate office buildings lend to the belief that more Quantum structures are on their way.

As a response to the rapid growth, American Eagle Outfitters could propose expanding with a corporate headquarters on the west coast. To save on design costs, a similar building to Quantum III could be constructed in Oakland, California. The new west coast headquarters must consider the large market the office space must tailor to—so two typical floor layouts will be added in QIII’s elevation.

(Note this in no way reflects the actual plans of American Eagle Outfitters and is proposed for the sole purpose of this structural depth.)

3.4.1 Gravity System

The floor plan on the new American Eagle Outfitters: Quantum building will also reflect the need for flexibility. Therefore, the dead and live loads applied on QIII will remain unchanged.

3.4.2 Lateral Force Resisting Elements

Given the seismic design considerations of California, a complete redesign of the lateral systems must be carried out. The original QIII design was in Pittsburgh, Pennsylvania; and was controlled by wind. Due to the large seismic induced forces present in California, lateral systems must be scaled up significantly. Column, brace, and girder sections must all increase as well. Special care will be taken in designing the details for the new Quantum building to ensure safety of the occupants in the event of an earthquake.

Moving the building to a new location presents many new factors when considering a lateral system redesign. The possibility of requiring additional vertical trusses will be met considering the effect of each truss on the existing open floor plan. Also, the higher cooling loads necessary in Oakland can result in the rooftop mechanical unit loads being increased. As a result, seismic acceleration and equivalent loads can grow. As with any engineering task, construction economics will be a considerable factor in the redesign of the lateral systems. The redesign of the lateral force resisting system will take account of all these factors throughout the following pages.

3.4.3 Design Goals and Scope

Due to the inherent complexities of moving a building design to a new site, the goal is to reach an adequate preliminary design for the lateral force resisting system. In this respect, building geometry, redundancy, and the development of plastic hinges throughout the vertical trusses will be taken into account. The lateral force resisting systems will be designed based on strength. Additionally, a preliminary drift evaluation under both wind and seismic loads will be determined to solidify the controlling case.

Overall, the scope of this study is to gain an understanding of design methods used in the architectural engineering field. With experience in East Coast design methods, the move to West Coast provides the daunting task of designing lateral systems to resist earthquake induced loads. The three technical reports completed last fall shrink in comparison to this study on a number of issues. With that said, the following pages outline the precautions taken to design a building to resist and withstand earthquake induced forces, not only to allow the safety of building occupants, but those people inhabiting and travelling through neighboring sites.

3.5 Proposed Gravity System

3.5.1 Gravity Framing

As stated earlier, dead and live loads remain unaltered from the previous Quantum III design. The result is a gravity system not unlike the existing structural sandwich. RAM Structural System was used to obtain the preliminary gravity beams, girders, and columns.

Two typical floors, each at 13'-8" were inserted above the fourth floor. The result was the minor increasing of lower level column sections. Also, the sections that were designated as part of the frame system were altered to be gravity members alone. This provided the minimal allowable design for girders and columns entering into the lateral force resisting system, satisfying the requirement for all frame girders to withstand gravity forces neglecting the truss braces. Shown below is a simple comparison of existing versus new gravity members throughout QIII's structure.

Gravity Member Designs							
Level		Column F3		Girder C3-D3		Infill Beam	
Existing	New	Existing	New	Existing	New	Existing	New
Roof	Roof	W12x40	W12x40	W21x44	W21x45		
5th	7th	W12x53	W12x53	W24x55 [28]	W24x68 [24]	W18x35 [16]	W16x31 [18]
4th	6th	W12x53	W12x53				
3rd	5th	W12x72	W12x72				
2nd	4th	W12x72	W12x72				
	3rd		W12x96				
	2nd		W12x96				

Figure 17 – Gravity Member Comparison

3.5.2 Gravity Frame Detailing

At this point, the level of detail in the gravity system is sufficient to conduct a preliminary lateral force resisting system design. To continue with the depth, a certain number of details were neglected because of their minimal impact on the lateral frame design:

1) Torsion of beams and girders eccentrically supporting shell elements
2) Infill beams around floor openings
3) Reinforced exterior masonry walls at the service entrance on the first floor

3.6 Proposed Lateral Frame Design

3.6.1 New Wind Criteria

Oakland, California has different wind criteria which are outlined below. The actual wind force calculations were completed using an Excel spreadsheet adapted from Technical Report 1. They are available in Appendix B.1.

Assumptions

Building Height (h)	96.64' to Roof T.O.S.
Basic Wind Speed (3 second gust)	85
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)	0.85, 0.88

3.6.2 Wind Design Methodology

Wind pressures were determined using Microsoft Excel (1), and then plotted on a 2-D scale model of the building in AutoCAD. Using the inquiry function, the area of building enclosure was determined and multiplied to find equivalent forces (2). The wind forces were lumped at each floor level, and overturning moment and base shear were calculated in Excel based on each floor's height (3). At this point, lumped wind shears were applied on the diaphragm of an ETABS building model (4). Story drifts were then printed from ETABS, and inserted into another Excel spreadsheet that checked they meet serviceability requirements (5). The methodology is outlined below, and the applicable graphs and output for each step of the process is available in Appendix B.1.

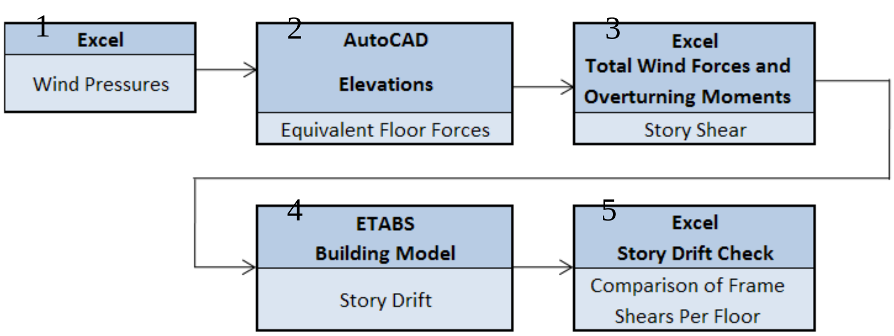


Figure 18 – Wind Analysis Methodology

3.6.3 Wind Story Shears and Overturning Moments

A comparison of North-South and East-West wind was performed to determine which would control story drift. Wind pressures are not assumed to control the strength of lateral force resisting braced frames. Therefore, shears are found to analyze the wind story drift limitation of H/400. Below are the equivalent story shears lumped at each floor level.

Total Wind Forces and Overturning Moments - North-South Wind													
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)	Overturning Moment M (k-ft)
Min	ft	Max	ft						in^2	ft^2			
0	0	81.3	6.77	9.03	-8.34	17.37	1	0	0	0	0.00	0.00	0.0
81.3	6.77	163	13.5	9.03	-8.34	17.37	2	162.5	214337	1488	25.86	52.66	713.1
163	13.5	180	15	9.03	-8.34	17.37			46165	320.6	5.57		
180	15	240	20	9.60	-8.34	17.94			158280	1099	19.71		
240	20	245	20.4	10.06	-8.34	18.40			11871	82.44	1.52		
245	20.4	300	25	10.06	-8.34	18.40	3	326.5	146409	1017	18.70	56.03	1524.5
300	25	327	27.2	10.45	-8.34	18.79			69907	485.5	9.12		
327	27.2	360	30	10.45	-8.34	18.79			83756	581.6	10.93		
360	30	409	34	11.10	-8.34	19.44			127943	888.5	17.28		
409	34	480	40	11.10	-8.34	19.44	4	490.5	188617	1310	25.47	59.32	2424.70
480	40	491	40.9	11.64	-8.34	19.98			27699	192.4	3.84		
491	40.9	573	47.7	11.64	-8.34	19.98			216316	1502	30.01		
573	47.7	600	50	11.64	-8.34	19.98	5	654.5	72545	503.8	10.06	61.28	3342.25
600	50	655	54.5	12.09	-8.34	20.43			143771	998.4	20.40		
655	54.5	720	60	12.09	-8.34	20.43			172789	1200	24.52		
720	60	737	61.4	12.49	-8.34	20.83			43527	302.3	6.30		
737	61.4	819	68.2	12.49	-8.34	20.83	6	818.5	216316	1502	31.29	60.91	4154.24
819	68.2	840	70	12.49	-8.34	20.83			48473	336.6	7.01		
840	70	901	75	12.85	-8.34	21.19			153597	1067	22.60		
901	75	960	80	12.85	-8.34	21.19	7	982.5	156961	1090	23.09	66.44	5439.84
960	80	983	81.9	13.17	-8.34	21.51			59355	412.2	8.87		
983	81.9	1065	88.7	13.17	-8.34	21.51			230825	1603	34.48		
1065	88.7	1080	90	13.17	-8.34	21.51	Roof	1147	26380	183.2	3.94	34.90	3334.33
1080	90	1147	95.5	13.47	-8.34	21.81	Roof		204445	1420	30.96		
1147	95.5	1303	109	13.47	-8.34	21.81	Roof - Stair	1147	4392	30.5	0.67	87.39	4771.06
1147	95.5	1291	108	30.31	-20.20	50.51	Windscreen		120960	840	42.43		
1147	95.5	1195	99.6	29.82	-19.88	49.70	Parapet		128352	891.3	44.30		
											Totals	478.92	25704.0

Figure 19 – North-South Wind Shears and Overturning Moments

Total Wind Forces and Overturning Moments - East-West Wind											
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)	Overturning Moment M (k-ft)
ft	ft						in^2	ft^2			
0	6.77	9.44	-8.91	18.35	1	0	0	0	0.00	0.00	0.0
6.77	15	9.44	-8.91	18.35	2	162.5	224188	1556.9	28.57	48.90	662.1
15	20	10.52	-8.91	19.43			139920	971.67	18.88		
20	20.38	10.93	-8.91	19.84			10494	72.875	1.45		
20.38	25	10.93	-8.91	19.84	3	326.5	129426	898.79	17.83	53.88	1466.1
25	30	11.61	-8.91	20.52			139920	971.67	19.94		
30	34.04	11.61	-8.91	20.52			113102	785.43	16.12		
34.04	40	11.61	-8.91	20.52	4	490.5	166738	1157.9	23.76	55.33	2261.79
40	47.71	12.17	-8.91	21.08			215710	1498	31.57		
47.71	50	12.17	-8.91	21.08	5	654.5	64130	445.35	9.39	57.14	3116.78
50	60	12.64	-8.91	21.55			279840	1943.3	41.89		
60	61.38	13.06	-8.91	21.97			38478	267.21	5.87		
61.38	70	13.06	-8.91	21.97	6	818.5	241362	1676.1	36.83	58.72	4005.06
70	75.04	13.43	-8.91	22.34			141086	979.76	21.89		
75.04	80	13.43	-8.91	22.34	7	982.5	138754	963.57	21.53	61.93	5070.78
80	89.17	13.77	-8.91	22.68			256520	1781.4	40.40		
89.17	90	13.77	-8.91	22.68	Roof	1146.5	23320	161.94	3.67	32.53	3107.67
90	96.46	14.08	-8.91	22.99			180730	1255.1	28.85		
96.46	100	14.08	-8.91	22.99	Roof - Stair	1146.5	6120	42.5	0.98	58.39	3187.72
100	109.5	14.63	-8.91	23.54			12750	88.542	2.08		
96.46	108.5	30.31	-20.20	50.51	Windscreen		43350	301.04	15.21		
96.46	108.5	30.31	-20.20	50.51			2550	17.708	0.89		
96.46	100.5	29.82	-19.88	49.70	Parapet		113664	789.33	39.23		
									Totals	**426.83**	**22878.0**

Figure 20 – East West Wind Shears and Overturning Moments

As you can see in Figure 19, North-South wind forces are greater, and will control the wind drift check of American Eagle Outfitters: Quantum III. A conservative estimate of the building weight resulted in a factor of over 60 against overturning. This is due to the large volume of the building in comparison to the surface area wind can act on. The overturning calculation is available in Appendix B.2.

3.6.4 Wind Induced Story Drift

The story drift of Quantum III as a result of wind induced forces was minimal at most. Since wind was not assumed to control story drift or strength design of the vertical trusses, they were designed using seismic loads. After a satisfactory preliminary design was achieved in ETABS, wind forces were applied on the model and drift was calculated. The minimum allowed story drift was equivalent to 0.40625 inches at the first floor. With large seismic force resisting vertical trusses, wind induced drift was limited to less than 1/1000th of an inch for a single story. This reinforces the assumption that seismic forces not only control the design of the lateral system but dominate it. The study of wind forces on AEO: QIII did not progress beyond this stage to allow ample time to analyze the complexities of earthquake induced forces.

3.6.5 New Seismic Criteria

As shown below, the seismic coefficients for California vary greatly from that of Pennsylvania. In order to meet code requirements for seismic design category E, the AISC Seismic Design Manual was used. Since American Eagle Outfitters: Quantum III contains both eccentrically braced frames and concentric braced frames, the conservative Response Modification Factor, Over-strength Factor, and Deflection Amplification factors were used. These values were that for special steel concentric braced frames. Supporting calculations are in Appendix B.3.

Occupancy Category	II
Seismic Use Group	II
Importance Factor (I)	1.0
Location	12th St., Oakland, California
Mapped Spectral Response Accelerations	
S_s = 1.522 g	
S_1 = 0.6 g	
Site Class	D
Site Class Factors	
F_a = 1. 0	
F_v = 1.5	
S_{MS}	1.522
S_{M1}	0.9
S_{DS}	1.015
S_{D1}	0.6
Seismic Design Category	E
Braced Frames are "Special Steel Concentric Braced Frames"	
Response Modification Factor (R)	6
Over-strength Factor (W_o)	2.0
Deflection Amplification Factor (C_d)	5.0

Seismic Response Coefficient (C_t)	0.02
Period Coefficient	0.75
Seismic Coefficient (C_s)	0.1054
Building Period (T)	0.949

3.6.6 Additional Lateral Frames

From the start of the lateral system redesign it was understood that the five frames present throughout American Eagle Outfitters: Quantum III will not be sufficient for seismic forces. To provide for redundancy and achieve an adequate preliminary design, a number of locations for additional braced frames were investigated. Existing vertical trusses are designated with a VT and potential new trusses are designated with an NT. See Figure 21 below.

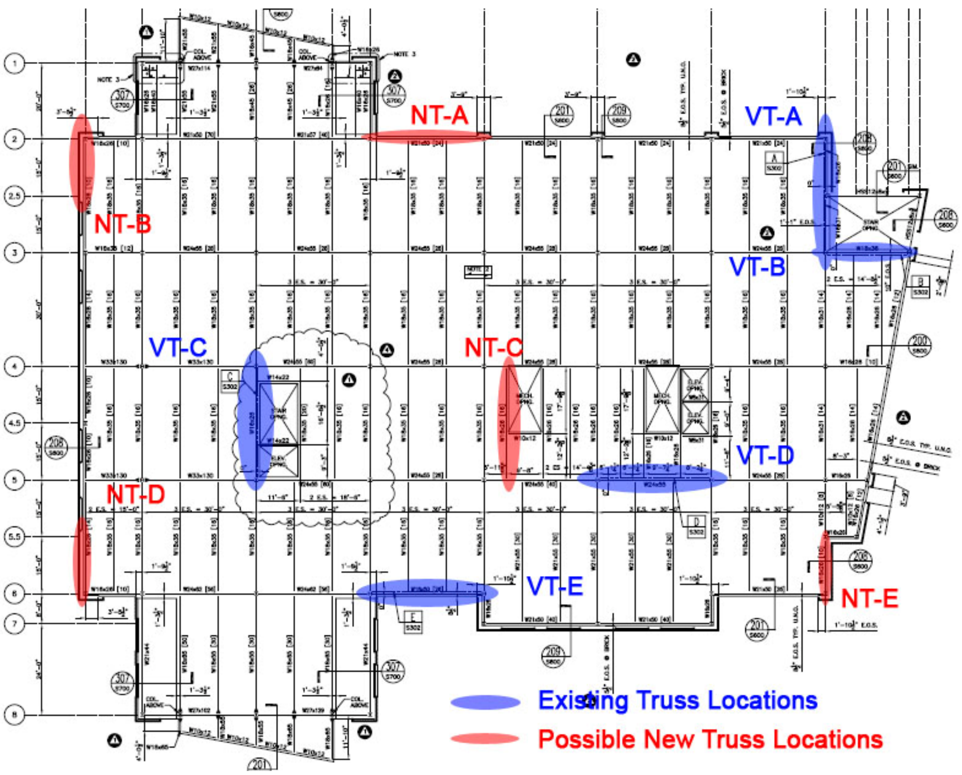


Figure 21 – Existing and Potential New Vertical Truss Locations

Most direct shear will be taken by the most rigid frames, so VT and NT-C would dominate the design in the y direction. VT-D, E, and NT-A are all 30' span trusses and will provide excellent shear resistance and redundancy in the x direction. NT-B, D, and E all span 15', and are therefore less efficient to resist story shears. However, their placement on the building shell maximizes their ability to resist torsional shears. Because the lateral force resisting systems are placed so asymmetrically, there exists the possibility of torsional irregularities. Not only could this increase the apparent seismic forces on the building through the redundancy factor and torsional shears, but can cause equivalent lateral force analysis to be not permitted.

3.6.6.1 Vertical Truss Elevations

As shown to the right, the proposed trusses B, D and E are slimmer, reducing their efficiency in resisting story shears. The X-bracing scheme also is inefficient in the number of connections it requires. On a per story basis, an X-braced frame requires five connections to be detailed whereas an inverted V-truss such as NT-A and C require only three. In a seismic controlled region such as Oakland, California; the detailing would vastly increase the building cost.

To combat the amount of detailing required NT-B, D, and E should be changed to inverted V-trusses beyond this preliminary design. In addition, the elevations below demonstrate the need for foundation detailing at the base of NT-B and D. They appear to be “floating”. Be assured this is not the case; the slab on grade is directly below the end of the truss outlined in blue. Therefore, the walls shown below are a combination of structural and retaining walls. Special reinforcing details are required to insure shear and axial forces are transferred to foundations and piles. (Note: Image below is of original QIII elevation and is used to demonstrate the foundation requirements below trusses NT-B and D.)

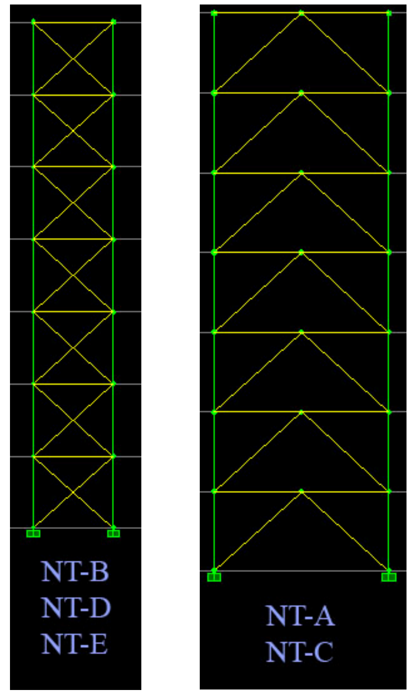


Figure 22 – Proposed Truss Elevations

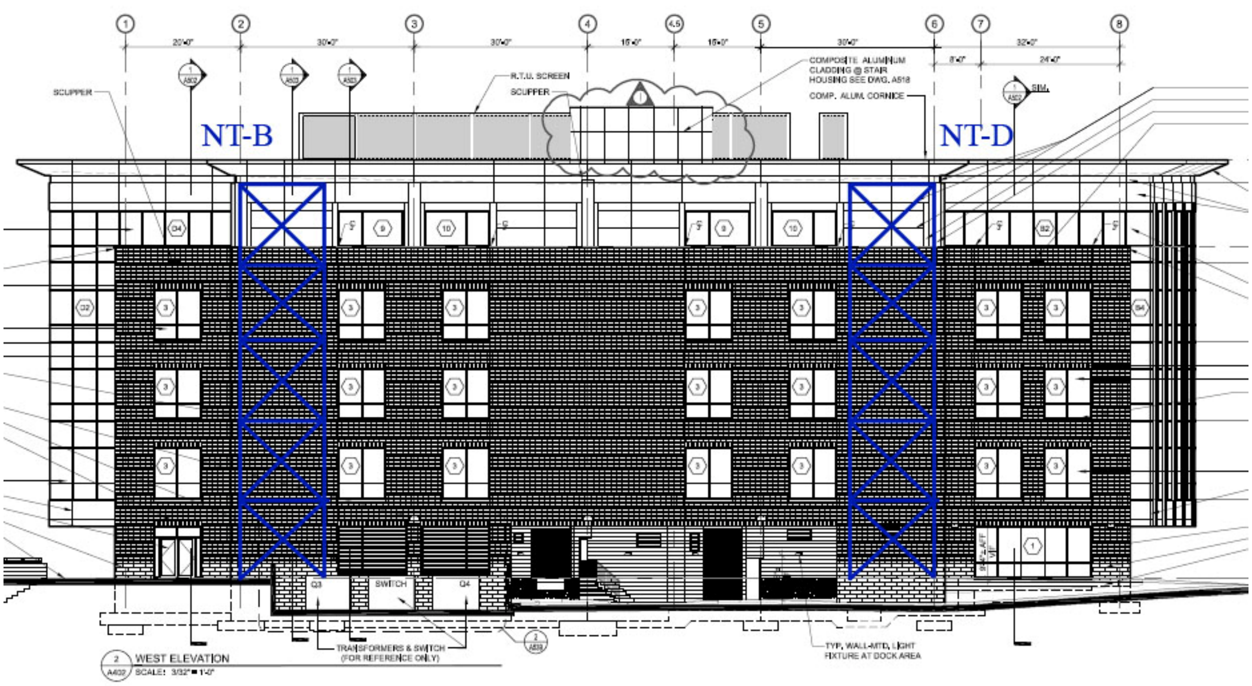


Figure 23 – West Elevation and NT-B and D

3.6.7 Seismic Design

A number of differing methodologies were employed in determining frame location and sizes for QIII. To get to the current preliminary design, the author went through over five possible designs of the lateral system, and with each iteration, discovering more efficient design methods. All methods employed RAM Structural System for story weights and SCBF beam gravity designs. Excel was used to determine equivalent seismic story forces. These forces were then compared to ETABS calculated results. Each method diverged in its approach to design the lateral system after this point. These anomalies in approach are outlined in Sections 3.6.7.2 and 3.6.7.3.

3.6.7.1 Seismic Story Shears

Utilizing story weights obtained from an updated RAM Structural System Model, equivalent seismic story forces and shears were found. By applying the respective building period and seismic coefficient (C_s), the forces, story shears, and overturning moments shown below were obtained. Also, the Excel and hand calculations were compared to ETABS model results shown in Figure 25.

