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Technical Report III

Simmons College School of Management, Boston, Ma

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Executive Summary: Technical Report III

The Simmons College School of Management is a newly constructed five story educational facility located in Boston, Massachusetts. The building is 65,000 SF and sits on the south east corner of a five level below grade parking garage. Accommodations have been made in the original design for a future expansion of the building which would top out at a nine story building.

The below grade parking garage is a post tensioned concrete system with a slurry wall as the exterior foundation wall system. Interior columns are W14 shapes extend into the ground to form load bearing element foundations. At the plaza level provisions were made for the use of a crane in the construction of the above grade building. The five story building is steel with composite floors and primarily uses wide flange shapes.

Originally the building was designed under the Massachusetts State Building Code, Sixth Edition. This report used ASCE 7-05 as the primary code to develop the loading and strength requirements for the structure. Therefore, it is expected that there will be variations between the original design and the analysis that appears in this report.

The third technical report assessed the design lateral loading that the building would experience and the adequacy of the lateral load resisting system. Torsion was an important parameter to consider for the resistance of lateral loading in the building. This was a result of both the building geometry and the eccentricity between the point of load application and the center of rigidity of the structure. The building geometry would likely result in a diagonal loading to be critical to the design of the lateral system. A combination wind load case was considered in order to recognize this as a design consideration on the structure.

Steel braced frames in combination with steel moment frames provide the resistance to lateral forces acting on the building. A preliminary relative stiffness analysis was performed to identify an approximate lateral load distribution to each frame. Due to the arrangement of frames at varying angles throughout the building this assessment only provided preliminary distributions. Further analysis was performed with 3D structural modeling programs to fully investigate the effects of all lateral loading conditions.

The analysis of the building displayed that the structure was designed to adequately resist all lateral loading as well as meet the serviceability requirements of the building. Both wind and seismic drift values were within the limitations of code and accepted industry standards.

Introduction

The Simmons College School of Management is a newly completed five story educational facility to be located on the Simmons College campus in Boston, Massachusetts. The $63 million building which was completed in December of 2008 was designed by Cannon Design.

As part of the project a five level below grade parking structure was provided to replace the parking lot that previously occupied the site. This relocation of parking allowed for the creation of a new green space quad to serve the school.

When the building was completed it achieved the LEED Gold rating by the USGBC. The project received 40 LEED points which included recognition for significant reductions in water and energy usage.

The project includes design considerations for a future building expansion to be topped out at nine stories. This design parameter was considered from the beginning of the design process including the original geotechnical evaluation of the site.

Structural Systems

Foundations

The below grade parking structure was constructed by the top down method with the installation of a slurry wall and load bearing elements (LBE) prior to excavation. Slurry wall panels have varying widths ranging from 10'-0" to 25'-0" with the typical panel width being 24'-0". Penetration of the 10'-0" centerbite into marine sands on site ranges from 1'-0" to 43'-0" depending on the bearing capacity demands of the wall section. See Figure 4 for typical slurry wall panel elevation.

Load bearing elements are constructed with W14 columns from the garage embedded in concrete shafts. Depths of the concrete shafts are divided into four categories summarized in Figure 1. W14 column embedment into the concrete shafts ranges from 16' to 27'. Typical shear studs are 4" long ¾" diameter and arranged in patterns of eight, ten, or 12 studs per foot seen in Figure 2. See Figure 4 for typical LBE configuration below the slab on grade.

LBE INSTALLATION CRITERIA CATEGORIES	
CATEGORY 1	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT
CATEGORY 2	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT OR MINIMUM EMBEDMENT OF TWO (2) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF TEN (10) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSITS
CATEGORY 3	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSIT
CATEGORY 4	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF BEDROCK DEPOSIT

Figure 1 Typical LBE Configuration

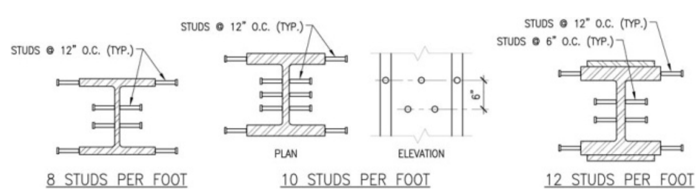


Figure 2 Typical LBE Configuration

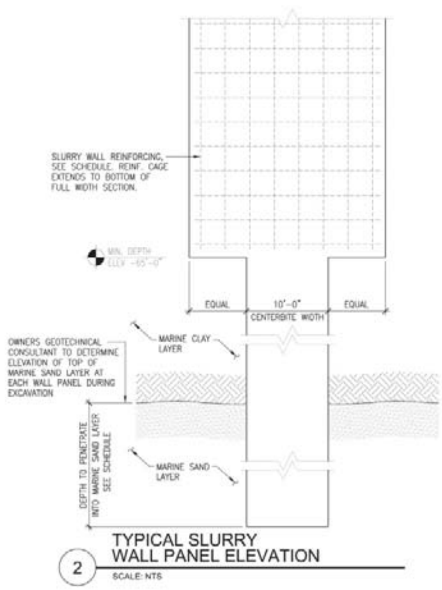


Figure 3 Slurry Wall Foundation Detail

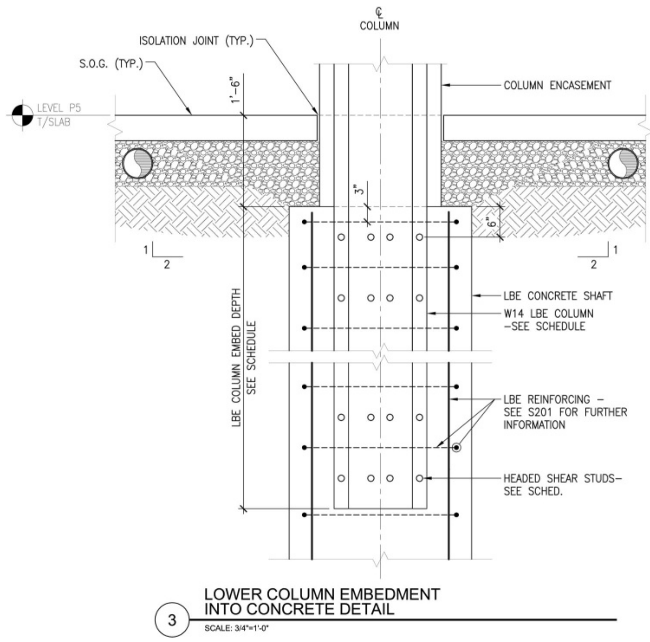


Figure 4 Load Bearing Element Foundation Detail

Beneath the area of the superstructure that is not located on top of the parking garage .365" thick, 10.75" diameter concrete filled steel pipe piles are used for foundations at column locations. Arrangements of piles include three, four, five, and eleven pile configurations. This foundation type is used below the braced frame which will be assessed for its load carrying capacity in the following sections. See Figure 5 for a typical layout of the pipe pile foundation.

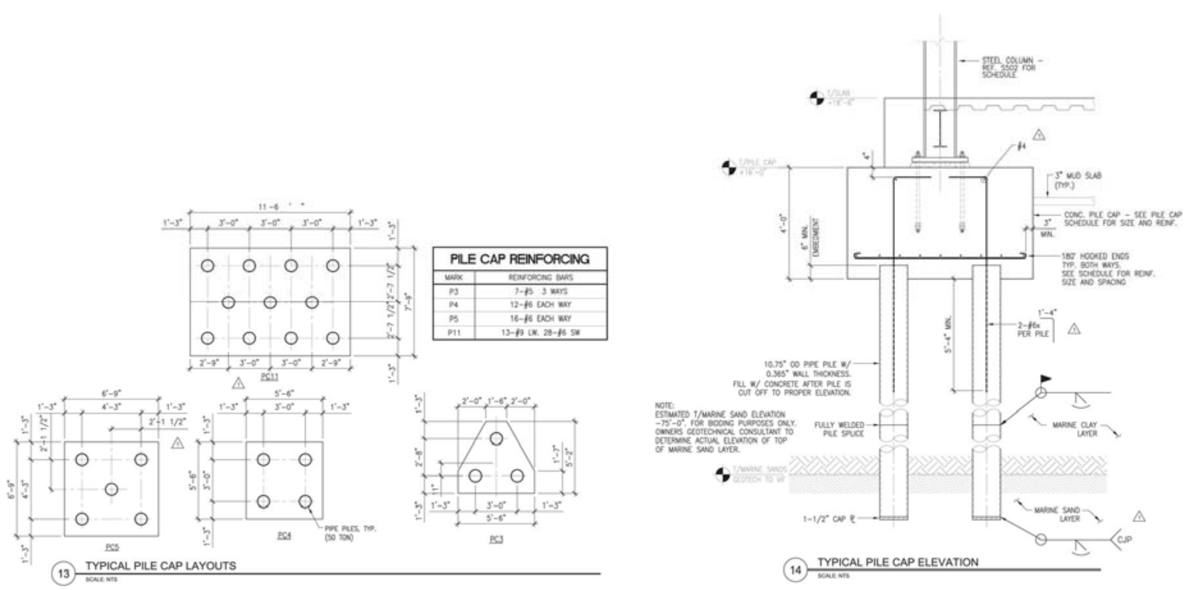


Figure 5 HSS Pile Foundation Detail

Floor Systems

Post tensioned concrete slabs are utilized for the typical floor system in the sub grade parking garage. Slab thickness in levels P1 through P4 is 14" with 6500 psi concrete. Bay sizes in the parking garage range from 36'x32' to 42'x49'.

Banded reinforcement spans in the north south direction of the parking garage plan with the typical bottom drape in each tendon meeting the minimum concrete cover at 1.75 inches. The typical force after all losses in these tendons is 1600 kips. Distributed reinforcement is placed in the east west direction at a maximum of 48 inches on center. At the column connections various patterns of stud rail arrangements and additional mild reinforcement are provided. For the lower four parking levels steel columns are encased in concrete to form a round 2'-8" diameter round column. The post tensioned slabs provide the permanent lateral bracing for the foundation slurry wall to resist the lateral soil pressures.

At the plaza and first floor level the structural floor system changes from post tensioned concrete to steel beams with composite floor slabs. In the main quad area typical bay sizes remain the same. Typical horizontal framing in this area ranges from W24x76 beams with 52 shear studs to W36x135 beams with 80 shear studs. Three inch deck with 9" of 3000psi concrete is typical for all horizontal surfaces at the main quad space. Plate girders are used to transfer load from superstructure columns above this level to the columns extending through the parking garage. All plate girders are 48 inches deep with weights from 330 to 849 lb/ft.

The use of steel beams with composite action is continued in the floor framing of the building above grade. See the third floor framing in Figure 6 for a typical plan and framing layout.

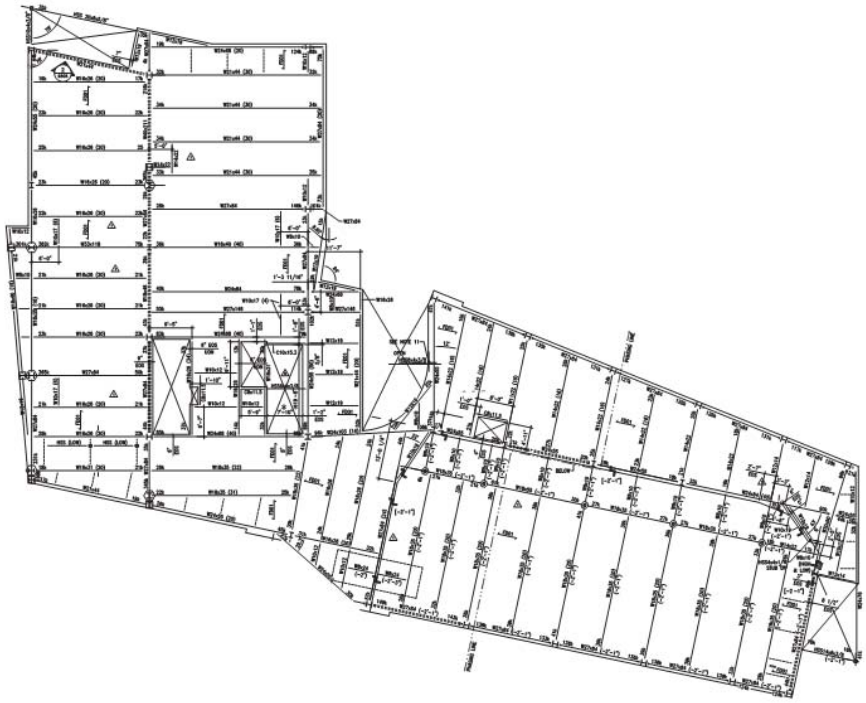


Figure 6 Second Floor Framing Layout

Columns

Typical column sections for the superstructure of the Simmons College School of Management are wide flange sections with some usage of hollow structural steel (HSS) sections. Wide flange sections are all W14s with weights varying from 43 to 109 lb/ft. The most commonly used wide flange column is a W14X90. HSS sections are either HSS6x6 or HSS8x8. In addition to carrying gravity loads the majority of the columns participate in the lateral force resisting systems as part of either the moment frames or braced frames.

