Science Center Research Park
3711 Market St.
Philadelphia, PA

The Pennsylvania State University Department of Architectural Engineering Senior Thesis 2009-2010

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[TECHNICAL REPORT 3]

Lateral System Analysis

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EXECUTIVE SUMMARY

The Science Center Research Park is a 401,032 GSF mixed-use building and is approximately 144 feet tall. It currently has the largest green roof in the city of Philadelphia. The building includes offices, wet labs, retail space, and a 500 car parking garage. The structure is made up of steel construction, and composite deck. Lateral support is provided by steel braced frames using HSS steel shapes for cross-bracing. The ground floor is a reinforced slab on grade with grade beams, and drilled caissons that support the buildings columns.

Technical report 3 is a report is an analysis of the existing lateral system of the Science Center Research Parks building. The purpose is to gain an understanding of how lateral loads are distributed, and to verify that a load path exists. This report uses current standards and the governing standards to analyze the existing Lateral system. It includes modeling using ETABS, and checks on strength, drifts and overturning moments.

Topics covered in technical report 3, but not limited to:

- *Gravity and Lateral Loads*
- *Load Paths*
- *Computer Analysis*
- *Drift*
- *Overturning Issues*
- *Lateral Spot Checks*

In conclusion, the controlling load combination was found to be: 1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)

The controlling wind load case was found to be load case 1 which includes 100% of the North-South or East-West wind loads. Also, all the drifts do to lateral forces were found to be acceptable and were less than the limitations for wind and seismic drifts found in ASCE 7-05.

The overturning moments were found to be offset by the moment cause by the building self weight. The critical shear forces were used in ETABS to calculate the overturning moments for the building. All the member spot checks were found to be acceptable. Lateral member spot checks were done and it was found that the design of the members was acceptable.

At the end of this report is an appendix that contains all the calculations for the loads stated above.

INTRODUCTION

The Science Center Research Park is an addition to the growing research/science development in the University City area. "The Science Center is the nation's preeminent destination for early-stage life science companies across the globe", said Pradip Banerjee. The building includes offices, wet labs, retail space, and a 500 car parking garage. It is covered by glass curtain wall, stone, and a brick veneer along the Market Street facade.

Technical Report 3 analyzes the existing lateral system for the Science Center Research Park building in order to gain a better understanding of how wind and seismic loads are distributed. Conclusion will be made on the validity of structural members designed for the lateral system.

CODE

CODE / REFERENCES

- ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*
- IBC 2006 *International Building Code*
- ACI 318-08 *Building Code Requirements for Structural Concrete*
- AISC 13th Edition Steel Construction Manual

Note: The following codes and references were used in the original design and in this report. All references are up-to-date building design standards.

DRIFT CRITERIA

Allowable Building Drift = H/400

Inner-Story Drift

Wind = h/400 to h/600

Seismic = 0.015h

LOAD COMBINATIONS

The following load combinations were used in the combination of factored gravity and lateral loads. These combinations were used for the 3D model analysis done using ETABS. The four different wind load cases stated below were also used when considering these load combinations.

1. 1.4(Dead)
2. 1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
3. 1.2(Dead) + 1.6(Roof Live) + (1.0(Live) + 0.8(Wind))
4. 1.2(Dead) + 1.6(Wind) 1.0(Live) + 0.5(Roof Live)
5. 1.2(Dead) + 1.0(Seismic) +1.6(Wind)
6. 0.9(Dead) + 1.6(1.6(Wind)
7. 0.9(Dead) +1.0(Seismic)