Seismic Base Shear

Level	h_x (in)	h_x (ft)	h_x^k	W	W * h_x^k	C_{vx}	F	V	M	∑M
Roof	1146.50	95.54	265.917	1420	377655.3	0.146	311.34	311.34	29745.96	29745.96
7	982.50	81.88	220.117	3140	691057.6	0.267	569.71	881.05	46645.01	93290.02
6	818.50	68.21	176.009	3136	551963	0.213	455.04	1336.09	31037.52	124327.5
5	654.50	54.54	133.852	3141	420361.3	0.162	346.55	1682.64	18901.26	143228.8
4	490.50	40.88	94.022	3143	295511.5	0.114	243.62	1926.26	9957.992	153186.8
3	326.50	27.21	57.121	3148	179809.8	0.069	148.24	2074.49	4033.249	157220
2	162.50	13.54	24.307	3155	76683.93	0.030	63.22	2137.71	856.0834	158076.1
1	0.00	0.00	0.000	0	0	0.000	0.00	2137.71	0	158076.1
			Totals	**20281.9**	**2593043**	**1**	**2137.71**		**141177.1**	

		Cs	W (kips)		Total Force	
V = C_s * W =		0.1054	20281.9	=	**2137.71226**	**k**

T	k
0.50	1
0.95	**1.2245**
2.50	2

	Lower Bound	Exact	Upper Bound	Use
Cs =	0.05	0.169	0.1054	0.1054

Figure 24 – Seismic Base Shears

Seismic Base Shear Comparison			
Level	Hand Calculated k	ETABS k	Percent Difference
Roof	311.34	327.1	4.82
7	881.05	917.92	4.02
6	1336.09	1391.37	3.97
5	1682.64	1755.67	4.16
4	1926.26	2013.2	4.32
3	2074.49	2170.67	4.43
2	2137.71	2238.14	4.49

Figure 25 – Seismic Base Shear Comparison

3.6.7.2 Design A

Elevation and Framing

The layout used for the first design included all existing trusses as well as NT-B, C, and D. To place NT-C, columns moved less than 6' to be flush with the mechanical space opening shown in Figure 21. Beams that framed into this column were slightly elongated or shortened and had minimal effect on the beam design or structural sandwich.

Methodology

The first design involved trial and error through sizing and resizing frame members in ETABS. As expected, there are many faults with this approach. First, the systematic increasing of member sections to resist lateral loads proved to be fundamentally flawed. After adding NT-B and D, all y axis frame sections were simultaneously increased. In effect, by increasing the column sections of VT and NT-C, their stiffness increased as well. Therefore more seismic shear was distributed to this frame. This resulted in ever-increasing section sizes, never producing an adequate framing layout.

At this point in study, it was found that taking a counter-intuitive approach to lateral design was necessary. By downsizing the most rigid braced frame, more story shear is filtered to, in this case, NT-B and D. When all members finally passed the preliminary ETABS design, most columns for exterior wall trusses were a staggering W14x730. Conversely, interior truss column sections were W14x370 or smaller. When lateral frame dead and live loads were applied, these interior column sections were too small for combined loading. At this point, this design method was proved inadequate and other means were pursued.

3.6.7.3 Design B

This design on American Eagle Outfitters: Quantum III was the most in depth analysis performed for the structural depth. It utilized Excel spreadsheets, ETAB's, and RAM Structural System to get preliminary frame member sizes based on criteria outlined in *Methodology*.

Elevation and Framing

Due to the high relative stiffness of frames VT and NT-C and the apparent gravity loads, these trusses proved inadequate for preliminary design. If sections increased, more shear force would cause them to fail; decreased sections meant failure under gravity loads and minor combination loading. Therefore, both of these were removed. The remaining frames in Design B are shown below.

Figure 26 – Design B Frame Locations

V-trusses are researched as an alternative for X-braced frames NT-B and D due to the increased number of connections required. At 15' long, the member sizes and number of connections required for X-braces create a massive frame that is not efficient or economic. Inverted V-trusses interrupt vertical load paths of the braces and therefore require more shear strength in beams. The author believed this to be an adequate sacrifice to avoid more connection details. The elevation for NT-B and D is at right.

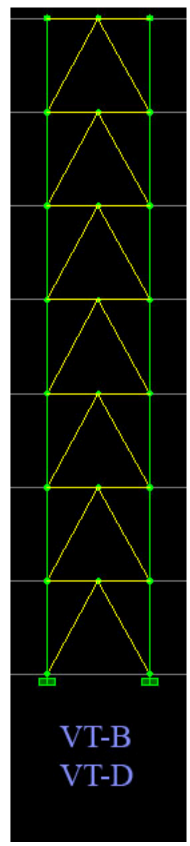


Figure 27 – Design B VT-B and D Elevation

Methodology

Design B utilized the full design process shown below to achieve a preliminary lateral framing design for American Eagle Outfitters: Quantum III. The flowchart has step by step descriptions and Appendix B.3 has each spreadsheet utilized in Design B. Had more time been available, further analysis would be performed. Further considerations past what is covered in this methodology is outlined in 3.6.7.4.

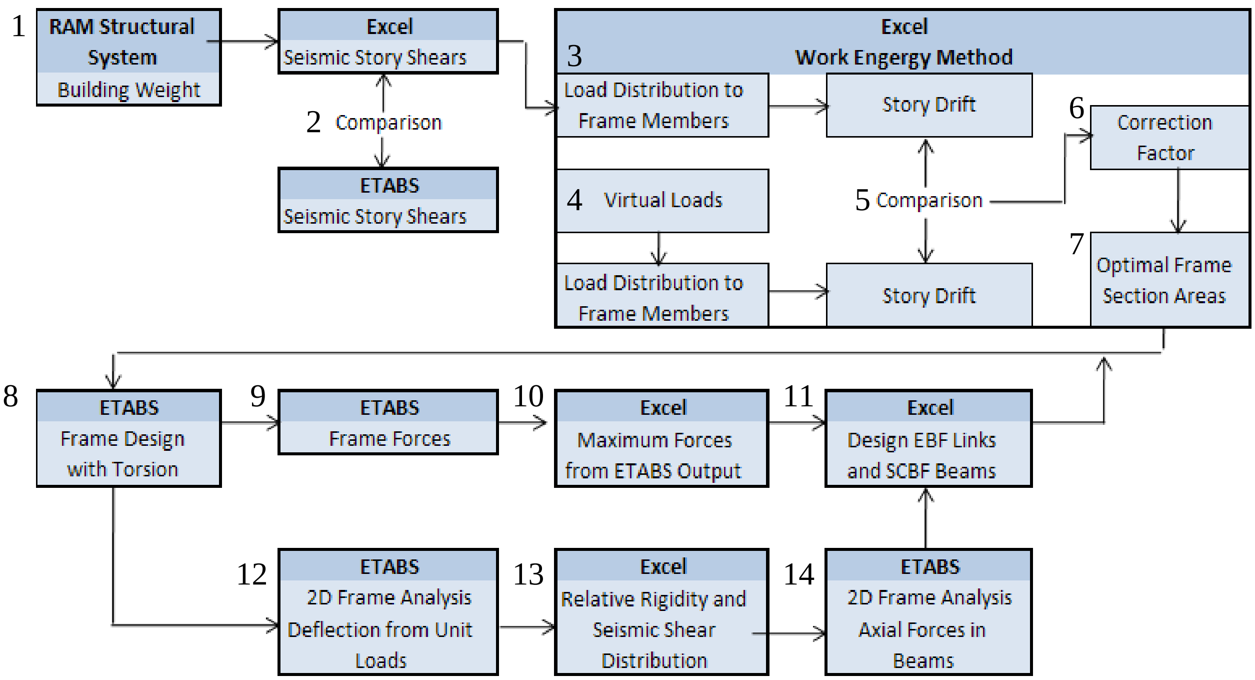


Figure 28 – Design B Methodology

As outlined previously, RAM Structural System was used to find story weights and add them into Excel (1). Story shears, calculated in Excel, were compared to those found in ETABS (2). The seismic shear forces, determined from Excel, were then divided by the number of trusses acting in each orthogonal direction. For frames running in the x-direction in Figure 26, total seismic story shear was divided by three. This assumes each frame is equally rigid and neglects torsion. For frames running in the y-direction, the seismic story shear was divided by two. NT-B and D are significantly less rigid and therefore provide less resistance to seismic shears as VT-A (3).

Using work-energy method, preliminary column sizes were found based on allowable drift. An Excel spreadsheet was developed to analyze virtual loads acting on each vertical truss, and calculate their expected story drift (4). The members optimal, cross sectional areas were then determined based on their allowable seismic drift and equivalent lateral forces through a correction factor (5-7). An example spreadsheet for this procedure is available in Appendix B.3.

The required frame sections were then put into an ETAB's model, and torsional effects were taken into consideration. Utilizing strength design, all members were sized against the 50 load cases ETABS considered (8). Frame forces were then input to Excel, which would locate the maximum shear and moment on beams (10). Frame designs were inserted to another,

separate ETABS model to find frame beam axial forces (12-14). Finally, utilizing more Excel spreadsheets, eccentric brace frame (EBF) links and special concentric braced frame (SCBF) beams were designed (12). The last steps (8-14) were an iterative process to optimize the design.

Results

The truss elevations to the left and on the next page display the wide flange sections used for Quantum III's lateral force resisting system. It was found that the effectiveness of a SCBF was attributed to: 1) its column sizes, 2) brace strength, and 3) beam size. It was in this order that frame sections were designed. Due to local buckling issues, only certain wide flange sizes could be used in seismic regions. The frames contain all allowable wide flange shapes as outlined in the AISC Seismic Design Manual. Utilizing ETAB's, braces were optimized through numerous iterations of the framing layout and member sizes.

The presence of W14x426's reinforce the author's belief on NT-B and D: their half-bay length greatly reduces the efficiency of the frame. With a smaller moment arm to each column, the bending force each truss can withstand is severely decreased. Larger member sections are needed to achieve the same strength as a full-bay length.

Large beam sizes are the direct result of brace sizing. With inverted V-trusses, beams must be designed to withstand 100 percent of the tension brace yield strength and 30 percent of compression brace nominal strength. The result is a large magnitude vertical force on the beam. In this design, shear forces could exceed 1000 kips.

As with shear forces, a beam's strength is determined by the area of the web alone. It is required that shear reinforcing is placed within the web to increase the cross sectional area resisting the shear forces. This will lead to an economic frame girder design. Another obvious fix for this problem is to allow members to transfer that vertical force on the beam, i.e. make the frame have multi-story X-braced frames. Continuous load paths transfer seismic force throughout the frame, allowing all members to supply their full cross sectional area for strength. By continuing design in this fashion, the uneconomic design of the beams shown in Figure 30 and Figure 31 can be eliminated. Figure 29 displays the stress ratio key for all frame elevations.

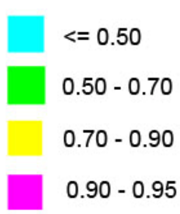


Figure 29 – Stress Ratio Key

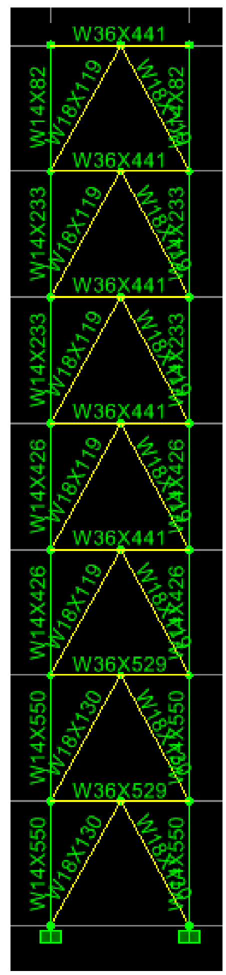
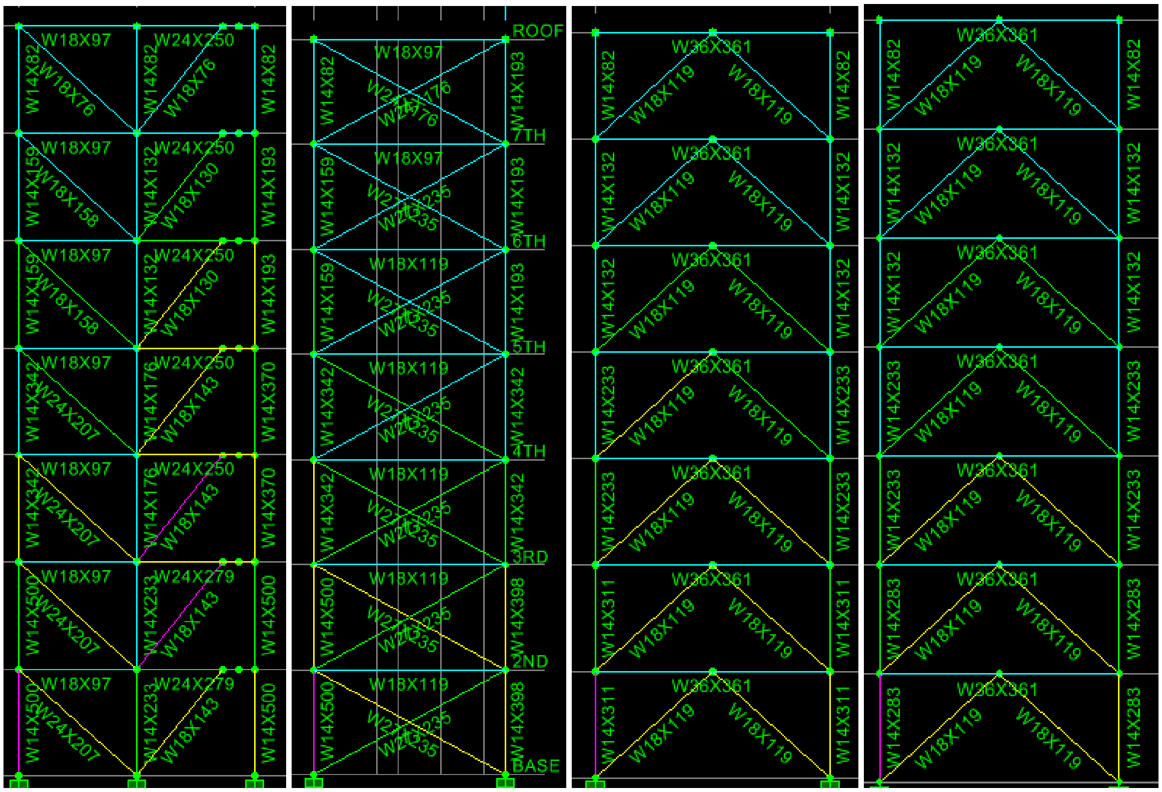


Figure 30 – NT-B and NT-D Elevation



VT-A *VT-B* *VT-D* *VT-E*

Figure 31 – Vertical Truss Elevations Under Controlling Loads

3.6.7.4 Continuing Design

The level of detail in this design was considered sufficient to move onto the architectural and mechanical breadths. Due to time constraints and the complexity of designing lateral systems to resist seismic shear, engineering of the lateral force resisting systems could not be carried further. The author recognizes the following items need to be engineered to develop a working lateral system that could be used in a building like Quantum III. Had more time been available, these items could have been investigated.

1. EBF beam design outside of the link
2. EBF and SCBF beam shear reinforcing design
3. EBF and SCBF connection details

Furthermore, the heavy beams used throughout inverted V-trusses in the current design are unacceptable. They are uneconomical and inefficient as are all inverted V-trusses in American Eagle Outfitters: Quantum III. For the next iteration, these frames should be modified into two story X-braced frames to achieve uninterrupted vertical load paths. Another option would be to add shear reinforcing to aid the web in resisting these large magnitude forces. As a result, the beam designs will decrease in size dramatically. Alternatives to NT-B and NT-D should also be

considered. Their lower rigidity in comparison to VT-A, B, D, and E not only makes them inefficient in terms of member sizes, but allows the diaphragm to rotate much more on the west side of the building relative to the east.

Eccentric braced frame beam links require shear reinforcing at the ends of the link and intermittently. A design of one instance of this was performed, but it was for a preliminary design not consistent with Design B.

3.6.7.5 Redundancy and Irregularities

Currently, the design does not contain any torsional irregularities. If the structure were to have this irregularity, the equivalent lateral force procedure would not be permitted to use in the design of Quantum III. The only irregularity the structure has is a re-entrant corner, requiring the increase of seismic forces by 25 percent for connection of diaphragms to vertical elements. The removal of one brace or connection within these frames does not reduce the strength of any story by more than 33 percent either. Therefore, the redundancy factor, ρ, remains 1.0.

3.7 Impact of Redesign

The addition of two floors in American Eagle Outfitters: Quantum III will change a number of factors throughout the structural system. Foundations will increase with larger building mass. Piles capacity can be increased, and their original capacity is outlined in 3.1.1 Geotechnical and Foundation Concerns. Gravity columns at the lower levels will increase as well.

As a result of the two additional floors, more wind and seismic overturning is present. With a high volume building like QIII, the factor of safety against wind overturning is large. In this case, it exceeds 60! Conversely, high volume buildings have higher mass each floor, lowering the factor of safety against seismic overturning. For Quantum III, the factor is only 10 against seismic overturning. This is still great enough to have no concerns of building overturning.

Finally, the new heating and cooling loads found in the mechanical system breadth require larger equipment on the roof. The original structural design was considered conservative its approach: two 35,000 pound units were expected to be placed on the roof. The structural system was designed for two 40,000 pound units in RAM Structural System. Since the building masses were obtained from this model, the impact of the new rooftop units is negligible to the equivalent seismic lateral forces and lateral and gravity design.

3.8 Structural Conclusion

The design was a success through providing the author with numerous design challenges never encountered in classroom work. Goals included learning the subtleties of seismic controlled lateral design. Considering the amount of detail this analysis went into, this was accomplished. Only a portion of design criteria were touched on because of the numerous detailing requirements in seismic regions. More so, this laid the foundation for the continuing education in lateral design that will be experienced in the workforce.

4. Architectural Breadth

4.1 Existing Building Architecture and Proposed Changes

As stated in the Building Background Section, the architectural taste of American Eagle Outfitters: Quantum III is characteristic of Pittsburgh, and materials imply this sense of place. New materials will be researched and analyzed based on their adequacy for building shell redesign. Quantum III's façade will be reevaluated to fit the scene of the surrounding architecture in Oakland, CA and local materials can be emphasized to give a sense of place.

With the addition of floors, shell architecture changes as a result of scale. Parapets and pedestals may need resized or redesigned based on this. Also, due to the structural requirements of a seismic controlled region, additional lateral resistance frames will be strategically placed within the interior and building shell to limit architectural interference. The preference for frame location is on the building shell. The interior of American Eagle Outfitters: Quantum III is open to allow for tenant fit out. Therefore, the focus of the architectural redesign is in the shell.

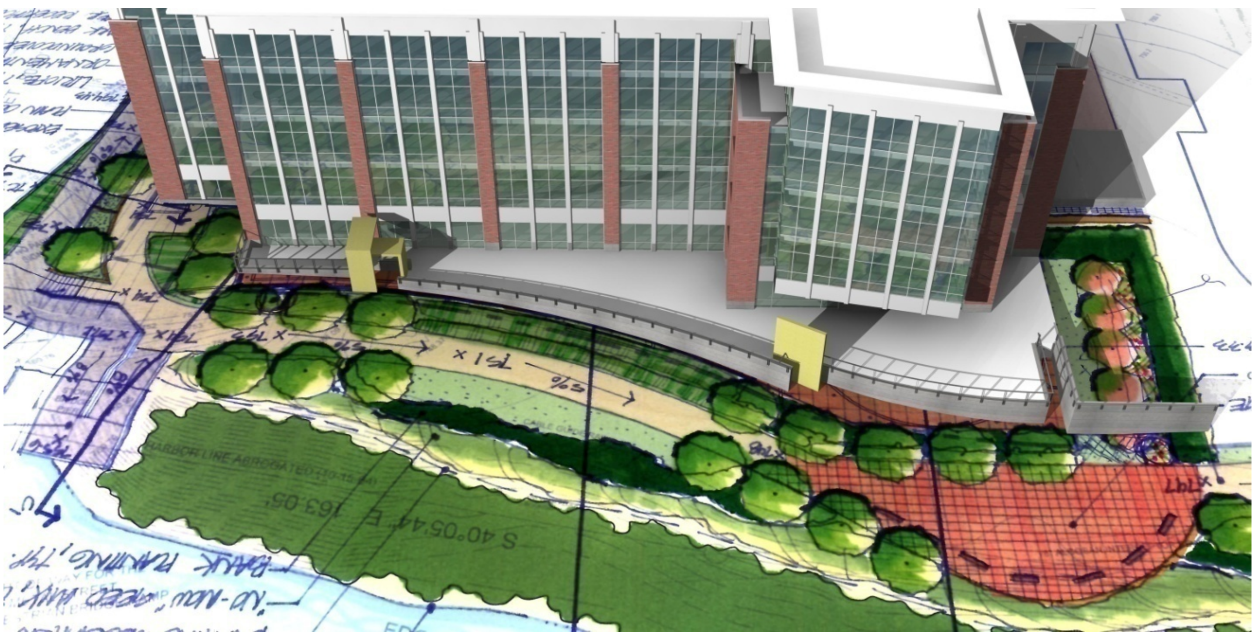


Figure 32 – North QIII Façade

4.2 Possible Frame Locations

As outlined in the structural depth, additional lateral frames must be added to American Eagle Outfitters: Quantum III if it is to resist California's seismic demands. The effects of each frame location on building architecture were weighed against the frame's usefulness in lateral strength. Following is the procession of designs considered and the corresponding architectural issues that arose as a result.

