TECHNICAL REPORT III

University Academic Center

Eastern USA

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Structural Option

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Executive Summary

This report focused on the lateral force resisting system used in the University Academic Center. The lateral system consists of 15 total frames, most concentrically braced and some moment frames. Analysis of this system was done using ETABS modeling to facilitate calculations. Data for torsion, story drift, and displacements was interpreted from the ETABS results and compared to current code provisions.

University Academic Center is an asymmetric building with multiple roof levels stepping down from a five story office wing to the single story library area. This variation in form provides an appealing architecture, but complicates the structural design, creating eccentricities and requiring more consideration into framing to minimize displacements.

The framing for this building is a structural composite steel system of mostly wide flange members with HSS diagonal bracing members in the lateral resisting system. Lateral forces are resisted by a combination of 15 braced frames spread throughout the building with the majority located in the central classroom wing. Due to the rotation of several of the braced frames hand calculations would prove more difficult so computer modeling software was used in a large part of this report.

Preliminary determination of load values was shown including snow and drift loads, dead loads, live loads, wind loads in generalized N-S and E-W directions, and seismic loads. The lateral loads were then input into the ETABS model to analyze maximum cases in story drift and overall displacements as well as torsional effects on the building.

Story drifts and displacements were all determined to be within code limitations with maximum drift and displacements occurring under wind case 3 loading. Maximum displacement occurred in an E-W shift of 0.9722 in at the 5th floor roof. Maximum story drift also occurred in this direction at the ground floor with a story drift of 0.2598 in.

ETABS output showed a large eccentricity resulting in center of mass and center of rigidity displacement on all levels in the range of 20-40 feet on average. This was checked with quick hand calculations and assumed accurate. Such an eccentricity results in high torsional effects which could be easily seen in the ETABS model animation. Despite these rotations University Academic Center still maintains enough rigidity to stay within code limitations for displacements.

Member checks were also done to verify individual members were not overstressed in the model assumptions. Those members checked remained stable based on loading determined by ETABS output, further validating the results of this study.

Introduction

Located in the eastern United States, the University Academic Center is a 192,000 square foot building designed to house a library resource center, dining area, 45 classrooms, and over 120 offices. Other key features include a 5-story atrium and multiple roof gardens.

The layout of the building consists of three main sections. The northern 3-story section contains mostly dining and classroom areas. In the center of the building, a 4 story section houses the library and the majority of classrooms, as well as acting as the main entrance. The southern end of the building consists almost entirely of office spaces. On either side of the center section are the vertical circulation cores which also provide access to the roof gardens.

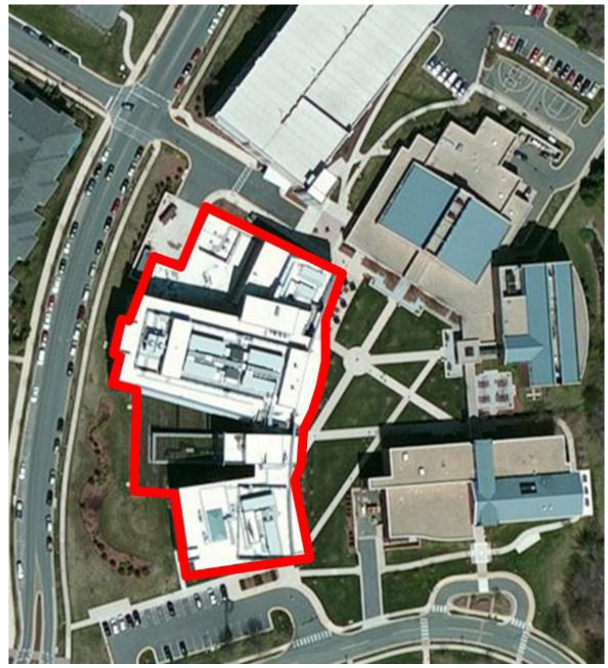


Photo taken from Bing Maps

There are 4 main types of building façade implemented in this building. The 3 and 5-story sections of the building have a brick façade with cast stone bands running horizontally across the brick surface. Glass curtain walls are used in the vertical circulation located on either side of the 4-story section. The 4-story section's façade is mostly metal panels. There is also glazed CMU used to accent the other façade types at various places.

Through the use of multiple energy saving techniques the University Academic Center holds a LEED gold rating. This includes energy efficient HVAC equipment and the use of natural daylighting, as well as shading devices, to help minimize energy consumption. All these features, along with the roof gardens, provide a "green" learning environment. LEED credits were also gained through site design to minimize storm water runoff, use of recyclable and local materials, and the addition of bike racks and on site showering facilities to promote alternative modes of transportation.

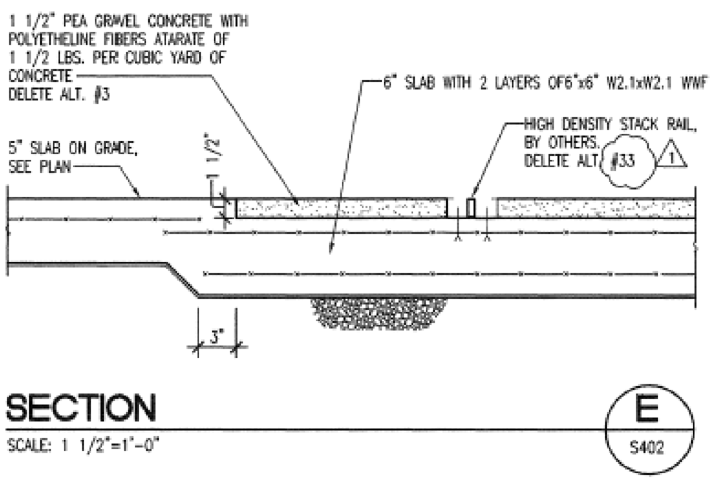
Structural Overview

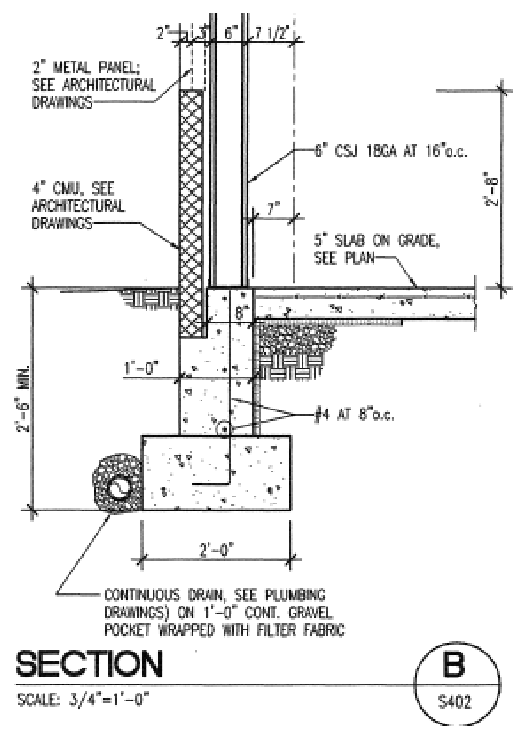
The University Academic Center is a steel framed building with composite metal decking supported by a foundation of spread footings and slab-on-grade. The building resists lateral forces by a combination of braced and moment frames.

Foundation

Based on the 2002 geotechnical report taken, footings for University Academic Center are designed for an allowable bearing capacity of 3,000 psf. Footings are placed on undisturbed soil or on structurally compacted fill. The bottoms of exterior footings are 2'-6" below grade.

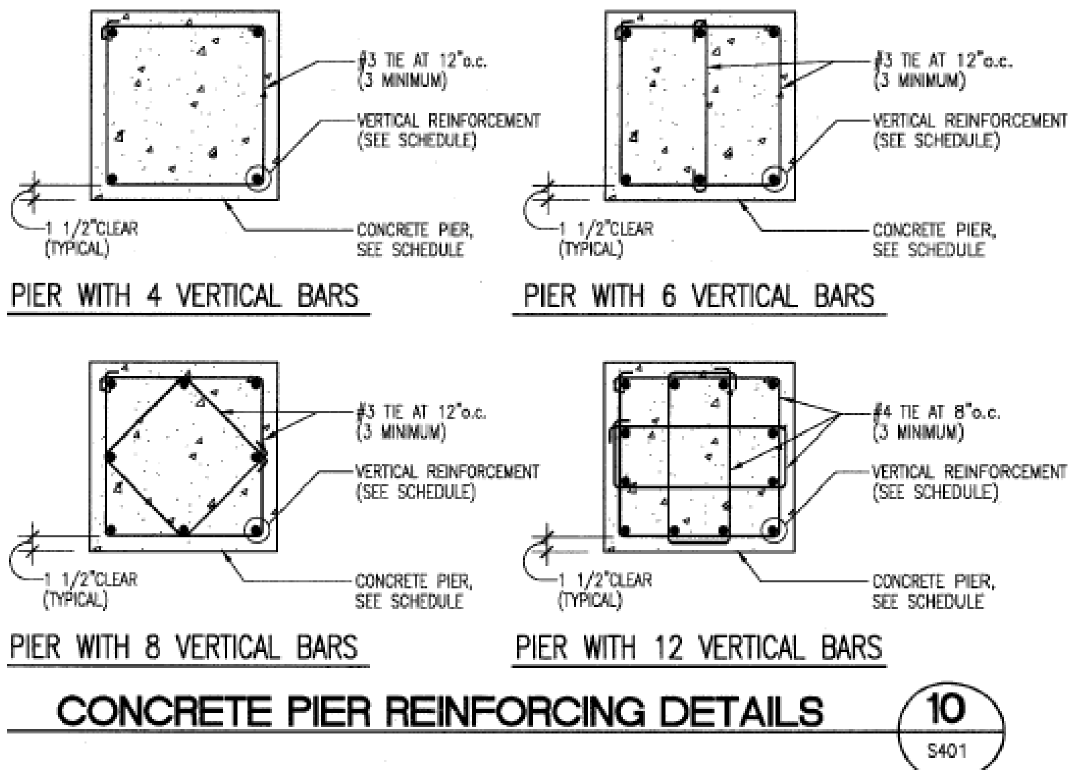
Slab-on-grade sits on a coarse granular fill material compacted to 95% of maximum density as defined by ASTM D1557 modified proctor test. The slab-on-grade is designed as 5" thick concrete reinforced with 6"x6", W1.4xW1.4 WWF. This is the reinforcement for all slab-on-grade except for the area located under the library stacks which is 6" thick concrete reinforced with 2 layers of 6"x6", W2.1xW2.1 WWF.