Once the building column loads have been transferred by the plate girders W14 column sections continue to carry the load through the parking garage. Weights vary from 159 to 398 lbs/ft. In two different locations W14x398 with side plates or W14x500 columns are used. Here all columns below the first parking garage level are encased in concrete to form a 2’-8” diameter round column.

Supplementary Structural Systems

Two supplementary structural systems are used in the building in addition to the main load carrying elements. At the roof a braced frame screen is used to hide the penthouse and mechanical equipment. HSS sections are used for vertical and horizontal members while angles form the diagonal bracing.

In the parking garage reinforced concrete members are used to form the ramp access to all parking levels. Edge beams span the length of the length of the ramp with a 12 inch slab bridging the 21’-2” for the driving surface. Girders are 2’-7” deep and span below the slab at columns locations.

Lateral Systems

Two structural systems are used in the Simmons College School of Management to resist lateral forces applied to the building. In the north south direction of the building steel braced frames carry lateral loads. The lateral force resisting system in the east west direction is a combination of steel braced frames and steel moment frames. Locations of steel braced frames can be seen in Figure 7 and steel moment frames are noted in Figure 8. The number of steel braced frames used is reduced in the upper floors of the building. In some areas of the building, moment frames are used on in locations on upper floors where braced frames were present on the floors below. See Figure 9 for the identification of each lateral element in the building. Appendix B includes the elevations of each of the lateral frames. The majority of the braced and moment frames transfer load at their bases to transfer girders which frame to the garage columns and carry load to the foundations. An area of note is the offset of the moment frame B/MF-EW-3 and MF-EW-4. The moment frame changes from column line ZE to ZD. Lateral loads are then transferred to the moment frame on column line ZE by W33x141 beams. The effect of offsetting a moment frame is one area of the lateral system that may need some additional research.

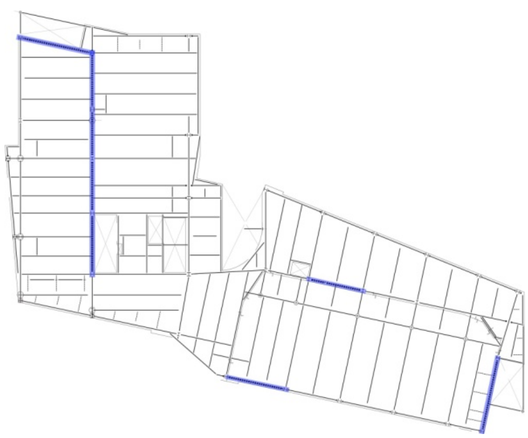


Figure 7 Braced Frame Locations

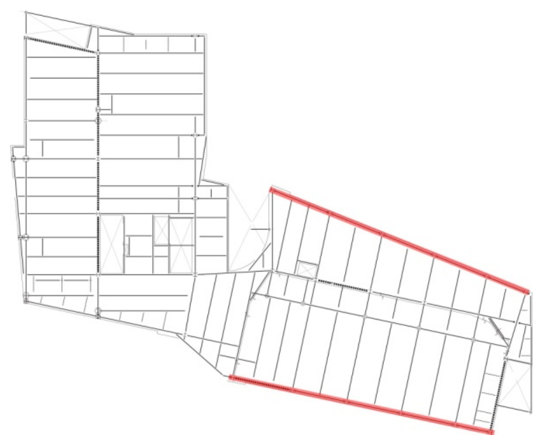


Figure 8 Moment Frame Locations

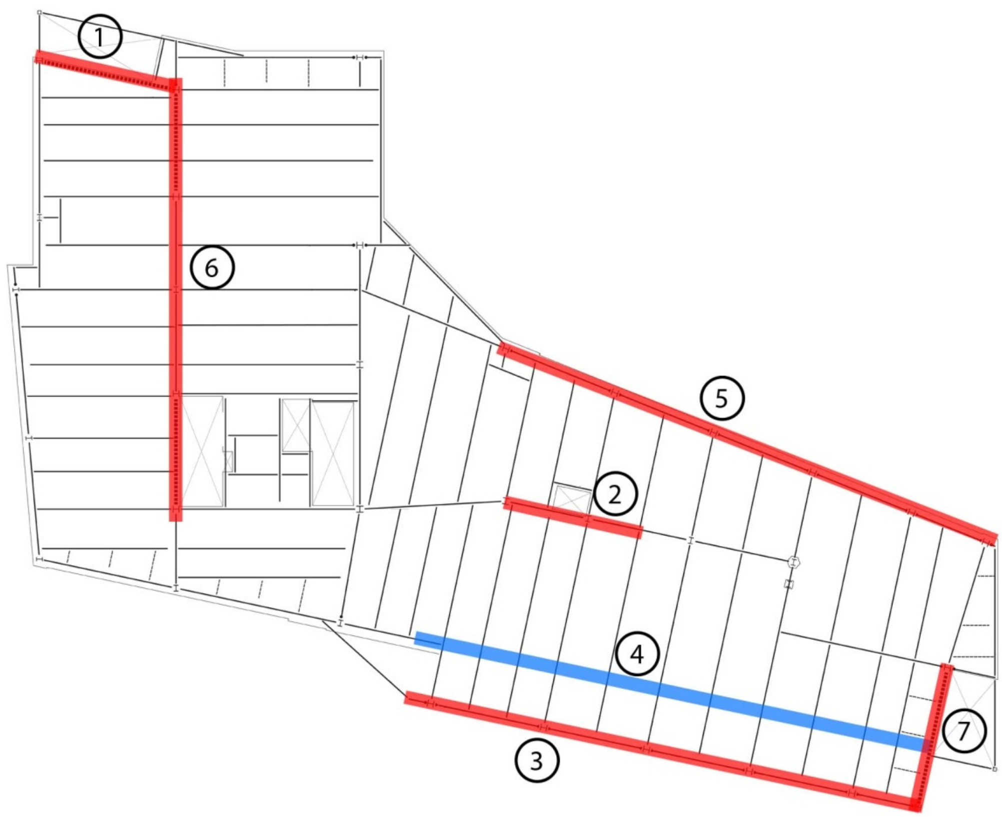


Figure 9 Lateral Frame Identifications

1. BF-EW-1
2. BF-EW-2
3. B/MF-EW-3
4. MF-EW-4 (offset of B/MF-EW-3 at 5th story)
5. MF-EW-5
6. BF-NS-1
7. BF-NS-2

At all levels the concrete floor deck forms a ridged diaphragm which transfers lateral load to either the braced or moment frames. The amount of force that each lateral load resisting element receives is dependent on that element's relative stiffness in the system. An assessment of the relative stiffness of each lateral force resisting element is presented in the following sections.

Due to the arrangement of the lateral elements throughout the building, the effect of torsion becomes increasingly important. When lateral loads are applied to the building all elements participate in the resistance of load even when the loads are applied in the primary directions. This causes a deviation in the force distribution from the anticipated forces determined by relative stiffness.

Code Requirements

Design Codes

Building Code, Design Loads:	Massachusetts State Building Code CMR 780 6th Addition
Reinforced Concrete:	American Concrete Institute (ACI) 318
Structural Steel:	American Institute of Steel Construction (AISC)

Substitute Codes for Thesis

Building Code:	International Building Code (IBC) 2006
Building Loads:	American Society of Civil Engineers (ASCE) 7-05
Structural Steel:	American Institute of Steel Construction (AISC) 13th Edition 2005
Reinforced Concrete:	American Concrete Institute (ACI) 318-08

Building Loads

Dead Loads

(All Values in PSF)

FD01	43.2
FD02	42.7
FD03	69.0
FD04	96.8
PT floor slab	175
Structural Steel	Per AISC Manual
Green Roof	100
Superimposed Dead loads:	
MEP	10
Partitions	20
Finishes/Misc.	5
Curtain Wall	10

Live Loads

(All Values in PSF)

Space:	Design Value	ASCE 7-05
Parking Floors	50	40
Plaza	100	100
	300 Construction	
Exit Corridors	100	100
Stairs	100	100
Lobbies	100	100
Typical Floor	50	50 (office load)
Corridors above 1st Floor	80	80
Roof Garden	100	100
Flat Roof	-	20
Mechanical Areas	150	

Lateral Loads

Lateral loads acting on the structure were determined according to ASCE 7-05. The original loading for the building was in accordance with the sixth addition of the Massachusetts State Building Code. This is one source of variance that is observed between design loads that those calculated in this report. Seismic loads were the controlling lateral force on the building. Both base shear and overturning moment values for seismic design were higher than the values for wind design.

Wind Load Analysis

Wind loads were calculated using method two, the analytical procedure from section 6.5 of ASCE 7-05. Given the configuration of the building, loads were assumed to act on projected widths of the building. In this technical report the wind lad was analyzed in the primary directions as seen in Figure 10. Additional steps were taken to consider diagonal wind load cases which will be discussed in the following sections.

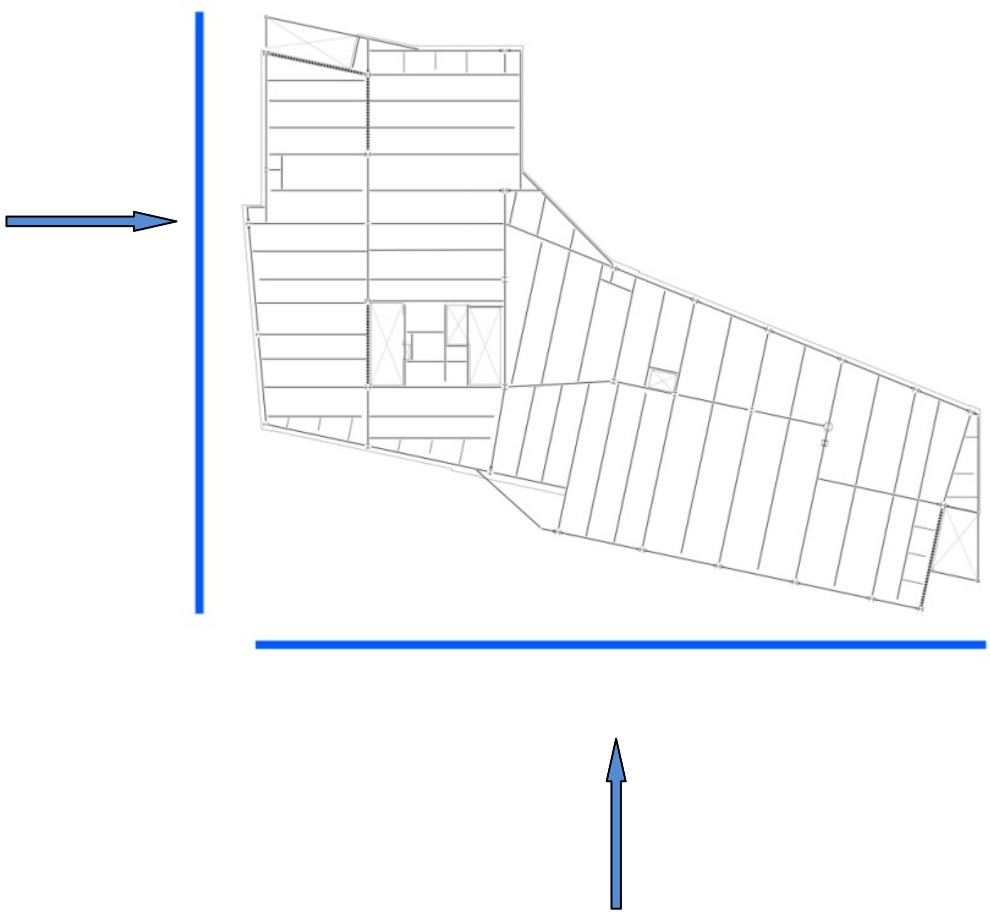


Figure 10 Wind Loading Directions

Design Wind pressures p EAST WEST direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.57	5.78	27.35	15.79
	60	30.6	20.56	5.78	26.34	14.78
	50	29.2	19.62	5.78	25.40	13.84
	40	27.4	18.41	5.78	24.19	12.63
	30	25.2	16.93	5.78	22.71	11.15
	25	23.8	15.99	5.78	21.77	10.21
	20	22.3	14.99	5.78	20.77	9.21
	15	20.5	13.78	5.78	19.56	8.00
Leeward	All	32.1	-8.09	5.78	-2.31	-13.87
Side	All	32.1	-18.87	5.78	-13.09	-24.65
Roof	70.5	32.1	-24.26	5.78	-18.48	-30.04
	70.5	32.1	-13.48	5.78	-7.70	-19.26
	70.5	32.1	-8.09	5.78	-2.31	-13.87