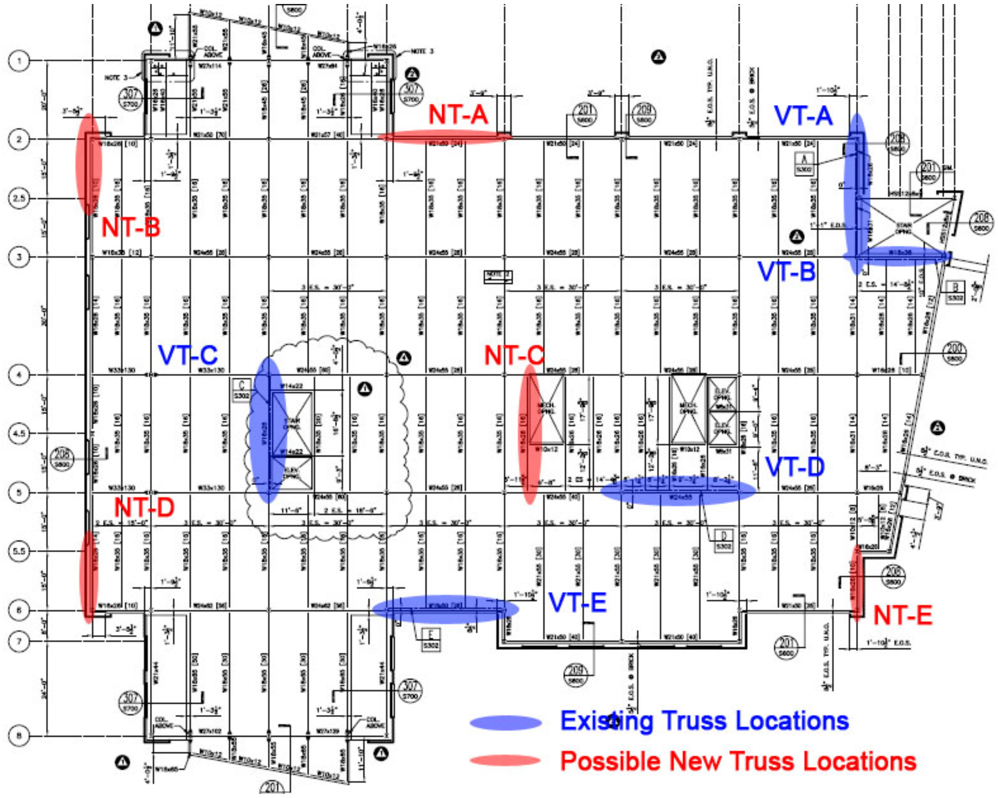


Figure 33 – Existing and New Truss Locations

All proposed lateral frame locations do not interfere with the open floor layout of Quantum III. The only possible additional frame location within the building is beside a core area where NT-C is. There are currently no doors accessing this wall of the core, and the only possible interference can be with mechanical systems and ducts.

Note all following building elevations are for the existing AEO: QIII. They are provided to demonstrate where architectural interferences may occur. For the new, increased elevation, all top story elements (such as aluminum paneling around columns) are assumed to transfer to the new QIII's top story.

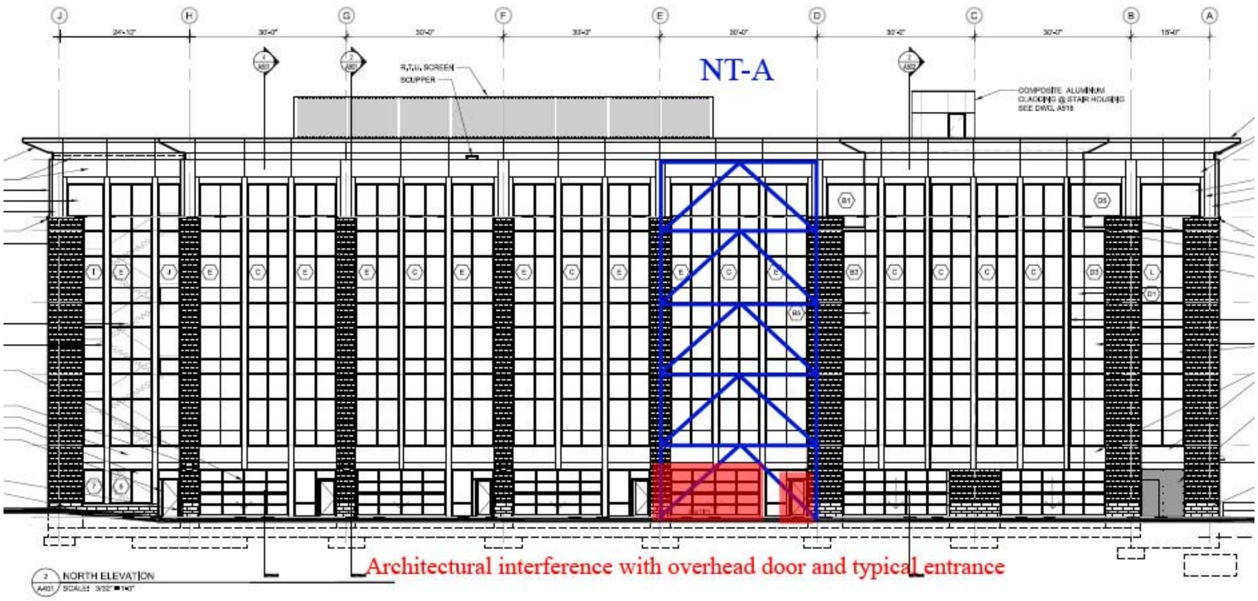


Figure 34 – North Elevation and NT-A

All locations save NT-A have minimal façade interference. As shown above, the base level intersects with an overhead door and windows. A frame at this location would render this door useless and block an entrance. Due to the door being placed on the corner of the bay, even an eccentric frame could not avoid this obstruction. Although it is an excellent display of structure, this is the least desirable location for a new truss. NT-B, D, and E are all located on brick exterior walls so as to avoid curtain wall conflicts. The top story façade is composed of aluminum panels at these locations, so lateral framing will not hinder the transparency of Quantum III.

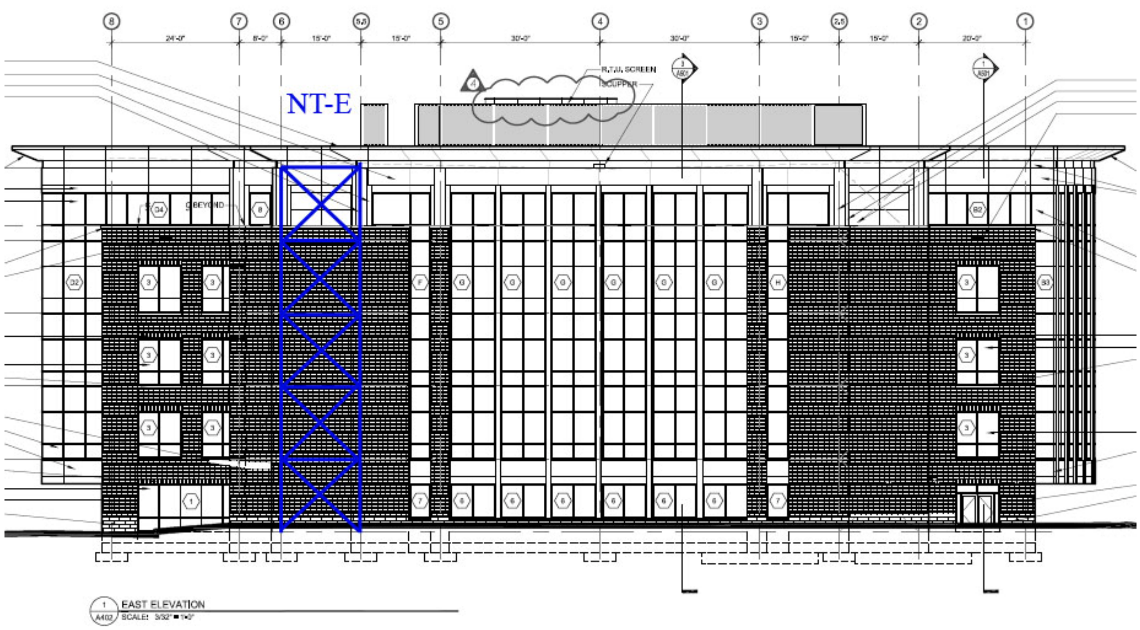


Figure 35- East Elevation and NT-E

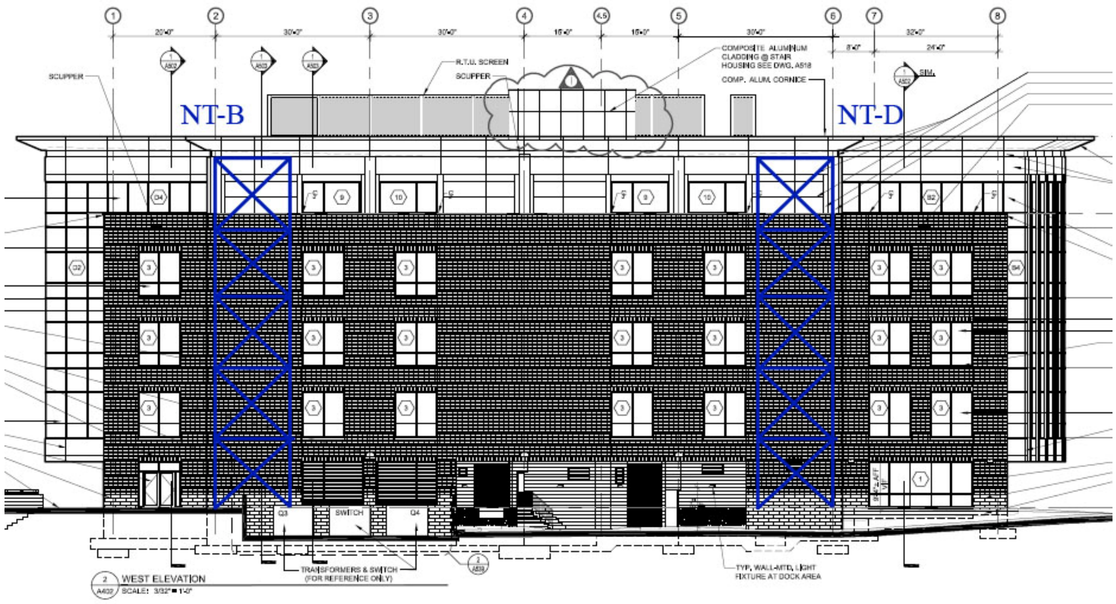


Figure 36 - West Elevation and VT-B and D

As shown in Figure 36 - West Elevation and VT-B and D NT-B and D do not obstruct any architectural features of American Eagle Outfitters: Quantum III. Outlined on the previous page, the façade at the top of the elevation is composed entirely of composite aluminum panels. In effect, no curtain wall systems or windows are blocked by the addition of these frames. Additionally, the two proposed trusses in the above figure appear to be “floating”. QIII’s ground level is exactly where the blue truss outline ends. Slab on grade is at this plane, so the walls below are a combination of retaining and structural walls.

4.3 Final Frame Layout

As shown at right, the final frame layout utilizes NT-B, D, VT-A, B, D, and E. No more curtain wall facades are obstructed by the new frame layout than in the original American Eagle Outfitters: Quantum III design.

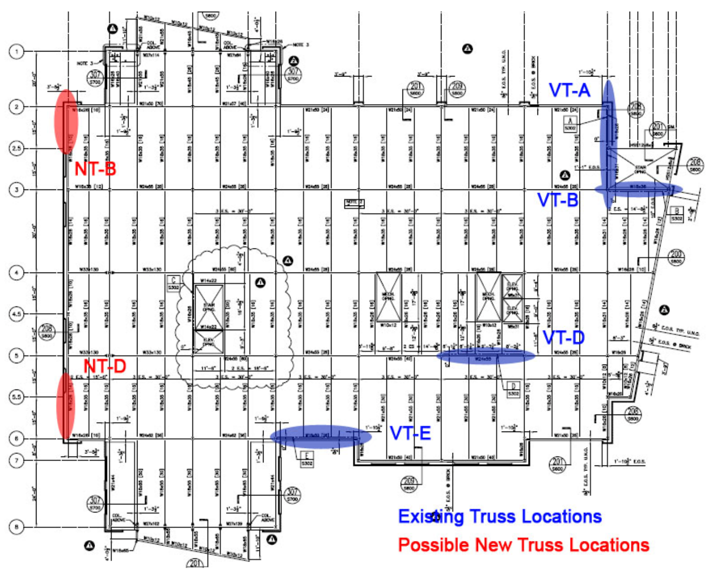


Figure 37 – Final Frame Layout

4.4 Shell Redesign

The shell redesign began with the scaling of vertical and horizontal elements of Quantum III. Originally, the author was going to double the height of the building, keeping it barely below the ASCE 7-05 seismic limit of 160'. It was proposed to have ten stories and a rooftop mechanical level. At this point rescaling of column and massing element widths would have been a typical architectural consideration. However, the final addition of floors to QIII was limited at two. Although this new design is roughly 140 percent of the original building height, massing element rescaling was negligible.

Looking at the new height and scaling analytically, typical interior brick vertical elements would change from 4' to 6'. Preliminary AutoCAD drawings were made to analyze this, and the difference between the two was minute. Therefore, the smaller elements (such as the continuous aluminum panels running up the façade) would change from 1' width to 1'-3.5". This is obviously negligible. Columns in both elevations below are the same width to demonstrate the minor difference in scale of columns and massing elements.

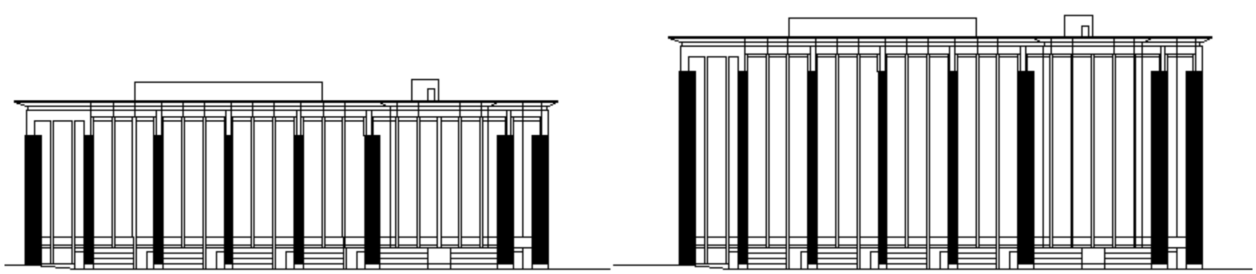


Figure 38 – North Building Elevation for Existing and New Quantum III

Therefore, the scope of the façade redesign only extends to materials and frame location. The location of mass and transparency elements will not change. In other words, the building elevation increases but location of existing elements such as brick walls will still provide mass at their current location.

4.4.1 Oakland Architecture

More so than typical San Francisco and Bay Area architecture, Oakland was defined by the progression of transportation development. Whereas San Francisco was tied by carriage and ship traffic only, the transcontinental railroad had tied Oakland in with the rest of the country, making it a hub of manufacture and development. It was considered the prime suburb of San Francisco and remained ever close to surpassing the city across the bay leading up to the 20th century.

After the 1906 earthquake that destroyed much of San Francisco's residencies and businesses, an influx of people, business, and manufacture moved to Oakland. Oakland had minor damage compared to San Francisco which made it a prime location for the displaced Americans. The influx after the earthquake led to rapid growth and development but it was too

much for the city to accommodate. Consequently, the city could not handle the overload of people and business—most of those that had moved left within two years, leaving an over abundance of newly constructed housing.

Overall, the constant movement of people, both from across the bay and across the country led to the mixed aura of Oakland. Much of the housing constructed after the 1906 earthquake still stands today, and adds to the aura of the city. Also, Oakland's continued expansion of its transportation systems allowed for architectural tastes from all over the country to be left within the city. As Gertrude Stein exclaimed about Oakland: "There's no there there." With hints of California Bungalow, Chicago Prairie School, Classic Revival, English Tudor and recent developments around Lake Merrit, the remark gains ever more bearing on the feel of Oakland. (Winter, 1973, updated 1985)

4.4.2 Façade Assemblies

The current focus is "green" design. It is the tying factor between architects and structural, mechanical, and electrical engineers. From the façade to energy systems to the interior lights, all trades are wrapped into one common goal: energy conservation. This goal will help drive the design of the shell and aid in material selection.

To begin the redesign of the shell, the author researched buildings in Oakland, architecture in Oakland, and factors the climate can have on the building shell. As it turns out, Oakland is in an extreme precipitation zone, where rainfall can exceed 60" per year. To minimize leakage and rain damage in Quantum III, particular caution should be exercised in barrier construction. First, materials must be relatively vapor permeable. Due to the effect of seasons on the building, drying can exist both in and out of the wall; changing the direction of vapor and heat flow. Additionally, interior and exterior side-permeable air barriers are required to limit moisture transport. Where massing elements are present, weather barriers should be installed. This will prevent moisture and precipitation from passing the exterior layers of the shell system. Also, glass and curtain walls should be installed insuring all insulation makes a firm connection to the glass. (Architects, 2007)

The amount of glass in Quantum III's façade adds significant light to the building interior while also increasing cooling requirements. By controlling the amount of sunlight entering the building, certain spectra can be provided to aid in office tasks while limiting the radiation transfer. This can be done to achieve an architecturally and visually appealing façade. Glass panes can be glazed to match the tones of the building while achieving energy efficiency.

Another factor to consider in wall assemblies is the systems resistance to racking. Connections should be designed to withstand seismic accelerations and windows should be designed to withstand shattering as well. This is especially important in high seismic probability zones.

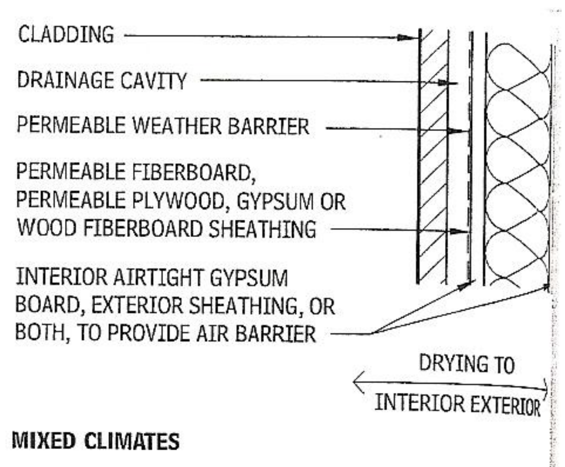


Figure 39 – Mixed Climate Wall Assembly (Architects, 2007)

4.4.3 Façade Redesign

The façade will achieve the feel of a modern high rise while uniting the city of Oakland with the water it borders. Blue toned glass will be coupled with aluminum paneling to invoke balance between buildings such as Oakland City Center (Eric Mueller AE Senior Thesis 2007) and the bay. The rendering below emphasizes the north façade of American Eagle Outfitters: Quantum III.

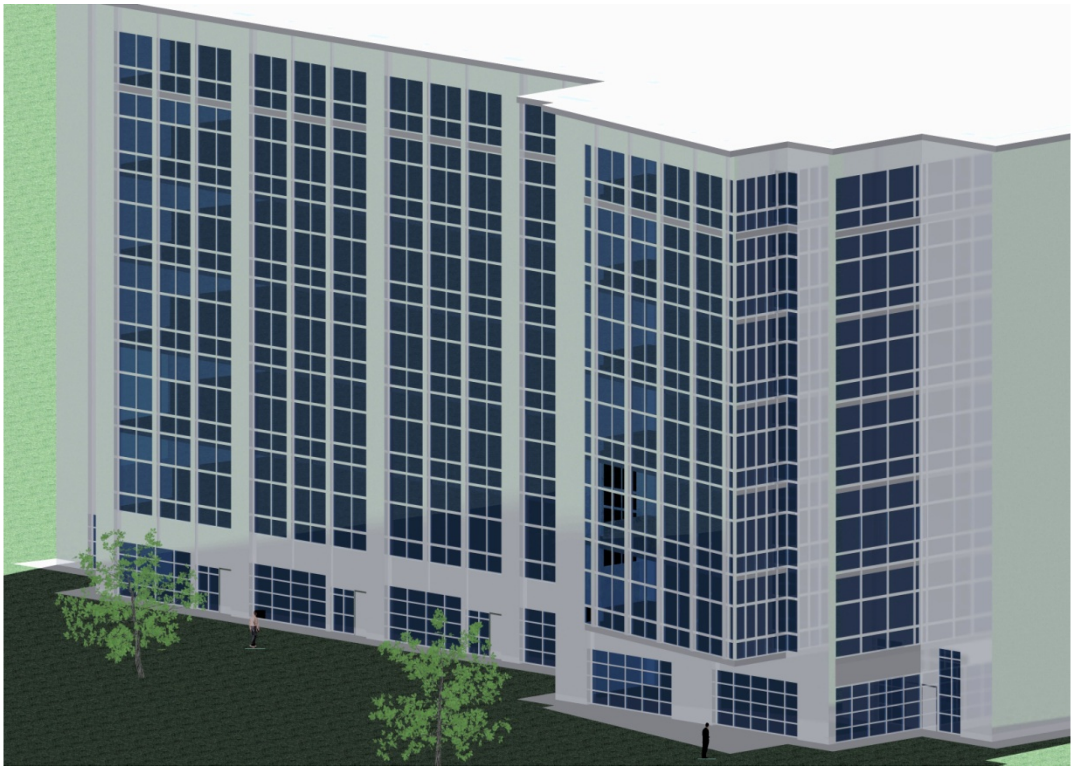


Figure 40 – North Façade Rendering

5. Mechanical Breadth

Numerous factors altering the design of American Eagle Outfitters: Quantum III mechanical systems call for a complete re-evaluation of heating and cooling requirements. First, the building was relocated from Pittsburgh, Pennsylvania to Oakland, California. Second, wall assemblies and glass properties changed to accommodate the new location and architecture. Finally, two stories were added to the building elevation to allow for American Eagle Outfitters increased office space demand. The following outlines the steps taken to determine the input for Trane TRACE 700 software and the results it produced.

5.1 Design Goals

The climate in Oakland, California differs greatly from that of Pittsburgh, Pennsylvania. The first and foremost objective is to achieve new heating and cooling requirements for the marine climate of Oakland and the increased building size. On top of this, the efficiency of Quantum III can be evaluated based on shell permeability and/or heat loss of windows. A comparison between the new and existing systems can be made to determine the success of the new design or options for further investigation.

5.2 Existing Systems

As outlined in 2.8 Mechanical Systems, two 35,000 pound rooftop air handling units provide a total 120,000 CFM. Heat recovery wheels are installed and operate at 64% efficiency for cooling and 77% efficiency for heating. The system is designed to use 36,000 CFM, or 30%, outside air. The boiler room is located on the fifth floor, simplifying HVAC system layout by placing the units and boiler room close vertically and horizontally. Hot water is supplied via two pumps operating at 66% efficiency, pumping 250 gpm. There are typically two VAV boxes per floor, regulating air flow vertically throughout the building.

TRACE determined through location input that the peak load on the cooling coil would occur in July at approximately 3 PM, and the outside air dry bulb temperature would be 86 degrees and wet bulb would be 71 degrees. The peak heating coil load occurs in January-February at 1 AM, with an outside dry bulb temperature of 5 degrees.

The total number of people in the building permitted by code is 1,508. In the new design the total was increased by a factor of 7/5 to account for the new floors. Typical floor gross square feet is 30,550; and this is the estimate used for every new floor of QIII.

5.3 New Shell Assemblies

It was concluded through the architectural breadth that a spectrically sensitive double glazed window should be used for curtain walls. Although more expensive, the window would more than make up it's upfront cost with future cooling load savings. Figure 41 and Figure 42 show the window that was substituted. The original window was assumed to have a U-factor of 0.50 whereas the new double glazed window can deliver 0.30. The airspace between the two panes adds to the insulating quality of each curtain wall and therefore reduces heat transmittance.

