151 First Side

Technical Assignment 3

December 19th, 2007

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Structural Option

AE 481w – Senior Thesis
The Pennsylvania State University

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Executive Summary

Report Summary:

The purpose of this third technical report is to analyze the current lateral force resistance system of 151 First Side in Pittsburgh, PA. 151 First Side achieves its lateral force resistance through a combination of ordinary concentric braced framing and moment connections. The building was originally designed to only use ordinary concentric braced framing, but due to a change in architectural plan the framing was altered to its current state. The parking levels rely solely on two sets of braced frames. Moment connections were used in many areas of the residential levels so that none of the rentable space would have a diagonal brace within it. This resulted in diagonal braces near the central core with three sets of moment connections in the N-S direction and two sets in the E-W direction.

Lateral loads are transferred from the façade to the framing and into the floor system. Since the Hambro floor system creates a rigid diaphragm, the loads are taken from the floor and applied to the lateral frames as both a moment at the moment connections and as an axial compression force at the braced frames. These loads are carried through the columns and distributed through the foundation to the surrounding soil.

A computer model was created using RAM Structural System to further analyze this system. While checking drift values it was found that the model was providing errant data. The sections which are missing from this report will be fully analyzed during the next phase of my Thesis and this report will be updated accordingly.

Structural System

Foundation:

The foundation was designed based on soil reports prepared by Engineering Mechanics, Inc. and Ackenheil Engineering, Inc., dated April, 2002 and July 1, 2005 respectively. Due to the close proximity of the Monongahela River pressure injected auger cast piles, 18” in diameter were used. Pile tips were placed at an elevation of 674’-0”, which gives an average length of 52’. Each pile has a capacity of 120 tons. Pile caps are made of concrete with a 28 day strength of f'_c = 3000psi.

Slab on Grade:

The sub-basement and basement floors consist of slab on grade at elevations 725’-0” and 728’-0” respectively. Slabs are made from 5” of concrete with a 28 day strength of f'_c = 4000psi and are reinforced with 6x6 w2.1 x w2.1 welded wire fabric. Concrete was placed above 4” of AASHTO 57 well graded compacted granular stone.

Structural Frame:

The structural framing is made of steel W shapes. Beams range from W10 to W16 with the most common size being a W14x61. The columns are W12 shapes with weights ranging from 40 to 336 pounds per linear foot. Common column splices occur at every second floor.

Floor and Roof System:

The parking levels on the first three stories as well as the terrace level have poured concrete floors. All parking floors are 4” of light weight concrete on a 2” 20ga. galvanized composite metal deck with the exception of some highly loaded areas of the ground floor in which there is a 6” slab. The 4” sections on the parking levels are reinforced with #4 rebar spaced at 12” in both the bottom and the top of the slab with the top bars continuing for ¼ of the span length past the supports. The 6” sections contain 6x6-W2.9xW2.9 welded wire fabric while the terrace level has 6x6-W1.4xW1.4 welded wire fabric for its reinforcement.

The residential and mechanical levels, as well as the roof, contain an MD200 composite floor joist system provided by Hambro. A typical floor plan can be found in figure 1. There is a 3¼" thick slab made from concrete with a 28 day strength of f'_c=4000psi. Reinforcing within the concrete is a 6x6-W2.9xW2.9 welded wire mesh. The concrete is supported by 22ga. 1½" galvanized steel deck. Joist depth is 16" unless otherwise noted. The top chord is an "S' shape piece of cold-rolled, ASTM A 1008, Grade 50, 13ga. steel which works as both a compressive member as well as a shear connector while the bottom chord is made of two steel angles. Both chords have a minimum F_y=50,000psi. The web is formed from 7/16" hot-rolled steel bars with an F_y=44,000psi. The roof is also topped with a waterproof membrane.