East
West

Pressure	height	width	moment arm	Shear	overturning moment
8.1	70.5	140	35.25	79.95	2818.13
13.8	15	140	7.5	28.98	217.35
15	5	140	17.5	10.50	183.75
16	5	140	22.5	11.20	252.00
16.9	5	140	27.5	11.83	325.33
18.4	10	140	35	25.76	901.60
19.6	10	140	45	27.44	1234.80
20.6	10	140	55	28.84	1586.20
21.6	10	140	65	30.24	1965.60
				254.74	9484.76

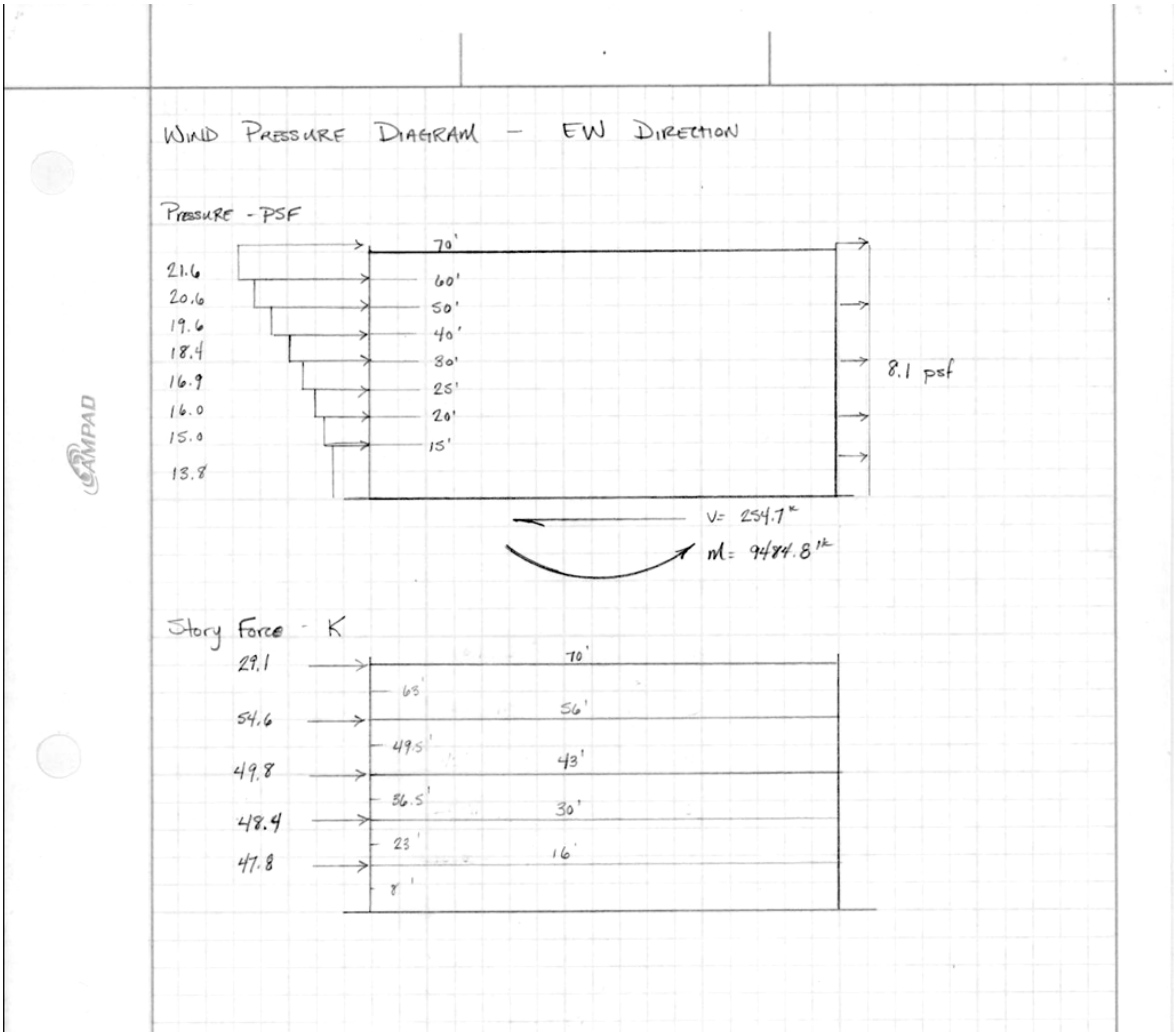


Figure 11 Wind Pressures, East-West

Design Wind pressures p NORTH SOUTH direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.06	5.78	26.84	15.28
	60	30.6	20.07	5.78	25.85	14.29
	50	29.2	19.16	5.78	24.94	13.38
	40	27.4	17.97	5.78	23.75	12.19
	30	25.2	16.53	5.78	22.31	10.75
	25	23.8	15.61	5.78	21.39	9.83
	20	22.3	14.63	5.78	20.41	8.85
	15	20.5	13.45	5.78	19.23	7.67
Leeward	All	32.1	-13.16	5.78	-7.38	-18.94
Side	All	32.1	-18.42	5.78	-12.64	-24.20
Roof	70.5	32.1	-31.58	5.78	-25.80	-37.36
	70.5	32.1	-18.42	5.78	-12.64	-24.20
	70.5	32.1	-18.42	5.78	-12.64	-24.20

North South

Pressure	height	width	moment arm	Shear	overturning moment
13.2	70.5	180	35.25	167.51	5904.66
13.5	15	180	7.5	36.45	273.38
14.6	5	180	17.5	13.14	229.95
15.6	5	180	22.5	14.04	315.90
16.5	5	180	27.5	14.85	408.38
18	10	180	35	32.40	1134.00
19.2	10	180	45	34.56	1555.20
20.1	10	180	55	36.18	1989.90
21.1	10	180	65	37.98	2468.70
				387.11	14280.06

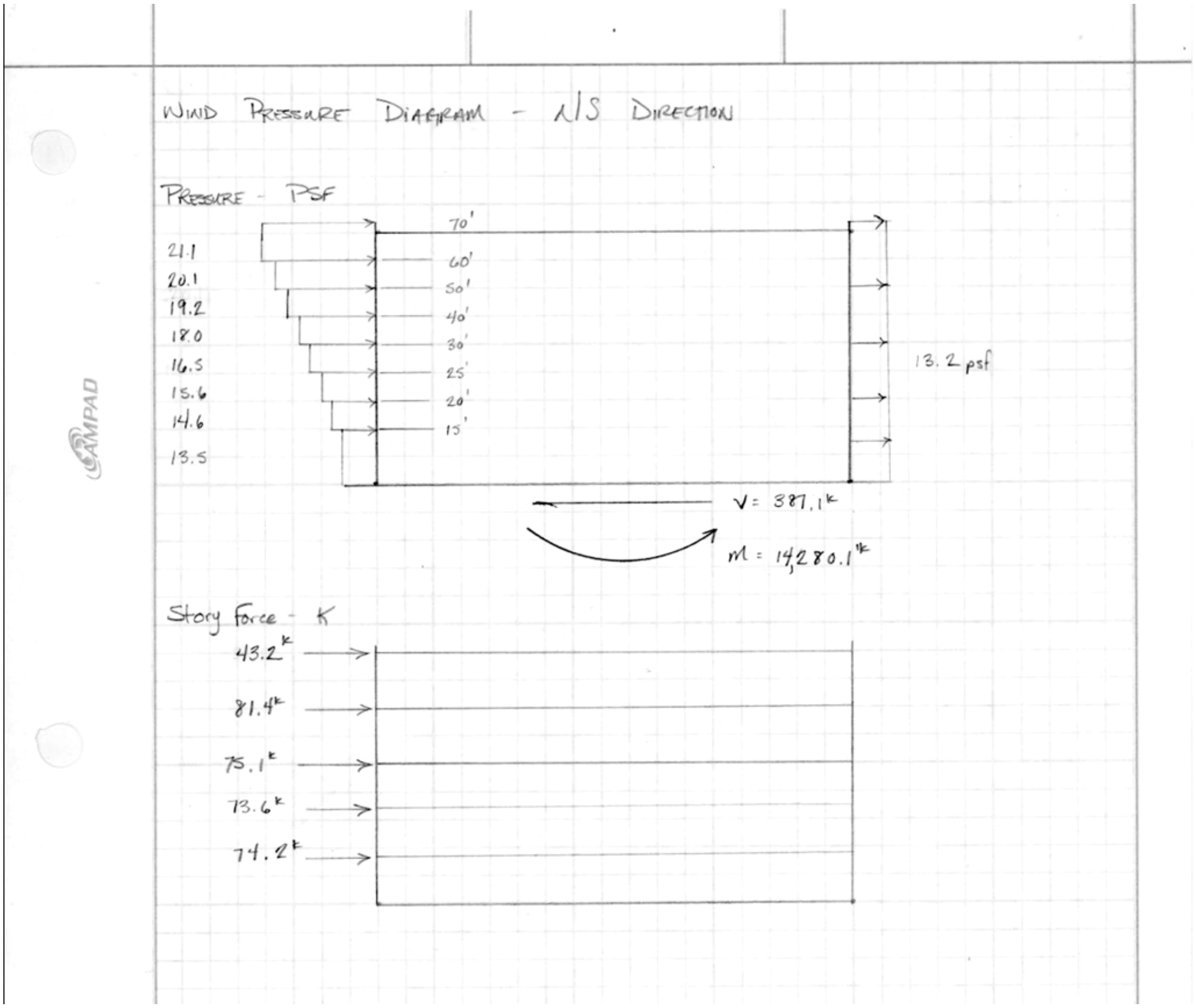


Figure 12 Wind Pressures, North-South

Seismic Load Analysis

Seismic loads, similar to the wind loads, were determined in accordance with ASCE 7-05 rather than the Massachusetts State Building Code. As a result some differences are present in the design calculations and those presented in this report. Site class E was used as a conservative approximation for the soil classification. This was determined to be the closest to the S3 soil classification that was used during design. The R-factor in each direction was determined to be a 5 when using the Massachusetts State Building Code. ASCE 7-05 categorizes the lateral systems differently which resulted in an R-factor of 6 in the EW direction and 3.25 in the NS direction. The ground motion acceleration values used in this report were determined with the USGS Ground Motion Parameter Calculator. Given these differences in the design procedures and those used in this evaluation, variance between final loadings can be expected.

Seismic Forces in the North/South Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	258.18	258.18	17899.62
5	1832	56	102592.0	0.34	373.46	631.64	20913.53
4	1438	43	61834.0	0.21	225.09	856.72	9678.80
3	1449	30	43470.0	0.14	158.24	1014.96	4747.19
2	1404	15.66	21986.6	0.07	80.04	1095.00	1253.36
				Total:	1095.00		54492.51

Seismic Forces in the East/West Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	127.32	127.32	8827.21
5	1832	56	102592.0	0.34	184.17	311.49	10313.52
4	1438	43	61834.0	0.21	111.00	422.49	4773.11
3	1449	30	43470.0	0.14	78.04	500.53	2341.08
2	1404	15.66	21986.6	0.07	39.47	540.00	618.10
				Total:	540.00		26873.02

Load Combinations

Lateral load combinations that would apply to this building determined from ASCE 7-05. These loads are listed below as well as the load case inputs for ETABS. Load cases which include dead and live loads would need to be combined through additional analysis methods. ETABS allowed for the assessment of the four wind load cases from section 6.5.12.3 of ASCE 7-05 to determine the critical loading of the structure. The 3D model was only developed to model the lateral system and did not include the effects of gravity loads.