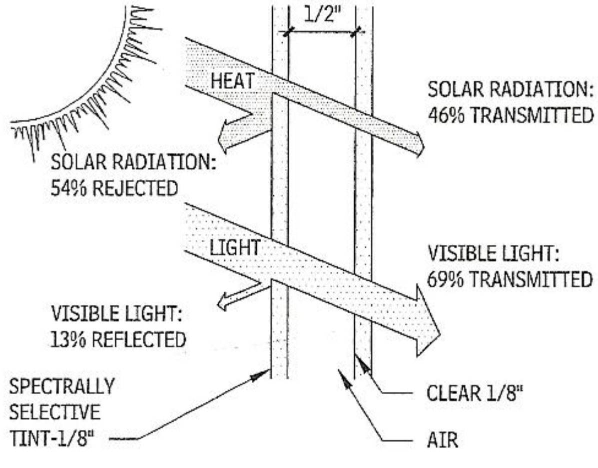


Figure 41 – Window Transmitting Properties (Architects, 2007)

CHARACTERISTIC	EXAMPLE 1	EXAMPLE 2	EXAMPLE 3	EXAMPLE 4	EXAMPLE 5	EXAMPLE 6	EXAMPLE 7	EXAMPLE 8
General glazing description	Single-glazed clear	Double-glazed clear	Double-glazed bronze	Double-glazed clear	Double-glazed low-E	Double-glazed spectrally selective	Triple-glazed clear	Triple-glazed low-E superwindow
Layers of glazing and spaces (outside to inside)	1/8" clear	1/8" clear	1/8" bronze	1/8" clear	1/8" clear	1/8" low-E (0.04)	1/8" clear	low-E (0.08) on 1/8" clear
		1/2" air	1/2" air	1/2" air	1/2" argon	1/2" argon	1/2" air	1/2" krypton
		1/8" clear	1/8" clear	1/8" clear	low-E (0.20) on 1/8" clear	1/8" clear	1/8" clear	1/8" clear
							1/2" air	1/2" krypton
							1/8" clear	low-E (0.08) on 1/8" clear
Center of glass								
U-factor	1.11	0.49	0.49	0.49	0.30	0.24	0.31	0.11
Solar heat gain coefficient	0.86	0.76	0.62	0.76	0.74	0.41	0.69	0.49
Shading coefficient	1.00	0.89	0.72	0.89	0.86	0.47	0.81	0.57
Visible transmittance	0.90	0.81	0.61	0.81	0.74	0.72	0.75	0.68
Frame								
Type	Aluminum, no thermal break	Aluminum, thermal break	Aluminum, thermal break	Wood or vinyl	Wood or vinyl	Wood or vinyl	Wood or vinyl	Insulated vinyl
U-factor	1.90	1.00	1.00	0.40	0.30	0.30	0.30	0.20
Spacer	—	Aluminum	Aluminum	Aluminum	Stainless steel	Stainless steel	Stainless steel	Insulated
Total window								
U-factor	1.30	0.64	0.64	0.49	0.33	0.29	0.34	0.15
Solar heat gain coefficient	0.79	0.65	0.55	0.58	0.55	0.31	0.52	0.37
Visible transmittance	0.69	0.62	0.47	0.57	0.52	0.51	0.53	0.48
Air leakage								
Cubic ft/minimum per linear foot of crack	0.65	0.37	0.37	0.37	0.10	0.10	0.10	0.05
Cubic ft/minimum per sq ft of unit	0.98	0.56	0.56	0.56	0.15	0.15	0.15	0.08

Source: Carmody, Selkowitz, and Heschong, *Residential Windows—New Technologies and Energy Performance*, 1996.

Figure 42 – Window Assembly U-Factors (Architects, 2007)

The total heat loss/gain is heavily dependent on each kind of wall assembly, and how much of the total building perimeter it covers. As with windows, this can significantly impact the required cooling and heating capacity. The total area of glass per typical story is 49.7 percent; accounting for between 5 to 10 percent mullions and aluminum paneling. This percentage is taken as a portion of the total perimeter of Quantum III: which was a total of 615 LF.

5.4 Oakland Climatic Data

The maximum cooling load for Oakland is expected to be at the same time as Pennsylvania but the outside air dry bulb temperature is 98 degrees with the wet bulb temperature at 70 degrees. This is significantly higher than PA and higher temperatures occur more frequently throughout the year. On top of that, the lowest temperature Oakland experiences is 32 degrees Fahrenheit, reinforcing the fact that Oakland experiences higher temperatures throughout the year.

5.5 Results

TRACE results for the existing system came within 0.4 percent of the design cooling coil airflow at 125,439 CFM. Therefore, the assumptions for typical floor glass area and window U-factors was accurate. The software calculated the total existing cooling coil capacity at 286.2 tons or 3,434.8 MBtu/hr. Entering temperature for cooling is averaged at 78.3 degrees with leaving air temp at 59 degrees. See below for the breakdown of cooling loads per element.

A significant portion of the cooling load is attributed to the occupants of Quantum III. Almost 20 percent is attributed to body heat alone. Also, the 7 percent of the cooling load from "miscellaneous" can be attributed to computer and other electronic equipment exhaust. Since the building is open to allow for tenant fit-out, the partition load is zero.

COOLING COIL PEAK

Peaked at Time: Mo/Hr: 7 / 15
Outside Air: OADB/WB/HR: 86 / 71 / 95

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)
Envelope Loads				
Skylite Solar	0	0	0	0.00
Skylite Cond	0	0	0	0.00
Roof Cond	0	87,770	87,770	2.56
Glass Solar	433,559	0	433,559	12.62
Glass Cond	102,275	0	102,275	2.98
Wall Cond	77,317	17,260	94,577	2.75
Partition	0		0	0.00
Exposed Floor	0		0	0.00
Infiltration	0		0	0.00
Sub Total ==>	613,151	105,031	718,182	20.91
Internal Loads				
Lights	500,482	125,121	625,603	18.21
People	679,500		679,500	19.78
Misc	234,601	0	234,601	6.83
Sub Total ==>	1,414,583	125,121	1,539,704	44.83
Ceiling Load	230,151	-230,151	0	0.00
Ventilation Load	0	0	906,455	26.39
Ov/Undr Sizing	0		0	0.00
Exhaust Heat		-26,844	-26,844	-0.78
Sup. Fan Heat			185,836	5.41
Ret. Fan Heat		111,502	111,502	3.25
Duct Heat Pkup		0	0	0.00
Reheat at Design			0	0.00
Grand Total ==>	2,257,886	84,657	3,434,834	100.00

Figure 43 – TRACE Existing Cooling Coil Results

The existing heating load requirements are displayed below. Note the percentage of heating and cooling requirements attributed to glass. Glass conduction accounts for 20.48 percent of all required heating and 15.6 percent of cooling loads.

HEATING COIL PEAK

Mo/Hr: 13 / 1
OADB: 5

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-76,641	2.48
Glass Solar	0	0	0.00
Glass Cond	-633,320	-633,320	20.48
Wall Cond	-136,725	-170,552	5.52
Partition	0	0	0.00
Exposed Floor	0	0	0.00
Infiltration	0	0	0.00
Sub Total ==>	-770,045	-880,513	28.47
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-110,468	0	0.00
Ventilation Load	0	-2,095,258	67.75
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		-116,642	3.77
Additional Reheat		0	0.00
Grand Total ==>	-880,513	-3,092,413	100.00

Figure 44 – TRACE Existing Heating Coil Results

The following two figures represent the cooling and heating coil requirements for the new Quantum III located in Oakland, California. All data returned from TRACE is available in Appendix D.

COOLING COIL PEAK

Peaked at Time: Mo/Hr: 7 / 15
Outside Air: OADB/WB/HR: 98 / 70 / 65

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)
Envelope Loads				
Skylite Solar	0	0	0	0.00
Skylite Cond	0	0	0	0.00
Roof Cond	0	277,480	277,480	6.25
Glass Solar	732,080	0	732,080	16.50
Glass Cond	163,302	0	163,302	3.68
Wall Cond	135,656	29,412	165,068	3.72
Partition	0		0	0.00
Exposed Floor	0		0	0.00
Infiltration	0		0	0.00
Sub Total ==>	1,031,038	306,892	1,337,930	30.15
Internal Loads				
Lights	700,675	175,169	875,844	19.74
People	677,250		677,250	15.26
Misc	328,442	0	328,442	7.40
Sub Total ==>	1,706,367	175,169	1,881,536	42.41
Ceiling Load	482,061	-482,061	0	0.00
Ventilation Load	0	0	770,700	17.37
Ov/Undr Sizing	0		0	0.00
Exhaust Heat		-26,756	-26,756	-0.60
Sup. Fan Heat			295,982	6.67
Ret. Fan Heat		177,589	177,589	4.00
Duct Heat Pkup		0	0	0.00
Reheat at Design			0	0.00
Grand Total ==>	3,219,466	150,833	4,436,981	100.00

Figure 45 – TRACE New Cooling Coil Results

COOLING COIL PEAK

Peaked at Time: Mo/Hr: 7 / 15
Outside Air: OADB/WB/HR: 98 / 70 / 65

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)
Envelope Loads				
Skylite Solar	0	0	0	0.00
Skylite Cond	0	0	0	0.00
Roof Cond	0	277,480	277,480	6.25
Glass Solar	732,080	0	732,080	16.50
Glass Cond	163,302	0	163,302	3.68
Wall Cond	135,656	29,412	165,068	3.72
Partition	0		0	0.00
Exposed Floor	0		0	0.00
Infiltration	0		0	0.00
Sub Total ==>	1,031,038	306,892	1,337,930	30.15
Internal Loads				
Lights	700,675	175,169	875,844	19.74
People	677,250		677,250	15.26
Misc	328,442	0	328,442	7.40
Sub Total ==>	1,706,367	175,169	1,881,536	42.41
Ceiling Load	482,061	-482,061	0	0.00
Ventilation Load	0	0	770,700	17.37
Ov/Undr Sizing	0		0	0.00
Exhaust Heat		-26,756	-26,756	-0.60
Sup. Fan Heat			295,982	6.67
Ret. Fan Heat		177,589	177,589	4.00
Duct Heat Pkup		0	0	0.00
Reheat at Design			0	0.00
Grand Total ==>	3,219,466	150,833	4,436,981	100.00

Figure 46 – TRACE Heating Coil Results

The new cooling loads for American Eagle Outfitters: Quantum III reflect the changes made to the system. Total required capacity is now 369.8 tons as opposed to 286.2 tons, or 129 percent more than the original design. When comparing the total increase in floor area, the increase is roughly 140 percent. Relatively speaking, the new QIII is more efficient than the original design.

This can be attributed in part to the new curtain wall system. Although it is difficult to compare the adequacy of the windows for two separate locations, we can look at the total percent of loads it contributed to both heating and cooling. The existing design in Pittsburgh, Pennsylvania had windows account for (20.48+12.62+2.98) or 36.08 percent of all heating and cooling loads. The new, more efficient double glazed windows account for (12.69+16.5+3.68) or 32.87 percent of all heating and cooling. The difference is small but can add up to significant savings over the course of many years.

5.6 Mechanical Breadth Conclusion

The new design for AEO: QIII requires a significant increase in the rooftop mechanical unit capacity, and therefore their size. This can be expected considering the increase in building size and capacity. To continue with the mechanical design, partition and infiltration loads could be added to the TRANE mechanical model, producing more accurate results of the final cooling and heating coil requirements. Next, the units could be sized with estimates made about their weight on the roof system. This would be added to the structural design to create an up-to-date model of the building behavior under seismic loads. Finally, the costs and savings per ton could be evaluated and weighed against the increased upfront cost of installing the new window system.

Redesign was successful on the basis that the new design is more efficient than the one present in Pittsburgh. This is partially the result of replacing the curtain panels with a more efficient glass construction. As stated before the upfront cost increases, but would eventually pay for it in savings from heating and cooling requirements.

6. Conclusion

The redesign of American Eagle Outfitters: Quantum III was a success in a number of reasons for each of the structural, architectural, and mechanical studies performed:

1. The author gained invaluable knowledge of the design considerations in seismically controlled regions. This applies to both the design of EBF and SCBF systems and how system symmetry can aid in design.

2. Preliminary design was completed to a level of detail. The shear capacities of EBF beam links and SCBF girders were taken into account to obtain member sections. Calculations on one attempted system included shear reinforcing for links as well.

3. Through numerous iterations, the economic design of a lateral frame was performed. Although girders in inverted-V frames are heavy and should contain shear reinforcing, the column and brace sizes were determined through over five possible framing layouts. These layouts each took into account story drift limitations, P-delta effects, and torsion.

4. Structural interference with building architecture was minimal. Two frames were added in exterior bays where the façade displayed mass, limiting interference with the curtain walls and open plan.

5. Façade scaling was analyzed. No changes were made on the basis that minimal elevation change caused negligible effects to the perceived scaling of the building.

6. A redesign for the façade was presented. It proposed eliminating brick columns and replacing them with aluminum paneling—a more common façade element in Oakland and Bay Area California.

7. Mechanical systems were re-evaluated for Oakland and the increased building size. Heating and cooling loads were obtained and evaluated based on efficiency.

8. Building windows were changed to allow for heating and cooling savings. This resulted in the increased efficiency of the building and less heat loss through curtain wall systems.

The Architectural Engineering Senior Thesis has been the culmination of five years of intense study. It is the product of countless hours of research, design, and redesign. It has taught me the worth of design guides and the aide of colleagues, peers, and professionals. Above all else, it has been an invaluable tool in preparation for entering the field as an Architectural Engineer.

Appendix A – Gravity Loads

A.1 Dead Loads

Dead Loads			
Component	Typical Floor	Roof	Mechanical Roof
Concrete Slab	38		38
Metal Decking		2	
Flooring/Ceiling	3	4	3
M/E/P	7	10	7
Rigid Insulation		9	
Membrane		2	
Total Dead Load	48	27	48

Figure 47 – 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 48 – Mechanical Unit Surface Loads

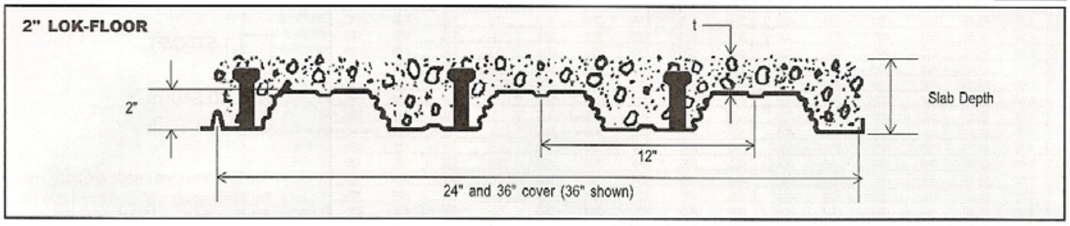
Wall Loads

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)

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 115 pcf concrete

United Steel Deck, Inc.

LRFD



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.^{4}); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.^{3}); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.43
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.52
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.61
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.69
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3990	0.87

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.^{2} per foot of width. **Vol.** is the volume of concrete in ft.^{3} per ft.^{2} needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.^{2}. S_c is the section modulus of the "cracked" concrete composite slab; in.^{3} per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.^{4} per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

COMPOSITE PROPERTIES													
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	34	1.00	4.4	28.13	4270	6.32	8.46	8.56	0.023
	5.00	46.44	37.5	0.333	38	1.18	6.0	33.12	4610	6.03	8.09	8.19	0.027
	5.25	49.53	40.0	0.354	41	1.27	6.9	35.69	4790	5.90	7.93	8.02	0.029
	5.50	52.61	42.6	0.375	43	1.36	7.9	38.29	4970	5.77	7.77	7.86	0.032
	6.00	58.78	48.0	0.417	48	1.55	10.1	43.58	5340	5.55	7.49	7.58	0.036
	6.25	61.87	50.8	0.438	50	1.65	11.3	46.26	5540	5.45	7.36	7.45	0.038
	6.50	64.95	53.6	0.458	53	1.75	12.7	48.97	5730	5.36	7.24	7.32	0.041
	7.00	71.12	59.5	0.500	58	1.94	15.7	54.44	6150	5.18	7.01	7.10	0.045
	7.25	74.21	61.9	0.521	60	2.04	17.4	57.20	6310	5.10	6.91	6.99	0.047
	7.50	77.29	64.3	0.542	62	2.14	19.2	59.97	6480	5.05	6.81	6.89	0.050
20 gage	4.50	48.60	32.6	0.292	34	1.20	4.8	33.77	4560	7.42	9.71	10.03	0.023
	5.00	56.18	37.5	0.333	38	1.42	6.5	39.80	5030	7.07	9.28	9.59	0.027
	5.25	59.96	40.0	0.354	41	1.53	7.4	42.91	5210	6.91	9.09	9.39	0.029
	5.50	63.75	42.6	0.375	43	1.64	8.5	46.05	5390	6.76	8.91	9.20	0.032
	6.00	71.32	48.0	0.417	48	1.87	10.9	52.47	5760	6.49	8.57	8.86	0.036
	6.25	75.11	50.8	0.438	50	1.99	12.2	55.73	5960	6.37	8.42	8.70	0.038
	6.50	78.90	53.6	0.458	53	2.10	13.7	59.02	6150	6.26	8.27	8.55	0.041
	7.00	86.47	59.5	0.500	58	2.34	16.9	65.67	6570	6.05	8.00	8.27	0.045
	7.25	90.26	61.9	0.521	60	2.46	18.7	69.03	6730	5.95	7.87	8.14	0.047
	7.50	94.05	64.3	0.542	62	2.58	20.6	72.41	6900	5.89	7.75	8.01	0.050
19 gage	4.50	55.85	32.6	0.292	34	1.38	5.1	38.67	4560	8.35	10.55	10.91	0.023
	5.00	64.68	37.5	0.333	38	1.63	6.9	45.61	5240	7.94	10.10	10.43	0.027
	5.25	69.10	40.0	0.354	41	1.75	7.9	49.19	5590	7.76	9.89	10.22	0.029
	5.50	73.52	42.6	0.375	43	1.88	9.0	52.83	5790	7.59	9.69	10.01	0.032
	6.00	82.35	48.0	0.417	48	2.15	11.6	60.25	6160	7.29	9.33	9.64	0.036
	6.25	86.77	50.8	0.438	50	2.28	13.0	64.02	6360	7.15	9.16	9.47	0.038
	6.50	91.19	53.6	0.458	53	2.42	14.5	67.83	6550	7.02	9.00	9.30	0.041
	7.00	100.03	59.5	0.500	58	2.69	17.9	75.53	6970	6.78	8.71	9.00	0.045
	7.25	104.44	61.9	0.521	60	2.83	19.8	79.42	7130	6.67	8.57	8.86	0.047
	7.50	108.86	64.3	0.542	62	2.97	21.8	83.33	7300	6.59	8.44	8.72	0.050
18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.00	72.04	37.5	0.333	38	1.81	7.3	50.72	5240	8.75	10.84	11.20	0.027
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	5.50	82.00	42.6	0.375	43	2.10	9.5	58.78	5950	8.35	10.41	10.76	0.032
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050
16 gage	4.50	62.08	32.6	0.292	34	1.88	6.0	42.99	4560	10.49	12.57	12.99	0.023
	5.00	72.04	37.5	0.333	38	2.22	8.0	50.72	5240	9.96	12.03	12.43	0.027
	5.25	77.02	40.0	0.354	41	2.40	9.2	54.72	5590	9.72	11.78	12.18	0.029
	5.50	82.00	42.6	0.375	43	2.58	10.5	58.78	5950	9.50	11.55	11.94	0.032
	6.00	91.95	48.0	0.417	48	2.94	13.4	67.07	6700	9.11	11.13	11.50	0.036
	6.25	96.93	50.8	0.438	50	3.13	15.0	71.29	7090	8.93	10.94	11.30	0.038
	6.50	101.91	53.6	0.458	53	3.32	16.8	75.55	7490	8.76	10.75	11.11	0.041
	7.00	111.87	59.5	0.500	58	3.71	20.6	84.17	8150	8.45	10.40	10.75	0.045
	7.25	116.85	61.9	0.521	60	3.90	22.8	88.52	8310	8.31	10.24	10.59	0.047
	7.50	121.83	64.3	0.542	62	4.10	25.1	92.91	8480	8.22	10.09	10.43	0.050

2" LOK-FLOOR

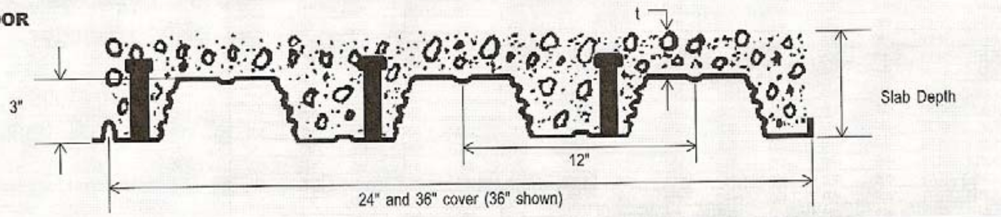
38

Figure 49 – Roof Composite Roof Deck
(United Steel Deck, 2003)

United Steel Deck, Inc.