Drawings provided by Skanska

The columns in the University Academic Center bear on piers ranging in size depending on loading and connection type. These piers are a minimum of 8" ranging to a maximum depth of 3'-9". The piers come in 4 types: 4, 6, 8, and 12 vertical bar piers. Footings also range in size under the columns with a maximum 19'x19' under a single column.

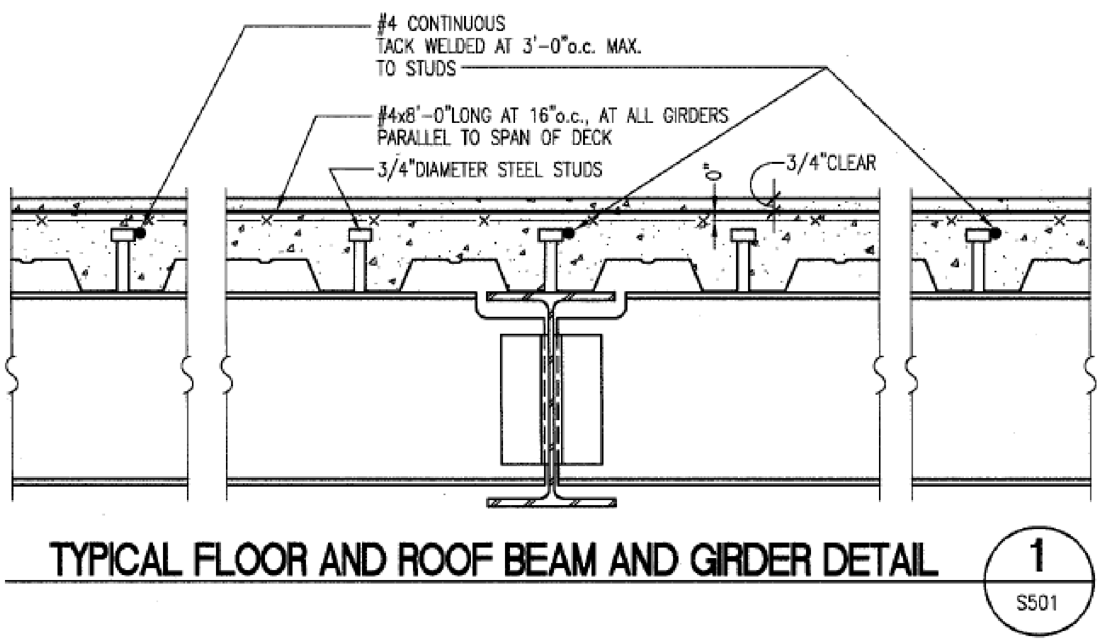


Drawings provided by Skanska

Floor and Roof Systems

The University Academic Center utilizes a composite metal deck flooring system. This includes 2" composite 20 gage deck with ribs 12" o.c. and 1.5" type B, wide rib 20 gage deck. All metal deck is designed to be continuous over 3 spans. Floor system also includes shear studs and lightweight concrete topping varying based on location and loading.

Roofing systems also varies due to some areas like the roof gardens and mechanical spaces of greater loading. Decking for roofs includes both 2" composite 18 gage deck with ribs 12" o.c. and 1.5" type B, wide rib 20 gage deck, covered by a built up roof and rigid insulation.



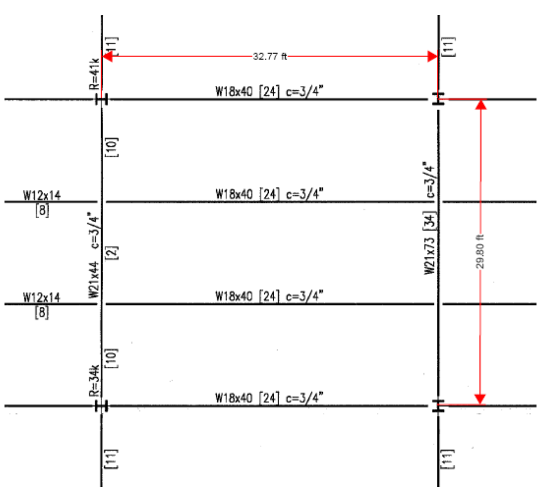
Drawings provided by Skanska

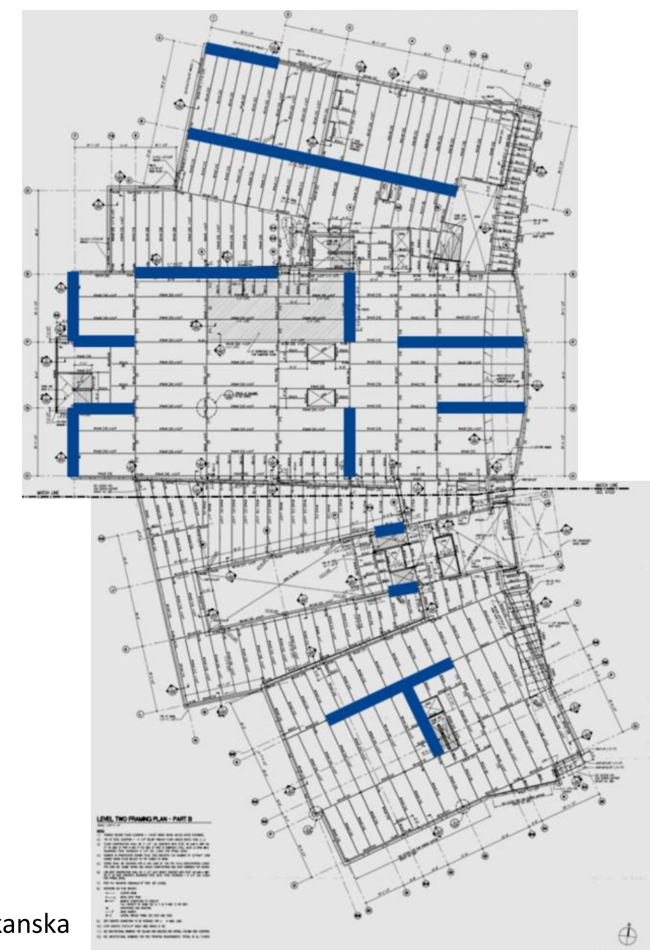
Framing System

The framing system for the University Academic Center includes C-shapes, HSS members, and Wide Flange members with the majority being W-shapes. Gridlines are set at multiple angles with bay sizes varying throughout the building. Areas with consistent framing between floors are located in the classroom wing in the central section of the building and the office spaces on the south side. The gravity system transfers vertical loads due to dead, live, and snow loading across a floor or roof deck, into beams and girders, and is take as axial force in columns to the foundation.

Lateral System

The lateral system for this building includes braced frames of varying heights and types located throughout the building. Below is a plan view of University Academic Center with the 15 lateral braced frames shown in blue. These frames resist the forces on the building due to wind and seismic loading. The wind loads are taken into the floor diaphragm from the façade and distributed amongst the bracing based on relative stiffness. The frames in turn transfer these loads to the foundation. A braced framing system is logical with a steel building given the lightweight paired with relative stiffness. Where shear walls would limit the circulation throughout the building, using knee braces, as University Academic Center does in multiple locations, allows for more useable space. Braced frames are also stiffer than moment framing alternatives and cheaper to construct.





Drawings provided by Skanska

Codes and Standards

As Designed:

- 2000 ICC International Building Code
- 2000 ICC International Mechanical Code
- 2000 ICC International Plumbing Code
- 2000 ICC International Fuel-Gas Code
- 2000 ICC International Fire Code
- 2000 ICC International Energy Conservation Code
- 2000 NFPA Life Safety Code
- 2000 Americans with Disabilities Act – Accessibility Code
- 1999 National Electrical Code
- AIC 318 "Building Code Requirements for Structural Concrete"
- AIC 530 "Building Code Requirements for Masonry Structures"
- AISC Manual of Steel Construction (locally approved edition)
- ANSI "Structural Welding Code"

Thesis Calculations:

- American Society of Civil Engineers (ASCE) 7-10
- AISC Steel Construction Manual, 14th Edition
- ACI 318-11
- Vulcraft steel deck catalog

Design Loads

Dead Loads

Dead loads are estimated based off material weights found in the AISC Steel Construction Manual since no values were given on drawings except for weights of rooftop units which range from 8,000-45,000 lbs. Deck weight is compared to similar weights in Vulcraft catalog based on topping thickness and deck type.

Dead Loads	
Description	Load (psf)
Framing	10
Superimposed DL	10
MEP	10
Composite Deck	
3.25" LCW topping	42
4.75" LCW topping	50
5" NWC topping	70
Roof Garden	80
Façade	
Brick	40
Glass	10
Metal Panel	15

Live loads

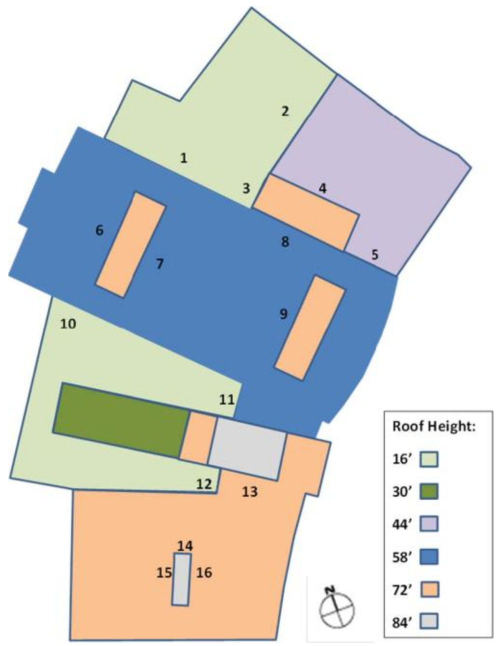
Live load values were given on the drawings. These values are shown along with the values given in ASCE7-10 in the table below. Where values are not given in one source the value from the other source was used in calculations. Likewise, when differing values are present the larger of the two was used in thesis calculations.