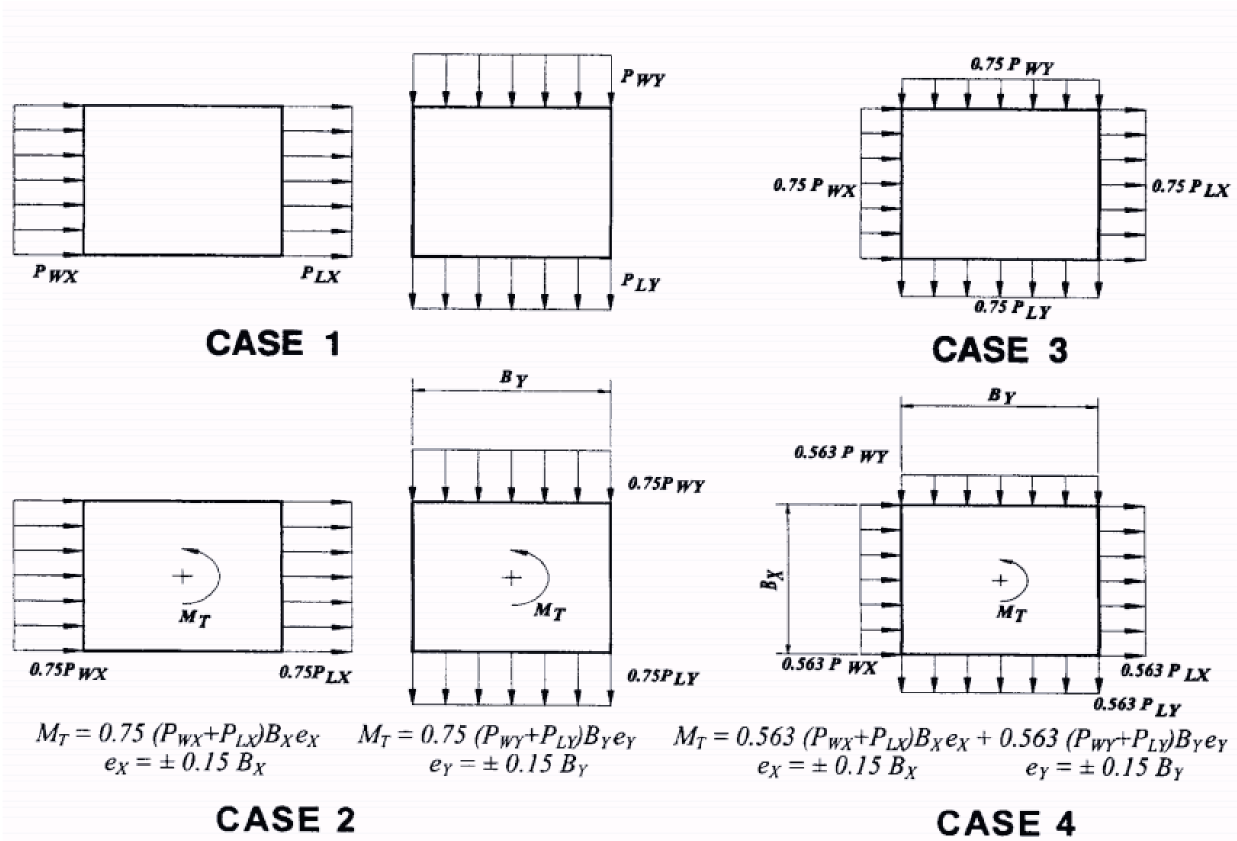
Wind Load Cases

<u>Case 1</u>: 100 % of the wind forces in the north-south or east-west direction

<u>Case 2</u>: 75 % of the north-south or east-west direction applied with torsion

<u>Case 3</u>: 75 % of the north-south and east-west direction applied simultaneously

<u>Case 3</u>: 56.3 % of the north-south and east-west direction applied simultaneously with torsion



Note: The above criteria were taken from ASCE 7-05.

MATERIAL

CONCRETE

Slabs on grade	f'_c = 4000 psi
Slab on steel deck	f'_c = 4000 psi
Drilled caissons	f'_c = 3000 psi
Foundation walls, piers &grade beams	f'_c = 4000 psi
Steel column encasement	f'_c = 3000 psi

STRUCTURAL STEEL

W – Shapes	ASTM A992
Bars, rods and plates	ASTM A36 (UNO)
All other structural shapes	ASTM A36
Pipes	ASTM A53, Grade B
Cold-formed hollow structural sections (tubing)	ASTM A500, Grade B
High strength bolts	ASTM A325
Deformed bar anchors	ASTM A706 Low Carbon
Anchor rods	ASTM A36
Shear connectors (headed)	ASTM A108, Grade 1010 to 1020

GRAVITY AND DESIGN LOADS

DEAD LOADS

Concrete	150 pcf
Light Weight Concrete	115 pcf
Partitions	20 psf
M.E.P.	5 psf
Finishes and Misc.	3 psf
Roof Deck	2.6 psf
Rigid Insulation	4 psf

LIVE LOADS

Corridors, Lobbies & Exits	100 psf
Labs / Offices	100 psf
Garage	40 psf
Mechanical Equip. Rooms	150 psf
Roof	30 psf

Note: The above loads were used in the original design and in this report.

SNOW LOADS

Snow load were calculated to determine whether the roof design is sufficient to carry the applied snow load. Table 1 contains all the design values required to calculate the flat roof snow load. When the applied snow load was compared to the existing designed snow load it was noticeable that designers had used a larger value. All the design criteria is the same used by the designers, but the designers used a more conservative value for the snow load. Snow drift was not calculated, but it vary depending on the different roof levels of the building.

Flat Roof	
p_g	30 psf
c_e	1.0 (Terrain Category B)
c_t	1
I	1.0 (Category II)
p_f	21 psf
	using $pf = 0.7c_ec_tIp_g$ (Eq. 7-1)