LRFD

3" LOK-FLOOR



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.[4]); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.[3]); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.[2] per foot of width. **Vol.** is the volume of concrete in ft.[3] per ft.[2] needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.[2]. S_c is the section modulus of the "cracked" concrete composite slab; in.[3] per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.[4] per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10[6] psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{½}A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

DECK PROPERTIES

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.7	0.505	0.797	0.454	0.500	718	2190	0.49
20	0.0358	2.1	0.610	0.993	0.583	0.620	1020	3220	0.59
19	0.0418	2.4	0.710	1.158	0.708	0.726	1350	4310	0.69
18	0.0474	2.8	0.810	1.324	0.832	0.832	1720	4880	0.79
16	0.0598	3.5	1.020	1.666	1.045	1.045	2540	6130	0.99

COMPOSITE PROPERTIES

	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	5.50	52.80	37.6	0.333	38	1.27	7.6	35.57	4810	8.06	10.49	10.83	0.023
	6.00	59.89	42.0	0.375	43	1.46	9.7	40.92	5120	7.70	10.06	10.39	0.027
	6.25	63.43	44.3	0.396	46	1.56	10.9	43.68	5280	7.54	9.86	10.18	0.029
	6.50	66.97	46.6	0.417	48	1.66	12.1	46.49	5440	7.39	9.67	9.99	0.032
	7.00	74.05	51.3	0.458	53	1.86	15.0	52.24	5770	7.11	9.33	9.63	0.036
	7.25	77.59	53.8	0.479	55	1.97	16.6	55.17	5950	6.99	9.17	9.47	0.038
	7.50	81.13	56.3	0.500	58	2.07	18.3	58.14	6120	6.87	9.02	9.31	0.041
	8.00	88.22	61.3	0.542	62	2.29	22.0	64.15	6470	6.68	8.73	9.02	0.045
	8.25	91.76	63.9	0.563	65	2.40	24.1	67.20	6660	6.61	8.60	8.88	0.047
	8.50	95.30	66.6	0.583	67	2.50	26.3	70.27	6840	6.54	8.47	8.75	0.050
20 gage	5.50	62.81	37.6	0.333	38	1.51	8.1	42.29	5250	9.35	11.75	12.14	0.023
	6.00	71.37	42.0	0.375	43	1.73	10.4	48.61	5870	8.92	11.27	11.65	0.027
	6.25	75.65	44.3	0.396	46	1.85	11.7	51.89	6180	8.73	11.06	11.43	0.029
	6.50	79.92	46.6	0.417	48	1.97	13.0	55.23	6470	8.55	10.85	11.21	0.032
	7.00	88.48	51.3	0.458	53	2.21	16.1	62.07	6800	8.23	10.48	10.82	0.036
	7.25	92.76	53.8	0.479	55	2.34	17.8	65.57	6980	8.08	10.30	10.64	0.038
	7.50	97.03	56.3	0.500	58	2.46	19.6	69.10	7150	7.94	10.13	10.47	0.041
	8.00	105.59	61.3	0.542	62	2.72	23.6	76.28	7500	7.72	9.82	10.15	0.045
	8.25	109.87	63.9	0.563	65	2.85	25.7	79.92	7690	7.64	9.67	9.99	0.047
	8.50	114.15	66.6	0.583	67	2.98	28.0	83.59	7870	7.56	9.53	9.85	0.050
19gage	5.50	72.04	37.6	0.333	38	1.72	8.7	48.35	5250	10.47	12.73	13.16	0.023
	6.00	82.00	42.0	0.375	43	1.98	11.0	55.60	5870	9.98	12.23	12.64	0.027
	6.25	86.97	44.3	0.396	46	2.12	12.4	59.36	6180	9.77	11.99	12.40	0.029
	6.50	91.95	46.6	0.417	48	2.25	13.8	63.20	6510	9.56	11.78	12.17	0.032
	7.00	101.91	51.3	0.458	53	2.53	17.0	71.08	7170	9.19	11.37	11.75	0.036
	7.25	106.89	53.8	0.479	55	2.68	18.8	75.10	7510	9.02	11.18	11.56	0.038
	7.50	111.87	56.3	0.500	58	2.82	20.7	79.17	7860	8.87	11.00	11.37	0.041
	8.00	121.83	61.3	0.542	62	3.12	24.9	87.46	8570	8.62	10.67	11.02	0.045
	8.25	126.81	63.9	0.563	65	3.27	27.2	91.65	8780	8.52	10.51	10.86	0.047
	8.50	131.78	66.6	0.583	67	3.42	29.6	95.89	8960	8.43	10.36	10.71	0.050
18 gage	5.50	80.96	37.6	0.333	38	1.94	9.1	54.28	5250	11.48	13.61	14.07	0.023
	6.00	92.32	42.0	0.375	43	2.23	11.6	62.43	5870	10.94	13.07	13.51	0.027
	6.25	98.00	44.3	0.396	46	2.38	13.0	66.67	6180	10.70	12.83	13.26	0.029
	6.50	103.68	46.6	0.417	48	2.53	14.5	70.99	6510	10.48	12.59	13.01	0.032
	7.00	115.04	51.3	0.458	53	2.85	17.9	79.88	7170	10.07	12.16	12.57	0.036
	7.25	120.72	53.8	0.479	55	3.01	19.8	84.42	7510	9.88	11.96	12.36	0.038
	7.50	126.40	56.3	0.500	58	3.17	21.8	89.03	7860	9.71	11.77	12.16	0.041
	8.00	137.76	61.3	0.542	62	3.51	26.2	98.39	8570	9.43	11.42	11.80	0.045
	8.25	143.44	63.9	0.563	65	3.68	28.6	103.15	8930	9.33	11.25	11.62	0.047
	8.50	149.12	66.6	0.583	67	3.85	31.1	107.94	9300	9.23	11.09	11.46	0.050
16 gage	5.50	80.96	37.6	0.333	38	2.36	10.1	54.28	5250	13.04	15.20	15.71	0.023
	6.00	92.32	42.0	0.375	43	2.72	12.8	62.43	5870	12.43	14.61	15.10	0.027
	6.25	98.00	44.3	0.396	46	2.90	14.3	66.67	6180	12.15	14.34	14.82	0.029
	6.50	103.68	46.6	0.417	48	3.09	16.0	70.99	6510	11.89	14.08	14.55	0.032
	7.00	115.04	51.3	0.458	53	3.48	19.7	79.88	7170	11.42	13.60	14.06	0.036
	7.25	120.72	53.8	0.479	55	3.68	21.7	84.42	7510	11.21	13.38	13.83	0.038
	7.50	126.40	56.3	0.500	58	3.89	23.9	89.03	7860	11.01	13.17	13.61	0.041
	8.00	137.76	61.3	0.542	62	4.30	28.7	98.39	8570	10.69	12.78	13.20	0.045
	8.25	143.44	63.9	0.563	65	4.51	31.3	103.15	8930	10.57	12.59	13.01	0.047
	8.50	149.12	66.6	0.583	67	4.72	34.1	107.94	9300	10.46	12.41	12.83	0.050

3" LOK-FLOOR

Figure 50 – Typical Floor Composite Deck
(United Steel Deck, 2003)

A.2 Live Loads

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	

Appendix B – Lateral Loads

B.1 Wind Loads

Gust Effect Factor	
$\bar{z}$	57.32
$L_{\bar{z}}$	558
Q (plan north-south)	0.83
Q (plan east-west)	0.91
$I_{\bar{z}}$	0.18
g_Q	3.4
g_v	3.4
G (plan north-south)	0.85
G (plan east-west)	0.88

Internal Pressure Coefficients	
+ GCpi	- GCpi
0.18	-0.18

Table 6-2		
Exposure	C	
α	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

Locality Input			
Basic Wind Speed	V =	85	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 =	95.54	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	

Figure 52 – Wind Input

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	0.49	<=	48	-0.90	-0.18
		h/2 to h	- 0.9, - 0.18		48	96	-0.90	-0.18
		h to 2h	- 0.5, - 0.18		96	191	-0.50	-0.18
		> 2h	- 0.3, - 0.18		>	191	-0.30	-0.18
	>= 1.0	0 to h/2	- 1.3, - 0.18		<=	48	-1.30	-0.18
		> h/2	- 0.7, - 0.18		>	48	-0.70	-0.18
East to West	<= 0.5	0 to h/2	- 0.9, - 0.18	0.43	<=	48	-0.90	-0.18
		h/2 to h	- 0.9, - 0.18		48	96	-0.90	-0.18
		h to 2h	- 0.5, - 0.18		96	191	-0.50	-0.18
		> 2h	- 0.3, - 0.18		>	191	-0.30	-0.18
	>= 1.0	0 to h/2	- 1.3, - 0.18		<=	48	-1.30	-0.18
		> h/2	- 0.7, - 0.18		>	48	-0.70	-0.18

Figure 53 – Wind Pressure Coefficients

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	1.25	19.71	13.35
20	**0.90**	0.90	1.25	19.71	14.18
25	**0.94**	0.95	1.25	19.71	14.86
30	**0.98**	0.98	1.25	19.71	15.44
40	**1.04**	1.04	1.25	19.71	16.41
50	**1.09**	1.09	1.25	19.71	17.20
60	**1.13**	1.14	1.25	19.71	17.87
70	**1.17**	1.17	1.25	19.71	18.46
80	**1.21**	1.21	1.25	19.71	18.98
90	**1.24**	1.24	1.25	19.71	19.46
100	**1.26**	1.27	1.25	19.71	19.90
120	**1.31**	1.32	1.25	19.71	20.68
140	**1.36**	1.36	1.25	19.71	21.36
160	**1.39**	1.40	1.25	19.71	21.97

Figure 54 – Wind q Factor Calculation

MWFRS Design Pressures						
Walls						
	Wind Direction		**Pressures (lb/ft^2)**			
Leeward	North/South		P =	-8.34	±	3.55
	East/West		P =	-8.26	±	3.55
Side			P =	-12.20	±	3.55
	Wind Direction	**Height (feet)**	**Pressures (lb/ft^2)**			
Windward	North-South	0-15	P =	9.03	±	3.55
		20	P =	9.60	±	3.55
		25	P =	10.06	±	3.55
		30	P =	10.45	±	3.55
		40	P =	11.10	±	3.55
		50	P =	11.64	±	3.55
		60	P =	12.09	±	3.55
		70	P =	12.49	±	3.55
		80	P =	12.85	±	3.55
		90	P =	13.17	±	3.55
		100	P =	13.47	±	3.55
		120	P =	13.99	±	3.55
		140	P =	14.45	±	3.55
		160	P =	14.87	±	3.55
	East-West	0-15	P =	9.44	±	3.55
		20	P =	10.03	±	3.55
		25	P =	10.52	±	3.55
		30	P =	10.93	±	3.55
		40	P =	11.61	±	3.55
		50	P =	12.17	±	3.55
		60	P =	12.64	±	3.55
		70	P =	13.06	±	3.55
		80	P =	13.43	±	3.55
		90	P =	13.77	±	3.55
		100	P =	14.08	±	3.55
		120	P =	14.63	±	3.55
		140	P =	15.11	±	3.55
		160	P =	15.54	±	3.55

Figure 55 – MWFRS Design Pressures

MWFRS Design Pressures										
Roof										
	Wind Direction	**Distance From Windward Wall (feet)**	**Pressures (lb/ft^2)**							
Windward	North-South	0 to 34	P =	-15.00	±	3.55	or	-0.64	±	3.55
		34 to 68	P =	-15.00	±	3.55	or	-0.64	±	3.55
		68 to 137	P =	-8.34	±	3.55	or	-0.64	±	3.55
		over 137	P =	-5.00	±	3.55	or	-0.64	±	3.55
	East-West	0 to 34	P =	-15.69	±	3.55	or	-0.64	±	3.55
		34 to 68	P =	-15.69	±	3.55	or	-0.64	±	3.55
		68 to 137	P =	-8.72	±	3.55	or	-0.64	±	3.55
		over 137	P =	-5.23	±	3.55	or	-0.64	±	3.55

Parapet							
	GC_{pn}	K_p	q_p	Pressures (lb/ft2)			
Windward	1.5	1.26	19.88	P =	29.82	±	3.55
Leeward	-1	1.26	19.88	P =	-19.88	±	3.55

Windscreen							
height =	12	feet					
	GC_{pn}	K_w	q_w	Pressures (lb/ft2)			
Windward	1.5	1.29	20.20	P =	30.31	±	3.55
Leeward	-1	1.29	20.20	P =	-20.20	±	3.55

Figure 56 – MWFRS Design Pressures

Total Wind Forces and Overturning Moments - East-West Wind

Height Above Grade (ft)	Height Above Grade (ft)	Wind Pressure (Windward)	Wind Pressure (Leeward)	Total Wind Pressure	Level	T.O.S. Height	Total Area per Level and Pressure (in^2)	Total Area per Level and Pressure (ft^2)	Force	Total Level Force F (k)	Overturning Moment M (k-ft)
0	6.77	9.44	-8.91	18.35	1	0	0	0	0.00	0.00	0.0
6.77	15	9.44	-8.91	18.35	2	162.5	224188	1556.9	28.57	48.90	662.1
15	20	10.52	-8.91	19.43			139920	971.67	18.88		
20	20.38	10.93	-8.91	19.84			10494	72.875	1.45		
20.38	25	10.93	-8.91	19.84	3	326.5	129426	898.79	17.83	53.88	1466.1
25	30	11.61	-8.91	20.52			139920	971.67	19.94		
30	34.04	11.61	-8.91	20.52			113102	785.43	16.12		
34.04	40	11.61	-8.91	20.52	4	490.5	166738	1157.9	23.76	55.33	2261.79
40	47.71	12.17	-8.91	21.08			215710	1498	31.57		
47.71	50	12.17	-8.91	21.08	5	654.5	64130	445.35	9.39	57.14	3116.78
50	60	12.64	-8.91	21.55			279840	1943.3	41.89		
60	61.38	13.06	-8.91	21.97			38478	267.21	5.87		
61.38	70	13.06	-8.91	21.97	6	818.5	241362	1676.1	36.83	58.72	4005.06
70	75.04	13.43	-8.91	22.34			141086	979.76	21.89		
75.04	80	13.43	-8.91	22.34	7	982.5	138754	963.57	21.53	61.93	5070.78
80	89.17	13.77	-8.91	22.68			256520	1781.4	40.40		
89.17	90	13.77	-8.91	22.68	Roof	1146.5	23320	161.94	3.67	32.53	3107.67
90	96.46	14.08	-8.91	22.99			180730	1255.1	28.85		
96.46	100	14.08	-8.91	22.99	Roof - Stair	1146.5	6120	42.5	0.98	58.39	3187.72
100	109.5	14.63	-8.91	23.54			12750	88.542	2.08		
96.46	108.5	30.31	-20.20	50.51	Windscreen		43350	301.04	15.21		
96.46	108.5	30.31	-20.20	50.51			2550	17.708	0.89		
96.46	100.5	29.82	-19.88	49.70	Parapet		113664	789.33	39.23		
									Totals	**426.83**	**22878.0**

Figure 57 – Wind Forces and Overturning Moments - E-W Wind

Total Wind Forces and Overturning Moments - North-South Wind													
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)	Overturning Moment M (k-ft)
Min	ft	Max	ft						in^2	ft^2			
0	0	81.3	6.77	9.03	-8.34	17.37	1	0	0	0	0.00	0.00	0.0
81.3	6.77	163	13.5	9.03	-8.34	17.37	2	162.5	214337	1488	25.86	52.66	713.1
163	13.5	180	15	9.03	-8.34	17.37			46165	320.6	5.57		
180	15	240	20	9.60	-8.34	17.94			158280	1099	19.71		
240	20	245	20.4	10.06	-8.34	18.40			11871	82.44	1.52		
245	20.4	300	25	10.06	-8.34	18.40	3	326.5	146409	1017	18.70	56.03	1524.5
300	25	327	27.2	10.45	-8.34	18.79			69907	485.5	9.12		
327	27.2	360	30	10.45	-8.34	18.79			83756	581.6	10.93		
360	30	409	34	11.10	-8.34	19.44			127943	888.5	17.28		
409	34	480	40	11.10	-8.34	19.44	4	490.5	188617	1310	25.47	59.32	2424.70
480	40	491	40.9	11.64	-8.34	19.98			27699	192.4	3.84		
491	40.9	573	47.7	11.64	-8.34	19.98			216316	1502	30.01		
573	47.7	600	50	11.64	-8.34	19.98	5	654.5	72545	503.8	10.06	61.28	3342.25
600	50	655	54.5	12.09	-8.34	20.43			143771	998.4	20.40		
655	54.5	720	60	12.09	-8.34	20.43			172789	1200	24.52		
720	60	737	61.4	12.49	-8.34	20.83			43527	302.3	6.30		
737	61.4	819	68.2	12.49	-8.34	20.83	6	818.5	216316	1502	31.29	60.91	4154.24
819	68.2	840	70	12.49	-8.34	20.83			48473	336.6	7.01		
840	70	901	75	12.85	-8.34	21.19			153597	1067	22.60		
901	75	960	80	12.85	-8.34	21.19	7	982.5	156961	1090	23.09	66.44	5439.84
960	80	983	81.9	13.17	-8.34	21.51			59355	412.2	8.87		
983	81.9	1065	88.7	13.17	-8.34	21.51			230825	1603	34.48		
1065	88.7	1080	90	13.17	-8.34	21.51	Roof	1147	26380	183.2	3.94	34.90	3334.33
1080	90	1147	95.5	13.47	-8.34	21.81	Roof		204445	1420	30.96		
1147	95.5	1303	109	13.47	-8.34	21.81	Roof - Stair	1147	4392	30.5	0.67	87.39	4771.06
1147	95.5	1291	108	30.31	-20.20	50.51	Windscreen		120960	840	42.43		
1147	95.5	1195	99.6	29.82	-19.88	49.70	Parapet		128352	891.3	44.30		
											Totals	**478.92**	**25704.0**

Figure 58 – Wind Forces and Overturning Moments – N-S Wind

Wind Story Drift									
Story	Item	Load	Point		Story Height	Story Drift		Allowable Drift	Conclusion
			X	Y	Z	X	Y		
			in	in	in	in	in	in	
ROOF	Max Drift X	DSTLD1	780	-142	1146.5	6.3E-05		0.41	OK
ROOF	Max Drift Y	DSTLD1	2638.5	1464	1146.5		5.2E-05	0.41	OK
7TH	Max Drift X	DSTLD1	780	-142	982.5	6.2E-05		0.41	OK
7TH	Max Drift Y	DSTLD1	2638.5	1464	982.5		5.7E-05	0.41	OK
6TH	Max Drift X	DSTLD1	780	-142	818.5	0.00005		0.41	OK
6TH	Max Drift Y	DSTLD1	2638.5	1464	818.5		4.9E-05	0.41	OK
5TH	Max Drift X	DSTLD1	780	-142	654.5	3.7E-05		0.41	OK
5TH	Max Drift Y	DSTLD1	2638.5	1464	654.5		3.5E-05	0.41	OK
4TH	Max Drift X	DSTLD1	780	-142	490.5	2.7E-05		0.41	OK
4TH	Max Drift Y	DSTLD1	2638.5	1464	490.5		0.00003	0.41	OK
3RD	Max Drift X	DSTLD1	780	-142	326.5	1.5E-05		0.41	OK
3RD	Max Drift Y	DSTLD1	2638.5	1464	326.5		0.00002	0.41	OK
2ND	Max Drift X	DSTLD1	1260	384	162.5	5E-06		0.40625	OK
2ND	Max Drift Y	DSTLD1	2638.5	1464	162.5		1.5E-05	0.40625	OK
ROOF	Max Drift X	DSTLD2	780	-142	1146.5	9.7E-05		0.41	OK
ROOF	Max Drift Y	DSTLD2	2638.5	1464	1146.5		0.00008	0.41	OK
7TH	Max Drift X	DSTLD2	780	-142	982.5	9.6E-05		0.41	OK
7TH	Max Drift Y	DSTLD2	2638.5	1464	982.5		8.9E-05	0.41	OK
6TH	Max Drift X	DSTLD2	780	-142	818.5	7.7E-05		0.41	OK
6TH	Max Drift Y	DSTLD2	2638.5	1464	818.5		7.6E-05	0.41	OK
5TH	Max Drift X	DSTLD2	780	-142	654.5	5.7E-05		0.41	OK
5TH	Max Drift Y	DSTLD2	2638.5	1464	654.5		5.5E-05	0.41	OK
4TH	Max Drift X	DSTLD2	780	-142	490.5	4.2E-05		0.41	OK
4TH	Max Drift Y	DSTLD2	2638.5	1464	490.5		4.8E-05	0.41	OK
3RD	Max Drift X	DSTLD2	780	-142	326.5	2.4E-05		0.41	OK
3RD	Max Drift Y	DSTLD2	2638.5	1464	326.5		3.1E-05	0.41	OK
2ND	Max Drift X	DSTLD2	1260	384	162.5	7E-06		0.40625	OK
2ND	Max Drift Y	DSTLD2	2638.5	1464	162.5		2.4E-05	0.40625	OK

Figure 59 – Wind Story Drift

The spreadsheet above represents only a portion of the actual drift checks performed for American Eagle Outfitters: Quantum III. Over 20 load cases were taken into account resulting in a spreadsheet over 300 cells long. See book for full checks.

B.2 Seismic Loads

BUILDING IRREGULARITIES - HORIZ. PG 1 SMPJ

① TORSIONAL IRREGULARITIES

- LOAD CASE WITH MAX ROTATION: QUAKEXY1
  RZ = -0.00045 RAD

- BUILDING CORNER DISPLACEMENTS:

  ELEVATIONS:

  - NORTH: LEFT = 3.625344" RIGHT = 2.436467" (Y)
  - EAST: TOP = 2.570868" BOTTOM = 2.570968" (X)
  - SOUTH: LEFT = 3.625344" RIGHT = 2.436467 (Y)
  - WEST: SAME AS EAST

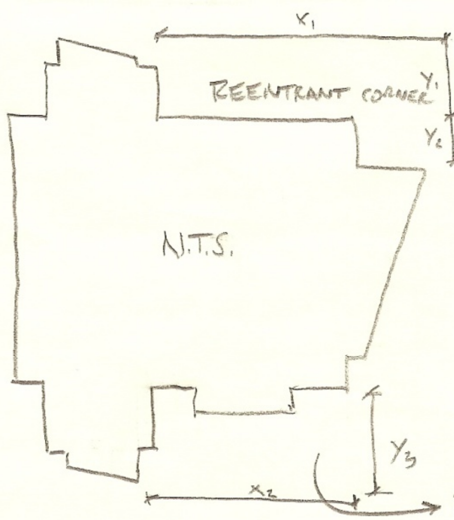
- EAST TORSIONAL IRREGULARITY:

$$\frac{\Delta_1 + \Delta_2}{2} = \frac{3.625344 + 2.436467}{2} = 3.03091''$$

$$\frac{3.625344}{3.03091} = 1.19611 < 1.2$$ OK ✓ BUT CLOSE

∴ NO TORSIONAL IRREGULARITY

② RE-ENTRANT CORNER IRREGULARITIES



$L_x = 219.854'$
$L_y = 195.67'$

$x_1 = 144.854'$
$y_1 = 31.833'$
$y_2 = 15'$
$x_2 = 120'$
$y_3 = 32'$

$0.15L_x = 32.978'$
$0.15L_y = 29.351'$

REENTRANT CORNER

BUILDING IRREGULARITIES   PG 2   SMPJ

② CONTINUED

∴ RE-ENTRANT CORNERS ARE PRESENT

↳ SEIS. DESIGN FORCES INCREASED BY 25% FOR CONNECTIONS OF DIAPHRAGMS TO VERTICAL ELEMENTS AND TO COLLECTORS & CONNECTIONS OF COLLECTORS TO VERTICAL ELEMENTS

③ DIAPHRAGM DISCONTINUITY

∴ DOES NOT EXIST BY INSPECTION

④ & ⑤ OUT OF PLANE OFFSETS & NONPARALLEL SYSTEMS

∴ IRREGULARITIES DO NOT EXIST BY INSPECTION

BUILDING IRREGULARITIES - VERT.   PG 1   SMPJ

① STIFFNESS SOFT STORY

STIFFNESS INCREASES SIGNIFICANTLY AS YOU PROGRESS DOWN THE BUILDING

∴ NO IRREGULARITY BY INSPECTION

② - ⑤ IRREGULARITIES DO NOT EXIST BY INSPECTION

SEISMIC CALCULATIONS SMPJ PG 1

- OCCUPANCY CAT. : $\boxed{\text{CATEGORY II}}$ PEOPLE ARE NOT CONGREGATED
- SPECTRAL RESPONSE ACCELERATION:

$S_S = 1.522$ (USGS 12TH ST. OAKLAND, CA 94607)
$S_1 = 0.6$

- SITE CLASS: ASSUME D (DATA UNKNOWN)
- SITE CLASS FACTORS: $F_a = 1.0$
$F_v = 1.5$

$S_{MS} = F_a S_S = 1.0(1.522) = 1.522$
$S_{M1} = F_v S_1 = 1.5(0.6) = 0.9$

$S_{DS} = 2/3\ S_{MS} = 1.015$
$S_{D1} = 2/3\ S_{M1} = 0.6$

- IMPORTANCE FACTOR: $I = 1.0$
- SEISMIC DESIGN CATEGORY:

$S_1 > 0.75$ ∴ CATEGORY E (ASCE 7-05 11.6)

- BUILDING FRAME SYSTEM: SPECIAL STEEL CONC FRAMES

$R = 6$
$\Omega_0 = 2$
$C_d = 5$

FOR CATEGORY E: $h < 160'$

ACTUAL HT = $96.458'$ $< 160'$ OK

- FIND T

$C_t = 0.02$ } ALL OTHER STR. SYSTEMS
$x = 0.75$

$T_a = 0.02(96.458 + 13.25)^{0.75} = 0.678$

SEISMIC CALCULATIONS

SMPJ PG 2

$S_{D1} = 0.6 \quad \therefore C_u = 1.4$

$T \leq C_u T_a = 1.4(0.678) = \boxed{0.949 \text{ sec} \quad \text{CONTROLS}}$

$T_{ETABS} = 1.1371 \text{ sec} \quad (3\text{-}27\text{-}08)$

- FIND C_S

$$C_{S,ACT} = \frac{S_{DS}}{R/I} = \frac{1.015}{6/1} = 0.169$$

$$C_S \leq \frac{S_{D1}}{T(R/I)} = \frac{0.6}{0.949(6/1)} = 0.1054 \qquad T = 0.949 \ll T_L = 8$$

$T_L = 8$ FIG 22-16 (OAKLAND, CA)

$C_S > 0.01$

$$C_S > \frac{0.5 S_1}{R/I} = \frac{0.5(0.6)}{6/1.0} = 0.05$$

$\therefore$ MAX $\left| \begin{array}{l} 0.01 \\ 0.05 \\ 0.169 \end{array} \right.$ BUT MIN $\left| \begin{array}{l} \boxed{0.1054} \leftarrow \text{CONTROLS} \\ 0.169 \end{array} \right.$

$\therefore C_S = 0.1054$

• SEISMIC: PERMITTED ANALYTICAL PROCEDURES PG 1 SMPJ

SEISMIC DESIGN CATEGORY E

STRUCTURE HAS HORIZONTAL IRREGULARITY (2) REENTRANT CORNER

⇒ IRREGULARITIES PERMITTED:

HORIZ: 2, 3, 4, & 5 OK ✓

VERT: 4, 5a, & 5b OK ✓

$\Rightarrow T < 3.5T_S$

$T_S = S_{D1}/S_{DS} = \frac{0.6}{1.015} = 0.591\ s$

$3.5T_S = 2.069\ s$

$T_{ETABS} = 1.2249\ s < 3.5T_S$ OK ✓

$T_{CONTROLLING} = 0.949s < 3.5T_S$ OK ✓

∴ EQUIVALENT LATERAL FORCE ANALYSIS PERMITTED

• RHO (ρ) FACTOR CHECK

↳ BRACED FRAMES

↳ NO TORSIONAL IRREGULARITIES PRESENT

↳ REMOVAL OF SINGLE BRACE OR CONNECTION DOES NOT RESULT IN 33% REDUCTION OF STRENGTH

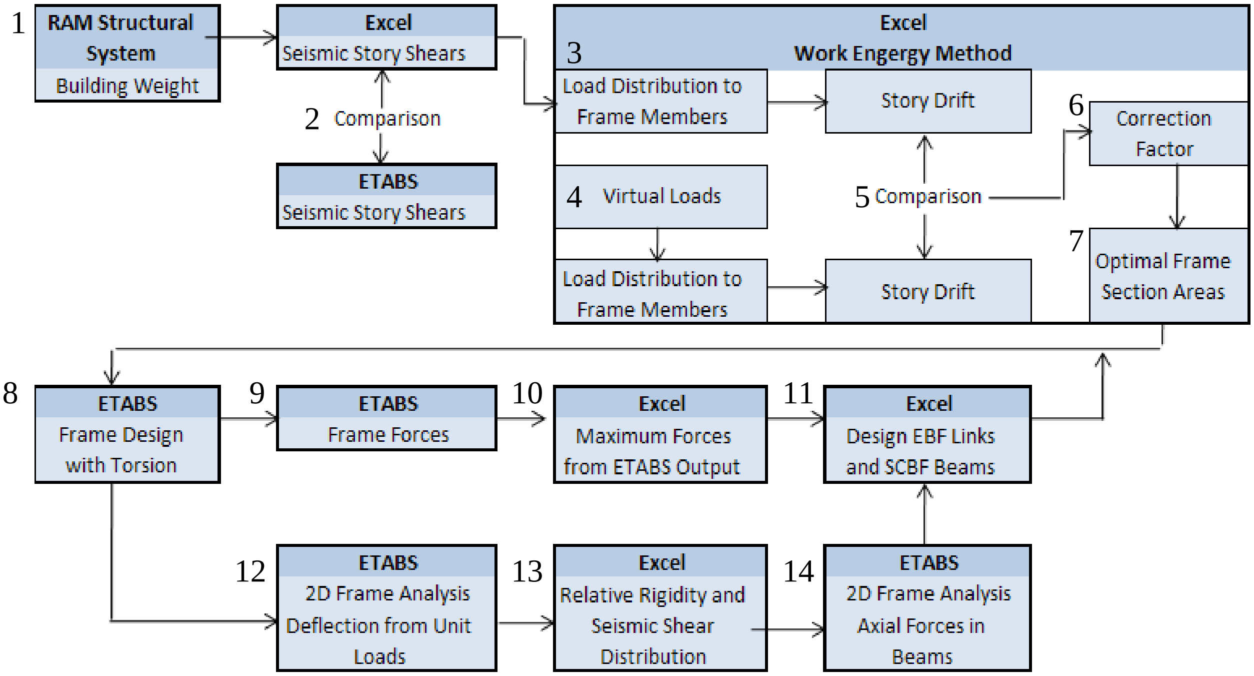


Figure 60 – Seismic Design Methodology

Building Weights Per Floor				RamSTEEL		7-Apr-08		
Total Level Weight k	Story Weight k		Area	Location X	Location Y			
1420.2	1381.5	42.904	229954	106.84	92.3	11.04	9.84	None
	38.7	1.201	7960	45	75.75	---	---	None
3139.5	3128	97.144	559007	94.57	88.33	10.95	9.87	None
	11.5	0.358	26	212.2	134.46	---	---	None
3136	3124.5	97.033	558801	94.54	88.35	10.95	9.87	None
	11.5	0.358	26	212.19	134.46	---	---	None
3140.5	3129	97.175	559521	94.54	88.33	10.95	9.87	None
	11.5	0.358	26	212.19	134.46	---	---	None
3143	3131.5	97.25	560020	94.54	88.36	10.95	9.87	None
	11.5	0.358	26	212.19	134.46	---	---	None
3147.9	3136.4	97.403	560907	94.55	88.36	10.95	9.87	None
	11.5	0.358	26	212.19	134.46	---	---	None
3154.8	3143.3	97.617	562234	94.54	88.33	10.95	9.87	None
	11.5	0.358	26	212.19	134.46	---	---	None

Figure 61 – RAM Building Weights (1)

Building Masses						
	Weight	Mass	Floor Area	Mass / Area		
			ETABS			
	k	(k s^2)/in	in^2			
Roof	1420.2	3.6793	4142910	8.8809E-07	8.8809E-07	1.2788E-04
7	3139.5	8.1334	4142910	1.9632E-06	1.9632E-06	2.8270E-04
6	3136	8.1244	4142910	1.9610E-06	1.9610E-06	2.8238E-04
5	3140.5	8.1360	4142910	1.9638E-06	1.9638E-06	2.8279E-04
4	3143	8.1425	4142910	1.9654E-06	1.9654E-06	2.8302E-04
3	3147.9	8.1552	4142910	1.9685E-06	1.9685E-06	2.8346E-04
2	3154.8	8.1731	4142910	1.9728E-06	1.9728E-06	2.8408E-04

Figure 62 – Building Masses (1)

Seismic Base Shear										
Level	h_x (in)	h_x (ft)	h_x^k	W	W * h_x^k	C_{vx}	F	V	M	ΣM
Roof	1146.50	95.54	265.917	1420	377655.3	0.146	311.34	311.34	29745.96	29745.96
7	982.50	81.88	220.117	3140	691057.6	0.267	569.71	881.05	46645.01	93290.02
6	818.50	68.21	176.009	3136	551963	0.213	455.04	1336.09	31037.52	124327.5
5	654.50	54.54	133.852	3141	420361.3	0.162	346.55	1682.64	18901.26	143228.8
4	490.50	40.88	94.022	3143	295511.5	0.114	243.62	1926.26	9957.992	153186.8
3	326.50	27.21	57.121	3148	179809.8	0.069	148.24	2074.49	4033.249	157220
2	162.50	13.54	24.307	3155	76683.93	0.030	63.22	2137.71	856.0834	158076.1
1	0.00	0.00	0.000	0	0	0.000	0.00	2137.71	0	158076.1
			Totals	20281.9	2593043	1	2137.71		141177.1	

		Cs	W (kips)		Total Force	
V = C_s * W =		0.1054	20281.9	=	2137.71226	k

T	k
0.50	1
0.95	1.2245
2.50	2

	Lower Bound	Exact	Upper Bound	Use
Cs =	0.05	0.169	0.1054	0.1054

Figure 63 – Seismic Base Shear (2)

Seismic Base Shear Comparison			
Level	Hand Calculated k	ETABS k	Percent Difference
Roof	311.34	327.1	4.82
7	881.05	917.92	4.02
6	1336.09	1391.37	3.97
5	1682.64	1755.67	4.16
4	1926.26	2013.2	4.32
3	2074.49	2170.67	4.43
2	2137.71	2238.14	4.49

Figure 64 – Seismic Base Shear Comparison (2)

Y Direction				
Frame	Load	Deflection	Stiffness	Relative Stiffness
	k	in	k/in	%
NT-B	10	0.120841	82.75337	0.12015
NT-C	10	0.051038	195.9324	0.284476
NT-D	10	0.120841	82.75337	0.12015
VT-A	10	0.059777	167.2884	0.242888
VT-C	10	0.062492	160.0205	0.232335
		Total	688.7481	

X Direction				
Frame	Load	Deflection	Stiffness	Relative Stiffness
	k	in	k/in	%
VT-B	10	0.055156	181.3039	0.319817
VT-D	10	0.051868	192.7971	0.340091
VT-E	10	0.051868	192.7971	0.340091
		Total	566.8981	

Figure 65 – Preliminary Frame Relative Rigidities (3)

These deflections were determined through iterations in ETABS. Using the following spreadsheet to determine optimal areas, then inputting to ETABS, the author found actual deflections. Then optimal areas were found again based on more accurate seismic shears.

Braced Frame:	D & E

Relative Stiffness =	0.3400913

Cd =	5

Story Shears - X Direction

Level	Height ft	Floor Height ft	Total Force k	Force per Level k	Story Force k	Story Shear k
Roof	96.46	14.58333	311.34	311.34	105.88	105.88
7	81.88	13.6667	881.05	569.710035	193.75	299.64
6	68.21	13.6667	1336.09	455.040022	154.76	454.39
5	54.54	13.6667	1682.64	346.547148	117.86	572.25
4	40.88	13.6667	1926.26	243.620603	82.85	655.10
3	27.21	13.6667	2074.49	148.235783	50.41	705.52
2	13.54	13.5417	2137.71	63.2184642	21.50	727.02

Member Loads

Level	Height	Floor Height	Story Force	Story Shear	Story Moment	Axial Forces					
						Actual			Virtual		
						Column	Girder	Brace	Column	Girder	Brace
	ft	ft	k	k	ft-k	k	k	k	k	k	k
Roof	96.46	14.58	105.88	105.88	1544.14282	0	52.94204	73.84	0	0	0.697355
7	81.88	13.67	193.75	299.64	4095.05559	51.471427	149.8187	202.68	0.486111	0.5	0.676411
6	68.21	13.67	154.76	454.39	6210.04757	187.97328	227.1963	307.36	0.941668	0.5	0.676411
5	54.54	13.67	117.86	572.25	7820.77278	394.97487	286.1251	387.08	1.397224	0.5	0.676411
4	40.88	13.67	82.85	655.10	8953.10311	655.66729	327.5518	443.12	1.852781	0.5	0.676411
3	27.21	13.67	50.41	705.52	9642.09194	954.10406	352.7586	477.22	2.308338	0.5	0.676411
2	13.54	13.54	21.50	727.02	9845.04948	1275.5071	363.5086	489.73	2.763894	0.5	0.673612

Bay Length, L =	30 ft
Virtual Load	1.00 k

Member Areas and Strains

Level	Height ft	Floor Height ft	Areas			Strain		
			Column	Girder	Brace	Column	Girder	Brace
Roof	96.46	14.58	0.00	0.00	7.18	0.0000	0.0000	0.0891
7	81.88	13.67	5.00	8.66	11.71	0.0582	0.1074	0.1453
6	68.21	13.67	13.30	10.66	14.42	0.0799	0.1323	0.1790
5	54.54	13.67	23.49	11.96	16.18	0.0951	0.1485	0.2009
4	40.88	13.67	34.85	12.80	17.31	0.1064	0.1589	0.2149
3	27.21	13.67	46.93	13.28	17.97	0.1150	0.1649	0.2230
2	13.54	13.54	59.37	13.48	18.16	0.1204	0.1674	0.2255

Elastic Modulus	Columns	29000 ksi
	Braces	ksi

Rho's and Deflections											
Level	Height	Floor	Rho					Deflection			
		Height	Column	Sum	Girder	Brace	Sum	Floor	Total	Amplified
	ft	ft		Column				in	in	in
Roof	96.46	14.58	0.0000	0.0026	0.000000	0.000710	0.003265	0.5714	4.4520	22.26
7	81.88	13.67	0.0003	0.0026	0.000655	0.001199	0.004409	0.7231	3.8807	19.40
6	68.21	13.67	0.0004	0.0022	0.000807	0.001476	0.004515	0.7405	3.1575	15.79
5	54.54	13.67	0.0005	0.0018	0.000905	0.001657	0.004350	0.7135	2.4170	12.09
4	40.88	13.67	0.0006	0.0013	0.000969	0.001773	0.004001	0.6562	1.7036	8.52
3	27.21	13.67	0.0006	0.0007	0.001005	0.001840	0.003514	0.5763	1.0474	5.24
2	13.54	13.54	0.0007	0.0000	0.001030	0.001869	0.002899	0.4711	0.4711	2.36

Optimum Areas									
Level	Height	Floor	Area			Correction	Optimal Areas		
		Height	Column	Girder	Brace	Factor	Column	Girder	Brace
	ft	ft							
Roof	96.46	14.58	0.00	0.00	7.18	0.96	0.00	0.00	6.90
7	81.88	13.67	5.00	8.66	11.71	0.96	4.81	8.32	11.26
6	68.21	13.67	13.30	10.66	14.42	0.96	12.79	10.25	13.86
5	54.54	13.67	23.49	11.96	16.18	0.96	22.59	11.50	15.56
4	40.88	13.67	34.85	12.80	17.31	0.96	33.51	12.31	16.65
3	27.21	13.67	46.93	13.28	17.97	0.96	45.13	12.77	17.28
2	13.54	13.54	59.37	13.48	18.16	0.96	57.09	12.96	17.46

Target Building Deflection	0.0200
Calculated Building Deflection	0.0192
Correction Factor	0.96

0.020hsx =	23.15

Figure 66 – Frame Preliminary Sizing (3-7)

Insert ETABS Point Deflections For EBF Below This								
Story	**Point**	**Load**	**UX**	**UY**	**UZ**	**RX**	**RY**	**RZ**
STORY7	11	LATERAL	0.1602	0	0.0207	0	0.00012	0
STORY6	11	LATERAL	0.1351	0	0.0198	0	0.00013	0
STORY5	11	LATERAL	0.1109	0	0.0183	0	0.00013	0
STORY4	11	LATERAL	0.0851	0	0.0152	0	0.00013	0
STORY3	11	LATERAL	0.0606	0	0.0127	0	0.00013	0
STORY2	11	LATERAL	0.0361	0	0.0088	0	0.00013	0
STORY1	11	LATERAL	0.0132	0	0.005	0	0.00011	0

Insert ETABS Point Deflections For SCBF Below This								
Story	**Point**	**Load**	**UX**	**UY**	**UZ**	**RX**	**RY**	**RZ**
STORY7	3	LATERAL	0.3083	0	0.0251	0	0.00028	0
STORY6	3	LATERAL	0.2549	0	0.025	0	0.0003	0
STORY5	3	LATERAL	0.2031	0	0.024	0	0.0003	0
STORY4	3	LATERAL	0.1521	0	0.0211	0	0.00028	0
STORY3	3	LATERAL	0.1033	0	0.0179	0	0.00026	0
STORY2	3	LATERAL	0.0596	0	0.0127	0	0.00022	0
STORY1	3	LATERAL	0.0234	0	0.0074	0	0.00018	0

Figure 67 – Actual Frame Deflection Data from ETABS (13)

Deflections shown in Figure 67 are based on actual model data from ETABS. First, optimal areas of members were determined; then inputting similar wide flange shapes into ETABS found actual deflections. In turn, these deflections produced more accurate relative rigidities, and therefore more accurate optimal areas.

Frame Relative Rigidities													
Level	**Load**			**Deflection**			**Rigidity**			**Total**	**Relative Rigidity (Percent)**		
	VT-A	NT-B	NT-D	VT-A	NT-B	NT-D	VT-A	NT-B	NT-D		VT-A	NT-B	NT-D
Roof	10	10	10	0.1602	0.3083	0.3083	62.42	32.44	32.44	127.29	0.4904	0.2548	0.2548
7	10	10	10	0.1351	0.2549	0.2549	74.02	39.23	39.23	152.48	0.4854	0.2573	0.2573
6	10	10	10	0.1109	0.2031	0.2031	90.17	49.24	49.24	188.64	0.4780	0.2610	0.2610
5	10	10	10	0.0851	0.1521	0.1521	117.51	65.75	65.75	249.00	0.4719	0.2640	0.2640
4	10	10	10	0.0606	0.1033	0.1033	165.02	96.81	96.81	358.63	0.4601	0.2699	0.2699
3	10	10	10	0.0361	0.0596	0.0596	277.01	167.79	167.79	612.58	0.4522	0.2739	0.2739
2	10	10	10	0.0132	0.0234	0.0234	757.58	427.35	427.35	1612.28	0.4699	0.2651	0.2651
						Total	1543.72	878.59	878.59				

Distribution of Seismic Shears				
Level	**Seismic Force**	**Force VT-A**	**NT-B**	**NT-D**
Roof	311.34	**152.67**	**79.33**	**79.33**
7	569.71	**276.56**	**146.58**	**146.58**
6	455.04	**217.51**	**118.77**	**118.77**
5	346.55	**163.54**	**91.50**	**91.50**
4	243.62	**112.10**	**65.76**	**65.76**
3	148.24	**67.03**	**40.60**	**40.60**
2	63.22	**29.71**	**16.76**	**16.76**

Figure 68 – Frame Actual Relative Rigidities (13)

SCBF Beams - Elevation 5

MAX SHEAR	**12.56**	**MAX MOMENT**	**1513.958**
ROW		ROW	5409

Insert ETABS Force Data Below This Row

Story	Beam	Load	Loc	P	V2	V3	T	M2	M3	Absolute Value of Shear	Absolute Value of Moment		
ROOF	B3	QUAKEX	7.15	0	0.54	0	-0.01	0	93.541	0.54	93.541	...	...
ROOF	B3	QUAKEX	28.756	0	0.54	0	-0.01	0	81.849	0.54	81.849	...	...
ROOF	B3	QUAKEX	50.363	0	0.54	0	-0.01	0	70.156	0.54	70.156	...	...
ROOF	B3	QUAKEX	71.969	0	0.54	0	-0.01	0	58.463	0.54	58.463	...	...
ROOF	B3	QUAKEX	93.575	0	0.54	0	-0.01	0	46.771	0.54	46.771	...	...
ROOF	B3	QUAKEX	115.181	0	0.54	0	-0.01	0	35.078	0.54	35.078	...	...
ROOF	B3	QUAKEX	136.788	0	0.54	0	-0.01	0	23.385	0.54	23.385	...	...
ROOF	B3	QUAKEX	158.394	0	0.54	0	-0.01	0	11.693	0.54	11.693	...	...
ROOF	B3	QUAKEX	180	0	0.54	0	-0.01	0	0	0.54	0	...	...
ROOF	B3	QUAKEX	180	0	0.54	0	0.011	0	0	0.54	0	...	...
ROOF	B3	QUAKEX	201.606	0	0.54	0	0.011	0	-11.693	0.54	11.693	...	...
ROOF	B3	QUAKEX	223.212	0	0.54	0	0.011	0	-23.385	0.54	23.385	...	...
ROOF	B3	QUAKEX	244.819	0	0.54	0	0.011	0	-35.078	0.54	35.078	...	...
ROOF	B3	QUAKEX	266.425	0	0.54	0	0.011	0	-46.771	0.54	46.771	...	...
ROOF	B3	QUAKEX	288.031	0	0.54	0	0.011	0	-58.463	0.54	58.463	...	...
ROOF	B3	QUAKEX	309.637	0	0.54	0	0.011	0	-70.156	0.54	70.156	...	...
ROOF	B3	QUAKEX	331.244	0	0.54	0	0.011	0	-81.849	0.54	81.849	...	...

Figure 69 – Max Shear and Moment

The above spreadsheet takes thousands of rows of data output from ETABS and finds the maximum shear and moment. The two columns of triple dots on the right are conditionally formatted to find where the shear and moment are maximum. This spreadsheet exists for each inverted V-truss and the eccentric braced frame.