ASCE 7-05 Lateral Load Cases

1.2D + 1.6(Lr or S or R) + 0.8W
1.2D + 1.6W + L + 0.5(Lr or S or R)
1.2D + E + L + 0.2S
0.9D + 1.6W + 1.6H
0.9D + E + 1.6H

Wind and seismic loads were determined for the building in the primary X and Y direction. Wind loads were applied to the building at the center of pressure while seismic loads were applied to the center of mass of each floor diaphragm. Using each load and the load cases, the following load combination inputs were developed for the 3D ETABS model. These combinations only include the unfactored lateral loads. The load factors and effects of gravity loads need to be assessed through additional analysis methods.

ETABS Load Combinations

±Ex
±Ey
±Wx
±Wy
±0.75Wx ±Mtx
±0.75Wy ±Mty
±0.75Wx ±0.75Wy
±0.563Wx ±0.563Wy ±0.563Mtx ±0.563Mty (Moments only considered acting in the same direction)

An additional wind load case was developed to be analyzed for its effects on the structure. Due to the geometry of the building a diagonal wind would likely cause the critical loading for this building. To develop this load the components of the X and Y wind pressures were added at a varying angle seen in Figure 13. The angle where the maximum pressure would occur was then derived by method shown below. In some lateral force resisting elements this additional loading is the critical wind load case, controlling over the ASCE 7-05 wind load cases. The distribution of lateral forces caused by this diagonal loading is presented in the following section.

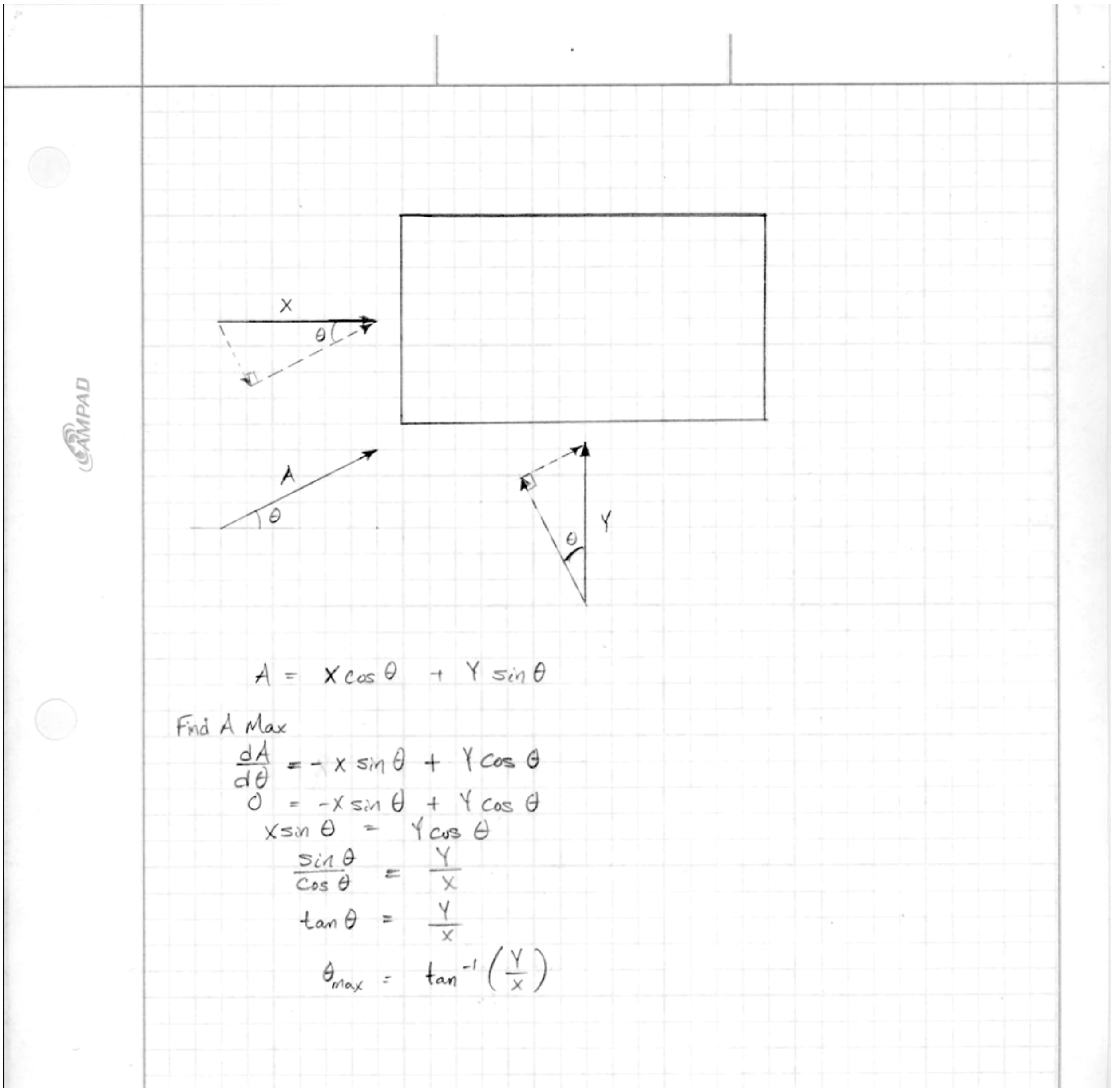


Figure 13 Diagonal Wind Combinations

Distribution of Lateral Loads

To approximate the distribution of lateral load, each lateral force resisting element was assessed along its primary line of action. This initial investigation into the stiffness of each frame is seen as only an estimation of how each element will perform in the building. The development of a full 3D model will be necessary to determine the fraction of lateral load that each element in the system will see. An alternative approach to the assessment of the non orthogonal frame arrangement, each element could be assessed along perpendicular axis with reductions in stiffness made according to the angle of the frame. The 12 degree offset was not deemed to be large enough to require this approach to the stiffness analysis.

The process used to determine the lateral stiffness was to apply a unit load on each frame and find the displacement. Stiffness, K, is then determined by the equation $P = K\Delta$. The lateral system remains relatively consistent from the fifth to the second stories with considerable changes in lateral elements in the first story. Therefore the stiffness was assessed at the fifth floor level and the second floor level. SAP 2000 was used to model the 2D frames. The modeling assumptions are listed below.

SAP 2000 2D Modeling:

1. All basses are pinned (See Note below)
2. Braces and beams not participating in moment frames have the 3-3 moment released
3. Rigid end offsets = 1.0 (moment frames)
4. Panel Zone Explicit Modeling (moment frames)
5. Equal Constraints at each floor level to model diaphragm constraints
6. Beam insertion points with modified stiffness: Top Center = -5.25

- Note: Many of the columns of the braced and moment frames sit on built up plate girders that transfer loads to the garage columns below. There would be no moment transfer from the columns to the girders which allowed from the basses to be modeled as pinned. The effect of vertical displacement at the supports was not explicitly modeled. The base conditions may be an area of further study in the building.

Lateral stiffness of each element at the fifth and second floor is summarized below. The label of each frame refers to the labeling presented in Figure 9. Relative stiffness is presented as a percentage of lateral loads that each element will likely see when applied along the elements primary axis. Due to the geometry of the building, the torsion shear component was not directly computed.

X Direction Lateral Elements

Frame	5th Fl. Defl.	k5	%P@5
BF-EW-1	0.00585	170.94017	63.0
B/MF-EW-4	0.014136	70.74137	26.1
MF-EW-5	0.03365	29.71768	10.9

Frame	2nd Fl. Defl.	k2	%P@2
BF-EW-1	0.000621	1610.3060	46.8
B/MF-EW-4	0.000838	1193.3174	34.7
BF-EW-2	0.001752	570.7763	16.6
MF-EW-5	0.015019	66.5823	1.9

Y Direction Lateral Elements

Frame	5th Fl. Defl.	k5	%P@5
BF-NS-1	0.002441	409.66817	61.8
BF-NS-2	0.003941	253.74270	38.2

Frame	2nd Fl. Defl.	k2	%P@2
BF-NS-1	0.000149	6711.4094	76.0
BF-NS-2	0.000471	2123.1423	24.0

ETABS 3D Building Model

It was determined to be important to develop a 3D building model to account for all effects of the building lateral system arrangement as well as the eccentric loading on the building. To perform the modeling ETABS was used to accurately determined the center of mass, center of rigidity, and distribution of lateral loads. Additionally the 3D model outputs the building's primary periods of vibration which can be used to develop dynamic response characteristics. The modeling procedure for ETABS was performed similar to that in SAP with the assumptions listed below.

ETABS Modeling Assumptions:

1. All basses are pinned
2. Braces and beams not participating in moment frames have the 3-3 moment released
3. Rigid end offsets = 1.0 (moment frames)
4. Panel Zone Explicit Modeling (moment frames)
5. Rigid Diaphragm Constraint at all levels
6. Diaphragm mass based on a typical 100psf floor dead weight
7. Beam insertion points with modified stiffness: Top Center = -5.25

Similar to the SAP modeling, the bases were assumed not to resist moment with the effects of vertical and horizontal displacements neglected. Typical floor dead weights including the exterior walls as a uniform distributed load ranged from 92 – 101 psf. For consistency a 100 psf floor load was used as the input for building mass. The critical building output from the ETABS model is summarized in the following charts and tables.

First Mode Period of Vibration

Period of Vibration (s)	
X	1.0123
Y	0.7343
Z	0.5465

Center of Mass and Center of Rigidity
Top Left of Floor Diaphragm as (0,0) See Figure 14

Story	XCM	YCM	XCR	YCR
STORY5	1025.94	-1032.845	982.088	-893.621
STORY4	908.083	-899.85	1076.668	-822.085
STORY3	908.083	-899.85	1014.08	-766.483
STORY2	908.083	-899.85	975.33	-726.267
STORY1	907.257	-907.941	770.768	-704.372

Figure 14 Center of Mass and Center of Rigidity

Distribution of Forces

WIND X

Fourth Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	-49.048	58.60
BF-EW-2	0	0.00
B/MF-EW-3	-19.882	23.75
MF-EW-4	-10.841	12.95
Total	-79.771	95.31
Input	-83.7	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	1.7337	
BF-NS-2	-18.7689	
Total	-17.0352	
Input	0	

First Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	-141.3366	61.56
BF-EW-2	-40.9378	17.83
B/MF-EW-3	-37.0904	16.15
MF-EW-4	-1.6442	0.72
Total	-221.009	96.26
Input	-229.6	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	2.2662	
BF-NS-2	-50.429	
Total	-48.1628	
Input	0	

WIND Y

Fourth Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	13.203	
BF-EW-2		
B/MF-EW-3	-3.8473	
MF-EW-4	0.6131	
Total	9.9688	
Input	0	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	-77.4489	62.16
BF-NS-2	-41.2595	33.11
Total	-118.7084	95.27
Input	-124.6	

First Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	26.4666	
BF-EW-2	1.9981	
B/MF-EW-3	2.7305	
MF-EW-4	1.0744	
Total	32.2696	
Input	0	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	-214.5905	61.75
BF-NS-2	-129.8134	37.36
Total	-344.4039	99.11
Input	-347.5	

WIND Combination

Fourth Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	-72.6391	48.20
BF-EW-2	0	0.00
B/MF-EW-3	-40.3057	26.75
MF-EW-4	-18.7899	12.47
Total	-131.7347	87.42
Input	-150.7	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	-87.3532	60.04
BF-NS-2	-81.8137	56.23
Total	-169.1669	116.27
Input	-145.5	

First Story

X-Direction Framing		
	Shear	% Fraction
BF-EW-1	-227.3073	54.50
BF-EW-2	-72.7881	17.45
B/MF-EW-3	-70.5568	16.92
MF-EW-4	-1.72	0.41
Total	-372.3722	89.28
Input	-417.1	

Y-Direction Framing		
	Shear	% Fraction
BF-NS-1	-244.6733	60.74
BF-NS-2	-241.796	60.03
Total	-486.4693	120.77
Input	-402.8	

When compared to the stiffness analysis of the lateral force resisting frames the building is performing as anticipated for the primary direction loading. It is interesting to note the load that is generated in the frames perpendicular to the line of load application. This directly results from the arrangement of frames at an angle to the load direction as well as eccentricity between load application and resistance.