TABLE 1 – Snow load design criteria

LATERAL LOADS

WIND LOADS

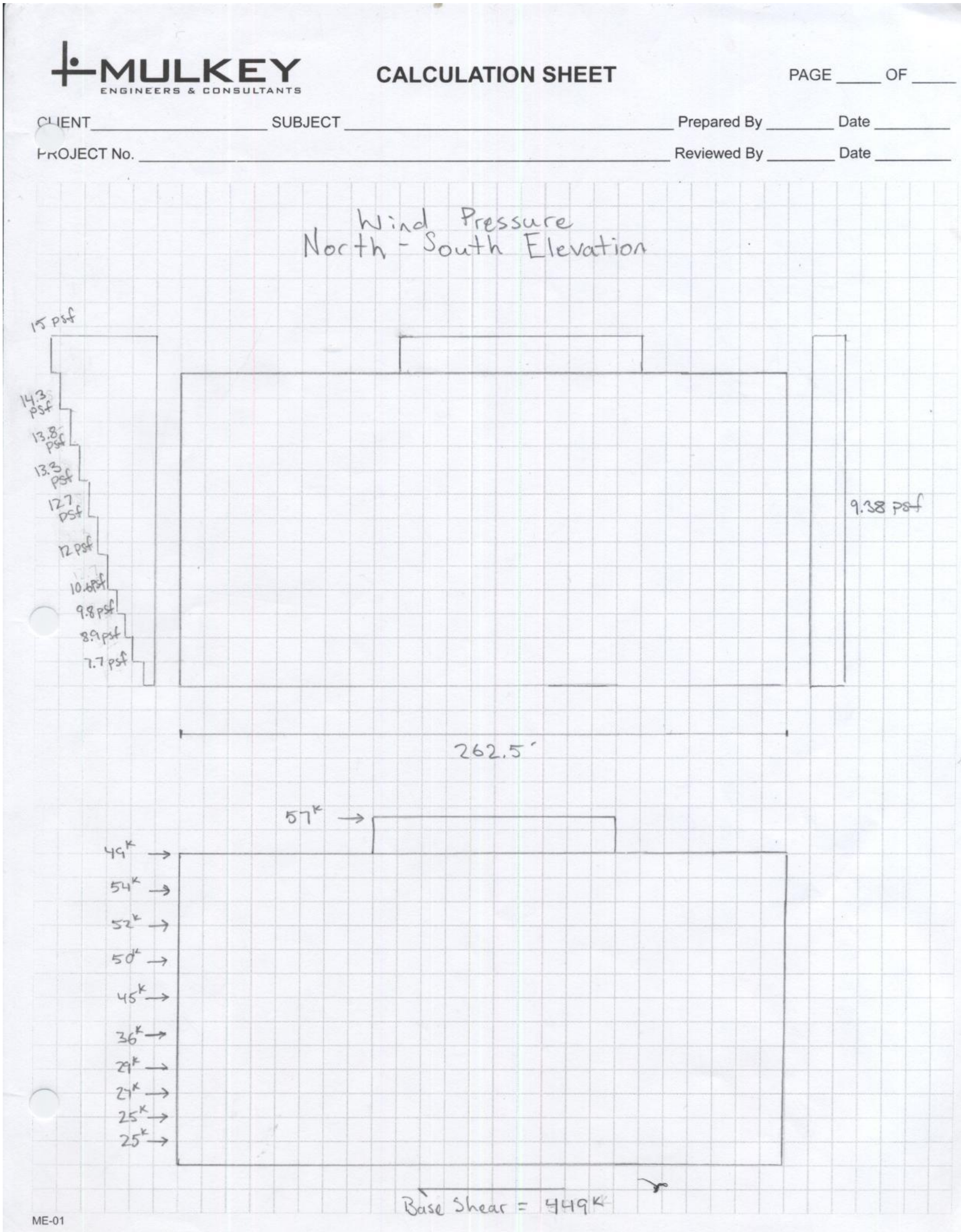
Wind loads were determined using ASCE 7-05 Section 6.5 which describes Method 2. The detailed analysis of the wind loads can be found in appendix B. Below are tables including wind factors and wind loads calculated for north-south and east-west elevations.

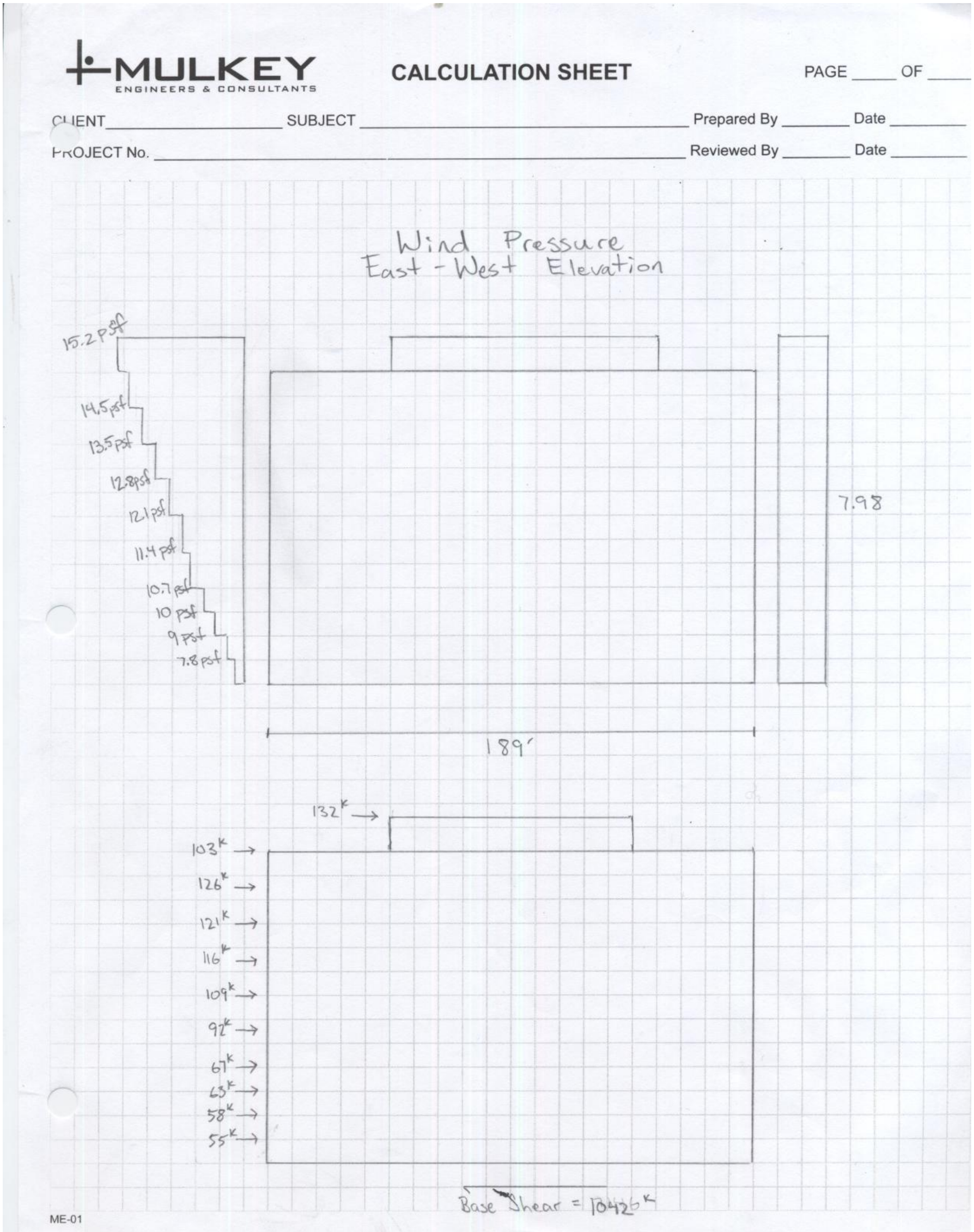
	Level	Height Above Ground (ft)	Kz	qz	Wind Pressures	
					N-S (psf)	E-W (psf)
windward	Pent House	147.5	1.10	21.41	15.00	15.19
	Roof Level	140.17	1.09	21.10	14.79	14.97
	T.O. Parapet	131.17	1.07	20.70	14.51	14.69
	10	125.5	1.05	20.44	14.33	14.51
	9	110.83	1.02	19.73	13.83	14.00
	8	96.17	0.98	18.95	13.28	13.45
	7	81.5	0.93	18.07	12.67	12.82
	6	66.83	0.88	17.08	11.97	12.12
	5	53.5	0.83	16.02	11.23	11.37
	4	43.5	0.78	15.10	10.59	10.72
	3	33.5	0.72	14.02	9.82	9.95
	2	23.5	0.65	12.67	8.88	8.99
	1	13.5	0.57	11.05	7.74	7.84
	Leeward	All	1.1	21.41	-9.38	-7.98