Live Loads		
Description	Designed Load (psf)	ASCE 7-10 Load (psf)
Slab on grade	100	100
Library slab on grade	150	150
Storage	125	125
Offices	50 + 20 (partition allowance)	50 + 15 (partition allowance)
Classrooms	40 + 20 (partition allowance)	50 + 15 (partition allowance)
Corridors (elevated floors)	80	80
Lobbies	100	100
Recreational areas	100	100
Mechanical/Electrical	125	N/A
Stairs	100	100
Chiller room	150 + equipment	N/A
Boiler room	200 + equipment	N/A
Roof	30	20
Roof Garden	N/A	100

Snow Loads

With the use of flat roofs on 6 different levels, the snow loading for University Academic Center will be an important consideration when designing the roof members. Both uniform snow loading and drifting must be factored into design.

Using ASCE7-10 to confirm the design loads used on the building were efficient, a flat roof snow load of 15.75 psf was calculated. According to the plans, the building was designed conservatively for a snow load of 20 psf.



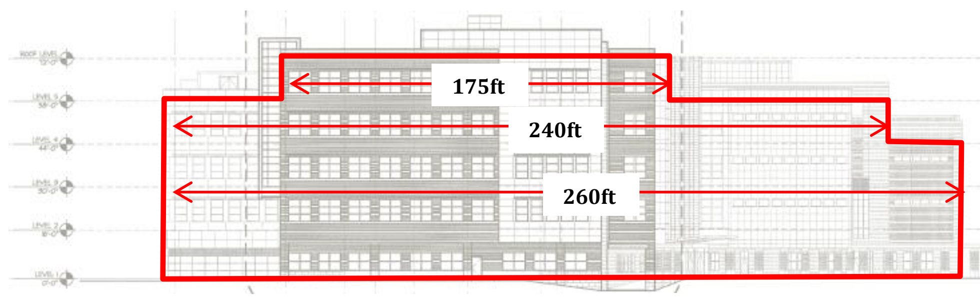
Basic snow drift calculations were also done to find the total snow loads including drift at 16 different locations of presumed maximum drift as well as when I_u=20 ft, the minimum length where drift calculations are necessary as defined in section 7.7.1. Snow is assumed unable to drift from one roof to another due to parapet walls. Calculation for drift around parapet walls may also be determined through the same procedure if required in future analysis. Resulting pressures are shown below and sample hand calculations can be found in the appendix.

Snow Drift Calculations					
Location	I_u (ft)	h_d (ft)	p_d (psf)	w (ft)	p_{tot} (psf)
-	20	1.00	17.32	4.02	33.07
1	100	2.52	43.40	10.06	59.15
2	62	1.98	34.15	7.92	49.90
3	90	2.39	41.23	9.56	56.98
4	61	1.96	33.86	7.85	49.61
5	80	2.25	38.90	9.02	54.65
6	46	1.69	29.08	6.74	44.83
7	109	2.62	45.23	10.49	60.98
8	94	2.44	42.12	9.77	57.87
9	109	2.62	45.23	10.49	60.98
10	103	2.55	44.02	10.21	59.77
11	118	2.72	46.96	10.89	62.71
12	116	2.70	46.59	10.80	62.34
13	101	2.53	43.61	10.11	59.36
14	33	1.39	24.00	5.56	39.75
15	63	2.00	34.44	7.98	50.19
16	49	1.75	30.11	6.98	45.86

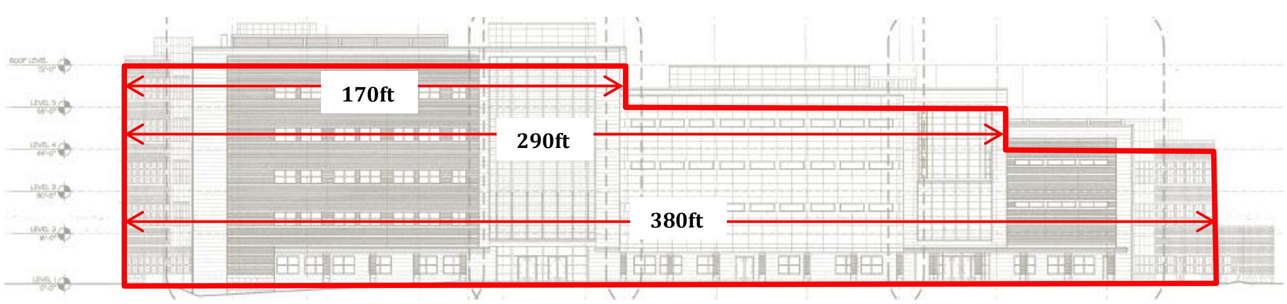
Wind Loads

Wind loads were calculated using the Directional Procedure found in ASCE7-10 Chapter 27. Preliminary values taken from the drawings along with detailed calculations in determining wind loads can be found in the hand calculations section of the appendix. An approximate building shape was taken for facilitating calculations based off the south and east elevations shown below. This simplification still required the determining of wind pressures for three levels. The wind pressures were then taken and converted into story forces for later use in lateral calculations including story drifts, max displacements, and overturning moment.

Based on the larger surface area in the N-S direction the forces at each story level are larger in the E-W wind direction. This translated into a larger base shear and larger overturning moment in the E-W wind direction.