As part of the design checks for the existing structure the building drift was analyzed under lateral loading. Building drift due to wind load was compared to the typical industry standard for wind drift, h/400. Seismic story drift was compared to ASCE 7-05 allowable story drift values from Table 12.12-1. It was important to address the total movement when assessing the drift values. Torsion in the building caused the drift and displacement in both the X and Y directions. Therefore the resultant of these two components was used to compare against accepted values. Both seismic and wind design checks were verified to meet code and industry standard. Below is a summary of the critical drift and displacement values.

Wind Building Drift

Story	Point	Load	DispX	DispY	DispTOT	Allowable Building Drift Δ=h/400
STORY5	84	+0.563Wx +0.563Wy +0.563Mtx +0.563Mty	0.4147	0.4195	0.589878	2.1

Seismic Story Drift

Story	Point	Load	DriftX	DriftY	DriftTOT	Allowable Drift Δ=0.015hx
STORY5	81	1EY	0.000729	0.003244	0.003325	2.4
STORY4	50	1EY	0.001048	0.002979	0.003158	2.34
STORY3	50	1EY	0.00086	0.002774	0.002904	2.34
STORY2	20	1EX	0.002342	0.000165	0.002348	2.565
STORY1	21	1EY	0.000417	0.002018	0.002061	2.82

Spot Checks

In order to check the adequacy of the existing structure one frame was chosen for a more in depth investigated of its load carrying capability. Braced frame BF-NS-2, as seen in Figure 15 below, was chosen for this additional study. Unlike the majority of the lateral load resisting elements in the building, this braced frame sits on spread footings rather than framing into the garage structure below. Therefore it was determined that uplift on this frame would be a design consideration worth investigating.

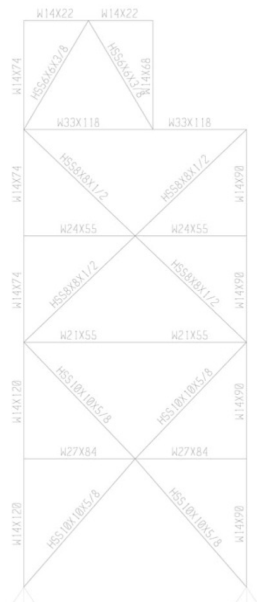


Figure 15 BF-NS-2 Frame Sections

To initially determine the critical wind and seismic load cases of for this braced frame the output of forces for a diagonal brace in the first story was selected. By displaying the maximum axial force that would be present in this brace it could then be determined which load case would deliver the most lateral load to this frame. The diagonal wind load case was the controlling wind load case for this braced frame. However, the controlling ASCE 7-05 wind load case was selected for the input wind load in the factored loading. The Y direction seismic loading caused greatest amount of load to be delivered to this frame.

The two uplift load cases were checked for the design of this braced frame, 0.9D ± 1.6W, and 0.9D ± 1.0E. Dead loads acting on the braced frame were applied factored along with the lateral loading according to the load combinations. Out of plane shear acting on this frame was not considered in this analysis. Each of the load cases caused the foundations to experience an uplift load. The load case 0.9D + 1.0E caused the largest uplift loading at the base of the frame. The resulting reactions can be seen in Figure 16. The driven pile foundation system would have the ability to resist against uplift caused by the lateral loading. However, the capacity of the foundations was not checked as part of this design verification.

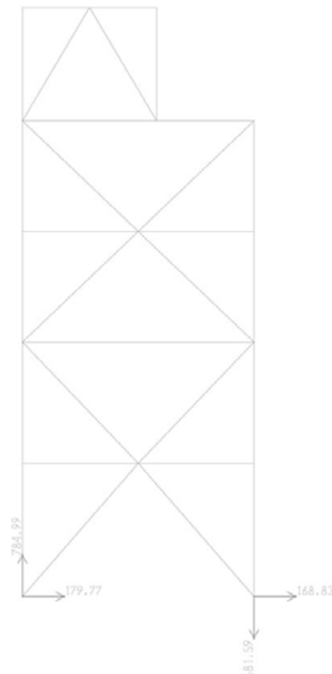


Figure 16 BF-NS-2 Critical Uplift Reactions

The first story diagonal members were checked under the most critical loading condition from the uplift load cases, 0.9D ± 1.6W, and 0.9D ± 1.0E. A factored compressive load of 275K and a factored tension load of 258K resulted from these load cases. It was determined that the HSS10X10X5/8 diagonal brace was adequately sized to resist the forces applied as a result of these load cases. Additional load cases were not checked for the critical loading of the members. Only the member capacity was considered at this time. Connection capacity was not considered in this report. Spot check calculation can be seen in Appendix E.

Conclusion

The structure of the Simmons College school of Management was assessed for the design lateral loading that the building would experience and the adequacy of the lateral load resisting system. As anticipated building torsion was an important parameter to consider for the resistance of lateral loading. This was a result of both building geometry and eccentricity between the point of load application and the center of rigidity of the structure.

Lateral loads were applied in accordance with ASCE 7-05. Additional load cases were considered for their effects when acting on the structure. The building geometry would likely result in a diagonal loading to be critical to the design of the lateral system. A combination wind load case was considered in order to recognize this as a design consideration on the structure.

The analysis of the building displayed that the structure was designed to adequately resist all lateral loading as well as meet the serviceability requirements of the structure. Both wind and seismic drift values were within the limitations of code and accepted industry standards. One braced frame was analyzed to verify its load carrying capacity. It was found that the members were designed to support the applied lateral loads.