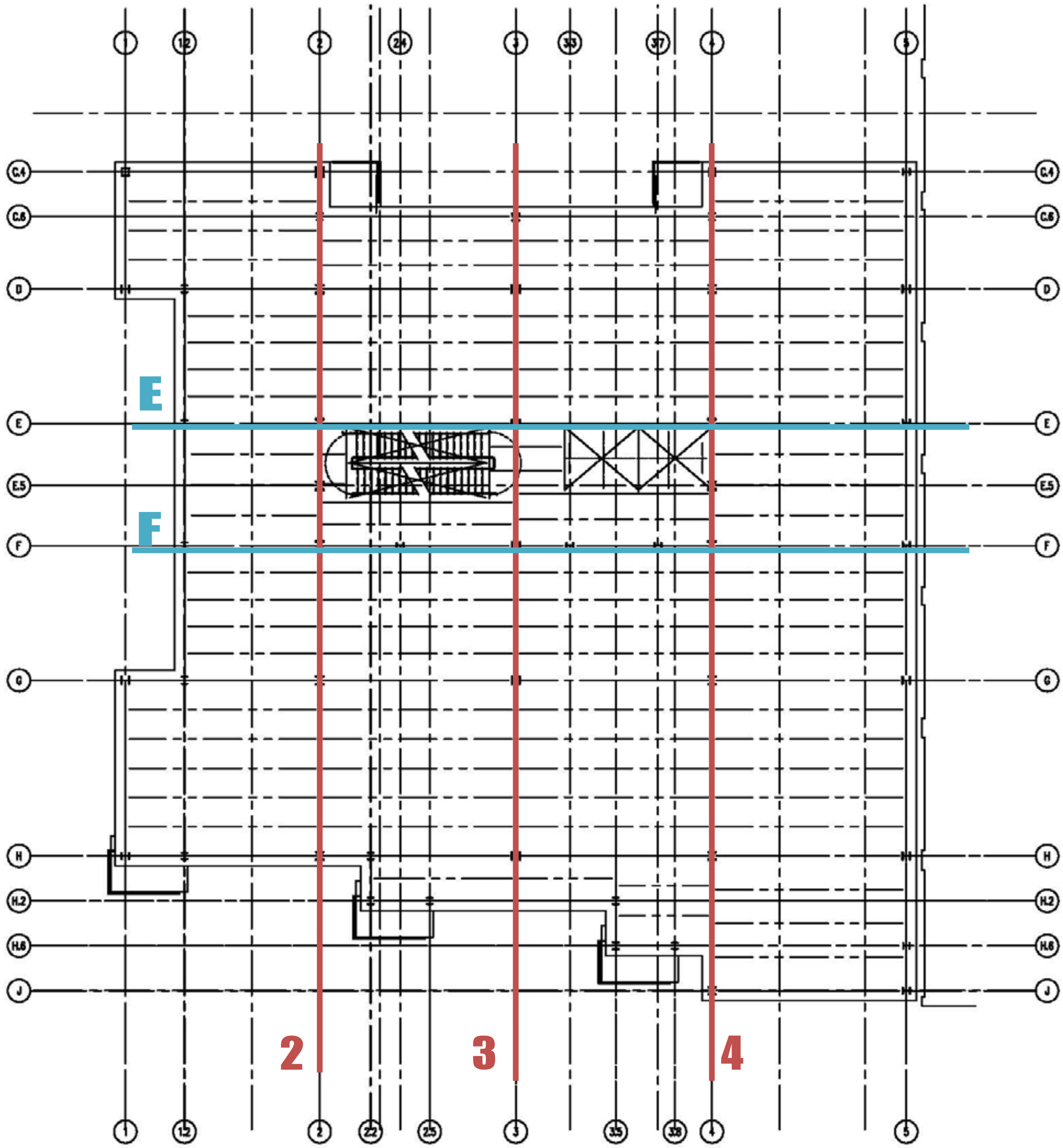


Figure 1

Lateral System:

The lateral system is composed of both braced frames as well as special moment frames. Lateral bracing is provided on column lines E and F (Figure 2) and column lines 2, 3, and 4 (Figure 3). Each of these column lines contain both moment connections and braced frames made of W12's or back to back channels.