Frame Sections

Beam	W36X361	
Brace	W18X119	
Column	W14X370	
Story h	164	in
Bay w	30	ft
Brace L	243.5077	in
lu, x	30	ft
lu, y	15	ft
Fy, brace	50	ksi
Fu, brace	65	ksi
Fy, beam	50	ksi
Fu, beam	65	ksi
E	29000	ksi

Forces

Pu =	522.47	k
Py =	5300	k
Vu =	12.56	k
Δx =		in
Mu =	1514	ft-k

Factors

Øb =	0.9
Øv =	0.9
Øc =	0.9

Figure 70 – SCBF Design Spreadsheet - Input

Beam Properties		
bf =	16.7	in
tf =	2.01	in
tw =	1.12	in
d =	38	in
Ag =	106	in2
Z =	1550	in3
rx =	15.6	in
ry =	3.85	in
I =	25700	in4

Brace Properties		
bf =	11.3	in
tf =	1.06	in
tw =	0.655	in
d =	19	in
Ag =	35.1	in2
Z =	262	in3
ry =	2.69	in

Flange Width Comparison: Beam vs. Brace				
			bf, beam > bf, brace	YES
bf, beam	=	16.7		
bf. brace	=	11.3	Beam Flange Adequate	

Element Slenderness - Beam				
	λ_f =	4.15422886	$\lambda_f < \lambda_{ps}$	YES
	λ_p =	9.15161188	Flanges are Compact	
	λ_w =	33.9285714		
	λ_p =	90.5527912	$\lambda_w < \lambda_{ps}$	YES
			Web is Compact	

Brace Axial Force	
Ry =	1.1
Pt =	**1930.5**
KL/r =	90.52331
Fe =	34.92826
Fcr =	27.46372
Pc =	**289.1929**

Unbalanced Vertical Beam Load	
Pty =	1300.17244
Pcy =	194.768556
Qb =	1105.40388

Additional Beam Axial Force	
Ptx =	1427.0185
Pcx =	213.77037
Pu =	820.39445

Unbraced Length Check				
			Lb < Lp	YES
Lp =	9.29	ft		
dc =	17.9			
Lb =	8.544167		Controlling Limit State is Yielding	

Flexural Strength				
			Mu < ØbMn	YES
Mn =	77500	ft-k		
ØbMn =	69750	ft-k		
Mu =	1514	ft-k	Beam is Adequate in Flexure	

Compression Strength				
			Pu < ØcPn	YES
KLx/rx	23.07692	**Controls**		
Klxy/ry	46.75325			
ØcFcr =	38.5	ksi		
ØcPn =	4081	k	Beam is Adequate in Compression	

Combined Loading			Pr/Pc < 0.2	NO
Pe1 =	56757.84			
Cm =	1	Combined Ratio		Limit
B2 =	1	0.348814655	<=	1
Pr =	1342.864			
B1 =	1.024233			
Mrx =	1550.689			
Pr/Pc =	0.329053	Beam is Adequate in Combined Loading		

Shear Strength		Vu < ØVn	YES
h/tw =	33.92857		
2.24*(E*Fy)^0.5 =	53.9463437		
Aw =	38.0576		
Vn =	1141.728		
Vu =	1117.964	Beam is Adequate in Shear	

Beam is Adequate

Figure 71 – SCBF Inverted V Beam Design

Link Element			Forces			Factors	
Beam	W24X279		Pu =	530.63	k	Øb =	0.9
Brace	W18X143		Py =	4100	k	Øv =	0.9
e	48	in					
Story h	162.5	in	Vu =	579.04	k		
Bay w	30	ft					
			Δx =	0.1412	in		
Fy, beam	50	ksi					
Fu, beam	65	ksi					
E	29000	ksi					

Beam Properties	
bf =	13.3
tf =	2.09
tw =	1.16
h =	26.7
Ag =	82
Z =	835

Flange Width Comparison: Beam vs. Brace			bf, beam > bf, brace	YES
bf, beam	=	13.3		
bf. brace	=	11.2	Beam Flange Adequate	

Figure 72 – EBF Beam Input and Design

Link Element Slenderness			
	λ_f =	3.181818	$\lambda_f < \lambda_{ps}$ YES
	λ_{ps} =	7.224957	**Flanges Meet Local Buckling Criteria**
	λ_w =	23.01724	
	C_a =	0.143802	Ca > 0.125 YES
	λ_{ps} =	58.96869	$\lambda_w < \lambda_{ps}$ YES
			Web Meets Local Buckling Criteria

Link Shear Strength				
	0.15Py =	615 k	Pu > 0.15Py NO	
	Aw =	26.1232 in2	**Beam Axial Force Can Be Neglected in Shear Strength Determination**	
	Vp =	783.696 k		
	Vpa =	777.1048 k	**If Beam Axial Force Must Be Included:**	**If Beam Axial Not Included:**
	Mp =	41750 ft-k		
	Mpa =	42889.03 ft-k	Vu < Va YES	Vu < Vp YES
	Va =	699.3943 k	**Beam Link is Adequate in Shear**	**Beam Link OK**

Allowable Link Length				
	ρ' =	0.916396		
	$\frac{Vp * e}{Mp}$	=	0.901016	**Link Behavior Dominated by Shear Behavior**
	ρ' * (Aw/Ag)	=	0.291941	e < emax YES
	emax =	85.23713		**Link Length is OK**

Allowable Link Rotation			
	1.6 * (Mp/Vp) =	85.23713	
	2.6 * (Mp/Vp) =	138.5103	
	Θa =	0.08	γp < Θa YES
	Θp =	0.000869	
	γp =	0.006517	**Link Rotation OK**

Beam Link is Adequate

Figure 73 – EBF Link Design

Cd =	5

I =	1

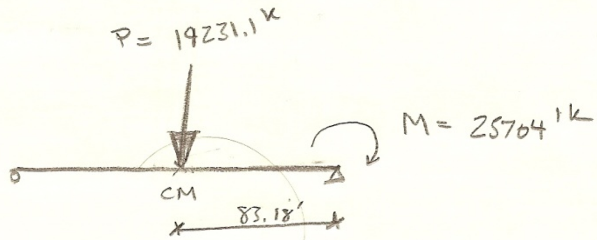
Siesmic Story Drift

Story	Load	Total Drift		Center of Mass		Story Height		Amplified Story Drift		Allowabl e Drift	Conclusion	
		UX	UY	X	Y	Z		X	Y		X	Y
ROOF	QUAKEX	1.8976	0.4132	1156.966	1064.648	1146.5		1.6055	0.4325	3.28	OK	OK
7TH	QUAKEX	1.5765	0.3267	1157.97	1065.287	982.5		1.7365	0.4505	3.28	OK	OK
6TH	QUAKEX	1.2292	0.2366	1158.218	1065.555	818.5		1.6	0.378	3.28	OK	OK
5TH	QUAKEX	0.9092	0.161	1157.048	1065.698	654.5		1.492	0.317	3.28	OK	OK
4TH	QUAKEX	0.6108	0.0976	1156.388	1066.125	490.5		1.2865	0.2525	3.28	OK	OK
3RD	QUAKEX	0.3535	0.0471	1156.881	1066.551	326.5		1.0615	0.16	3.28	OK	OK
2ND	QUAKEX	0.1412	0.0151	1157.853	1067.224	162.5		0.706	0.0755	3.25	OK	OK
ROOF	QUAKEXY1	1.8938	0.4415	1156.966	1064.648	1146.5		1.5965	0.4825	3.28	OK	OK
7TH	QUAKEXY1	1.5745	0.345	1157.97	1065.287	982.5		1.7255	0.499	3.28	OK	OK
6TH	QUAKEXY1	1.2294	0.2452	1158.218	1065.555	818.5		1.596	0.4035	3.28	OK	OK
5TH	QUAKEXY1	0.9102	0.1645	1157.048	1065.698	654.5		1.488	0.337	3.28	OK	OK
4TH	QUAKEXY1	0.6126	0.0971	1156.388	1066.125	490.5		1.287	0.2615	3.28	OK	OK
3RD	QUAKEXY1	0.3552	0.0448	1156.881	1066.551	326.5		1.064	0.159	3.28	OK	OK
2ND	QUAKEXY1	0.1424	0.013	1157.853	1067.224	162.5		0.712	0.065	3.25	OK	OK
ROOF	QUAKEXY2	1.9015	0.3849	1156.966	1064.648	1146.5		1.6145	0.383	3.28	OK	OK
7TH	QUAKEXY2	1.5786	0.3083	1157.97	1065.287	982.5		1.748	0.402	3.28	OK	OK
6TH	QUAKEXY2	1.229	0.2279	1158.218	1065.555	818.5		1.6045	0.352	3.28	OK	OK
5TH	QUAKEXY2	0.9081	0.1575	1157.048	1065.698	654.5		1.4955	0.2975	3.28	OK	OK
4TH	QUAKEXY2	0.609	0.098	1156.388	1066.125	490.5		1.2865	0.243	3.28	OK	OK
3RD	QUAKEXY2	0.3517	0.0494	1156.881	1066.551	326.5		1.0585	0.161	3.28	OK	OK
2ND	QUAKEXY2	0.14	0.0172	1157.853	1067.224	162.5		0.7	0.086	3.25	OK	OK
ROOF	QUAKEY	0.4358	3.0722	1156.966	1064.648	1146.5		0.522	2.804	3.28	OK	OK
7TH	QUAKEY	0.3314	2.5114	1157.97	1065.287	982.5		0.5035	2.856	3.28	OK	OK
6TH	QUAKEY	0.2307	1.9402	1158.218	1065.555	818.5		0.413	2.6615	3.28	OK	OK
5TH	QUAKEY	0.1481	1.4079	1157.048	1065.698	654.5		0.3345	2.323	3.28	OK	OK
4TH	QUAKEY	0.0812	0.9433	1156.388	1066.125	490.5		0.2365	2.0395	3.28	OK	OK
3RD	QUAKEY	0.0339	0.5354	1156.881	1066.551	326.5		0.1365	1.6245	3.28	OK	OK
2ND	QUAKEY	0.0066	0.2105	1157.853	1067.224	162.5		0.033	1.0525	3.25	OK	OK
ROOF	QUAKEYX1	0.4401	3.0405	1156.966	1064.648	1146.5		0.532	2.7485	3.28	OK	OK
7TH	QUAKEYX1	0.3337	2.4908	1157.97	1065.287	982.5		0.516	2.8015	3.28	OK	OK
6TH	QUAKEYX1	0.2305	1.9305	1158.218	1065.555	818.5		0.418	2.633	3.28	OK	OK
5TH	QUAKEYX1	0.1469	1.4039	1157.048	1065.698	654.5		0.3385	2.3005	3.28	OK	OK
4TH	QUAKEYX1	0.0792	0.9438	1156.388	1066.125	490.5		0.2365	2.029	3.28	OK	OK
3RD	QUAKEYX1	0.0319	0.538	1156.881	1066.551	326.5		0.1335	1.6255	3.28	OK	OK
2ND	QUAKEYX1	0.0052	0.2129	1157.853	1067.224	162.5		0.026	1.0645	3.25	OK	OK
ROOF	QUAKEYX2	0.4314	3.104	1156.966	1064.648	1146.5		0.5115	2.86	3.28	OK	OK
7TH	QUAKEYX2	0.3291	2.532	1157.97	1065.287	982.5		0.491	2.91	3.28	OK	OK
6TH	QUAKEYX2	0.2309	1.95	1158.218	1065.555	818.5		0.408	2.691	3.28	OK	OK
5TH	QUAKEYX2	0.1493	1.4118	1157.048	1065.698	654.5		0.3305	2.345	3.28	OK	OK
4TH	QUAKEYX2	0.0832	0.9428	1156.388	1066.125	490.5		0.237	2.05	3.28	OK	OK
3RD	QUAKEYX2	0.0358	0.5328	1156.881	1066.551	326.5		0.1395	1.6235	3.28	OK	OK
2ND	QUAKEYX2	0.0079	0.2081	1157.853	1067.224	162.5		0.0395	1.0405	3.25	OK	OK

Figure 74 – Seismic Drift

OVERTURNING PG 1 SMPJ

WIND



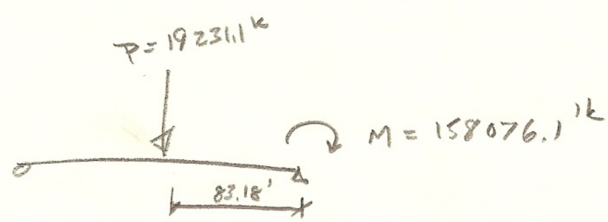
$Pd = 19231.1^{k} \; 83.18' = 1.6E6$

$M = 25704\ ^{'k}$

$\text{FACTOR OF SAFETY} = \dfrac{1.6E6}{25704} = 62.2 \gg 3$

OKAY ✓

SEISMIC



$\text{FACTOR OF SAFETY} = \dfrac{19231.1^{k} \cdot 83.18'}{158076.1\ ^{'k}}$

$= 10.12$ OK ✓

Figure 75 – Wind and Seismic Overturning Moments

Appendix C. Architectural Supplements

64 ELEMENT B: SHELL DESIGN CONSIDERATIONS

DESIGN CONSIDERATIONS

CLIMATE AND ENERGY

Of primary importance to the shell of a building is the mediation between the exterior and interior environment. Proper design and detailing of the building enclosure requires an understanding of the specific characteristics of both the desired interior environmental conditions and specific exterior environmental conditions, on both a macro and micro scale.

DEFINITIONS

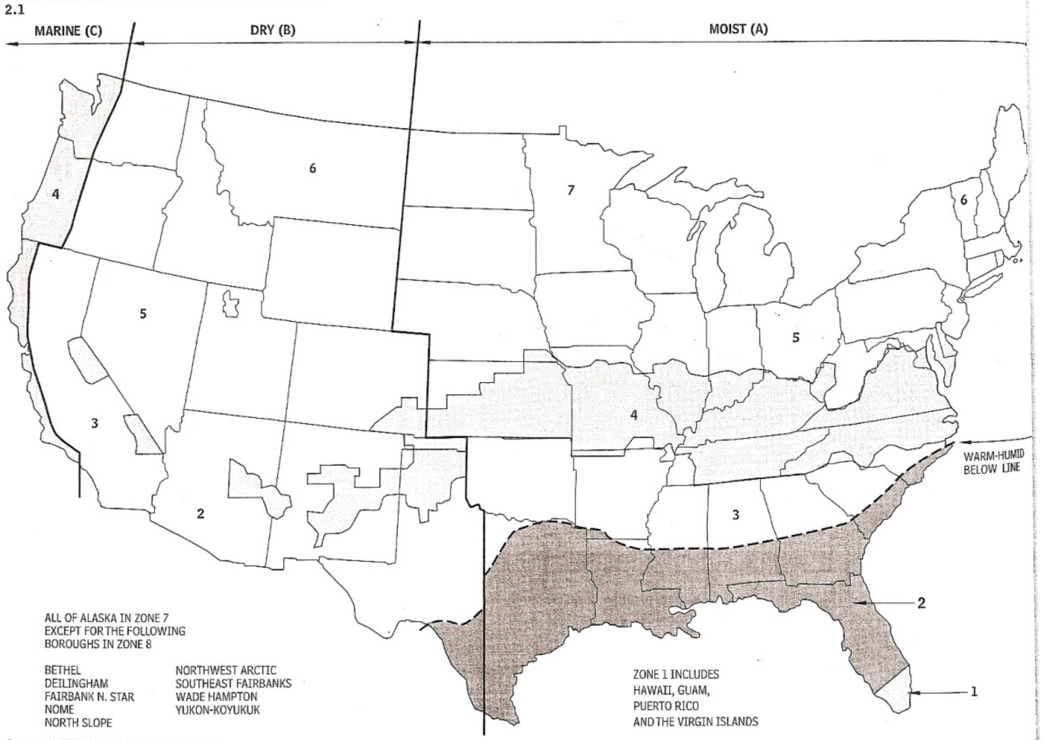
When reading the content of this chapter, keep in mind the following definitions of concepts and principles:

- *Air barriers*: Materials or combinations of materials that form a continuous envelope around all sides of the conditioned space to resist the passage of air. Joints, seams, transitions, penetrations, and gaps must be sealed. The air barrier must be capable of withstanding combined positive and negative wind load and fan and stack pressure without damage or displacement. The air barrier must be at least as durable as the overlying construction and be detailed to accommodate anticipated building movement. An air barrier may or may not be a vapor retarder.
- *Vapor barriers and retarders*: Without industrywide consensus, materials with a perm rating less than 1 are interchangeably called vapor barriers or vapor retarders (IBC and IEC 2003 use "vapor retarder"). More important than the term is to understand a few basic principles:
  - Vapor diffusion through materials with perm ratings less than 1 is nearly inconsequential, but even small gaps or holes can easily transport many times as much water vapor.
  - All materials have some greater or lesser degree of resistance to diffusion, and their placement in an enclosure assembly, whether intended as a retarder or not, will affect wetting and, more importantly, drying of an assembly.
- *Insulation*: A material that slows the flow of heat through conduction.
- *Radiant barriers*: A material, usually metallic or shiny, that reflects radiant thermal energy.
- *Weather barrier (water-resistant barrier)*: A material that is resistant to the penetration of water in the liquid state, or is waterproof. It may or may not be an air barrier or vapor retarder. The face of the weather barrier is sometimes called the *drainage plane*.
- *Barrier wall*: A wall assembly that resists moisture with a continuous waterproof membrane or with a plane of weather barrier material thick enough to prevent absorbed moisture penetrating to the interior.
- *Drained cavity wall*: A wall assembly with an outer water-[...]ding layer over an air cavity, and with a weather barrier. The [...]ity is flashed and weeped to drain incidental water.
- *Drainage plane wall*: A wall assembly with a continuous [...] resistant barrier under an outer water-shedding layer. The [...] a cavity limits the amount of water that can be quickly drai[...]
- *Pressure-equalized rainscreen wall*: A wall assembly [...] resists all the physical forces that can transport water acr[...] joint in the outer or "rainscreen" layer. Kinetic energy forces [...] controlled by venting a cavity behind the rainscreen and, [...] allowing the pressure differential across the joint to be eq[...]ized. An air barrier and compartmentalization of the cavity [...] required to control the pressure equalization. The cavi[...] flashed and weeped to drain incidental moisture.

EXTERIOR CLIMATIC INFLUENCE

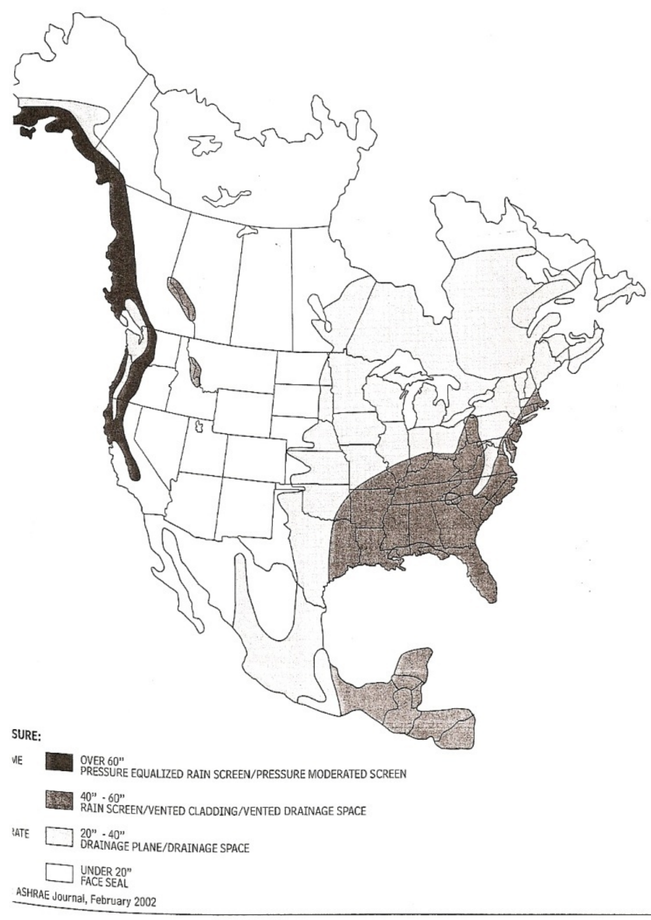
The United States has widely varying climates. More than the [...] ous extremes of Miami and Alaska are the subtler—and ju[...] important variations—within the contingent states. The A[...] ASHRAE/IESNA Standard 90.1 Map of Climate Zones for the U[...]

CLIMATE ZONES FOR UNITED STATES LOCATIONS
2.1



Source: ANSI/ASHRAE/IESNA Standard 90.1

Figure 76 – US Climate Zones (Architects, 2007)



DESIGN CONSIDERATIONS ELEMENT B: SHELL 65

reproduced in Figure 2.1 dictates zones based on heating
oling requirements. (Note: A simplified map of climatic zones
found in the book, *Moisture Control Handbook: Principles
actices for Residential and Small Commercial Buildings*, by
W. Lstiburek and John Carmody, 1996.) There are six zones
the continental states and Hawaii, plus two more for Alaska.
these zones are subzones for moist, dry, marine, and warm-

chapter will demonstrate, solutions appropriate for one
ay be totally unsuited for another. SEI/ASCE 7, "Minimum
Loads for Buildings and Other Structures," and other simi-
ndards establish the wind, snow, and seismic structural
n buildings. Again, there is wide variation in wind speed,
ll, and ground movement. In addition to the base loads,

localized conditions such as surrounding topography and adjacent buildings can cause wide variances in the environmental influences. Figure 2.2 shows the annual precipitation for North America. Suggested types of exterior enclosure systems that will meet the minimum level of service and reliability are correlated to the rainfall levels.

INTERIOR CLIMATIC INFLUENCE

Environmental conditions to be maintained within the building also influence the design of the shell. Buildings with requirements for high or low levels of humidity, tight temperature tolerances, pressure differentials to the exterior, high-reliability containment, acoustic isolation, protection from blast or forced entry, high indoor air quality, or other extraordinary requirements will require particular attention to system selection and detailing, in concert with consideration of the exterior climate.

HEAT, AIR, AND MOISTURE

In addition to the obvious structural loads, the building enclosure must resist the transfer of heat, air, and moisture (HAM). The laws of physics dictate that heat always flows from hot to cold. Air moves through building enclosures by passing through porous materials, or through holes and gaps in nonporous materials, based on differential air pressures. Moisture, as water in the liquid state (such as rain, snow, and groundwater), moves through enclosures by four methods: capillary action, surface tension, gravity, and kinetic energy (e.g., wind-driven rain). Moisture in the vapor state moves through enclosures from zones of higher to lower vapor pressures, by diffusion through solid materials or by air transport through holes.

CONTROL OF HAM

Control of the flow of HAM across the building enclosure is an interrelated problem, in that air movement can create the kinetic energy that pulls water through joints, dramatically reduce thermal insulation effectiveness, or cause massive vapor transport. Improper thermal insulation can cause condensation on uncontrolled surfaces.

To control HAM, three components must be considered separately: heat, air, and moisture.

Heat is most commonly controlled by thermal insulation. Keep in mind the following:

- Air movement around thermal insulation can seriously degrade its effectiveness, so avoid systems that ventilate the conditioned side of the thermal insulation.
- Radiant barriers may be effective, particularly in hot climates, but they must have an airspace on the warm side. Generally speaking, radiant barriers have virtually no insulating value and should not replace but, instead, enhance typical thermal insulation and conductive losses.
- Thermal short circuits can dramatically reduce the U-value of thermal insulation. The most common example is metal studs, which may reduce the effective value of thermal insulation between the studs by half.