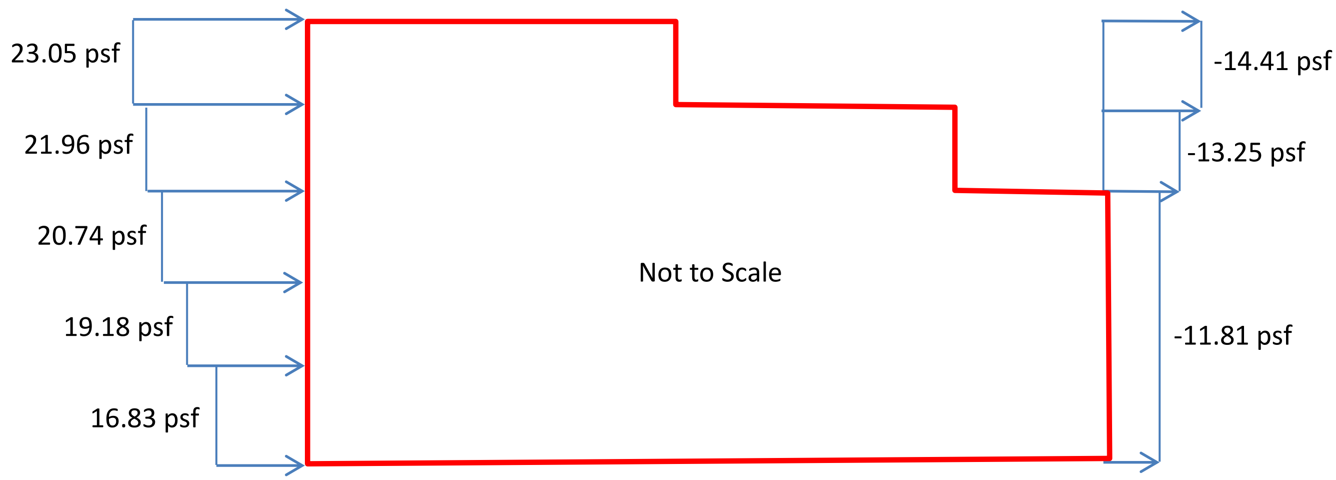


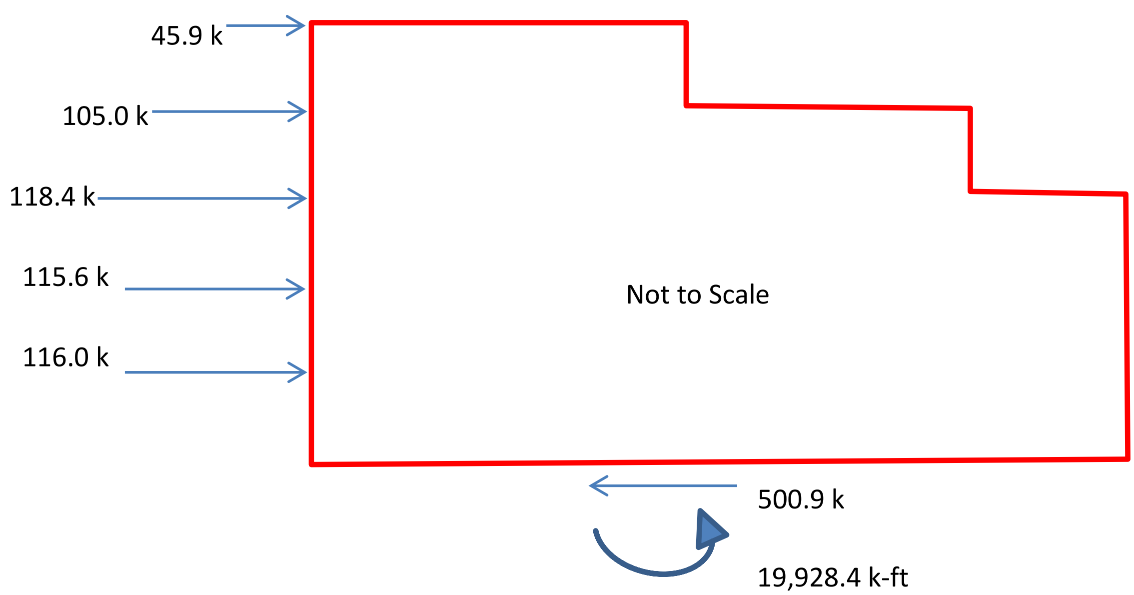
South Elevation provided by Skanska



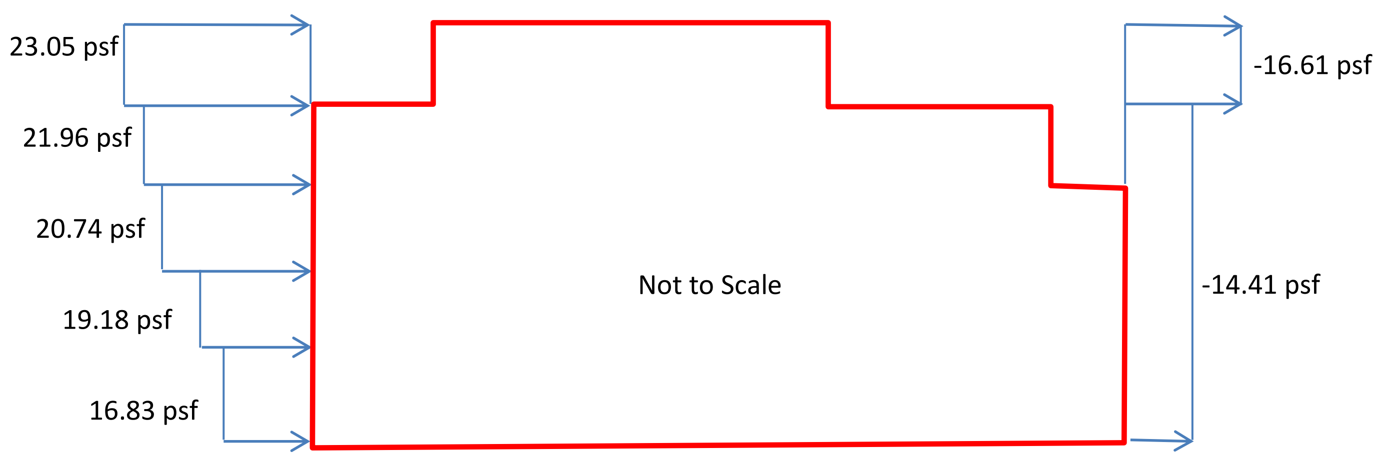
West Elevation provided by Skanska

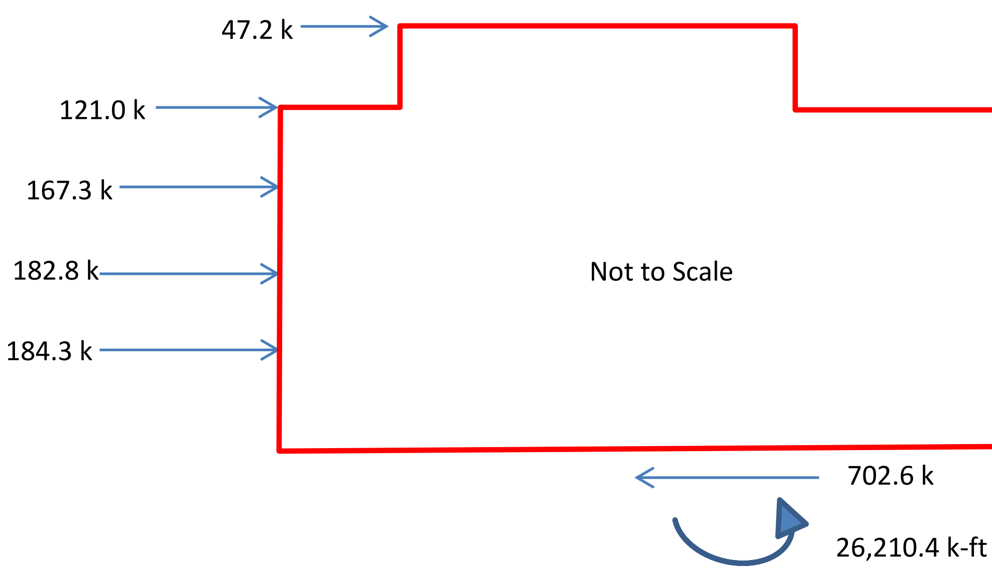
Wind Pressures (N-S)					
Location	Height (ft)	q (psf)	Cp	Wind Pressure (psf)	Internal Pressure (psf)
Windward	0-16	24.75	0.8	16.83	+/- 5.81
	16-30	28.20	0.8	19.18	+/- 5.81
	30-44	30.50	0.8	20.74	+/- 5.81
	44-58	32.29	0.8	21.96	+/- 5.81
	58-72	33.90	0.8	23.05	+/- 5.81
Leeward	0-44	33.90	-0.41	-11.81	+/- 5.81
	44-58	33.90	-0.46	-13.25	+/- 5.81
	58-72	33.90	-0.5	-14.41	+/- 5.81
Side	0-72	33.90	-0.7	-20.17	+/- 5.81





Wind Pressures (E-W)					
Location	Height (ft)	q (psf)	Cp	Wind Pressure (psf)	Internal Pressure (psf)
Windward	0-16	24.75	0.8	16.83	+/- 5.81
	16-30	28.20	0.8	19.18	+/- 5.81
	30-44	30.50	0.8	20.74	+/- 5.81
	44-58	32.29	0.8	21.96	+/- 5.81
	58-72	33.90	0.8	23.05	+/- 5.81
Leeward	0-44	33.90	-0.5	-14.41	+/- 5.81
	44-58	33.90	-0.5	-14.41	+/- 5.81
	58-72	33.90	-0.49	-16.61	+/- 5.81
Side	0-72	33.90	-0.7	-20.17	+/- 5.81

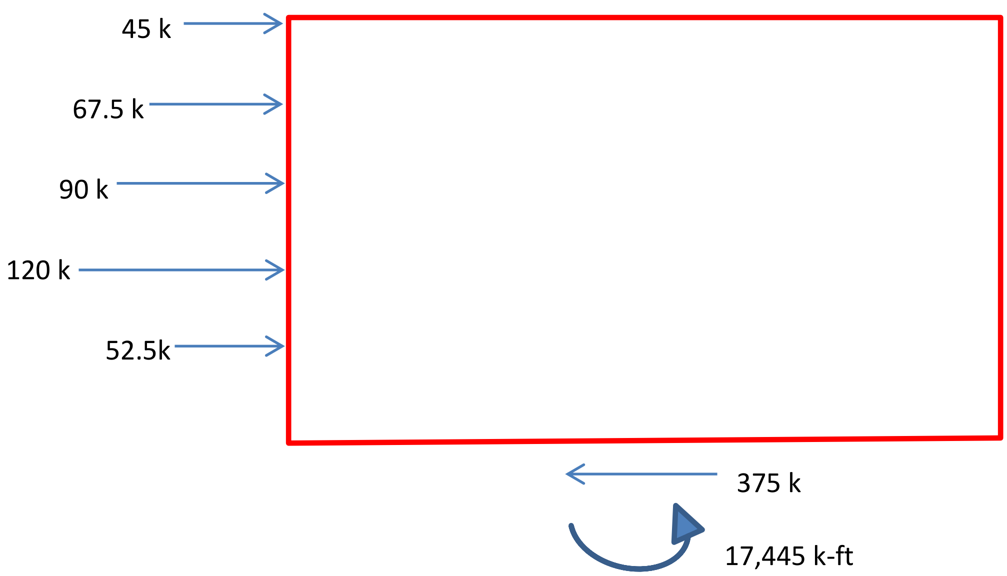




Seismic Loads

Seismic loading was designed using the Equivalent Lateral Force Procedure to follow the process used on the University Academic Center as stated in the drawings. Several design values were also given which when compared to the values calculated based on ASCE7-10 Equivalent Lateral Force Procedure, differed. However, both analyses resulted in similar base shear values. The as designed base shear is listed as 363 kip-ft, whereas the thesis calculated values came out to 377 kip-ft.

Seismic Load Calculation (N-S) & (E-W)						
Floor	Weight w_x (ft)	Height h_x (ft)	C_{vx}	Story Force F_x (kip)	Story Shear (kip)	Overturning Moment (kip-ft)
Ground	3,618	0	0	0	375	0
2	3,953	16	0.12	45	375	720
3	3,269	30	0.18	67.5	330	2,025
4	2,966	44	0.24	90	262.5	3,960
5	2,995	58	0.32	120	172.5	6,960
Roof	1,060	72	0.14	52.5	52.5	3,780
Total	17,861	-	1	375	-	17,445
Base Shear = 375 kip				Overturning Moment = 17,445 kip-ft		



Load Combinations

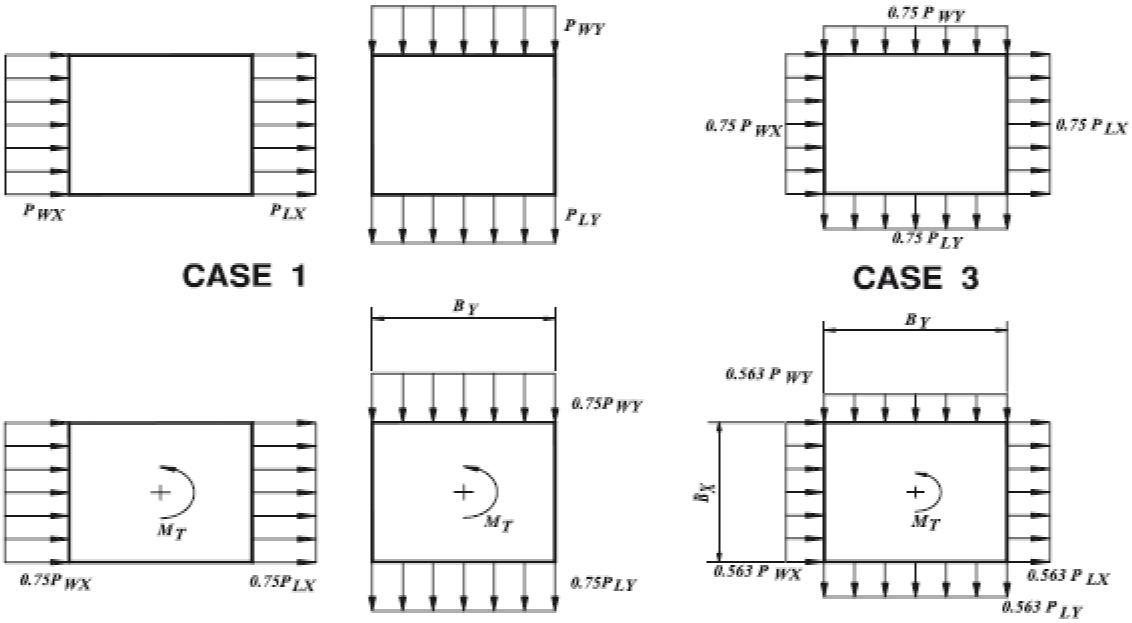
Load combinations taken from ASCE7-10 used in this report are shown below. Out of these load combinations only those containing wind and seismic loads need be considered since this portion of analysis only includes lateral effects on the structure. Load combinations 4 and 5 will govern for wind and seismic forces respectively. Load combinations 6 and 7 will control for evaluating overturning moments. In addition, the controlling wind load case taken from Figure 27.4-8 of ASCE7-10 must be determined as the wind load case used in the load combinations below.