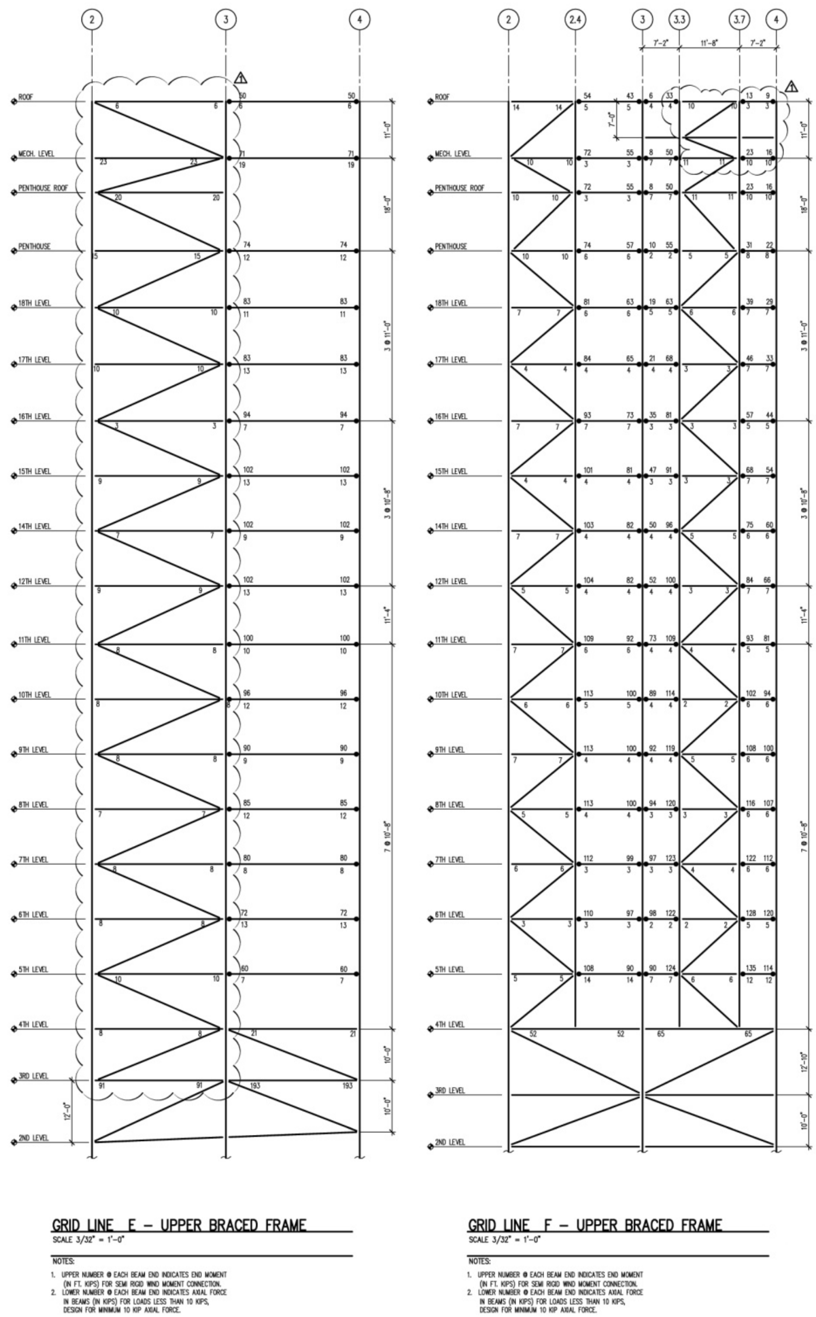


Figure 2

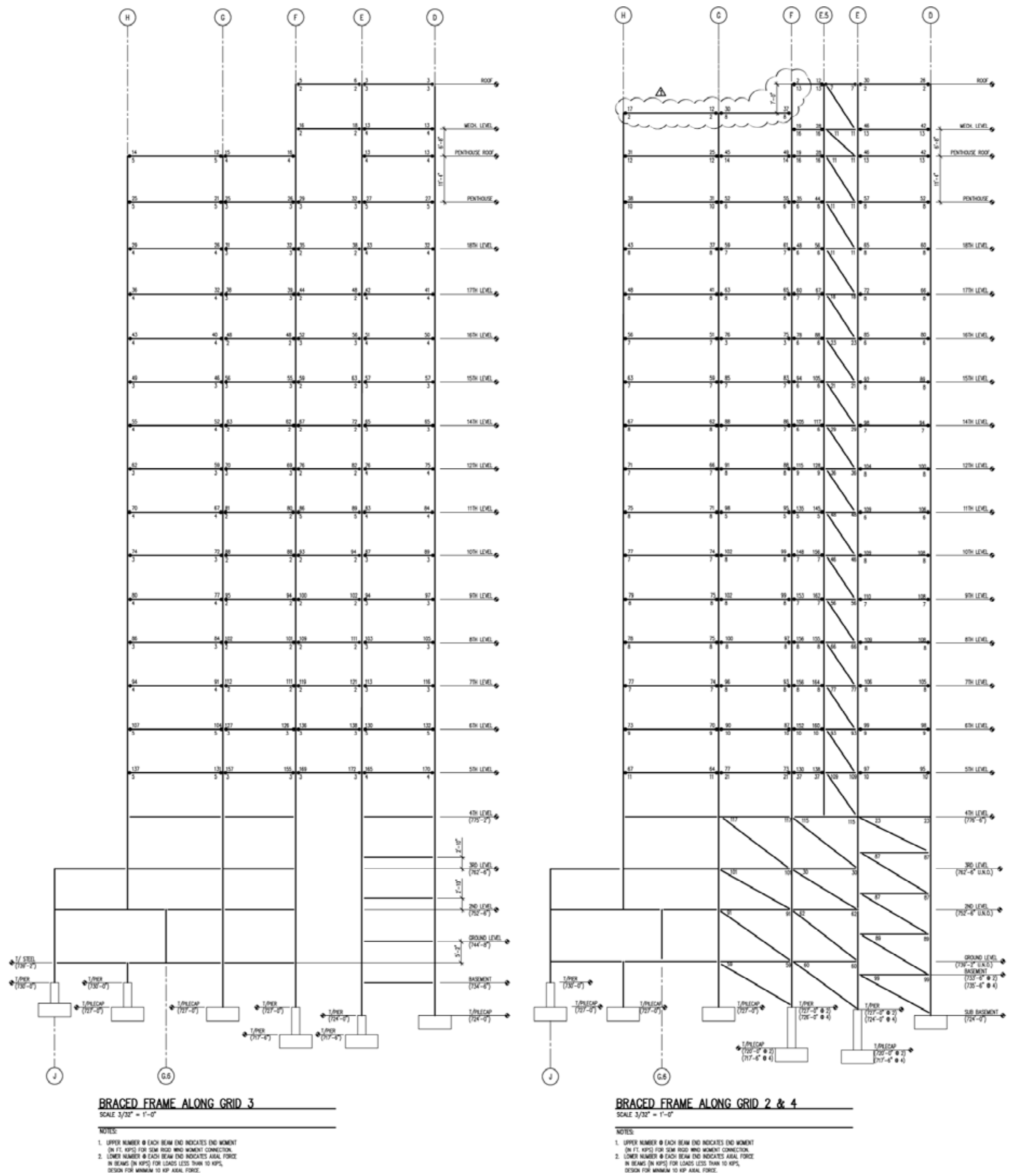

BRACED FRAME ALONG GRID 3
SCALE 3/32" = 1'-0"
NOTES:
1. UPPER NUMBER @ EACH BEAM END INDICATES END MOMENT (IN FT. KIPS) FOR SEMI RIGID WIND MOMENT CONNECTION.
2. LOWER NUMBER @ EACH BEAM END INDICATES AXIAL FORCE IN BEAMS (IN KIPS) FOR LOADS LESS THAN 10 KIPS, DESIGN FOR MINIMUM 10 KIP AXIAL FORCE.
BRACED FRAME ALONG GRID 2 & 4
SCALE 3/32" = 1'-0"
NOTES:
1. UPPER NUMBER @ EACH BEAM END INDICATES END MOMENT (IN FT. KIPS) FOR SEMI RIGID WIND MOMENT CONNECTION.
2. LOWER NUMBER @ EACH BEAM END INDICATES AXIAL FORCE IN BEAMS (IN KIPS) FOR LOADS LESS THAN 10 KIPS, DESIGN FOR MINIMUM 10 KIP AXIAL FORCE.

Figure 3

Codes

Building Code:

International Building Code (IBC), 2003 edition

Structural Concrete:

Building Code Requirements for Reinforced Concrete (ACI 318, latest edition)

Specifications for Structural Concrete (ACI 301, latest edition)

Steel Design:

Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC, 9th Edition)

Code of Standard Practice for Steel Buildings and Bridges (with exception of Section 4.2)

Building Design Loads:

ANSI/ASCE-7 2002

Design Loads

General Loads:

Floor Live Loads		
Load Area	**Design Load**	**Minimum Load (ASCE 7-05)**
Common Areas	100 psf	100 psf
Corridors	100 psf	100 psf
Parking	40 psf	40 psf
Residential	40 psf	40 psf
Mechanical	150 psf	n/a
Partition Allowance	20 psf where applicable	n/

Dead Loads	
Item	**Design Value**
Superimposed Dead Loads	
Mechanical , Electrical, Sprinkler	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf
Structure	Varies
Other Dead Loads	Where Applicable

Wind Loads:

The wind pressures and resulting base shear and overturning moment were calculated based on an exposure category B. The following spreadsheets give a detailed view of the pressure applied to each height level, and the corresponding floors. See the Appendix for my original calculations and diagrams regarding wind. Note that these values have not been compared to the original design values. This will be done when the original values have been obtained and will be included within the final report as part of the lateral system re-design.