Air transfer is controlled by a coordinated and continuous system of air barriers for all six sides of the enclosure (i.e., the lowest grade level, foundation walls, exterior walls, and the roof).

- Common approaches to wall air barriers are continuous membranes applied to sheathing and sealed to windows, doors, and penetrations.
- Below-grade assemblies can utilize either the concrete walls and slabs or applied waterproofing membranes.
- Most typical low-slope roof membranes will provide an air barrier, except for mechanically fastened systems that may not be able to resist all of the required loads.
- It is possible to design the gypsum board as an air barrier, if all joints and cracks are sealed.
- Many air barrier systems require a combination of a membrane and a structural panel to resist loading, such as spun-bond polyolefin membranes stapled to sheathing or bituminous membranes adhered to CMU.

Moisture management consists of controlling moisture entry, moisture accumulation, and allowing for drying.

- Perfect barriers to moisture are virtually impossible to achieve; therefore, it is important that measures taken to keep out moisture do not also trap moisture—for example, waterproofing membranes that trap thermal insulation between a vapor retarder.
- It is essential to maintain a balance of the moisture that is able to accumulate in an assembly between drying cycles. Accumulation and drying are extremely dependent on the local climate. Some materials such as wood-framed walls and masonry have the capacity to absorb relatively large quantities of moisture and to then later dry out without damage or deterioration. Other systems such as gypsum board on metal studs have very little capacity for the storage of moisture.

Figure 77 – US Rainfall Data
(Architects, 2007)

ELEMENT B: SHELL DESIGN CONSIDERATIONS

- The source of water is primarily rain, which should be limited by a reasonably detailed assembly based on the expected amount of precipitation. The precipitation map in Figure 2.2 shows recommended enclosure types along with the required performance to minimize water entry.
- Below grade, the primary source of moisture is through capillary action that can be controlled through membranes and capillary breaks.
- Sources of vapor may be in the interior or exterior environment. Vapor retarders have been the traditional method used to control vapor movement, but their use in mixed heating and cooling climates must be carefully evaluated to allow drying.
- Moisture control in the solid state (i.e., ice) depends on not letting liquid water freeze; or, if it does, allowing room for expansion. For example, cold roof surfaces that eliminate thawing also prevent ice buildup, and air-entrained concrete provides room for ice crystals to expand.

Figures 2.3 and 2.4 show details of wall assemblies that can be used for analysis of drying under various climatic conditions. The various assemblies are somewhat independent of the cladding type. Other wall assemblies, including face-sealed or massive barrier assemblies, should receive similar analysis of HAM control. Two useful tools for this purpose are:

- *Computerized modeling of wetting and drying of walls*: This is widely available and is very helpful to understanding moisture accumulation and drying. Analysis is recommended for large projects and any assembly that requires seasonal drying. Mixed climates may be the most difficult to predict by rule of thumb or empirical analysis. WUFI, developed by the Fraunhofer Institute for Building Physics in Germany with a North American version developed jointly with Oak Ridge National Laboratory (www.ornl.gov) is widely recognized modeling tool. Similar software is available through www.virtual-north.com/download/OrderForm.pdf and www.architects.org/emplibrary/HAMtoolbox.pdf.
- *Manual analysis of simple two-dimensional diagrams of wall sections*: This involves using temperature gradients plotted against dew point temperature or vapor-pressure gradients plotted against saturation pressure. For instructions refer to "Design Tools," by Anton TenWolde (Chapter 11 in the manual *Moisture Control in Buildings* [MNL18], Heinz R. Trechsel, editor, published by ASTM, 1994).

CONSIDERATIONS FOR CLIMATE ZONES

GENERAL

- Refer to specific information for each material for more information regarding selection criteria and proper detailing.
- Include only one vapor retarder in a wall assembly, and ensure that all other materials are increasingly permeable from the vapor retarder out.
- It is acceptable (and sometimes desirable) to provide more than one air barrier in a wall assembly.
- It is generally desirable to protect blanket insulation from air-washing with an air barrier on the cold side.

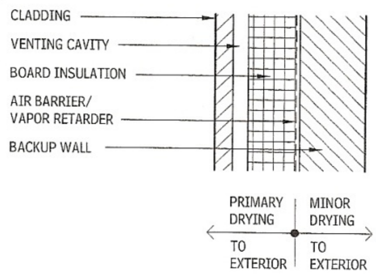
ALL CLIMATES

- Highly reliable enclosure system to control HAM in all climate zones, without relying on building mechanical systems to dry interior air.
- Thermal insulation located outside of structure and wall framing allows easy installation of continuous air barriers and vapor retarders.
- Thermal insulation must be continuous to prevent the vapor retarder from reaching the dew point.
- Excellent choice for masonry veneer over CMU or metal stud backup systems.
- If metal stud backup systems are used, do not place thermal insulation between the studs.
- Any paint or wall covering is allowed on interior finish.

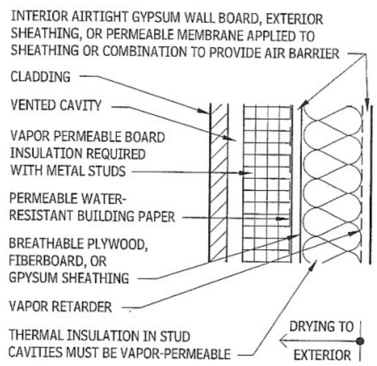
COLD CLIMATES (Zones 5 to 8)

- Materials should be progressively more permeable, because they are located closer to exterior face.
- Any paint or wall covering is allowed on interior finish.

ALL CLIMATES AND COLD CLIMATES
2.3



ALL CLIMATES



COLD CLIMATES

- Mechanical system is not required to dry interior air.
- Failure of the building paper may allow moisture accumulation that cannot be overcome by drying.
- Elements penetrating thermal insulation, (such as beams supporting a projecting canopy or the sump pan of roof drains) can cause condensation problems, unless they are insulated with closed-cell thermal insulation or a thermal insulation with a vapor retarder to keep moisture-laden air from getting to these surfaces. This is particularly true for occupancies with high humidity, (including residences, hospitals, museums, swimming pools,).

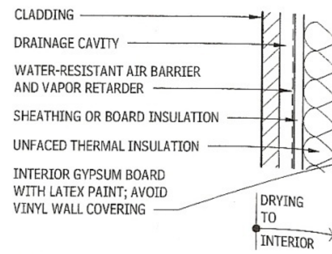
HOT CLIMATES (Zones 1, 2, and 3)

- The mechanical system must provide dehumidification of interior air for drying.
- Avoid any vapor-impermeable interior finishes (e.g., a vinyl wall covering that will trap moisture).
- A radiant barrier may be incorporated into the cavity.
- Taped joints in sheathing, board insulation, or a combination may provide air barrier.
- An air barrier is crucial to limit moisture transport through imperfections in the vapor retarder.

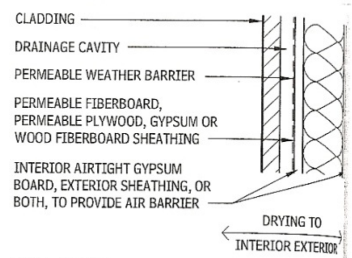
MIXED CLIMATES (Zones 3 and 4)

- All materials must be relatively vapor-permeable to allow drying in both directions, because seasons change direction of heat flow and vapor drive.
- Detail system with interior and exterior side-permeable air barriers to limit moisture transport and infiltration/exfiltration.
- May be possible to use board insulation with taped joints as sheathing, which will form a vapor retarder if board and blanket insulation have approximately the same U-value.

HOT, HUMID CLIMATES AND MIXED CLIMATES
2.4



HOT, HUMID CLIMATES



MIXED CLIMATES

SUSTAINABILITY AND ENERGY

The building shell should be a major part of the sustainable stra[tegy]. At a minimum, the shell should:

- Contribute to minimizing energy usage.
- Incorporate environmentally sensitive materials.
- Ensure good indoor air quality and occupant comfort.
- Be durable.

For high-performance building projects, the enclosure could [also] generate energy, return nutrients to the environment, and fil[ter] pollutants.

One area of special concern for the building shell is durability, e[ven] though it currently is not included in LEED evaluations in the Uni[ted] States. (It is included in Canadian LEED programs). The build[ing] superstructure and enclosure are frequently portions of the buil[d]ing that should last the longest and are the most difficult to rep[air] or replace. Buildings that perform well for many years slow[ly] reduce the consumption of resources and the wastestrea[m]. Failures of the enclosure can lead not only to water-damag[ed] materials needing repair or replacement but also to unnecessa[ry] long-term energy consumption, toxic mold, and sick buildings.

Buildings are major consumers of energy, so the enclosures sho[uld] be part of a strategy to reduce energy consumption. In fact, cre[at]ing a well-performing enclosure is considered to be the first st[ep] in reducing energy usage, ahead of other more sophisticat[ed] strategies, such as high-performance mechanical systems. A th[or]ough understanding of the interior and exterior environment[s] is paramount. For residential buildings in cold climates, heat lo[ss] through the enclosure may be the largest component of total en[er]gy consumption. For large commercial buildings in a moder[ate] environment, daylighting schemes may save more energy, even [if] they may result in an enclosure with lower thermal resistance.

Most jurisdictions require compliance with an energy conserv[ation] code. ASHRAE 90.1 and the International Energy Code (in vari[ous] editions) are common model codes. These minimum standar[ds] should be exceeded by 20 to 50 percent, if possible.

NOTES

2.3 and 2.4 Provide an air barrier in the assembly at one or more of the locations noted by properly detailing either the inner layer of gypsum board, the sheathing layer, or the permeable weather barrier. The inner gypsum board can be made an air barrier by sealing the perimeter, penetrations, and transitions to adjacent air barrier assemblies. The sheathing can be made an air barrier through similar means of sealing all joints, penetrations, and transitions. Using a membrane over the sheathering (either fluid-applied or sheet material) that is vapor permeable, weather-resistant, anmd airtight is extremely effective for providing an air barries with the added benefits of simple installation and inspection.

Consult th
- Design Vertica
- Moistu Reside Lstiburi
- Water by Willi

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DESIG

SELEC

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NOTES
2.5 a. Floors cles shall be lowing co not more for mech passeng b. The library

Figure 78 – Shell Design
(Architects, 2007)

Appendix D. Mechanical Breadth

MECHANICAL BREADTH — PG 1 SMPJ

1) TYPICAL FLOOR

↳ SQUARE FOOTAGE:

FIRST FLOOR	29 537 GSF	
$\boxed{\text{TYPICAL 2-4}}$	30,550 GSF	(x 3)
FIFTH	28,420 GSF	

↳ WALL LENGTH:

PLAN NORTH:

$15' + 30 \cdot 6 + 24.854 = 220$ LF

PLAN SOUTH: 205 LF

PLAN EAST/WEST:

$20' + 30 \cdot 4 + 8 + 24 + 11'\text{-}10 + 4 = 190$ LF

↳ HEIGHT OF LEVEL: TYPICAL 13'-8"

↳ OCCUPANCY: OFFICE

MAX # PPL: 1,508 PPL WHOLE

↳ % GLASS:

14.52
5.25

• EAST ELEVATION

$$A_{BRICK} = (58' \cdot 13.67' - 2 \cdot 8'^2) + (2 \cdot 4' \cdot 13.67') + (47' \cdot 13.67' - 8'^2)$$

$$= 1353 \text{ FT}^2$$

$$A_{glass} = 2 \cdot 8^2 + 8^2 + 13.67' \cdot 11' + 2 \cdot 4' \cdot 13.67' + 56' \cdot 13.67' + 4 \cdot 13.67'$$

$$= 1272 \text{ FT}^2$$

$$A_{MULLIONS \,\&\, ALUMINUM} = 0.1 \cdot A_{glass} = 127 \text{ FT}^2$$

$$\% \text{ GLASS} = \frac{1272 - 127}{1353 + 1272} = 0.44$$

MECHANICAL BREADTH PG 2 SMPJ

- WEST ELEVATION:

$$A_{BRICK} = \left[(124 + 52)13.67 - 7 \cdot 8^2\right] = 1958 \text{ FT}^2$$

$$A_{glass} = 4' \cdot 13.67 + 11 \cdot 13.67 + 7 \cdot 8^2 = 653 \text{ FT}^2$$

$$A_{mullions} = 0.05 A_{glass} = 0.05(653) = 33 \text{ FT}^2$$

$$\% \text{ GLASS} = \frac{653 - 33}{1958 + 653} = 0.18$$

- SOUTH ELEVATION:

$$A_{brick} = 7' \cdot 13.67' \cdot 3 + 64' \cdot 13.67 - 4 \cdot 8^2 + 11' \cdot 13.67'$$

$$= 1056 \text{ FT}^2$$

$$A_{glass} = 4 \cdot 8^2 + 50' \cdot 13.67 + 26 \cdot 13.67 \cdot 2$$

$$= 1650 \text{ FT}^2$$

$$A_{\substack{mullions \\ + aluminum}} = 0.1 A_{glass} = 165 \text{ FT}^2$$

$$\% A_{glass} = \frac{1650 - 165}{1056 + 1650} = 0.55$$

- NORTH ELEVATION

$$A_{brick} = (7' \cdot 4 + 4' \cdot 4)\, 13.67' = 602 \text{ FT}^2$$

$$A_{glass} = (18' + 26' \cdot 4 + 50' + 8')\, 13.67' = 2460 \text{ FT}^2$$

$$A_{\substack{mullions \\ + Aluminum}} = 0.1 A_{glass} = 246 \text{ FT}^2$$

$$\% \text{ GLASS} = \frac{2460 - 246}{2460 + 602} = 0.72$$

$$\text{TOTAL } \% \text{ GLASS} = \frac{1272 - 127 + 653 - 33 + 1650 - 165 + 2460 - 246}{1353 + 1272 + 1958 + 653 + 1056 + 1650 + 2460 + 602}$$

$$= 0.497 \cdot 100 = \boxed{49.7\ \%}$$

MECHANICAL BREADTH PG 2 SMPJ

↳ R-VALUES WALLS & ROOF

NOT AVAILABLE

↳ U-VALUES WINDOWS

ASSUME SPECTRICALLY SELECTIVE TINT DOUBLE GLAZED

↳ U = 0.24 → 0.3 (TABLE 2.491 ARCH. GRAPH. STANDARDS)

(2) BUILDING INFORMATION

- EXISTING
  - ↳ PITTSBURGH, PA
  - ↳ TOTAL FLOORS: 5
  - ↳ WINDOW SHADING UNKNOWN
  - ↳ ADJACENT BUILDINGS:
    - SIMILAR 3 TO 5 STORY BUILDINGS ACROSS STREET ON WEST SIDE ONLY

- NEW
  - ↳ OAKLAND, CA
  - ↳ TOTAL FLOORS: 7

EXISTING

System Checksums
By PSUAE

System - 001 — Variable Volume Reheat (30% Min Flow Default)

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak — Peaked at Time: Mo/Hr: 7 / 15; Outside Air: OADB/WB/HR: 86 / 71 / 95

Clg Space Peak — Mo/Hr: 9 / 13; OADB: 76

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0.00	0	0.00
Skylite Cond	0	0	0	0.00	0	0.00
Roof Cond	0	87,770	87,770	2.56	0	0.00
Glass Solar	433,559	0	433,559	12.62	680,221	33.76
Glass Cond	102,275	0	102,275	2.98	11,064	0.55
Wall Cond	77,317	17,260	94,577	2.75	55,129	2.74
Partition	0		0	0.00	0	0.00
Exposed Floor	0		0	0.00	0	0.00
Infiltration	0		0	0.00	0	0.00
Sub Total ==>	613,151	105,031	718,182	20.91	746,414	37.05
Internal Loads						
Lights	500,482	125,121	625,603	18.21	500,482	24.84
People	679,500		679,500	19.78	377,500	18.74
Misc	234,601	0	234,601	6.83	208,534	10.35
Sub Total ==>	1,414,583	125,121	1,539,704	44.83	1,086,517	53.93
Ceiling Load	230,151	-230,151	0	0.00	181,836	9.03
Ventilation Load	0	0	906,455	26.39	0	0.00
Ov/Undr Sizing	0		0	0.00	0	0.00
Exhaust Heat		-26,844	-26,844	-0.78		
Sup. Fan Heat			185,836	5.41		
Ret. Fan Heat		111,502	111,502	3.25		
Duct Heat Pkup		0	0	0.00		
Reheat at Design			0	0.00		
Grand Total ==>	2,257,886	84,657	3,434,834	100.00	2,014,767	100.00

HEATING COIL PEAK

Mo/Hr: 13 / 1; OADB: 5

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-76,641	2.48
Glass Solar	0	0	0.00
Glass Cond	-633,320	-633,320	20.48
Wall Cond	-136,725	-170,552	5.52
Partition	0	0	0.00
Exposed Floor	0	0	0.00
Infiltration	0	0	0.00
Sub Total ==>	-770,045	-880,513	28.47
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-110,468	0	0.00
Ventilation Load	0	-2,095,258	67.75
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		-116,642	3.77
Additional Reheat		0	0.00
Grand Total ==>	-880,513	-3,092,413	100.00

TEMPERATURES

	Cooling	Heating
SADB	60.4	90.6
Plenum	79.8	67.7
Return	75.8	70.0
Ret/OA	78.3	21.1
Fn MtrTD	0.1	0.0
Fn BldTD	0.3	0.0
Fn Frict	0.9	0.0

AIRFLOWS

	Cooling	Heating
Vent	30,200	30,200
Infil	0	0
Supply	129,216	40,138
MinStop/Rh	40,138	40,138
Return	129,216	40,138
Exhaust	30,200	30,200
Rm Exh	0	0
Auxiliary	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	23.4	75.2
cfm/ft²	0.85	0.26
cfm/ton	451.43	
ft²/ton	533.65	
Btu/hr·ft²	22.49	-20.24
No. People	1,510	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	286.2	3,434.8	2,581.0	125,439	78.3	64.8	74.5	59.0	55.8	64.5
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	286.2	3,434.8								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	152,750		
Part	0		
ExFlr	0		
Roof	30,550	0	0
Wall	56,350	16,905	30

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-1,351.6	40,138	59.0	90.6
Aux Htg	0.0	0	0.0	0.0
Preheat	-1,740.8	30,200	5.0	59.0
Humidif	0.0	0	0.0	0.0
Opt Vent	0.0	0	0.0	0.0
Total	-3,092.4			

Project Name:
Dataset Name: C:\CDS\TRACE700\Projects\Sam Thesis.trc

TRACE® 700 v4.1 calculated at 01:36 PM on 04/02/2008
Alternative - 1 System Checksums report Page 1 of 1

Figure 79 – TRACE Results

OAKLAND

System Checksums
By PSUAE

System - 001 — **Variable Volume Reheat (30% Min Flow Default)**

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak — Peaked at Time: Mo/Hr: 7 / 15; Outside Air: OADB/WB/HR: 98 / 70 / 65

Clg Space Peak — Mo/Hr: 8 / 15; OADB: 97

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0.00	0	0.00
Skylite Cond	0	0	0	0.00	0	0.00
Roof Cond	0	277,480	277,480	6.25	0	0.00
Glass Solar	732,080	0	732,080	16.50	810,512	27.28
Glass Cond	163,302	0	163,302	3.68	153,291	5.16
Wall Cond	135,656	29,412	165,068	3.72	138,293	4.65
Partition	0		0	0.00	0	0.00
Exposed Floor	0		0	0.00	0	0.00
Infiltration	0		0	0.00	0	0.00
Sub Total ==>	1,031,038	306,892	1,337,930	30.15	1,102,097	37.10
Internal Loads						
Lights	700,675	175,169	875,844	19.74	700,675	23.58
People	677,250		677,250	15.26	376,250	12.66
Misc	328,442	0	328,442	7.40	328,442	11.06
Sub Total ==>	1,706,367	175,169	1,881,536	42.41	1,405,367	47.30
Ceiling Load	482,061	-482,061	0	0.00	463,491	15.60
Ventilation Load	0	0	770,700	17.37	0	0.00
Ov/Undr Sizing	0		0	0.00	0	0.00
Exhaust Heat		-26,756	-26,756	-0.60		
Sup. Fan Heat			295,982	6.67		
Ret. Fan Heat		177,589	177,589	4.00		
Duct Heat Pkup		0	0	0.00		
Reheat at Design			0	0.00		
Grand Total ==>	3,219,466	150,833	4,436,981	100.00	2,970,955	100.00

HEATING COIL PEAK

Mo/Hr: 13 / 1; OADB: 32

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	-130,775	6.00
Glass Solar	0	0	0.00
Glass Cond	-276,707	-276,707	12.69
Wall Cond	-111,904	-138,840	6.37
Partition	0	0	0.00
Exposed Floor	0	0	0.00
Infiltration	0	0	0.00
Sub Total ==>	-388,612	-546,323	25.06
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	-157,712	0	0.00
Ventilation Load	0	-1,275,032	58.49
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		-358,424	16.44
Additional Reheat		0	0.00
Grand Total ==>	-546,324	-2,179,778	100.00

TEMPERATURES

	Cooling	Heating
SADB	61.9	77.6
Plenum	82.1	67.7
Return	75.8	70.0
Ret/OA	79.1	52.2
Fn MtrTD	0.1	0.0
Fn BldTD	0.3	0.0
Fn Frict	0.9	0.0

AIRFLOWS

	Cooling	Heating
Vent	30,100	30,100
Infil	0	0
Supply	203,381	64,185
MinStop/Rh	64,185	64,185
Return	203,381	64,185
Exhaust	30,100	30,100
Rm Exh	0	0
Auxiliary	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	14.8	46.9
cfm/ft²	0.95	0.30
cfm/ton	550.05	
ft²/ton	578.37	
Btu/hr·ft²	20.75	-10.19
No. People	1,505	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB/WB/HR °F	Enter DB/WB/HR °F	Enter DB/WB/HR gr/lb	Leave DB/WB/HR °F	Leave DB/WB/HR °F	Leave DB/WB/HR gr/lb
Main Clg	369.8	4,437.0	4,137.0	199,788	79.1	64.0	65.1	60.6	56.8	63.0
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	369.8	4,437.0								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	213,850		
Part	0		
ExFlr	0		
Roof	91,650	0	0
Wall	78,890	23,667	30

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-1,221.3	64,185	60.6	77.6
Aux Htg	0.0	0	0.0	0.0
Preheat	-958.5	30,100	32.0	60.6
Humidif	0.0	0	0.0	0.0
Opt Vent	0.0	0	0.0	0.0
Total	-2,179.8			

Project Name:
Dataset Name: C:\CDS\TRACE700\Projects\Sam Thesis.trc

TRACE® 700 v4.1 calculated at 01:42 PM on 04/02/2008
Alternative - 2 System Checksums report Page 1 of 1

Figure 80 – TRACE Results

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