2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	



$M_T = 0.75\ (P_{WX}+P_{LX}) B_X e_X$
$e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY}) B_Y e_Y$
$e_Y = \pm\ 0.15\ B_Y$

$M_T = 0.563\ (P_{WX}+P_{LX}) B_X e_X + 0.563\ (P_{WY}+P_{LY}) B_Y e_Y$
$e_X = \pm\ 0.15\ B_X$ $e_Y = \pm\ 0.15\ B_Y$

CASE 2

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Wind Load Cases

Case 1 (N-S)	
Story	Load (k)
5	45.9
4	105
3	118.4
2	115.6
1	116

Case 1 (E-W)	
Story	Load (k)
5	47.2
4	121
3	167.3
2	182.8
1	184.3

Case 2 (N-S)		
Story	Load (k)	M_T (k-ft)
5	34.4	83.8
4	78.8	209.5
3	88.8	193.7
2	86.7	184.4
1	87	170.4

Case 2 (E-W)		
Story	Load (k)	M_T (k-ft)
5	35.4	128.9
4	90.8	360.9
3	125.5	518.2
2	137.1	495.2
1	138.2	494.2

Case 3		
Story	(N-S) Load (k)	(E-W) Load (k)
5	34.4	35.4
4	78.8	90.8
3	88.8	125.5
2	86.7	137.1
1	87	138.2

Case 4			
Story	(N-S) Load (k)	(E-W) Load (k)	M_T (k-ft)
5	25.8	26.6	159.7
4	59.1	68.1	428.2
3	66.7	94.2	534.4
2	65.1	102.9	510.2
1	65.3	103.8	498.9

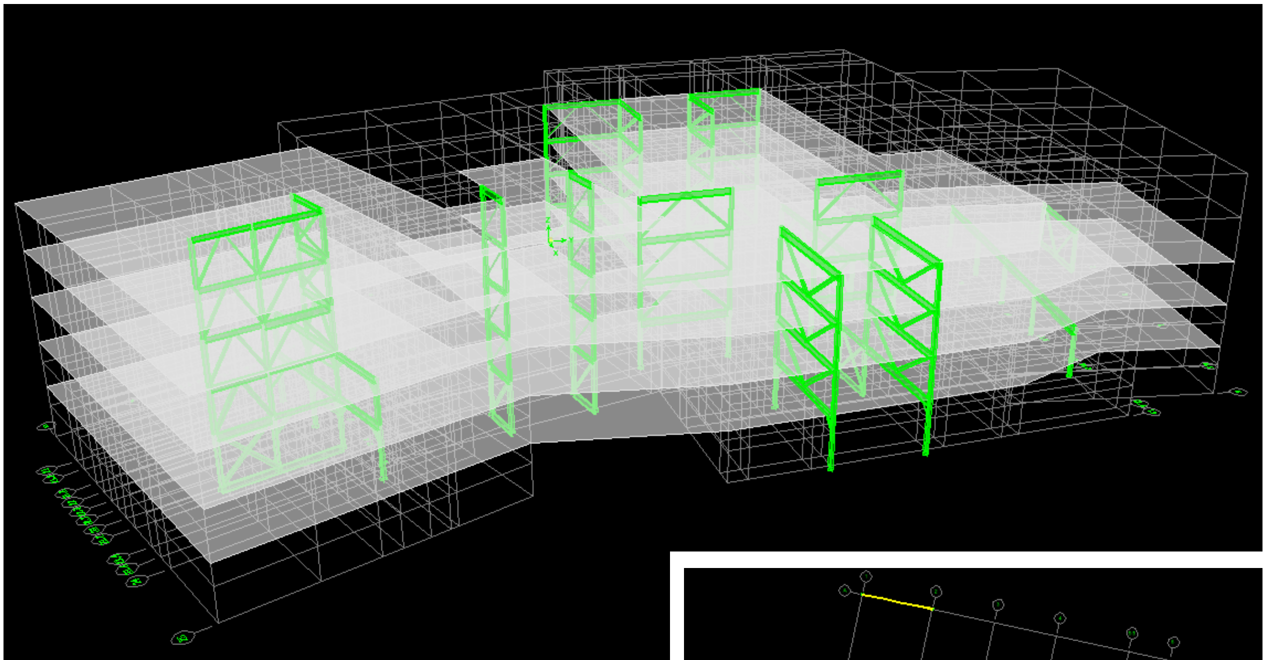
Seismic Load Cases

(N-S) & (E-W)	
Story	Load (k)
5	45
4	67.5
3	90
2	120
1	52.5

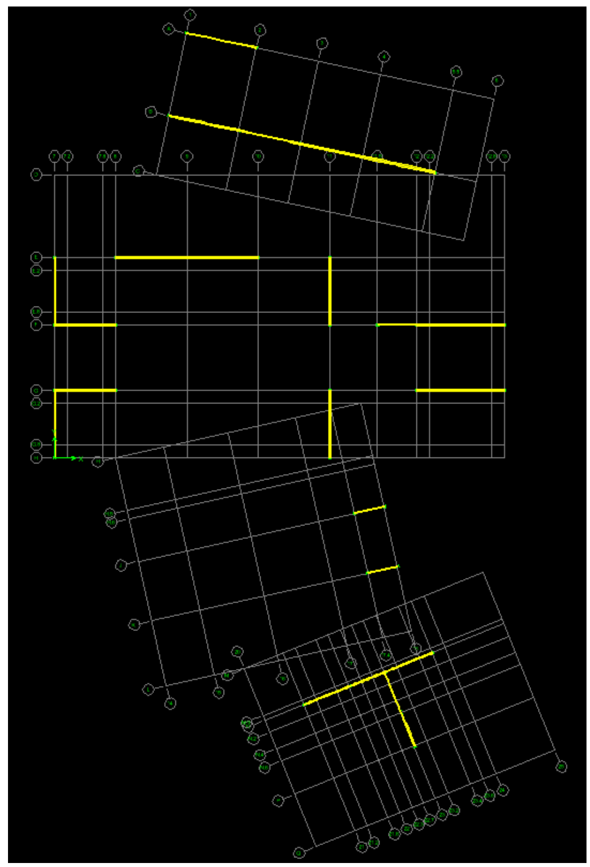
Overturning Moment

To ensure the foundations are adequate to prevent overturning, the weight of the foundation acting at center of mass was compared to the overturning moments resulting from the worst seismic and wind loading cases in each direction along with the factor of safety in the table below. Due to the large building area on the ground floor there is more than sufficient mass to resist overturning in even the weakest direction by a factor of 12.

Loading	Overturning Moment (k-ft)	Resisting Moment (k-ft)	Factor of Safety
Wind (N-S)	19,928	644,004	32.3
Wind (E-W)	26,210	314,766	12.0
Seismic (N-S)	17,445	644,004	36.9
Seismic (E-W)	17,445	314,766	18.0



ETABS 3D view



ETABS Plan View STORY1

ETABS Model

To analyze the lateral system of the University Academic Center a computer model was produced in ETABS. This allows for faster calculations and more precise values than can be easily obtained through hand calculations. Due to the angular offset of 6 of the lateral frames the computer model saved time in determining stability of the building.

First the gridlines were reproduced as found in the plans (see Appendix) and story levels added. Then each frame was modeled and member sizes added given in the construction documents (see Appendix). Only lateral resisting members were added for this model since only effects due to lateral forces will be investigated. The floor systems were modeled as rigid and given a mass as determined previously in calculations of building weight for seismic loads. Load cases were then added as calculated earlier for wind and seismic to determine the controlling cases.

Relative Stiffness

To determine the load distribution to each braced frame a 1000 kip unit load was applied in the x-axis (E-W) and y-axis (N-S). Section cuts were then taken through the frames at each level to determine what percent of the load was resisted by that frame. This load distribution will determine how crucial each frame is at resisting loads in that given axis. The braced frames rotated off the main axes were assumed to take load in both x and y axes. The story force for these frames was converted into equivalent forces acting along their primary axis to compare to drift values taken from ETABS. Due to the repetitiveness of these calculations only relative stiffness's for the first floor were calculated and used to find the buildings center of rigidity for comparison to the ETABS model.

This process resulted in a center of rigidity at point (32.490', 64.302'). ETABS calculated the center of rigidity at point (59.483', 63.331'). Variation in these two numbers is minimal along the y-axis so this value can be considered accurate. However, the large difference between the two values in the x-axis indicates an error. When reviewing the input the values for forces seen by BF-5 and BF-6 along the y-axis seem quite low considering the orientation of the frames act along the axis of loading. This was assumed to be the source of the error. Given more time, additional analysis would be done to find the exact error made. The ETABS model results were still considered accurate for further lateral study given the complexity of the analysis and time consumption had hand calculations been chosen instead.

STORY 1	(1000 kip along X-axis)				
Frame	Force Resisted (k)	Force[cos(Θ)]	Drift (in)	Stiffness (k/in)	Relative Stiffness
BF-1	192	0	-	0	0%
BF-2	195	0	-	0	0%
BF-3	-187	-187	0.00115	162609	10.0%
BF-4	-17	-17	0.00045	37778	2.3%
BF-5	-20	0	-	0	0%
BF-6	-19	0	-	0	0%
BF-7	-23	-23	0.00086	26744	1.6%
BF-8	-166	-166	0.00043	386047	23.8%
BF-9	181	177	0.00130	136154	8.4%
BF-10	-277	-274	0.00253	108300	6.7%
BF-11	-319	-315	0.00297	106060	6.5%
BF-12	-40	-37	0.00257	14397	0.9%
BF-13	-290	-104	0.00415	25060	1.5%
BF-14	16	15	0.00354	4237	0.3%
BF-15	-191	-191	0.00031	616129	38.0%
Sum	-965			1623515	

STORY 1	(1000 kip along Y-axis)				
Frame	Force Resisted (k)	Force[sin(Θ)]	Drift (in)	Stiffness (k/in)	Relative Stiffness
BF-1	-186	-186	0.00052	359073	38.8%
BF-2	-187	-187	0.00051	364522	39.4%
BF-3	-71	0	-	0	0%
BF-4	-8	0	-	0	0%
BF-5	36	36	0.00105	34417	3.7%
BF-6	35	35	0.00110	31963	3.5%
BF-7	-12	0	-	0	0%
BF-8	-22	0	-	0	0%
BF-9	190	40	0.00372	10753	1.2%
BF-10	-170	-27	0.00221	12217	1.3%
BF-11	-193	-30	0.00238	12605	1.4%
BF-12	-195	-70	0.00198	35354	3.8%
BF-13	-185	-173	0.00278	62230	6.7%
BF-14	23	5	0.00184	2717	0.3%
BF-15	100	0	-	0	0%
Sum	-845			925851	

The center of rigidity hand calculation, as well as a diagram orienting the location of both centers of rigidity is found in the appendix.