Pressure				
Wind from the North/South				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.72	0-15	-9.43	16.15
20	7.31	20	-9.43	16.74
25	7.78	25	-9.43	17.21
30	8.25	30	-9.43	17.68
40	8.96	40	-9.43	18.39
50	9.55	50	-9.43	18.98
60	10.02	60	-9.43	19.45
70	10.49	70	-9.43	19.92
80	10.96	80	-9.43	20.39
90	11.32	90	-9.43	20.75
100	11.67	100	-9.43	21.10
120	12.26	120	-9.43	21.69
140	12.85	140	-9.43	22.28
160	13.32	160	-9.43	22.75
180	13.79	180	-9.43	23.22
200	14.15	200	-9.43	23.58
250	15.09	250	-9.43	24.52

Pressure				
Wind from the East/West				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.68	0-15	-9.26	15.94
20	7.26	20	-9.26	16.53
25	7.73	25	-9.26	16.99
30	8.20	30	-9.26	17.46
40	8.91	40	-9.26	18.17
50	9.49	50	-9.26	18.75
60	9.96	60	-9.26	19.22
70	10.43	70	-9.26	19.69
80	10.90	80	-9.26	20.16
90	11.25	90	-9.26	20.51
100	11.60	100	-9.26	20.86
120	12.19	120	-9.26	21.45
140	12.77	140	-9.26	22.03
160	13.24	160	-9.26	22.50
180	13.71	180	-9.26	22.97
200	14.06	200	-9.26	23.32
250	15.00	250	-9.26	24.26

Wind from the North/South							
Floor	Height (Ft.)	Story Height (Ft.)	Trib. Area (Sf.)	P-total (psf)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
1 (ground)	0	0	0	16.15	0.00	**473.61**	**556969.93**
2	13.33	13.33	1242.50	16.15	20.07	473.61	6314.85
3	23.33	10.00	1215.88	17.21	20.93	453.55	10582.79
4	192.83	12.83	1251.38	18.39	23.01	432.62	83424.05
5	180.00	10.67	1136.00	18.98	21.56	409.61	73729.99
6	169.33	10.67	1136.00	19.45	22.10	388.05	65710.08
7	158.67	10.67	1136.00	19.92	22.63	365.96	58065.11
8	148.00	10.67	1136.00	20.39	23.17	343.33	50812.23
9	137.33	10.67	1136.00	20.75	23.57	320.16	43968.57
10	126.67	10.67	1136.00	21.69	24.64	296.59	37568.25
11	116.00	10.67	1171.50	21.69	25.41	271.95	31546.44
12	105.33	11.33	1171.50	22.28	26.10	246.54	25969.16
14	94.00	10.67	1136.00	22.28	25.31	220.44	20721.62
15	83.33	10.67	1136.00	22.75	25.84	195.13	16261.16
16	72.67	10.67	1153.75	22.75	26.25	169.29	12301.69
17	62.00	11.00	1171.50	23.22	27.20	143.04	8868.53
18	51.00	11.00	1171.50	23.22	27.20	115.84	5907.65
Penthouse	40.00	11.00	1544.25	23.58	36.41	88.63	3545.26
Mech. Level	29.00	18.00	1544.25	24.52	37.86	52.22	1514.52
Roof	11.00	11.00	585.75	24.52	14.36	14.36	157.98

North/South Direction:

Base Shear: 473.61 Kip

Overturning Moment: 556969.93 Ft.-Kip

Wind from the East/West							
Floor	Height (Ft.)	Story Height (Ft.)	Trib. Area (Sf.)	P-total (psf)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
1 (ground)	0	0	0	15.94	0.00	**468.27**	**550854.54**
2	13.33	13.33	1242.50	15.94	19.81	468.27	6243.61
3	23.33	10.00	1215.88	16.99	20.66	448.47	10464.19
4	192.83	12.83	1251.38	18.17	22.73	427.80	82494.47
5	180.00	10.67	1136.00	18.75	21.30	405.07	72912.39
6	169.33	10.67	1136.00	19.22	21.84	383.77	64984.40
7	158.67	10.67	1136.00	19.69	22.37	361.93	57426.38
8	148.00	10.67	1136.00	20.16	22.90	339.56	50255.38
9	137.33	10.67	1136.00	20.51	23.30	316.66	43488.44
10	126.67	10.67	1136.00	21.45	24.36	293.36	37159.44
11	116.00	10.67	1171.50	21.45	25.13	269.00	31203.98
12	105.33	11.33	1171.50	22.03	25.81	243.87	25688.08
14	94.00	10.67	1136.00	22.03	25.03	218.06	20497.85
15	83.33	10.67	1136.00	22.50	25.56	193.03	16086.03
16	72.67	10.67	1153.75	22.50	25.96	167.47	12169.50
17	62.00	11.00	1171.50	22.97	26.91	141.51	8773.53
18	51.00	11.00	1171.50	22.97	26.91	114.60	5844.52
Penthouse	40.00	11.00	1544.25	23.32	36.02	87.69	3507.53
Mech. Level	29.00	18.00	1544.25	24.26	37.46	51.67	1498.52
Roof	11.00	11.00	585.75	24.26	14.21	14.21	156.31

East/West Direction:

Base Shear: 468.27 Kip

Overturning Moment: 550854.54 Ft.-Kip

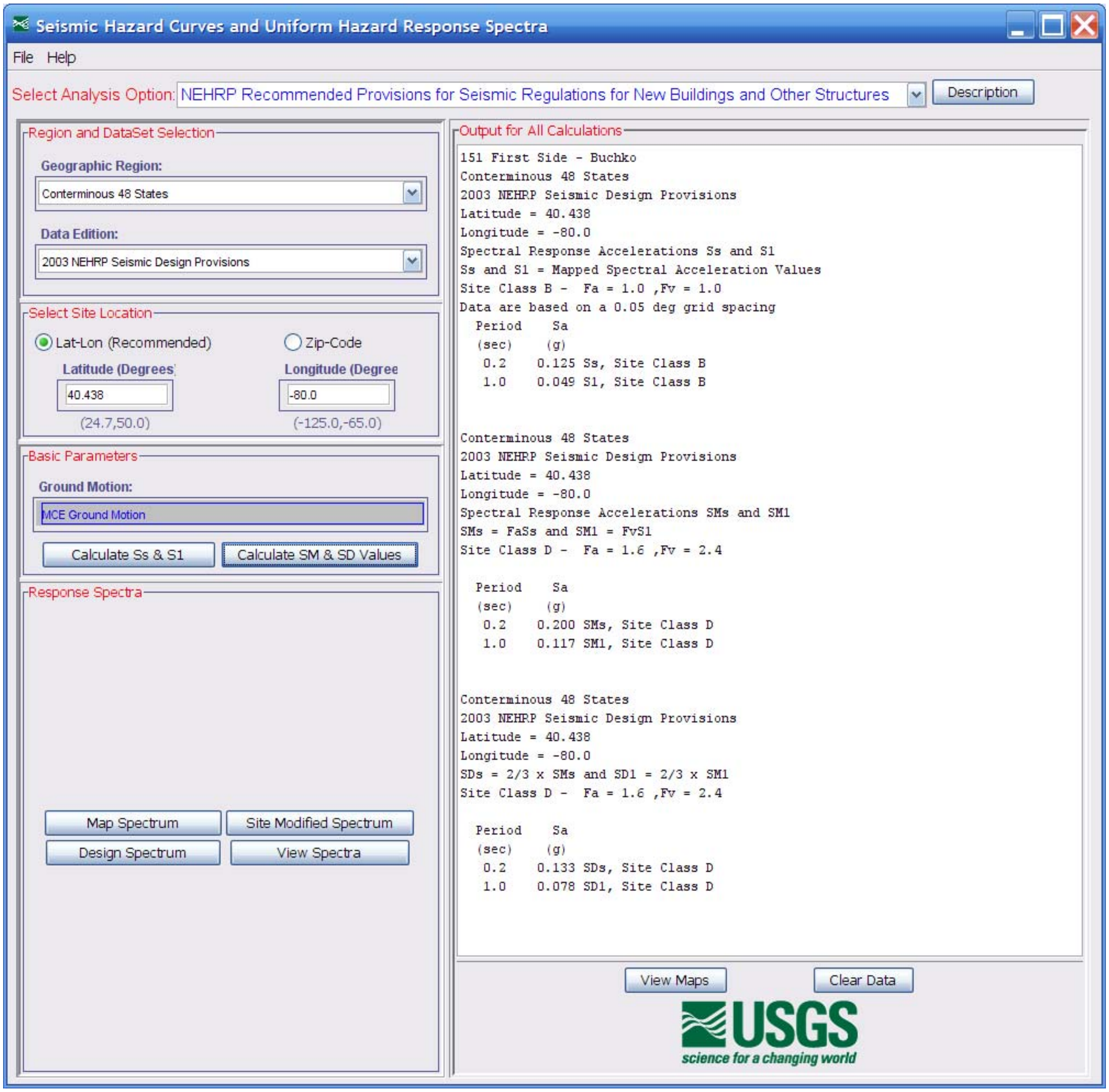
Seismic Loads:

Even though Pittsburgh is not known for its seismic activity, a simplified check has been performed to ensure that wind loading is indeed the controlling case. The building has been analyzed as a seismic design category B with ordinary concentric braced framing as its main seismic force resisting system. I have used software from the USGS website as an aid in calculating the required data. I have also preformed a vertical distribution of the seismic load. A sketch of the resultant loads can be found within the Appendix.

When I checked my value for the design base shear with that of the designer I noticed that mine was almost 1% off. When I investigated this further I found that the designer and I had started with different values for spectral response acceleration (S_1 and S_s). This can be accounted for based on the method of obtaining these values. I determined these values based on the output of the USGS software after inputting the longitude and latitude. It seems that the designer had used the then-current generic values for south eastern Pennsylvania. This discrepancy does not affect the overall design as both values are still less than the wind loads.

The following pages include a print out of the USGS website displaying the values that I have used for my analysis in addition to a spreadsheet showing the vertical distribution of the seismic load and final base shear.

Note that these values have not been compared to the original design values. This will be done when the original values have been obtained and will be included within the final report as part of the lateral system re-design.


Seismic Hazard Curves and Uniform Hazard Response Spectra
File Help
Select Analysis Option: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures
Description
Region and DataSet Selection
Geographic Region:
Conterminous 48 States
Data Edition:
2003 NEHRP Seismic Design Provisions
Select Site Location
Lat-Lon (Recommended)
Zip-Code
Latitude (Degrees
Longitude (Degree
40.438
-80.0
(24.7,50.0)
(-125.0,-65.0)
Basic Parameters
Ground Motion:
MCE Ground Motion
Calculate Ss & S1
Calculate SM & SD Values
Response Spectra
Map Spectrum
Site Modified Spectrum
Design Spectrum
View Spectra
Output for All Calculations
151 First Side - Buchko
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing
Period Sa
(sec) (g)
0.2 0.125 Ss, Site Class B
1.0 0.049 S1, Site Class B
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.200 SMs, Site Class D
1.0 0.117 SM1, Site Class D
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.133 SDs, Site Class D
1.0 0.078 SD1, Site Class D
View Maps
Clear Data
USGS
science for a changing world