Appendix A: Typical Layout

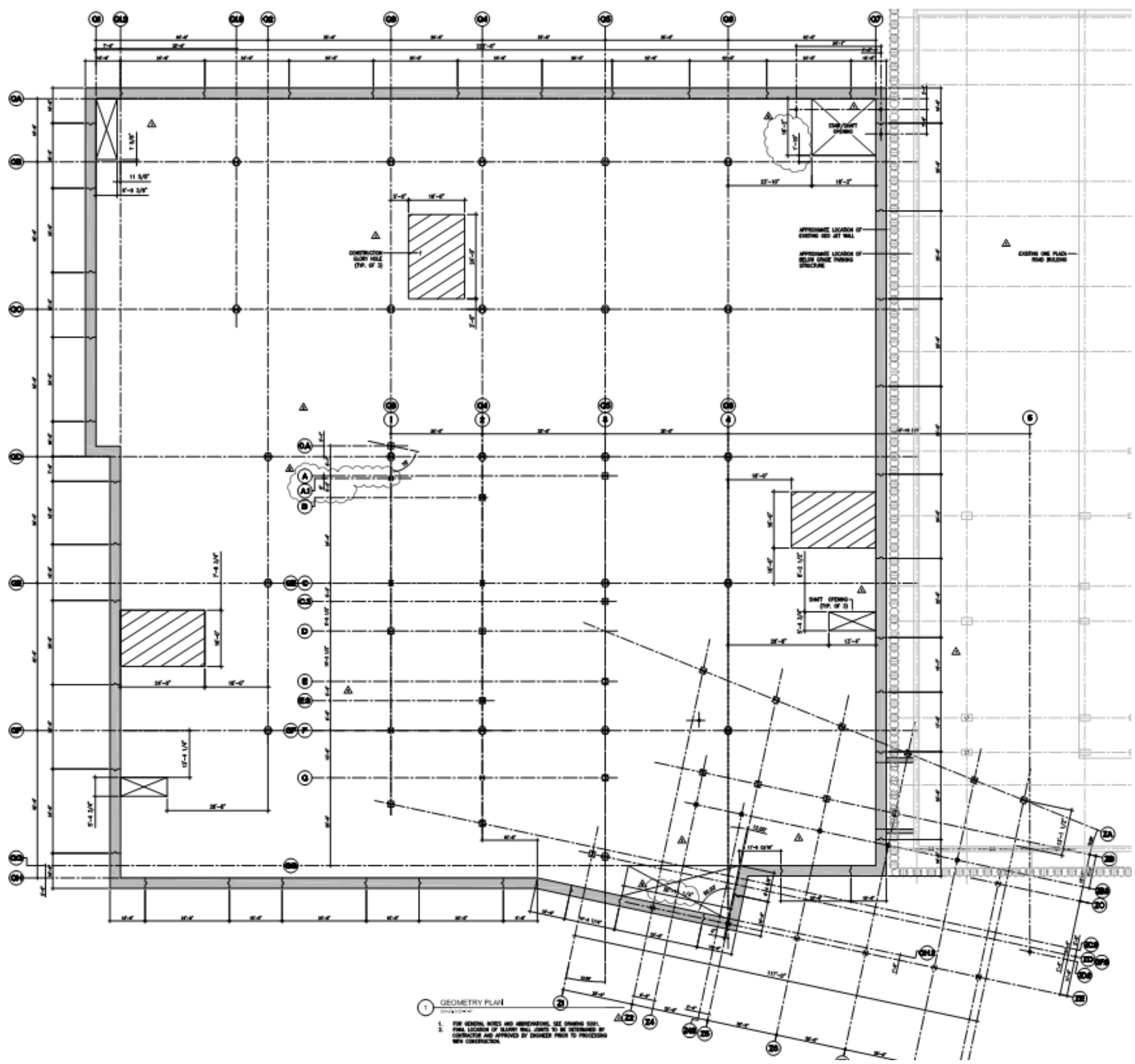


Figure 17 Sub Grade Parking Garage Layout

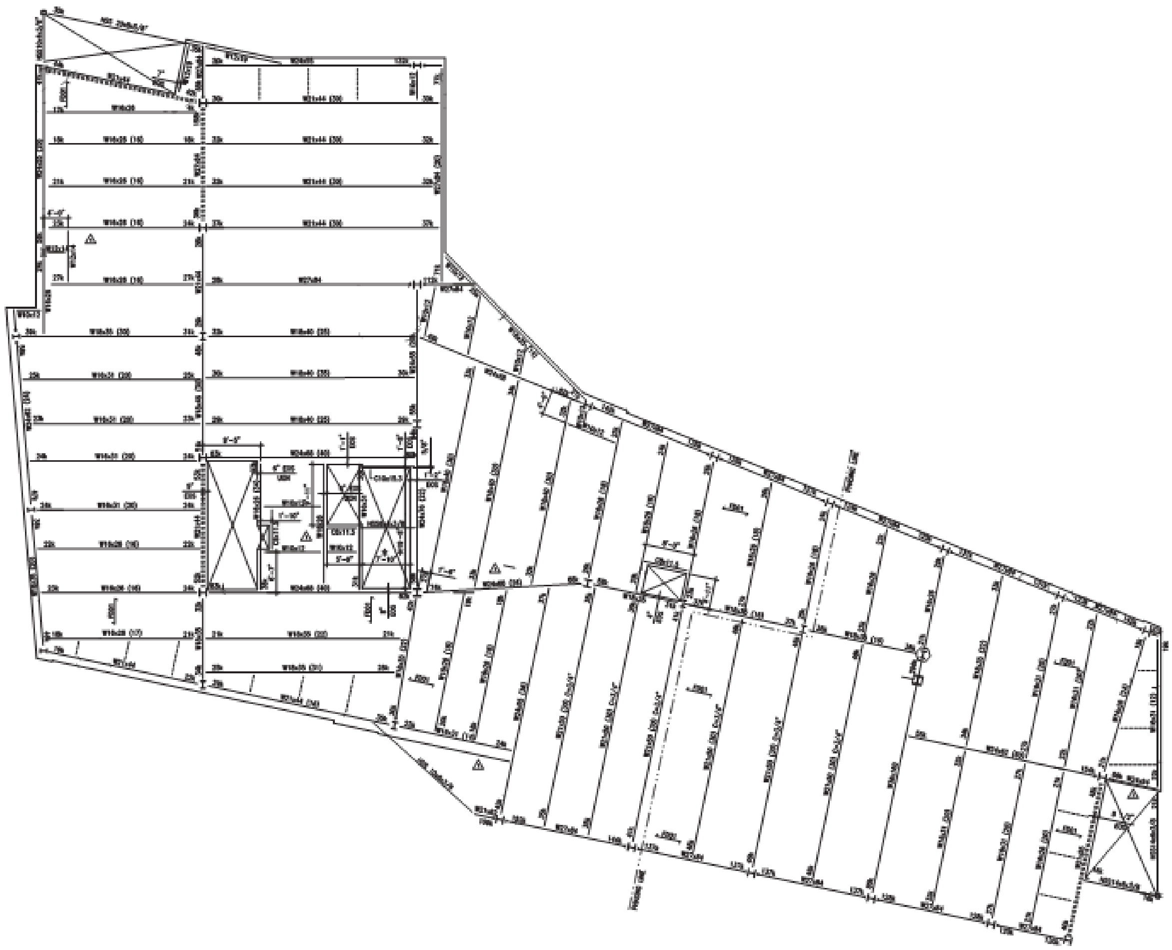


Figure 18 Typical Above Grade Building Framing

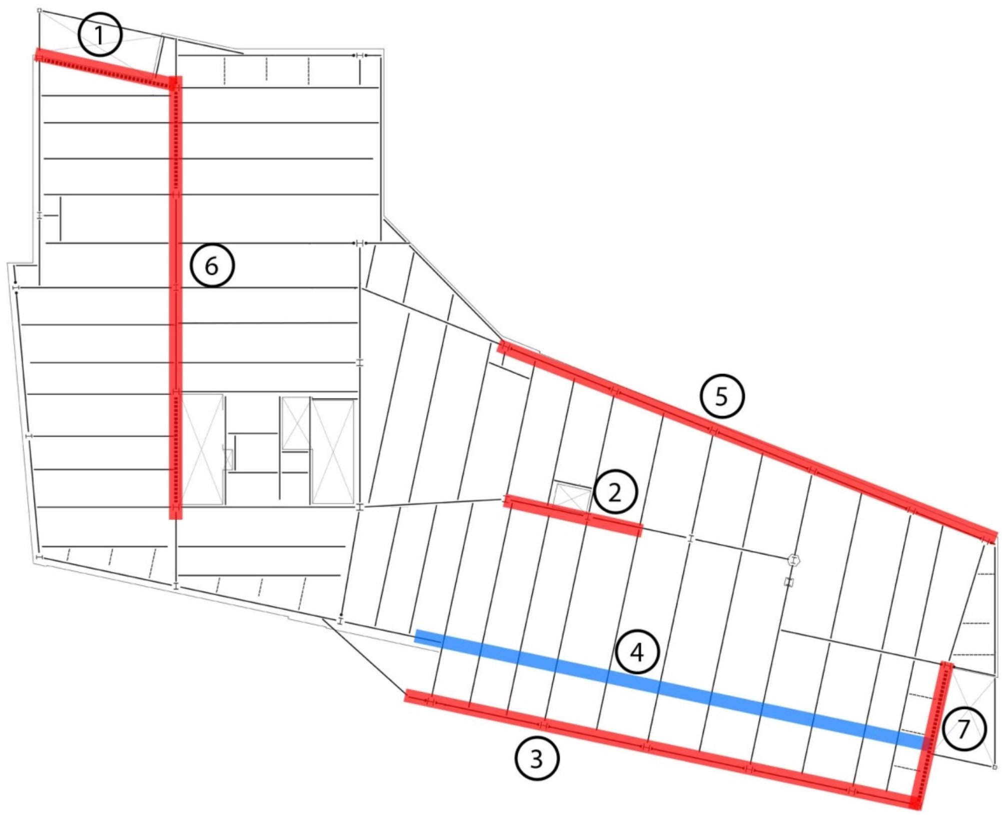


Figure 19 Lateral Frame Identifications

1. BF-EW-1
2. BF-EW-2
3. B/MF-EW-3
4. MF-EW-4 (offset of B/MF-EW-3 at 5th story)
5. MF-EW-5
6. BF-NS-1
7. BF-NS-2

Figure 20 Center of Mass and Center of Rigidity

Appendix B: Lateral Frame Elevations

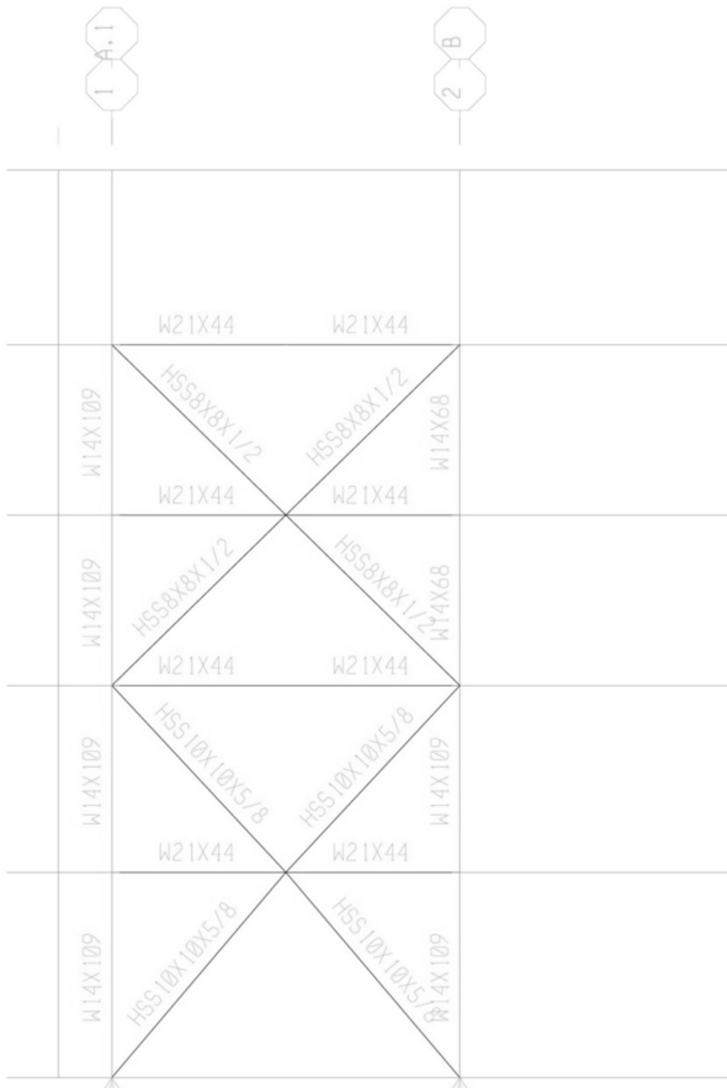


Figure 21 BF-EW-1

Figure 22 BF-EW-2

Figure 23 B/MF-EW-3

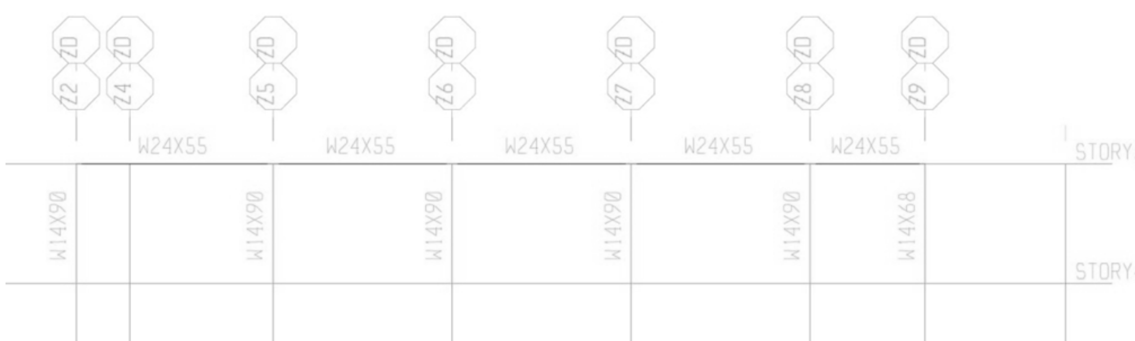


Figure 24 MF-EW-4

Figure 25 MF-EW-5

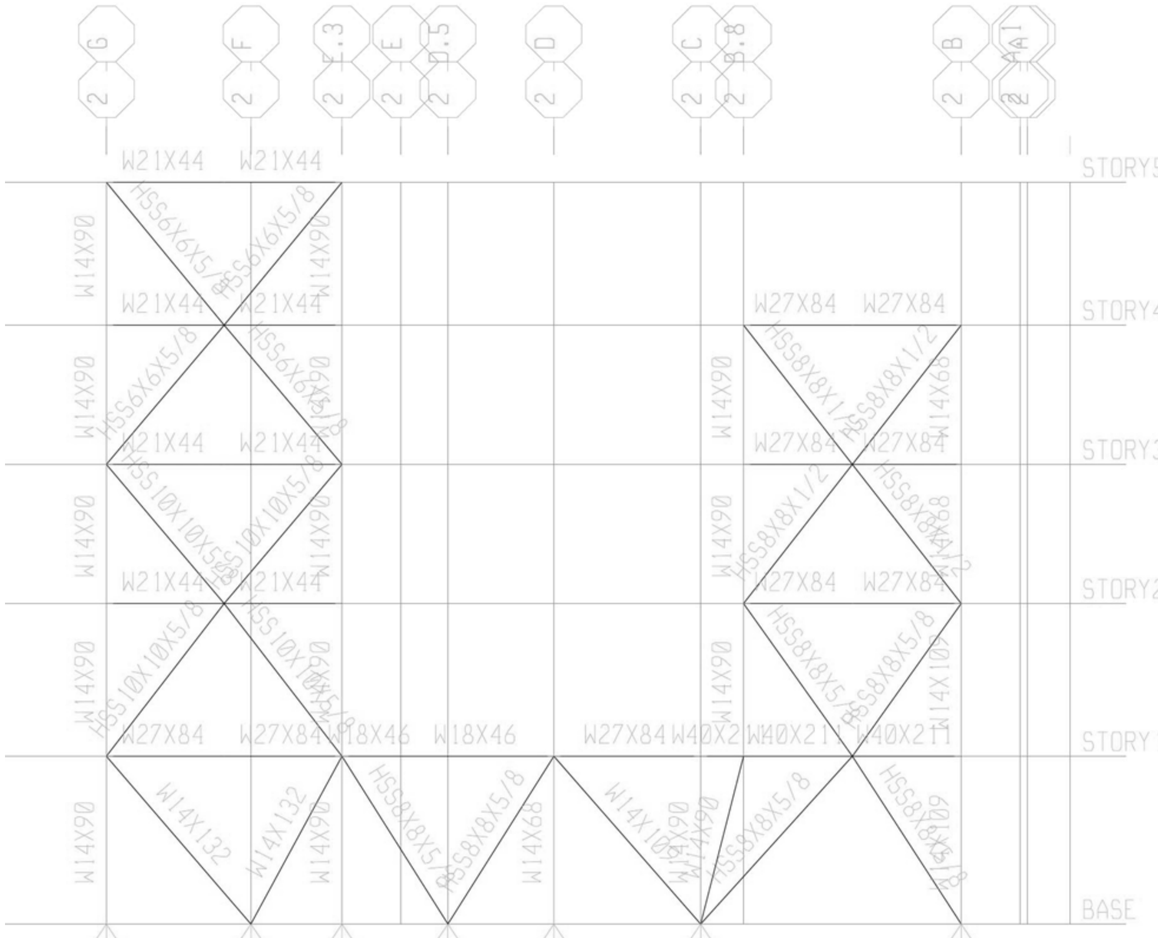


Figure 26 BF-NS-1

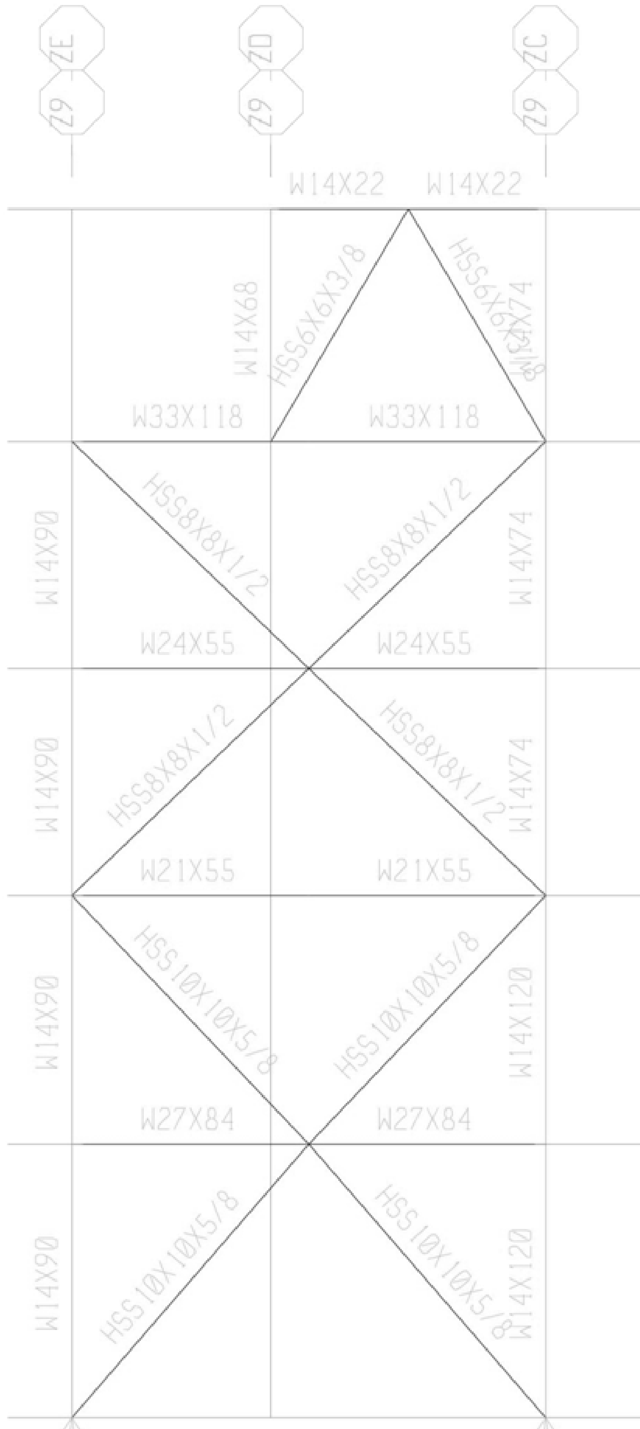


Figure 26 BF-NS-2

Appendix C: Wind Loads

Wind Loads - ASCE 7-05
Mean h = 70.5' > 60 ∴ Cannot Apply MFWRS
Method 2 - Analytical Procedure § 6.5

Simmons College - SoM
Design Loads
9/28/09
KTW

Location: Boston, Ma., Exposure B

Basic Wind Speed	V = 120 mph	Figure 6-1
Wind Directionality Factor	K_d = 0.85	Table 6-4
Occupancy Category: III		Table 1-1
Importance Factor I:	I = 1.15	Table 6-1
Not Located on Hill, Ridge, Escarpment	K_{zt} = 1.0	§ 6.5.7.1
Exposure Coefficient	$K_{z,h}$ = 0.89	Table 6-3

$q_p = 0.00256\, K_z K_{zt} K_d V^2 I$
$= 0.00256(0.89)(1.0)(0.85)(120)^2(1.15)$
$q_p = 32$ psf

q_p = 32 psf Eq. 6-15

Parapet Net Pressure Coefficient
WW GC_{pn} = 1.5 § 6.5.12.2.4
LW GC_{pn} = -1.0 " "

Design Pressure on Parapet
Windward: $p_p = q_p\, GC_{pn} = (32)(1.5)$
$p_p = 48$ psf

p_p = 48 psf (WW)

Leeward: $p_p = q_p\, GC_{pn}$
$= 32(-1.0)$

p_p = -32 psf (LW)

Natural Frequency
EW - Steel Moment Resisting Frame
$n_1 = 22.2/H^{0.8} = 22.2/(70.5)^{0.8} = 0.74$

NS/EW - Steel braced frame
Use (C6-17 (average value)
$n_1 = 100/H = 100/70.5 = 1.42 \geq 1$

Check Lower Bound
$n_1 = 75/70.5 = 100/70.5 = 1.06 \geq 1$

Since the average value and Lower bound are both greater than 1:

ASSUME STRUCTURE IS RIGID

$g_Q = g_v = 3.4$

$\bar{z} = 0.6h = 0.6(70.5)$
$\bar{z} = 42.3$ ft $> z_{min} = 30'$

$\bar{z}$ = 42.3 ft.

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{42.3}\right)^{1/6}$
$= 0.29$

$I_{\bar{z}}$ = 0.29

WIND LOADS 2

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{42.3}{33}\right)^{(1/3)}$$

$$L_{\bar{z}} = 347.6'$$

$L_{\bar{z}} = 347.6'$

$$Q_{E-W} = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$Q_{E-W} = \sqrt{\frac{1}{1+0.63\left(\frac{95+70.5}{347.6}\right)^{0.63}}} =$$

$Q_{E-W} = 0.85$

$$Q_{N-S} = \sqrt{\frac{1}{1+0.63\left(\frac{170+70.5}{347.6}\right)^{0.63}}} = 0.82$$