Torsional Effects

University Academic Center's bracing is irregular as well as its shape resulting in a large difference in center of mass and center of rigidity. These differences result in torsional forces that cause the building to torque when loaded. When calculating torsional effects on a building both the inherent torsion, as well as an accidental moment, are required to be found. This accidental moment is the effect due to asymmetric loads acting on the building that are unknown to the engineer. To account for this, a moment equivalent to that produced by an eccentricity of 5% the buildings length is added to the known moment. Below are tables showing the torsional effects on University Academic Center due to seismic loading in both the N-S and E-W directions. Wind loading will also produce torsional effects and could play a role in determining the controlling load case, however due to the redundancy in analytic procedure, is not listed in this report directly.

An investigation of University Academic Center's torsional effects shows that the building has a very large eccentricity on most floors causing the majority of torsion to stem from inherent torsion within the building due to asymmetry. The full effects this plays on drift and displacement are to be looked at later in this report.

Building Torsion due to N-S Seismic Loading							
	Force (k)	COR	COM	e_x (ft)	M_t (k-ft)	M_{ta} (k-ft)	M_{tot} (k-ft)
STORY 5	45	-61.275	-99.598	38.323	1,725	320	2,045
STORY 4	112.5	24.332	-6.236	30.568	3,439	1,288	4,727
STORY 3	202.5	30.186	12.665	17.521	3,548	2,319	5,867
STORY 2	322.5	40.256	9.968	30.288	9,768	3,693	13,461
STORY 1	375	63.311	17.758	45.553	17,082	4,294	21,376
						Total	47,476

Building Torsion due to E-W Seismic Loading							
	Force (k)	COR	COM	e_x (ft)	M_t (k-ft)	M_{ta} (k-ft)	M_{tot} (k-ft)
STORY 5	45	132.646	159.132	26.486	1,192	378	1,570
STORY 4	112.5	89.809	128.816	39.007	4,388	1,665	6,053
STORY 3	202.5	91.355	132.071	40.716	8,245	3,645	11,890
STORY 2	322.5	88.049	130.605	42.556	13,724	6,047	19,771
STORY 1	375	59.483	118.133	58.650	21,994	7,031	29,025
						Total	68,309

Story Drift and Displacement

University Academic Center is limited to a maximum story drift specified in ASCE7-10 as $0.02h_{sx}$ calculated at each story. It is also accepted practice to limit overall displacement to L/400 for a given building height L. ETABS can calculate story drifts and displacements for all load cases simultaneously and allow for quick comparison to determine the worst load case and ultimately the maximum story drifts and displacements seen by the building. Based on ETABS results wind case 3 controlled story drift and displacement. The tables below show the maximum drifts and displacements and if they are acceptable.

X-axis Loading					
	Story Displacement (in)	Story Drift (in/in)	Allowable Story Drift (in)	Load Case	OK?
STORY 5	0.9722	0.001350	0.28	Wind Case 3	ok
STORY 4	0.7454	0.000938	0.28	Wind Case 3	ok
STORY 3	0.5878	0.000879	0.28	Wind Case 3	ok
STORY 2	0.4400	0.001024	0.28	Wind Case 3	ok
STORY 1	0.2253	0.001353	0.32	Wind Case 3	ok
	L/400 = 2.16 in				

Y-axis Loading					
	Story Displacement (in)	Story Drift (in)	Allowable Story Drift (in)	Load Case	OK?
STORY 5	0.7831	0.001210	0.28	Wind Case 3	ok
STORY 4	0.5798	0.000733	0.28	Wind Case 3	ok
STORY 3	0.4567	0.000684	0.28	Wind Case 3	ok
STORY 2	0.3076	0.000680	0.28	Wind Case 3	ok
STORY 1	0.1890	0.001189	0.32	Wind Case 3	ok
	L/400 = 2.16 in				

Member Checks

To further verify the believability of the ETABS output, a member check was done of the most severely loaded bracing member located on the ground floor of BF-11. This member experienced an axial load of 174.53 k. Along with the bracing member the column this brace framed into was also checked for combined loading. Both members passed supporting the assumption the modeling data is accurate. These calculations can be found in the appendix.

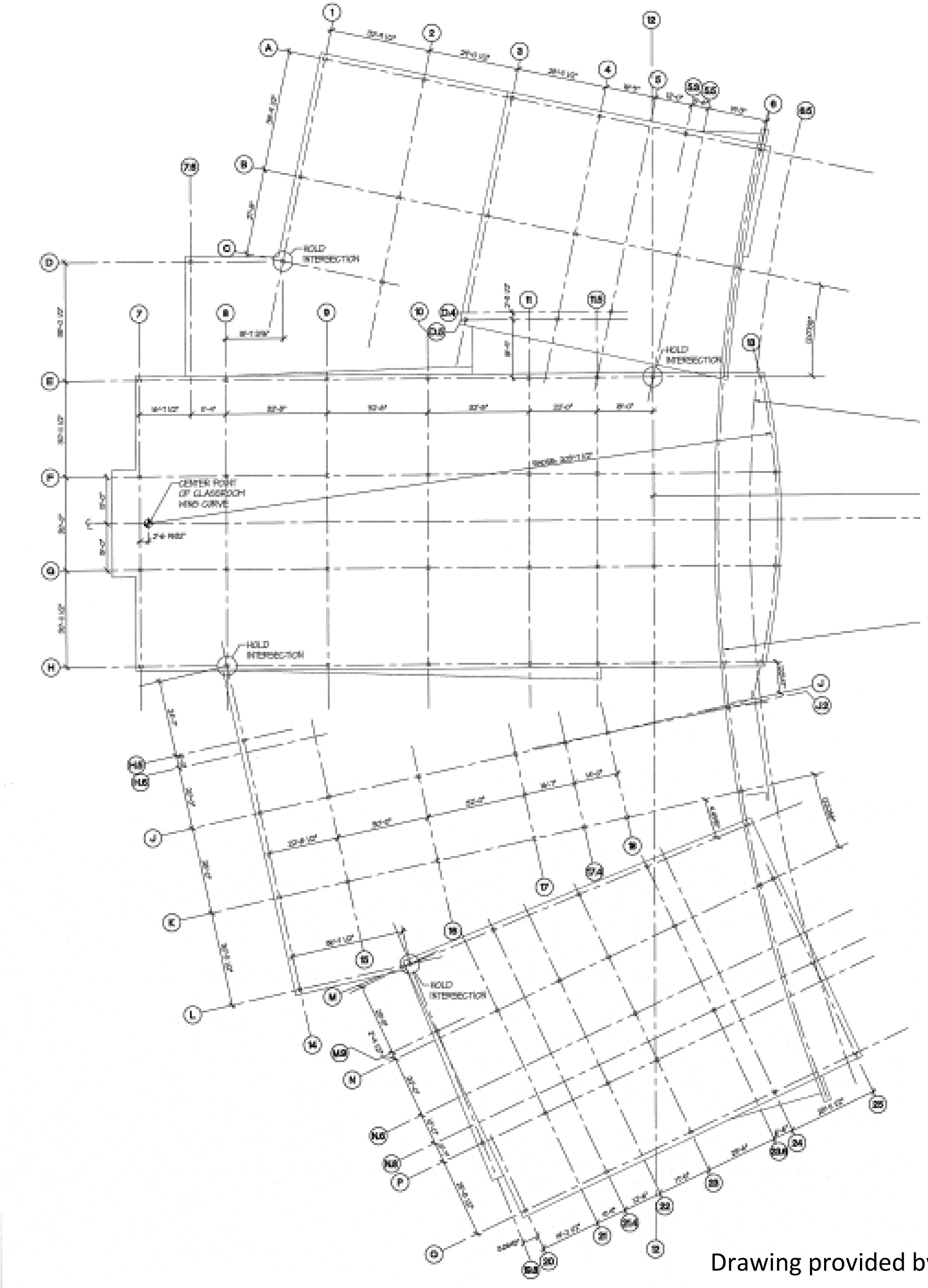
Conclusion

The lateral resisting system of University Academic Center proved to be efficient under the worst load case, wind load case 3. This design provided enough rigidity to meet displacement requirements while minimizing the reduction of useable space taken up by framing hidden in walls.

Overturning was concluded not to be of great concern in University Academic Center due to its large building footprint. The stacking effect of minimizing floor area as the height increases limits the severity of overturning moments. This design feature is both visually appealing and functional in reducing stresses on foundation due to uplift.

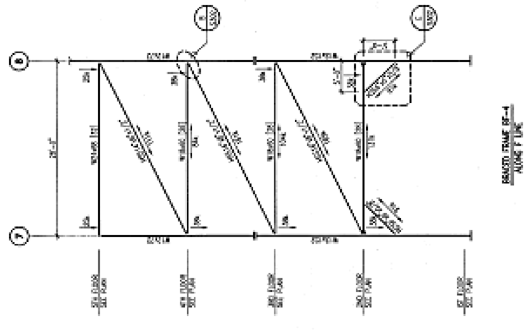
Torsional effects on the building were large due to the high eccentricities between center of mass and rigidity. Despite this torsion, no unreasonable displacements in the building were observed through the use of ETABS computer modeling software. The large eccentricities are undesirable and could be a possible point of interest in future study as to whether a redesign could reduce this torsion without drastically changing the building layout.