TABLE 2 – Wind Pressure at each level

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces			
					Load (kips)		Shear (kips)	
					N-S	E-W	N-S	E-W
Pent House	147.5	0						
Roof Level	140.17	7.33	7.33	7.33	57	132	57	132
T.O. Parapet	131.17	9						
10	125.5	5.67	5.67	7.33	49	103	107	236
9	110.83	14.67	7.33	7.33	54	126	161	361
8	96.17	14.67	7.33	7.33	52	121	213	483
7	81.5	14.67	7.33	7.33	50	116	263	599
6	66.83	14.67	7.33	6.67	45	109	308	708
5	53.5	13.33	6.67	5	36	92	344	800
4	43.5	10	5	5	29	67	372	867
3	33.5	10	5	5	27	63	399	930
2	23.5	10	5	5	25	58	424	988
1	13.5	10	5	6.75	25	55	449	1043
Total	147.5				449	1024	449	1024

TABLE 3 – Wind Loads: Shear and Moment at each level


MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE ___ OF ___
CLIENT
SUBJECT
Prepared By
Date
PROJECT No.
Reviewed By
Date
Wind Pressure
North - South Elevation
15 psf
14.3 psf
13.8 psf
13.3 psf
12.7 psf
12 psf
10.6 psf
9.8 psf
8.9 psf
7.7 psf
9.38 psf
262.5'
57k
49k
54k
52k
50k
45k
36k
29k
27k
25k
25k
Base Shear = 449k
ME-01


MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE ____ OF ____
CLIENT ____ SUBJECT ____ Prepared By ____ Date ____
PROJECT No. ____ Reviewed By ____ Date ____
Wind Pressure
East - West Elevation
15.2 psf
14.5 psf
13.5 psf
12.8 psf
12.1 psf
11.4 psf
10.7 psf
10 psf
9 psf
7.8 psf
7.98
189'
132k
103k
126k
121k
116k
109k
92k
67k
63k
58k
55k
Base Shear = 1042k
ME-01

SEISMIC LOADS

Seismic loads were determined using ASCE 7-05 chapters 11 and 12. The detailed analysis of the seismic loads can be found in appendix C. Building weight was calculated for each floor of the typical steel constructed building. The building weight includes the dead loads that are listed in the tables below.

Floor	Floor Area (ft^2)	Approx. Floor Dead Load (t_{slab}*150 pcf)	Floor Weight (lbs)	h/2 above (ft)	h/2 below (ft)	Column weight/length total (plf)	Column weight= height* weight/length (lbs)
Ground						11445	
1st	33,833	93.75	3171843	5	6.75	11498	134743.75
2nd	50,705	93.75	4753593	5	5	11566	115320.00
3rd	50,705	93.75	4753593	5	5	7385	94755.00
4th	50,705	93.75	4753593	5	5	7385	73850.00
5th	40,433	93.75	3790593	6.67	5	7205	84958.33
6th	34,439	93.75	3228656	7.33	6.67	4797	83211.33
7th	34,439	93.75	3228656	7.33	7.33	4797	70356.00
8th	30,439	93.75	2853656	7.33	7.33	2960	56884.67
9th	30,439	93.75	2853656	7.33	7.33	2960	43413.33
Pent house	6,437	93.75	603468	7.33	7.33	728	27045.33
Roof pent house level	21,509	93.75	2016468		7.33	2960	21706.67
Roof	6,437	93.75	603468		7.33	728	5338.67
Total			36007781				806244