Vertical Distribution of Seismic Load					
K=1.67 Vb=304.7					
Level	**wx (Kip)**	**hx (Ft.)**	**wxhx^1.67**	**Cvx**	**Fx (Kip)**
Roof	1304.04	216.17	10336846.93	0.1342	40.88
Mech. Level	1304.04	205.17	9473474.13	0.1230	37.47
Penthouse	1304.04	187.17	8126668.00	0.1055	32.14
18	1304.04	176.17	7344860.53	0.0953	29.05
17	1304.04	165.17	6595099.13	0.0856	26.08
16	1304.04	154.17	5878073.59	0.0763	23.25
15	1304.04	143.50	5214751.14	0.0677	20.62
14	1304.04	132.83	4583674.00	0.0595	18.13
12	1304.04	122.17	3985675.73	0.0517	15.76
11	1358.64	110.83	3529424.99	0.0458	13.96
10	1358.64	100.17	2980658.20	0.0387	11.79
9	1358.64	89.50	2469726.52	0.0321	9.77
8	1358.64	78.83	1998066.39	0.0259	7.90
7	1358.64	68.17	1567363.51	0.0203	6.20
6	1358.64	57.50	1179640.56	0.0153	4.67
5	1358.64	46.83	837396.93	0.0109	3.31
4	1358.64	36.17	543850.54	0.0071	2.15
3	1473.20	23.33	283650.10	0.0037	1.12
2	1473.20	13.33	111406.21	0.0014	0.44
1 (ground)	1473.20	0.00	0.00	0.0000	0.00
Totals	27025.08			1.00	304.70

Seismic Loading:

Base Shear: 304.7 Kip

Lateral Force Distribution

151 First Side achieves its lateral force resistance through a combination of ordinary concentric braced framing and moment connections. The building was originally designed to only use ordinary concentric braced framing, but due to a change in architectural plan the framing was altered to its current state. The parking levels rely solely on two sets of braced frames. Moment connections were used in many areas of the residential levels so that none of the rentable space would have a diagonal brace within it. This resulted in diagonal braces near the central core with three sets of moment connections in the N-S direction and two sets in the E-W direction.

Lateral loads are transferred from the façade to the framing and into the floor system. Since the Hambro floor system creates a rigid diaphragm, the loads are taken from the floor and applied to the lateral frames as both a moment at the moment connections and as an axial compression force at the braced frames. These loads are carried through the columns and distributed through the foundation to the surrounding soil.

Due to the somewhat complex nature of this dual system, a RAM Structural System model was created to further analyze the distribution of lateral forces and the effects they have on the building. The original design documents were converted into a 3d computer model which could be analyzed using RAM Frame. Unfortunately, as discussed more thoroughly in the next section, some issues have surfaced which will require a deeper look at the computer model as well as the program used to create it.

Drift

In the design of many of the larger buildings lateral drift is a major design criterion. For the comfort of the inhabitants, drift should be limited to a value of L/400, with L being the building height. For 151 First Side this would allow for approximately 7.5" of lateral drift.

It was during the analysis of story drift and overall building drift that I found what I believe to be an error with my computer model. RAM has calculated a 4" drift due to self weight and superimposed dead load. When I looked at the LRFD load combinations, I found a maximum drift of 38". Although I have not yet obtained the values calculated by the design engineer, I feel that they are much less than those calculated from my model. This has led me to review both the inputted values for my model as well as the software itself.

Upon further investigation, I have determined that the transfer girders are deflecting more than they should in the model (See Figure 4 on the next page). This deflection causes a vertical and lateral displacement of the columns above. I believe that it is this displacement that is causing the errant drift values. I have checked both the sizes of the transfer girders as well as the loading and have yet to find any wrong values. I have also checked many of the settings within RAM but still have not found any incorrect entries.

Although I have not found the precise cause of the discrepancy between my model and what I feel is an acceptable answer, I have learned of an issue which may be contributing to it. I have found that RAM will not always consider composite action on beams that are designated as "Frame Beams." Without composite action, it is possible that the transfer girders are unable to support an acceptable amount of the loading without significant deflections. It is also important to note that this deflection is a compounding issue. The more deflection that there is in the transfer girder, the greater the axial load from above will become. This will consequently result in a greater deflection in the transfer girder. Because of this "snowball effect" it is difficult to gage how far past the critical deflection value the transfer girder actually is.

I plan on doing a redesign of the lateral framing system for my final report. Because of this I will be performing a more detailed analysis of the current system and will be testing other 3d modeling programs. I will update this technical report as more suitable values are calculated.