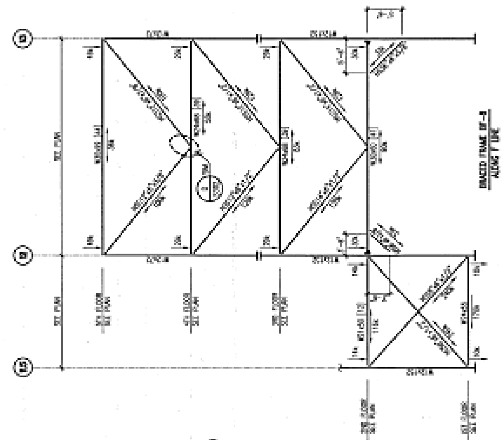
Appendix A: Gridline Layout

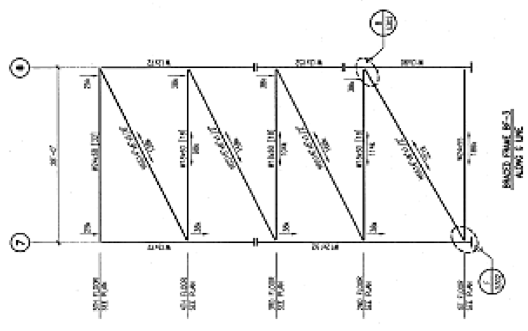


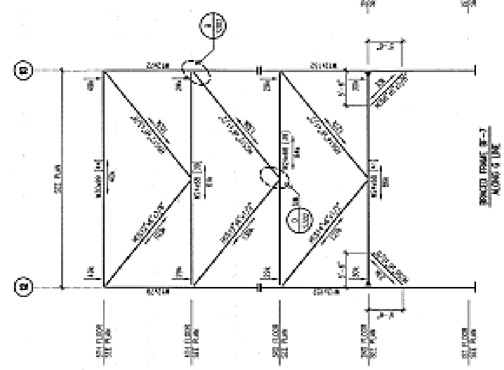
Drawing provided by Skanska

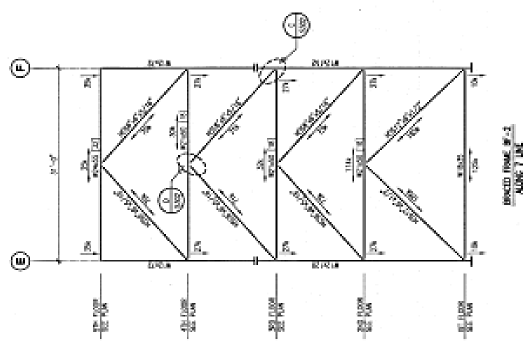
Appendix B: Braced Frames

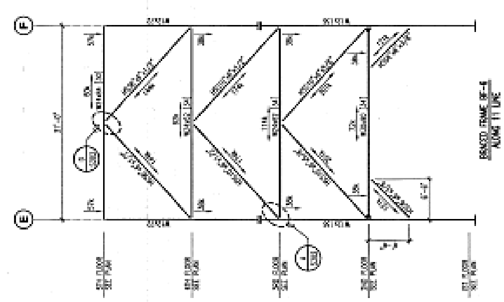


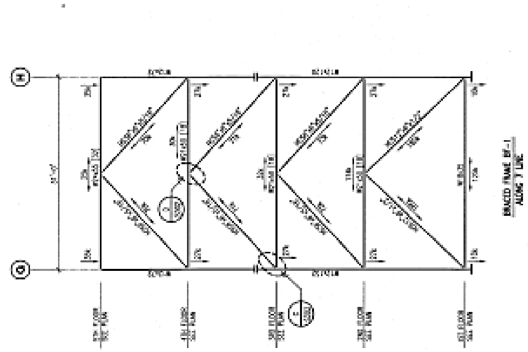


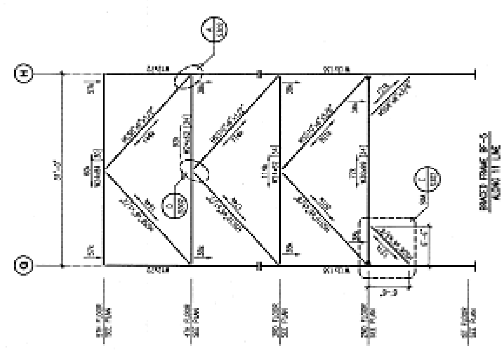




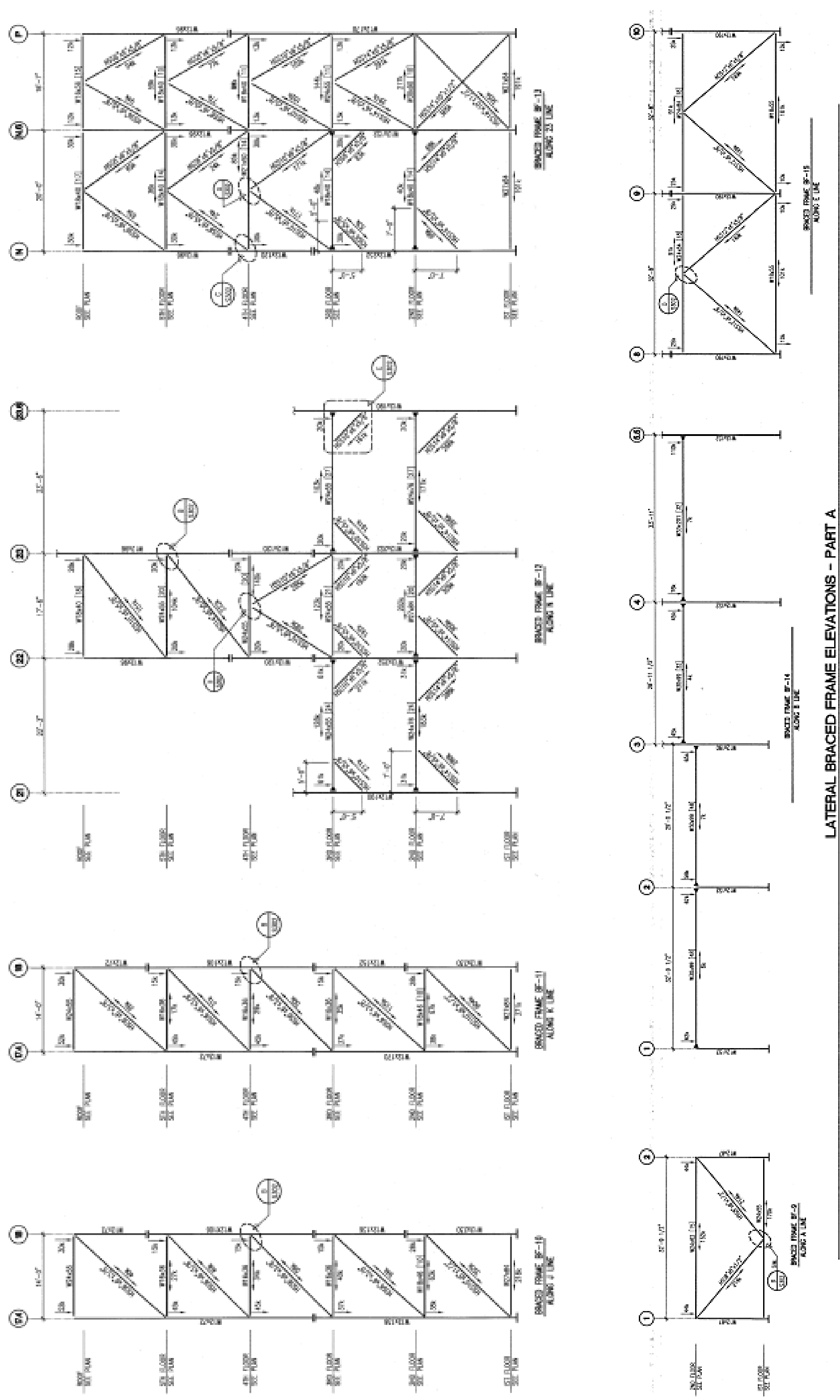








Drawings provided by Skanska



Drawings provided by Skanska

Appendix C: Hand Calculations

Snow Drift Calcs | Tech 3 | Alexander Altemose

Snow Load

Flat Roof Snow Load, P_f

$P_f = 0.7 C_e C_t I_s P_g$

$P_g = 25$ psf (Fig 7-1)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.0$ (Table 7-3)

$I_s = 1.0$ (Table 1.5-2)

$P_f = 0.7(0.9)(1.0)(1.0)(25)$

$P_f = 15.75$ psf (Calculated)

$P_f = 20$ psf (As Designed)

Snow Drift

$\gamma = 0.13 p_g + 14 = 17.25$ pcf

$h_b = \dfrac{P_s}{\gamma} = \dfrac{15.75}{17.25} = 0.91' > 0.2$

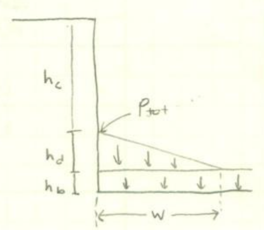
∴ calculate drift

→ Sample Calc for Snow drift

Location 1

$l_u = 100'$

$h_d = 0.43 \sqrt[3]{\underset{100'}{l_u}} \sqrt[4]{\underset{25\text{psf}}{P_g} + 10} - 1.5 = 2.52'$



Not to scale

$P_d = h_d \gamma = 2.52' (17.25\text{pcf}) = 43.47$ psf

$P_{tot} = P_f + P_d = 15.75 + 43.47 = 59.22$ psf

$h_c \sim 42' > h_d \longrightarrow w = 4 h_d = 10.08'$

Wind Calcs | Tech 3 | Alexander Altemose

Wind Loads → MWFRS (Directional Procedure) Analysis will be used [Steps outlined in Table 27.2-1]

Values given on drawings

basic wind speed (V) = 90 mph

wind load importance factor (I_w) = 1.15

risk category II

wind exposure C

internal pressure coefficient = 0.18

[Values are base on and older version of ASCE standard
Calculations are based off current version ASCE 7-10
Values will differ from design]

Basic Wind Parameters:

[Step 1] Risk Category II

[Step 2] V = 115 mph

[Step 3] K_d = 0.85

Exposure C

K_{zt} = 1 [Assumed based on lack of topographic concern on site]

G = 0.85

Enclosure = Fully Enclosed

GC_{pi} = ± 0.18

Wind Calcs | Tech 3 | Alexander Altemose

[Step 4]
[Step 5]

floor heights (ft)	K_z	q_z (psf)
0 - 16	0.86	24.75
16 - 30	0.98	28.20
30 - 44	1.06	30.50
44 - 58	1.12	32.29
58 - 72	1.18	33.90

Sample Calc

q_z @ 16' $= 0.00256(0.86)(1)(0.85)(115)^2 = 24.75$ psf [27.3-1]

[Step 6] External Pressure Coefficients (C_p)