TABLE 4 – Building Weight Tabulation

Floor	Approx. Beam weight (lbs)	Curtainwall (estimated length along perimeter) (ft)	Curtainwall height (ft)	Curtainwall weight (height*length* 15 psf)	Braced frame weight (lbs)
Ground					
1st	257591.00	913.5	10	137025	11064
2nd	257591.00	913.5	10	137025	7772
3rd	257591.00	913.5	10	137025	7239
4th	257591.00	913.5	10	137025	8639
5th	240266.00	913.5	13.33	182654.325	8639
6th	178765.00	913.5	14.67	201015.675	7772
7th	141120	850.5	14.67	187152.525	7041
8th	141120	850.5	14.67	187152.525	6773
9th	141120	819	14.67	180220.95	6439
Pent house	30240.00	378	14.67	83178.9	13207
Roof pent house level	7180.00				
Roof	92547.00				
Total	1910175			1569474.9	84585
Total Building Weight		40378.26 kips			

TABLE 4 – Building Weight Tabulation (continued)

Building forces including story and base shears were calculated after the tabulation of building weights. These forces are shown below in table 5.

Level	Story Weight w_x (Kips)	Height h_x (ft)	$w_x h_x^k$	Lateral Force F_x (Kips)	Story Shear V_x (Kips)	Moment M_x (ft-k)
1st	3712.27	13.5	50115.61125	6.05	286.42	81.68
2nd	5271.30	23.5	123875.5911	14.95	280.37	351.44
3rd	5250.20	33.5	175881.8256	21.23	265.41	711.31
4th	5230.70	43.5	227535.3956	27.47	244.18	1194.90
5th	4307.11	53.5	230430.4603	27.82	216.71	1488.28
6th	3699.42	66.83	247232.2559	29.85	188.89	1994.66
7th	3634.33	81.5	296197.5507	35.76	159.05	2914.28
8th	3245.59	96.17	312128.0481	37.68	123.29	3623.80
9th	3224.85	110.83	357410.0738	43.15	85.61	4782.07
Penthouse	757.14	125.5	95021.06791	11.47	11.47	1439.65
$\text{Roof}_{\text{penthouse level}}$	2045.36	125.5	256692.1048	30.99	30.99	3889.09
Roof	701.35	140.17	98308.84858	11.87	0	1663.56
Total					1892	

TABLE 5 – Seismic Load Tabulation

EXISTING STRUCTURAL SYSTEM

FOUNDATION

The foundation system is composed of cast-in-place reinforce concrete grade beams and piers. The deep foundation consists of drilled caissons that range from 3 to 5 feet in diameter, and 20 to 30 feet below grade. These caissons can carry loads up to 1900 kips depending on the size. The general thickness of the slab on grade is either 4 or 6inches depending on indication on plans, but is also 12 inches thick in some areas. The columns are also cast-in-place in some areas of the ground floor, but transfer to steel columns. All the concrete in the building has a compressive strength of 4000 psi; except for the caissons and steel column encasements have a compressive strength of 3000 psi.

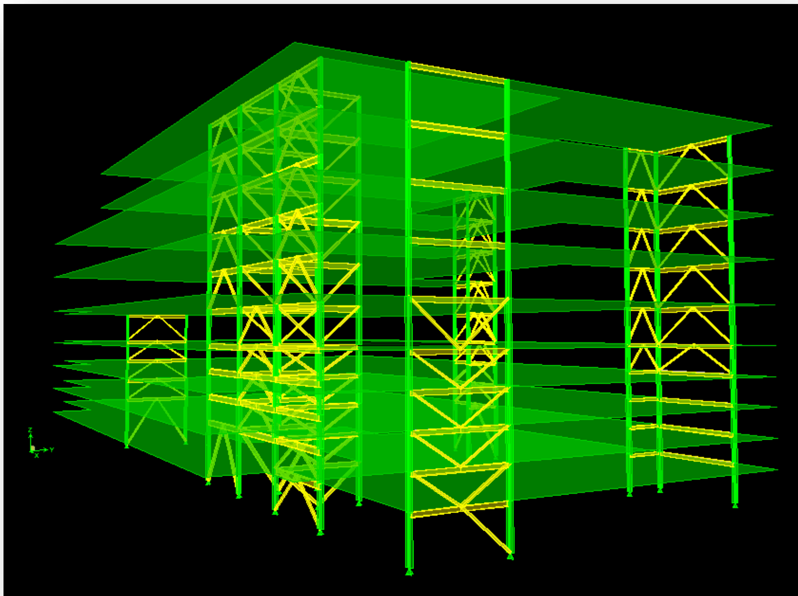
FLOOR SYSTEM

The floor system is a composite steel slab system on steel beams with a typical bay size of 31'6" x 31'6". The typical composite deck is composed of 6 inches of normal weight concrete and 1.5" – 18 gauge composite steel decking with ¾" studs. The floor is supported typically by W 18 x 40 beams and W 24 x 84 girders, but there are large amount of other W - shapes used. The roof consists of 1.5" – 18 gauge steel roof deck supported typically by W 16 x 26 beams and W 24 x 55 girders. Refer to typical bay layout and overall plan such as shown on page 20.