$Q_{N-S} = 0.82$

$$G = 0.925\left[\frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right]$$

$$G_{EW} = 0.925\left[\frac{1+1.7(3.4)(0.29)(0.85)}{1+1.7(3.4)(0.29)}\right] = 0.84$$

$G_{EW} = 0.84$

$$G_{N-S} = 0.925\left[\frac{1+1.7(3.4)(0.29)(0.82)}{1+1.7(3.4)(0.29)}\right] = 0.82$$

$G_{NS} = 0.82$

Velocity Pressure Coefficients

Height Above Ground	K_z
70	0.89
60	0.85
50	0.81
40	0.76
30	0.70
25	0.66
20	0.62
< 15	0.57

WIND LOADS 8

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$
$= 0.00256 K_z (1.0)(0.85)(120^2)(1.15)$
$= 36.03 K_z$

Height Above Ground	K_z	q_z (psf)
70	0.89	32.1
60	0.85	30.6
50	0.81	29.2
40	0.76	27.4
30	0.70	25.2
25	0.66	23.8
20	0.62	22.3
<15	0.57	20.5

Pressure Coefficient (E-W)
Windward Wall: $C_p = 0.8$ w/ q_z
Leeward Wall
$(L/B) = (170/95) = 1.8 \Rightarrow 2$ $C_p = -0.3$ w/ q_h
Side Walls $C_p = -0.7$ w/ q_h
Roof Pressures
$h/L = 70.5/170 = 0.41 < 0.5$
from WW edge to $h = 70.5$ $C_p = -0.9, -0.18$ w/ q_h
from 70.5 to $2h = 141$ $C_p = -0.5, -0.18$ w/ q_h
from 141 to 170 $C_p = -0.3, -0.18$ w/ q_h

Pressure Coefficients (N-S)
Windward Wall $C_p = 0.8$ w/ q_z
Leeward Wall
$(L/B) = (95/170) = 0.56$ $C_p = -0.5$ w/ q_h
Side Walls $C_p = -0.7$ w/ q_h
Roof Pressures
$h/L = 70.5/95 = 0.74$
$(h/2)(w) = 6000 \Rightarrow$ Reduction factor $= 0.9$
from WW edge to $h/2 = 35.25$ $C_p = -1.2, -0.18$ w/ q_h
from 35.25 to 95 $C_p = -0.7, -0.18$ w/ q_h

Windward walls, side walls, leeward walls, roofs
$q_i = q_h = 32.1$ psf

Internal Pressure Coefficient $GC_{pi} = \pm 0.18$

EAST - WEST
Windward Walls
$p_z = q_z G C_p - q_h GC_{pi}$
$= q_z(0.84)(0.8) - 32.1(\pm 0.18) = 0.672 q_z \pm 5.78 = p_z$

Leeward Side walls & Roof
$p_h = q_h G C_p - q_h (GC_{pi})$
$= C_p(32.1)(0.84) - (32.1)(\pm 0.18) = 26.96 C_p \pm 5.78 = p_h$

WIND LOADS 4

NORTH SOUTH

Windward Walls

$p_z = q_z(0.82)(0.8) - 32.1(\pm 0.18)$

$p_z = 0.656\, q_z \pm 5.78$

Leeward, Side walls & Roof

$p_h = C_p(32.1)(0.82) \pm 5.78$

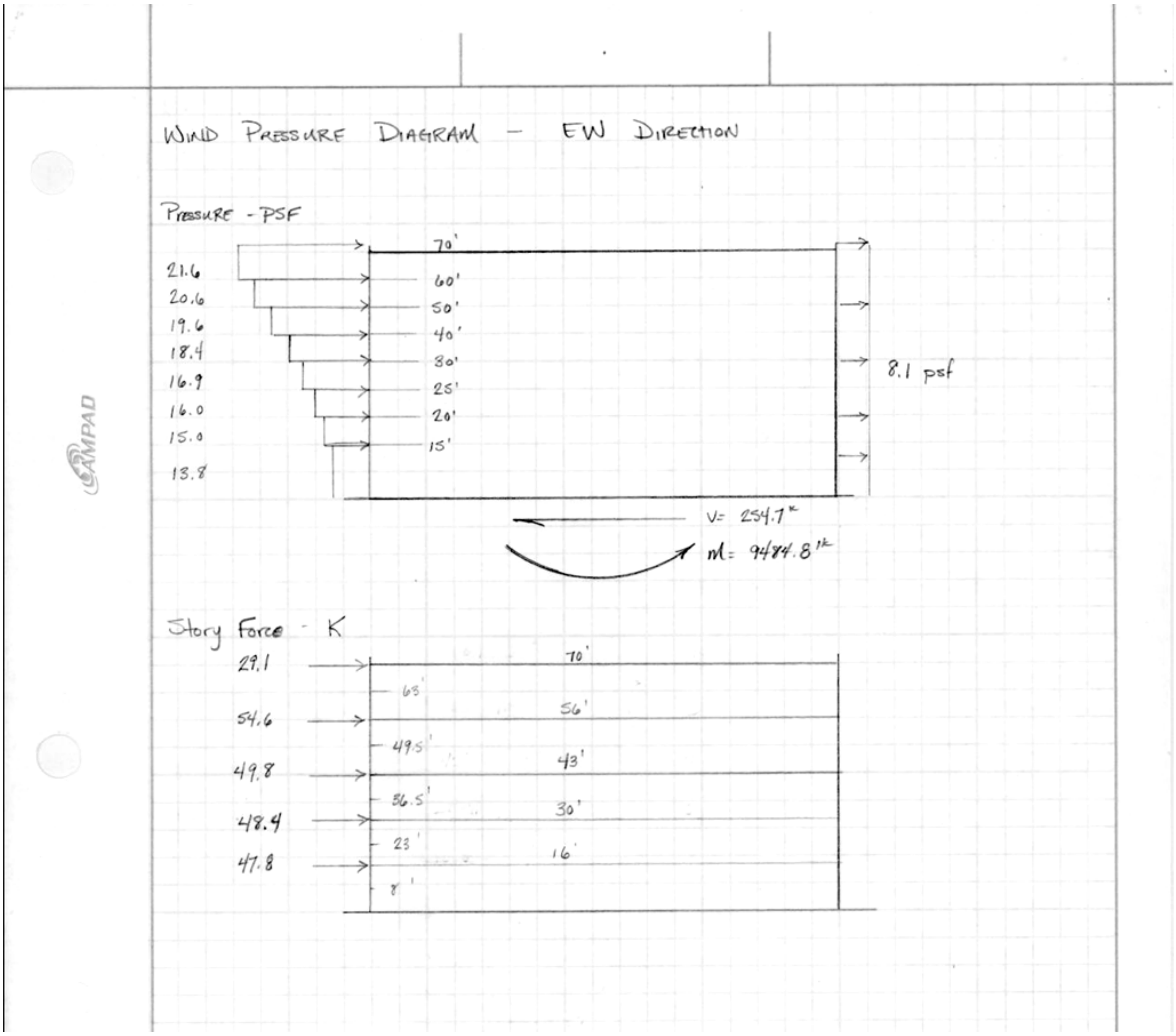
$p_h = 26.32\, C_p \pm 5.78$

Design Wind pressures p in the EAST WEST direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.57	5.78	27.35	15.79
	60	30.6	20.56	5.78	26.34	14.78
	50	29.2	19.62	5.78	25.40	13.84
	40	27.4	18.41	5.78	24.19	12.63
	30	25.2	16.93	5.78	22.71	11.15
	25	23.8	15.99	5.78	21.77	10.21
	20	22.3	14.99	5.78	20.77	9.21
	15	20.5	13.78	5.78	19.56	8.00
Leeward	All	32.1	-8.09	5.78	-2.31	-13.87
Side	All	32.1	-18.87	5.78	-13.09	-24.65
Roof	70.5	32.1	-24.26	5.78	-18.48	-30.04
	70.5	32.1	-13.48	5.78	-7.70	-19.26
	70.5	32.1	-8.09	5.78	-2.31	-13.87