0'-44'	N-S	E-W
L/B	1.46	0.68
Windward	0.8	0.8
Leeward	-0.41	-0.5
Side	-0.7	-0.7
44'-58'		
L/B	1.21	0.83
Windward	0.8	0.8
Leeward	-0.46	-0.5
Side	-0.7	-0.7
58'-72'		
L/B	1.03	0.97
Windward	0.8	0.8
Leeward	-0.49	-0.5
Side	-0.7	-0.7

Wind Calcs | Tech 3 | Alexander Altemose

[Step 7] $p = q\,GC_p - q_i(GC_{pi})$ [psf] [27.4-1]

Sample Calcs

N-S Windward 0'-16' $p = 24.75(0.85)(0.8) - 33.90(\pm 0.18) = 16.83$ psf ± 5.81 psf

N-S Leeward 44'-58' $p = 33.90(0.85)(-0.46) - 33.90(\pm 0.18) = -13.25$ psf ± 5.81 psf

N-S Side All $p = 33.90(0.85)(-0.7) - 33.90(\pm 0.18) = -20.17$ psf ± 5.81 psf

E-W Windward 58'-72' $p = 33.90(0.85)(0.8) - 33.90 \pm(0.18) = 23.05$ psf ± 5.81 psf

E-W Leeward All $p = 33.90(0.85)(-0.5) - 33.90(\pm 0.18) = -14.41$ psf ± 5.81 psf

E-W Side All $p = 33.90(0.85)(-0.7) - 33.90(\pm 0.18) = -20.17$ psf ± 5.81 psf

Seismic Calcs Tech 3 Alexander Altemose

Seismic Load

Data Given in Drawings

seismic use group : I

S_{DS} : 0.21 S_{D1} : 0.11

site class : D

seismic-force-resisting system : special steel concentrically braced frames

design base shear (V) : 363 kips

analysis procedure : equivalent lateral force method

Equivalent Lateral Force Procedure (per ASCE 7-10)

$V = C_s W$ (12.8-1)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ (12.8-2)

$I_e = 1.0$ (Table 1.5-2)

$R = 6$ (Table 12.2-1)

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

$S_{MS} = F_a S_s$ (11.4-1)

$S_s = 0.12$ (Fig 22-1)

$F_a = 1.6$ (Table 11.4-1)

↳ $S_{MS} = 0.192$

↳ $S_{DS} = 0.128$

↳ $C_s = 0.021$

Seismic Calcs Tech 3 Alexander Altemose

Seismic (cont'd)

$C_s \le C_s^* = \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ for $T \le T_L$ (12.8-3)

$T \cong T_a = C_t h_n^x$ (12.8-7)

$h_n = 72'$

$C_t = 0.02$ (Table 12.8-2)

$x = 0.75$ (Table 12.8-2)

↳ $T = T_a = 0.494\,s$

$T_L = 8\,s$ (Figure 22-12)

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

$S_{M1} = F_v S_1$ (11.4-2)

$S_1 = 0.05$ (Figure 22-2)

$F_v = 2.4$ (Table 11.4-2)

↳ $S_{M1} = 0.12$

↳ $S_{D1} = 0.08$

↳ $C_s^* = 0.027 > C_s = 0.21$ ✓ ok

$C_s = 0.044\, S_{DS} I_e \ge 0.01$ (12.8-5)

$C_s = 0.021 > 0.01$ ✓ ok

Seismic Calcs Tech 3 Alexander Altemose

Seismic (con'td)

Calculation of building weight (W)

Level 1 : W = 55,000 sf (5")(150 pcf) slab on grade
1,130' (16')(20 psf) $\frac{1}{2}$ facade ≃ 3,618 kip

Level 2 : W = 7000 sf (80 psf) green roof
7,000 sf (40 psf) typ. roof
41,000 sf (42 + 10 + 5 + 10) psf floor ≃ 3,953 kip
1130' (16')(20 psf) $\frac{1}{2}$
1320' (14')(20 psf) $\frac{1}{2}$ facade

Level 3 : W = 1,900 sf (80 psf) green roof
41,000 sf (42 + 10 + 5 + 10) psf floor ≃ 3269 kip
1320' (14') (20 psf) facade

Level 4 : W = 5,700 sf (40 psf) typ roof
35,300 sf (42 + 10 + 5 + 10) psf floor ≃ 2966 kip
1320' (14')(20 psf) $\frac{1}{2}$
1180' (14') (20 psf) $\frac{1}{2}$ facade
(8000 + 15000) Roof units

Level 5 : W = 19,300 sf (70 psf) roof
16,000 sf (42 + 10 + 5 + 10) psf floor ≃ 2995 kip
1180' (14') (20 psf) $\frac{1}{2}$
1000' (14') (20 psf) $\frac{1}{2}$ facade
16,000(2) + 25,000 + 40,000(3) + 45,000(2) roof units

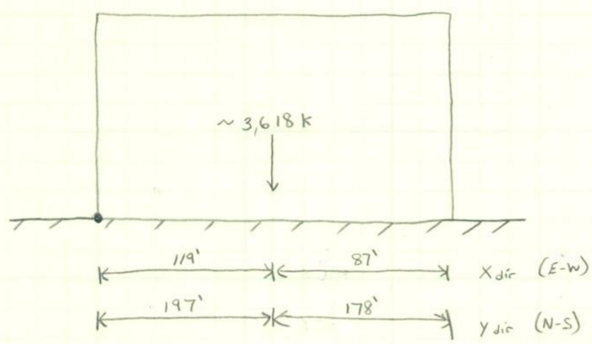
Level 6 : W = 14,000 sf (50 psf) roof
2,000 sf (42 + 10 + 5 + 10) psf floor
1000' (14')(20 psf) $\frac{1}{2}$ facade ≃ 1060 kip
18000(2) + 25,000(2) roof units

Total : W = 17861 kip

Base Shear : $V = C_s W = 0.021\,(17861) = 375$ kip

$F_x = C_{vx} V \qquad C_{vx} = \dfrac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$

Overturning Moments | Tech 3 | Alexander Altemose



(87') 0.9(3618k) = 283,289 k-ft

(178') 0.9(3618k) = 579,604 k-ft

Center of Rigidity | Tech 3 | Alexander Altemose

$$COR_x = \frac{38.8\%(0) + 39.4\%(0) + 3.7\%(126.0) + 3.5\%(126.0) + 1.2\%(92.5) + 1.3\%(151.0) + 1.4\%(157.1) + 3.8\%(173.0) + 6.7\%(164.9) + 0.3\%(174.3)}{100\%}$$

$COR_x = 32.490'$

ETABS $COR_x = 59.483'$

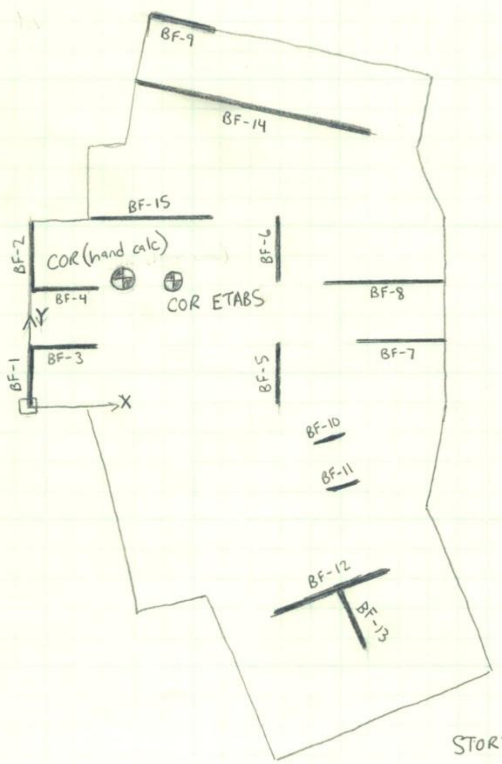
difference of $26.993'$

$$COR_y = \frac{10\%(31.0) + 2.3\%(61.0) + 1.6\%(31.0) + 23.8\%(61.0) + 8.4\%(194.9) + 6.7\%(-22.4) + 6.5\%(-49.8) + 0.9\%(-89.5) + 1.5\%(-98.3) + 0.3\%(157.0) + 38\%(92.0)}{100\%}$$

$COR_y = 64.302'$

ETABS $COR_y = 63.311'$

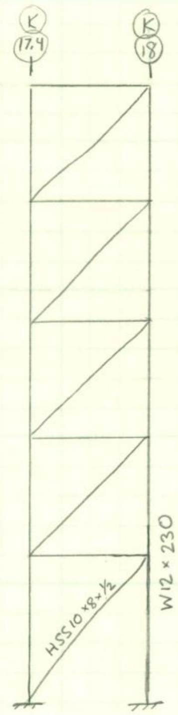
difference of $0.991'$



STORY 1 Plan View

Appendix D: Member Checks

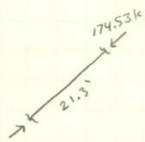
Member Checks | Tech 3 | Alexander Altemose



BF-11

HSS 10×8×1/2

$P_{MAX} = 174.53$ k (Wind Case 3) ETABS



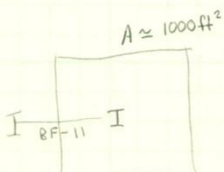
Tension

$\phi P_n = \phi F_y A_g = 0.9(46)(15.3) = \boxed{633\text{ k} > 174.53\text{ k}}$ ok

Compression

$\phi P_n = 399$ k @ $21.3' = KL$ (Table 4-3) AISC

$\boxed{399\text{ k} > 174.53\text{ k}}$ ok



DL = 72 psf

LL = 80 psf

SL = 30 psf

W12×230

$P_w = -200.6$ k (Wind Case 3) ETABS

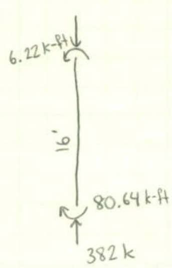
$P_D = 72$ k

$P_L = 80$ k

$P_S = 30$ k

$P_r = 1.2(72)(5) + 1.0(200.6) + 1.0(80)(4) + 0.5(30)(1) = 967.6$ k

$M_r = 80.64$ k-ft (Wind Case 3) ETABS



$p = 0.420 \times 10^{-3}$, $b_x = 0.626 \times 10^{-3}$ (Table 6-1) AISC

$pP_r + b_x M_r \leq 1.0$

$0.420 \times 10^{-3}(967.6) + 0.626 \times 10^{-3}(80.64) = \boxed{0.46 < 1.0}$

OK