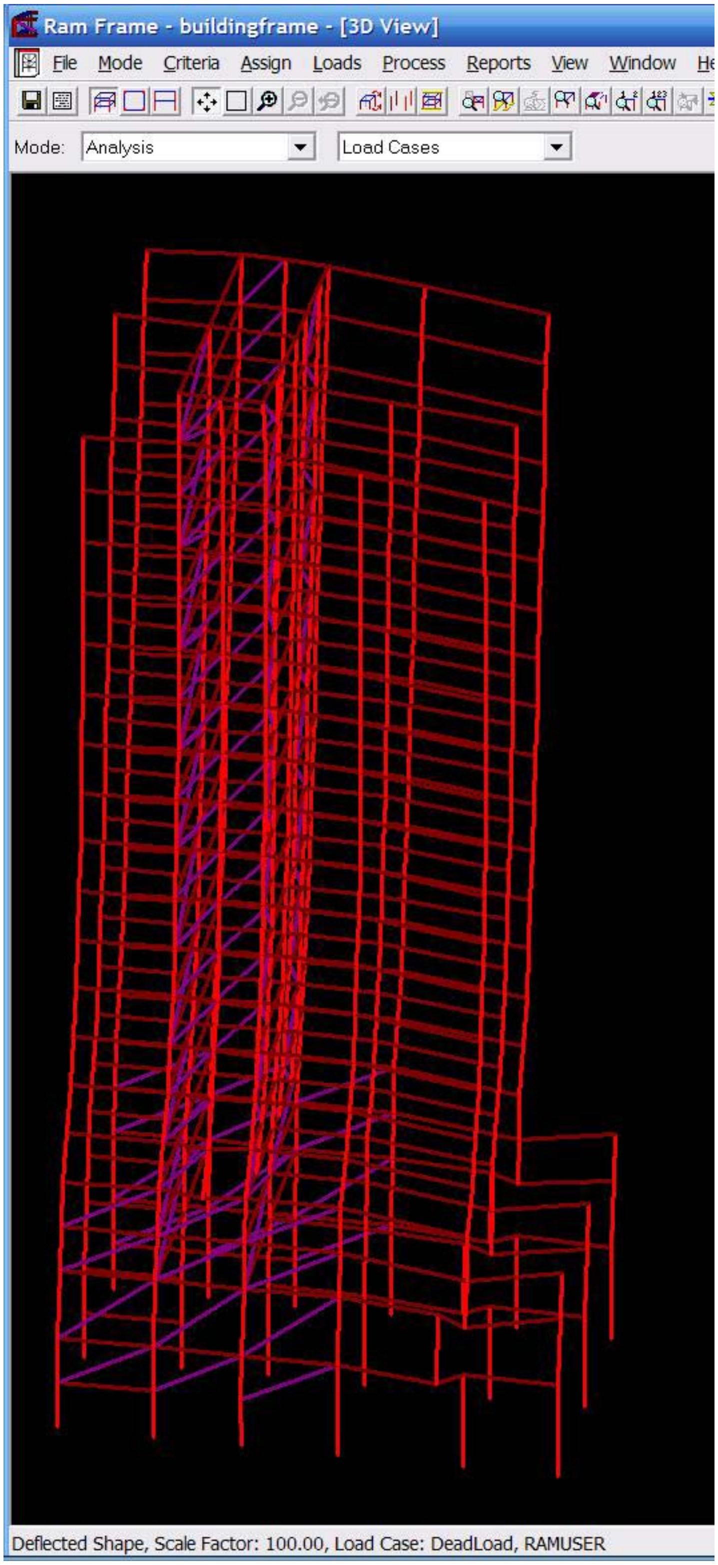

Ram Frame - buildingframe - [3D View]
File Mode Criteria Assign Loads Process Reports View Window
Mode: Analysis
Load Cases
Deflected Shape, Scale Factor: 100.00, Load Case: DeadLoad, RAMUSER

Figure 4

Torsion

Due to the relative symmetry and centralized location of the lateral frames, I do not feel that torsion will be a controlling factor. Due to the apparent errors in my computer model, I have postponed my torsion analysis. After the loads and model can be verified, I will check the torsion values. I will update this technical report after this information has been calculated.

Conclusions

Although I was unable to complete a full lateral stability and serviceability check, I have gained much knowledge of 151 First Side as well as structural computer programs.

- The lateral resistance for 151 First Side is provided through a combination of ordinary concentric braced frames as well as moment connections.

- Drift will control the lateral serviceability and wind will control the necessary lateral strength due to its location in Pittsburgh, PA and its height to mass ratio.

- Torsion will most likely not be a large factor in the design of 151 First Side. A more thorough check will be performed.

- While structural software can be a useful aid, it is necessary to know and understand the theory behind the software as well as its assumptions and limitations.

Appendix

151 First Side | Wind Loads pg 1/3 | William Buchko

Design Wind Speed $V = 90$ mph
Wind Importance factor $I = 1.0$
Wind Exposure Category B
Building Category II
$GCP_i = \pm 0.18$

Exposure Coefficients K_h and K_z

height	Components cladding	MWFRS
0-15'	0.7	0.57
20'	0.7	0.62
25'	0.7	0.66
30'	0.7	0.70
40'	0.76	0.76
50'	0.81	0.81
60'	0.85	0.85
70'	0.89	0.89
80'	0.93	0.93
90'	0.96	0.96
100'	0.99	0.99
120'	1.04	1.04
140'	1.09	1.09
160'	1.13	1.13
180'	1.17	1.17
200'	1.20	1.20
250'	1.28	1.28

$K_{zt} = 1.0$ Topographical factor
$K_d = 0.85$ wind direction factor

$T = 0.1(18) = 1.8$
$n = \frac{1}{T} = 0.55$ ∴ assume flexible structure

$$G_f = 0.925\left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z}\right)$$

$g_Q = 3.4$
$g_v = 3.4$
$g_R = \sqrt{2\ln(3600(1.8))} + \frac{0.577}{\sqrt{2\ln(3600(1.8))}}$
$g_R = 4.33$

$L_z = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
$= 320\left(\frac{128}{33}\right)^{1/3.0}$

$L_z = 502.78$

$I_z = c\left(\frac{33}{128}\right)^{1/6}$
$I_z = 0.239$

$\alpha = 7.0$
$z_g = 1200$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$l = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min} = 30$
$\bar{z} = .6(213.33) = 128'$

$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$
$\bar{V}_z = 0.45\left(\frac{128}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right)$
$\bar{V}_z = 83.36$

$n_h = 4.6 n_1 h / \bar{V}_z = 4.6(0.55)(213.33)/83.36 = 6.47$
$n_B = 4.6 n_1 B / \bar{V}_z = 4.6(0.55)\,^{106.5}_{138.33}/83.36 = 3.23$ N-S, 4.2 E-W
$n_L = 15.4 n_1 L / \bar{V}_z = 15.4(0.55)\,^{138.33}_{106.5}/83.36 = 14.06$ N-S, 10.82 E-W
$N_1 = n_1 L_z / \bar{V}_z = 0.55(502.78)/83.36 = 3.32$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.32)}{(1 + 10.3(3.32))^{5/3}} = 0.066$

$R_h = \frac{1}{6.47} - \frac{1}{(6.47)^2 2}\left(1 - e^{-2(6.47)}\right) = 0.143$

$R_B = \frac{1}{\,^{3.23}_{4.2}} - \frac{1}{2(\,^{3.23}_{4.2})^2}\left(1 - e^{-2(\,^{3.23}_{4.2})}\right) = 0.262$ N-S, 0.21 E-W

$R_L = \frac{1}{\,^{14.06}_{10.82}} - \frac{1}{2(\,^{14.06}_{10.82})^2}\left(1 - e^{-2(\,^{14.06}_{10.82})}\right) = 0.069$ N-S, 0.088 E-W

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.103$ N-S, 0.0967 E-W

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = 0.824 \text{ N-S},\ 0.816 \text{ E-W}$$

$$G_f = 0.925\left(\frac{1+1.7(0.239)\sqrt{3.4^2(Q^2)+4.33^2(R^2)}}{1+1.7(3.4)(0.239)}\right) \begin{array}{l} = 0.836 \text{ N-S} \\ = 0.831 \text{ E-W} \end{array}$$