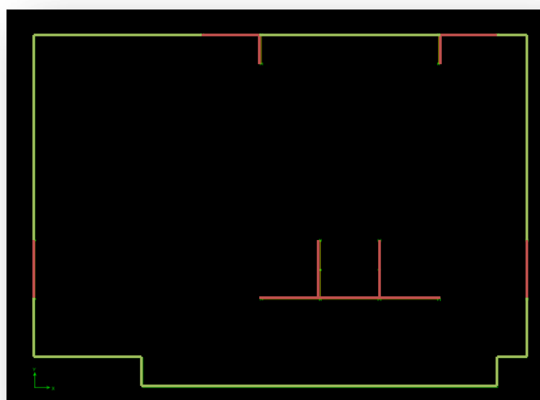
LATERAL SYSTEM

The lateral system is composed of braced frames strategically placed on each floor. The braced frame can be located in the walls of the main elevator and stairwell core in the center of the building, in some exterior walls, and in the exterior walls of the penthouse. The braces are hollow structural steel members. Typical brace members are HSS 8 x 8's and HSS 6 x 6's were used, but several different sizes were used. The shear at the end of the beams is typically 10 kips, unless indicated otherwise on the plans. Also, column splices transmit compression forces in end bearing with a minimum of 15 kips of shear. Two bays of the braced frames in the center core connect into the buildings foundation transfer the shear load. Refer to typical braced frame layout shown on page 22.

LATERAL SYSTEM ANALYSIS



Graphic of ETABS Model

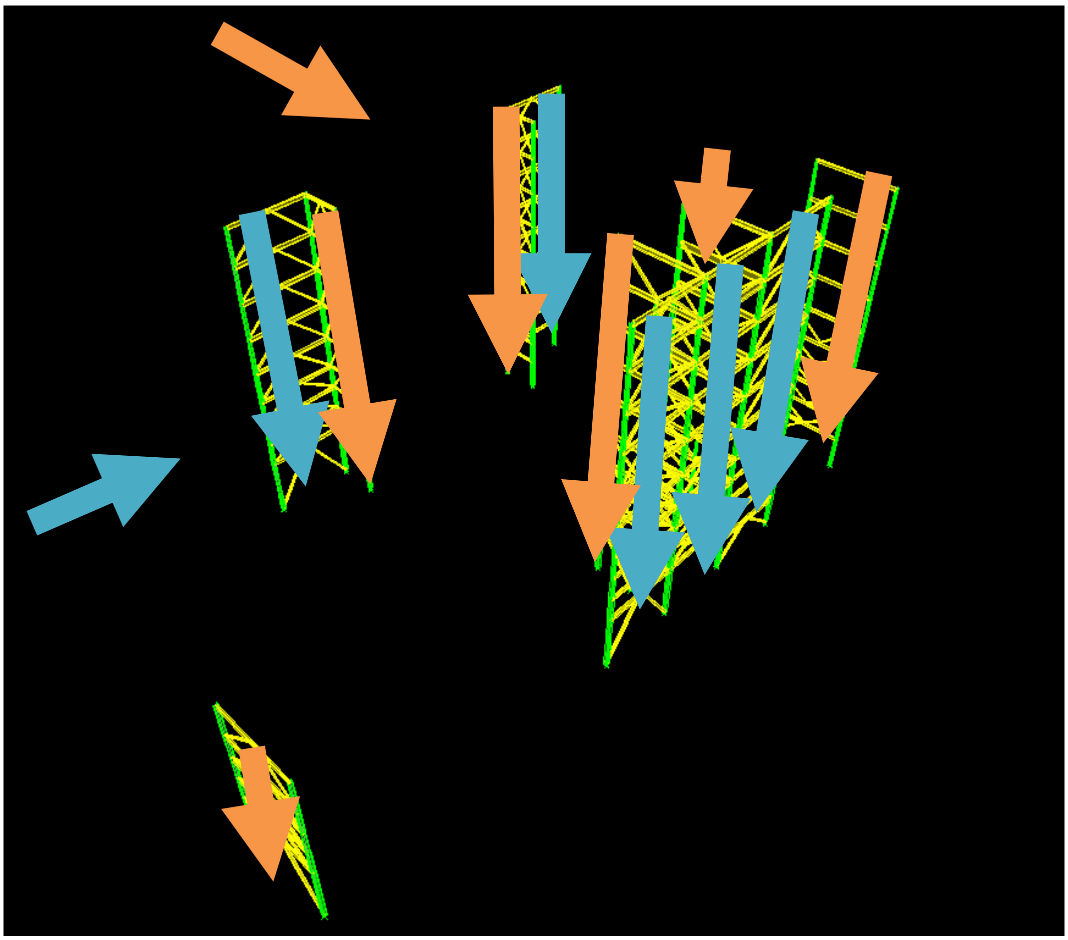


A computer model of the Science Center Research Park building was used to analyze the existing lateral system and the loads applied. An ETABS model was created including only the lateral elements and diaphragms. The reason is simplicity and the reductions of possible errors. The seismic loads were applied to the center of pressure. The ETABS model was used to calculate relative stiffness, wind and seismic drifts, the center of mass and rigidity, and overturning moments. The lateral loads were assumed to be transferred through the diaphragms into the lateral frames, and down to the base of the building.

Graphic above: Lateral System Layout

LOAD PATH

The lateral loads were assumed to be transferred through the diaphragms into the lateral frames, and down to the base of the building where the load is absorbed by the soils.