East
West

Pressure	height	width	moment arm	Shear	overturning moment
8.1	70.5	140	35.25	79.95	2818.13
13.8	15	140	7.5	28.98	217.35
15	5	140	17.5	10.50	183.75
16	5	140	22.5	11.20	252.00
16.9	5	140	27.5	11.83	325.33
18.4	10	140	35	25.76	901.60
19.6	10	140	45	27.44	1234.80
20.6	10	140	55	28.84	1586.20
21.6	10	140	65	30.24	1965.60
				254.74	9484.76

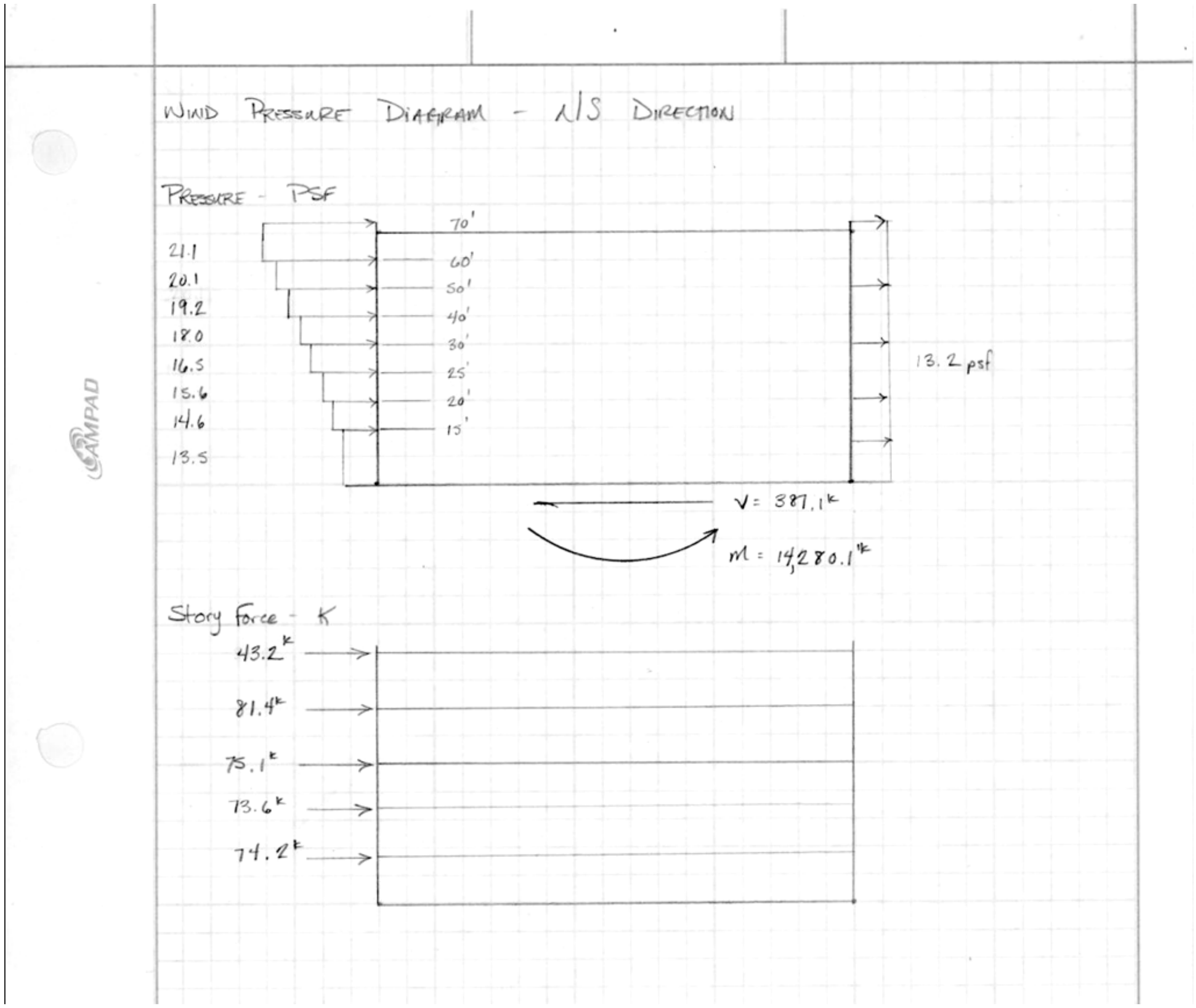

WIND PRESSURE DIAGRAM – EW DIRECTION
PRESSURE - PSF
70'
21.6
60'
20.6
50'
19.6
40'
18.4
30'
16.9
25'
16.0
20'
15.0
15'
13.8
8.1 psf
V= 254.7k
M= 9484.8'k
Story Force - K
29.1
70'
65'
54.6
56'
49.5'
49.8
43'
36.5'
48.4
30'
23'
47.8
16'
8'

Design Wind pressures p in the NORTH SOUTH direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.06	5.78	26.84	15.28
	60	30.6	20.07	5.78	25.85	14.29
	50	29.2	19.16	5.78	24.94	13.38
	40	27.4	17.97	5.78	23.75	12.19
	30	25.2	16.53	5.78	22.31	10.75
	25	23.8	15.61	5.78	21.39	9.83
	20	22.3	14.63	5.78	20.41	8.85
	15	20.5	13.45	5.78	19.23	7.67
Leeward	All	32.1	-13.16	5.78	-7.38	-18.94
Side	All	32.1	-18.42	5.78	-12.64	-24.20
Roof	70.5	32.1	-31.58	5.78	-25.80	-37.36
	70.5	32.1	-18.42	5.78	-12.64	-24.20
	70.5	32.1	-18.42	5.78	-12.64	-24.20

North South

Pressure	height	width	moment arm	Shear	overturning moment
13.2	70.5	180	35.25	167.51	5904.66
13.5	15	180	7.5	36.45	273.38
14.6	5	180	17.5	13.14	229.95
15.6	5	180	22.5	14.04	315.90
16.5	5	180	27.5	14.85	408.38
18	10	180	35	32.40	1134.00
19.2	10	180	45	34.56	1555.20
20.1	10	180	55	36.18	1989.90
21.1	10	180	65	37.98	2468.70
				387.11	14280.06


WIND PRESSURE DIAGRAM - N/S DIRECTION
PRESSURE - PSF
21.1
20.1
19.2
18.0
16.5
15.6
14.6
13.5
70'
60'
50'
40'
30'
25'
20'
15'
13.2 psf
V = 387.1k
M = 14,280.1'k
Story Force - K
43.2k
81.4k
75.1k
73.6k
74.2k

Appendix D: Seismic Loads

SEISMIC LOADS 1

Building Data
Location: Boston, Ma (Latitude 42.35; Longitude -71.1°)
Soil Classification: S_B Mass. State Bldg. Code, Site Class E Assumed as a conservative approximation.
Occupancy: III
Material: Structural Steel
Structural System
N-S: Ordinary Concentric Braced Frames
E-W: Dual System, Ordinary Moment Resisting frames with Ordinary Concentric Braced Frames.

Seismic Ground Motion Values
Mapped Accelerations: USGS Ground Motion Parameter Calculator — $S_s = 0.277$, $S_1 = 0.068$

Soil Modified Accelerations
Site Class E, $S_S = 0.277$ Table 11.4-1 interpolation — $F_a = 2.0$
Site Class E, $S_1 = 0.068 < 0.1$ Table 11.4-2 — $F_v = 3.5$
$S_{MS} = F_a S_S = 2.0(0.277) = 0.55$ — $S_{MS} = 0.55$
$S_{M1} = F_v S_1 = 3.5(0.068) = 0.24$ — $S_{M1} = 0.24$

Design Accelerations
$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.55) = 0.37$ — $S_{DS} = 0.37$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.24) = 0.16$ — $S_{D1} = 0.16$

Determine SDC
Check if $T_a < 0.8 T_s$
N-S Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec
E-W Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec — $T_a = 0.48$ sec
$T_s = S_{D1}/S_{DS} = 0.16/0.37 = 0.43$ — $T_s = 0.43$
T_a is not less than $0.8 T_s$ ∴ use 11.6-1 & 11.6-2
Table 11.6-1 SDS = C
Table 11.6-2 SDS = C — SDC = C

Determine Analytical Process
Check if $T < 3.5 T_s$
$3.5 T_s = 3.5(0.43) = 1.5 > T$

Determine if the Structure is Regular
Determine Response Modification Coefficient
E-W Direction (E1) No Limit — $R_{EW} = 6$
N-S Direction (B4) No Limit — $R_{N\text{-}S} = 3.25$
Importance Factor
$I = 1.25$ — $I = 1.25$
Long term period
$T_L = 6$ sec — $T_L = 6$ sec

SEISMIC LOADS 2

Seismic Response Coefficient

E-W Direction

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{(6/1.25)} = 0.077$$

$$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.16}{(0.48)(6/1.25)} = \underline{0.069}$$

$$\geq C_s = 0.01$$

E-W: $C_s = 0.069$

N-S Direction

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{3.25/1.25} = \underline{0.14}$$

$$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.37}{(0.48)(3.25/1.25)} = 0.28$$

$$\geq C_s = 0.01$$

N-S $C_s = 0.14$

Effective Seismic Weight

See Spread Sheet

$W = 7,820^k$

Base Shear

N-S Direction

$$V = C_s W = 0.14(7,820) = 1095^k$$

$V_{N\text{-}S} = 1095^k$

E-W Direction

$$V = C_s W = 0.069(7,820) = 540^k$$

$V_{EW} = 540^k$

Exponent for Structural Period

$$T_{EW} = T_{N\text{-}S} < 0.5 \quad \therefore k = 1.0$$

Seismic Forces in the North/South Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	258.18	258.18	17899.62
5	1832	56	102592.0	0.34	373.46	631.64	20913.53
4	1438	43	61834.0	0.21	225.09	856.72	9678.80
3	1449	30	43470.0	0.14	158.24	1014.96	4747.19
2	1404	15.66	21986.6	0.07	80.04	1095.00	1253.36
				Total:	1095.00		54492.51

Seismic Forces in the East/West Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	127.32	127.32	8827.21
5	1832	56	102592.0	0.34	184.17	311.49	10313.52
4	1438	43	61834.0	0.21	111.00	422.49	4773.11
3	1449	30	43470.0	0.14	78.04	500.53	2341.08
2	1404	15.66	21986.6	0.07	39.47	540.00	618.10
				Total:	540.00		26873.02

Appendix E: Spot Checks

Diagonal Member Spot Check

HSS 10x10 x 5/8 , $L = 21'$

Load Combination : $0.9D + E$

Factored Loads

Compression : 275^k

Tension : 258^k

Check Tension Yielding

$\phi P_n = \phi F_y A_g$

$\phi P_n = 0.9(50)(21.0)$

$\phi P_n = 945^k$

$\phi P_n > P_u = 258^k \therefore$ OK

Tension Rupture Not Considered At This Time.

Check Compression Capacity.

$r = 3.80$

$\frac{KL}{r} = \frac{(1.0)(21 \times 12)}{3.8} = 66.3$

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29,000}{50}} = 113$ $\quad 66.3 < 113 \therefore$ Inelastic

$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000)}{(66.3)^2} = 65.1$ ksi

$F_{cr} = \left[0.658^{(50/65.1)}\right](50) = 36.25$ ksi

$P_n = F_{cr} A_g = 36.25(21.0) = 761^k$

$\phi P_n = 0.9(761) = 685^k$

$\phi P_n > P_u = 275^k \therefore$ OK