Building is Enclosed
$GC_{pi} = \pm 0.18$

C_p windward = 0.8, use with q_z
C_p leeward = -0.5 N-S use with q_h
-0.494 E-W from interpolation

$q_z = 0.00256\, k_z (1.0)(0.85)(90^2)(1.0) = 17.6256(k_z)$

See spreadsheet for results

$q_h = 0.00256(1.28)(1.0)(0.85)(90^2)(1.0) = 22.56$

$p = qGC_p$ ignore internal pressure

for N-S use q from spreadsheet, $G_f = 0.836$, $C_{pww} = 0.8$, $C_{pLW} = -0.5$
for E-W use q from spreadsheet, $G_f = 0.831$, $C_{pww} = 0.8$, $C_{pLW} = -0.494$

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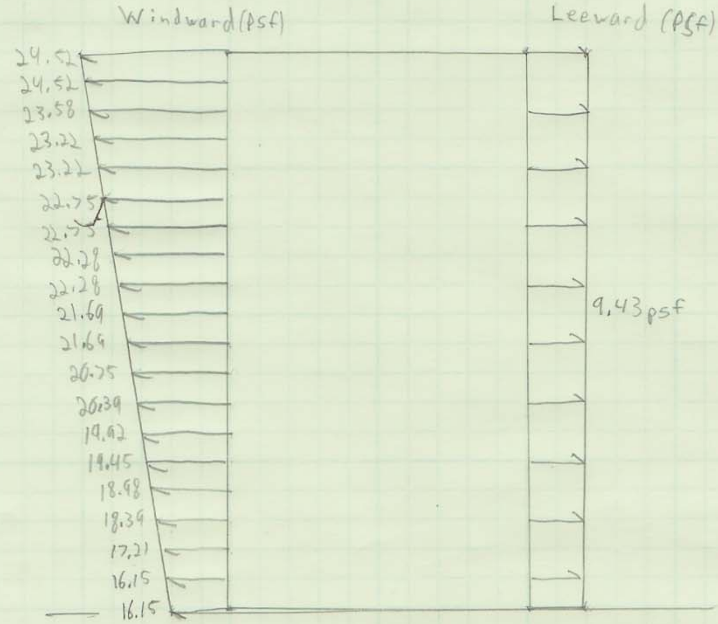
Pressure Diagram: Wind from N-S



Note: windward distribution is not linear

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Seismic Site Class ; D

Seismic Weight:
total Dead Load
25% Live Load for Storage
include Partition Load (20 psf)
Equipment Operating Weight
20% flat roof snow Load (if $P_f > 30$ psf

Using Software from USGS Website for Lat = 40.438 and Long = -80.0
(note that values are constant for all of zip code 15222

$S_S = 0.125$
$S_1 = 0.049$

From 11.4 (ASCE 7-05)
$F_a = 1.6$
$F_v = 2.4$

Note: Due to the complex combination of moment and braced frames, I will use the value of R provided by the designer which is Based off of ASCE 7-02.

From USGS Software
$S_{MS} = 0.200$
$S_{M1} = 0.117$
$S_{DS} = 0.133$
$S_{D1} = 0.078$

$I = 1.0$ (11.5-1)
$R = 5.5$ (Construction Documents)
SDC = B

C_t and x based on braced steel frame as per construction documents
$C_t = 0.03$
$x = 0.75$
$T_a = 0.03(213.33)^{.75}$
$T_a = 1.67s$

$C_u = 1.4$
$T = C_u T_a = 2.34s$
$T_L = 12$

$$C_s \geq S_{DS}/(R/I) = 0.133/(5.5/1) = 0.0242$$
$$\geq S_{D1}/(T(R/I)) = 0.076/(2.34(\tfrac{5.5}{1})) = 0.0061$$
$$\geq \frac{S_{D1}T_L}{T^2(R/I)} = \frac{0.078(12)}{(2.34)^2(\frac{5.5}{1})} = 0.031$$

$\underline{C_s = 0.01 \text{ min.}}$

Weight: Dead load + partition = 100 psf + 20 psf = 120 psf for Residential Levels
Dead load = 100 psf for parking Levels

$W = 120[9(10887) + 8(11322)] + 100[3(14732)] = 27025080$ lbs
$V_B = C_s W = 0.01(27025080) = \boxed{270.37k}$

Note: Designer used different C_s (0.019) assumed to be based on older values of Ss and S_1

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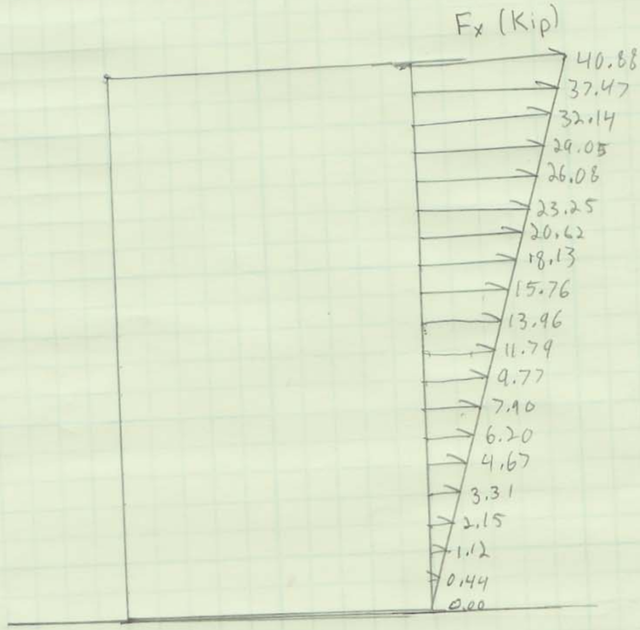
Vertical distribution of Seismic forces

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\Sigma w h^k}$

$k = 1.67$ by interpolation



Note: Fx distribution is not Linear