Lateral Loads in the Y-direction (North-South)

Lateral Loads in the X-directions (East-West)

Relative Stiffness

The relative stiffness for each frame per floor was tabulated using the ETABS model. A 1000 kip force was applied to the center of mass at the top level of the building. In order to calculate the relative stiffness in each frame, section cuts were made using the ETABS model to determine the shear forces. The tables below include the shear forces and tabulation of relative stiffness. It was confirmed that the total of the shear forces at each level roughly equal the story shear of 1000 kips. The relative stiffness was determined by taking the percentage of the 1000 kips that was resisted by the frame examined.

11th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-942	-0.942	1	0	0
A	0	0	3	0	0
			4	-496	-0.496
			5	-504	-0.504
			6	0	0
			10	0	0
Total	-942			-1000	

10th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-651	-0.651	1	-6	-0.006
A	-418	-0.418	3	-153	-0.153
			4	-416	-0.416
			5	-271	-0.271
			6	-154	-0.154
			10	0	0
Total	-1069			-1000	

9th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-615	-0.615	1	-5	-0.005
A	-369	-0.369	3	-161	-0.161
			4	-425	-0.425
			5	-246	-0.246
			6	-162	-0.162
			10	0	0
Total	-984			-999	

8th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-635	-0.635	1	-8	-0.008
A	-366	-0.366	3	-132	-0.132
			4	-511	-0.511
			5	-209	-0.209
			6	-139	-0.139
			10	0	0
Total	-1001			-999	

7th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-649	-0.649	1	-7	-0.007
A	-352	-0.352	3	-143	-0.143
			4	-469	-0.469
			5	-230	-0.23
			6	-150	-0.15
			10	0	0
Total	-1001			-999	

6th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-666	-0.666	1	-308	-0.308
A	-335	-0.335	3	-30	-0.03
			4	-157	-0.157
			5	-250	-0.25
			6	-25	-0.025
			10	-231	-0.231
Total	-1001			-1001	

5th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-642	-0.642	1	-265	-0.265
A	-361	-0.361	3	-69	-0.069
			4	-172	-0.172
			5	-232	-0.232
			6	-67	-0.067
			10	-195	-0.195
Total	-1003			-1000	

4th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-660	-0.66	1	-281	-0.281
A	-335	-0.335	3	-4	-0.004
			4	-215	-0.215
			5	-244	-0.244
			6	-3	-0.003
			10	-252	-0.252
Total	-995			-999	

3th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-648	-0.648	1	-246	-0.246
A	-342	-0.342	3	-6	-0.006
			4	-205	-0.205
			5	-325	-0.325
			6	-2	-0.002
			10	-214	-0.214
Total	-990			-998	

2th Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-719	-0.719	1	-296	-0.296
A	-294	-0.294	3	5	0.005
			4	-289	-0.289
			5	-228	-0.228
			6	-2	-0.002
			10	-191	-0.191
Total	-1013			-1001	

1ST Story					
Grid	X Force	% X	Grid	Y Force	% Y
F	-707	-0.707	1	-182	-0.182
A	-294	-0.294	3	-2	-0.002
			4	-397	-0.397
			5	-228	-0.228
			6	2	0.002
			10	-191	-0.191
Total	-1001			-998	

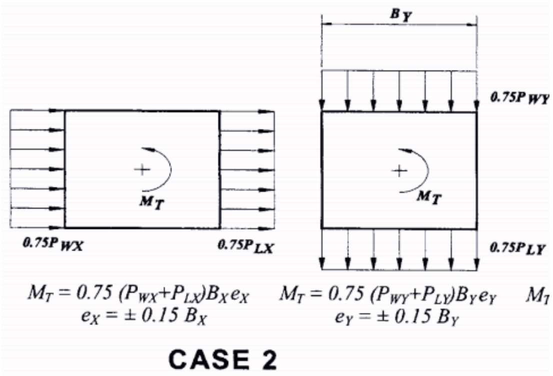
Figures 6 – 16: Relative Stiffness for frames resisting 1000 kips in the X and Y directions

Below is the output from ETABS for the center of mass and the center of rigidity for each level of the building. The coordinates do not change much due to the fact the building does not change radically in shape by level, and the upper stories are located in the center of the building.

Center of Mass		
Story	**XCM (ft)**	**YCM (ft)**
STORY11	168.08	93.71
STORY10	168.07	92.22
STORY9	168.11	92.44
STORY8	163.58	83.40
STORY7	163.60	83.42
STORY6	133.04	96.51
STORY5	132.98	96.52
STORY4	132.92	96.49
STORY3	132.94	96.48
STORY2	133.02	96.47
STORY1	187.25	120.67

Center of Rigidity		
Story	**XCR (ft)**	**YCR (ft)**
STORY11	164.71	57.94
STORY10	165.49	90.07
STORY9	164.03	88.80
STORY8	161.76	88.46
STORY7	157.67	90.13
STORY6	151.74	99.56
STORY5	148.31	97.27
STORY4	145.42	94.14
STORY3	147.28	90.84
STORY2	151.25	85.39
STORY1	190.59	72.33

Figures 17 & 18: Center of mass and Center of Rigidity



Story	B_x (in.)	e_x (in.)	B_y (in.)	e_y (in.)
1	3150	472.50	2268	340.20
2	3150	472.50	2268	340.20
3	3150	472.50	2268	340.20
4	3150	472.50	2268	340.20
5	3150	472.50	2268	340.20
6	3150	472.50	2268	340.20
7	2457	368.55	2268	340.20
8	2457	368.55	2268	340.20
9	2268	340.20	2079	311.85
10	2268	340.20	2079	311.85
11	1134	170.10	1134	170.10

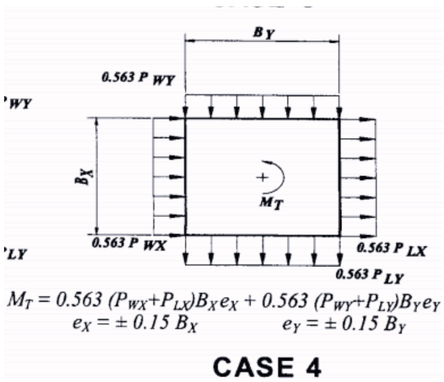
CASE 2 NORTH-SOUTH POS e				
Story	F_x	Fy	xccor	yccor
11	0	99	2187.1	1134
10	0	77.25	2356.2	1228.5
9	0	94.5	2356.2	1228.5
8	0	90.75	2290.1	1134
7	0	87	2290.1	1134
6	0	81.75	2047.5	1134
5	0	69	2047.5	1134
4	0	50.25	2047.5	1134
3	0	47.25	2047.5	1134
2	0	43.5	2047.5	1134
1	0	41.25	2583.0	1417.5

CASE 2 NORTH-SOUTH NEG e				
Story	F_x	Fy	xccor	yccor
11	0	99	1846.9	1134
10	0	77.25	1675.8	1228.5
9	0	94.5	1675.8	1228.5
8	0	90.75	1553.0	1134
7	0	87	1553.0	1134
6	0	81.75	1102.5	1134
5	0	69	1102.5	1134
4	0	50.25	1102.5	1134
3	0	47.25	1102.5	1134
2	0	43.5	1102.5	1134
1	0	41.25	1638.0	1417.5

CASE 2 EAST-WEST POS e				
Story	F_x	Fy	xccor	yccor
11	42.75	0	2017	1304.1
10	36.75	0	2016	1540.4
9	40.5	0	2016	1540.4
8	39	0	1921.5	1474.2
7	37.5	0	1921.5	1474.2
6	33.75	0	1575	1474.2
5	27	0	1575	1474.2
4	21.75	0	1575	1474.2
3	20.25	0	1575	1474.2
2	18.75	0	1575	1474.2
1	18.75	0	2110.5	1757.7

CASE 2 EAST-WEST NEG e				
Story	F_x	Fy	xccor	yccor
11	42.75	0	2017	963.9
10	36.75	0	2016	28.4
9	40.5	0	2016	916.7
8	39	0	1921.5	793.8
7	37.5	0	1921.5	793.8
6	33.75	0	1575	793.8
5	27	0	1575	793.8
4	21.75	0	1575	793.8
3	20.25	0	1575	793.8
2	18.75	0	1575	793.8
1	18.75	0	2110.5	1757.7

Figures 19 – 23: the effective coordinates for wind case 2



Story	B_x (in.)	e_x (in.)	B_y (in.)	e_y (in.)
1	3150	472.50	2268	340.20
2	3150	472.50	2268	340.20
3	3150	472.50	2268	340.20
4	3150	472.50	2268	340.20
5	3150	472.50	2268	340.20
6	3150	472.50	2268	340.20
7	2457	368.55	2268	340.20
8	2457	368.55	2268	340.20
9	2268	340.20	2079	311.85
10	2268	340.20	2079	311.85
11	1134	170.10	1134	170.10

CASE 4 NORTH-SOUTH e_x pos e_y pos				
Story	F_x	Fy	xccor	yccor
11	32.09	74.32	2187.10	1304.10
10	27.59	57.99	2356.20	1540.40
9	30.40	70.94	2356.20	1540.40
8	29.28	68.12	2290.10	1474.20
7	28.15	65.31	2290.10	1474.20
6	25.34	61.37	2047.50	1474.20
5	20.27	51.80	2047.50	1474.20
4	16.33	37.72	2047.50	1474.20
3	15.20	35.47	2047.50	1474.20
2	14.08	32.65	2047.50	1474.20
1	14.08	30.97	2583.00	1757.70

CASE 4 NORTH-SOUTH e_x neg e_y neg				
Story	F_x	Fy	xccor	yccor
11	32.09	74.32	1846.90	963.90
10	27.59	57.99	1675.80	28.40
9	30.40	70.94	1675.80	916.70
8	29.28	68.12	1553.00	793.80
7	28.15	65.31	1553.00	793.80
6	25.34	61.37	1102.50	793.80
5	20.27	51.80	1102.50	793.80
4	16.33	37.72	1102.50	793.80
3	15.20	35.47	1102.50	793.80
2	14.08	32.65	1102.50	793.80
1	14.08	30.97	1638.00	1757.70

CASE 4 EAST-WEST e_x pos e_y neg				
Story	F_x	Fy	xccor	yccor
11	32.09	74.32	2187.10	963.90
10	27.59	57.99	2356.20	28.40
9	30.40	70.94	2356.20	916.70
8	29.28	68.12	2290.10	793.80
7	28.15	65.31	2290.10	793.80
6	25.34	61.37	2047.50	793.80
5	20.27	51.80	2047.50	793.80
4	16.33	37.72	2047.50	793.80
3	15.20	35.47	2047.50	793.80
2	14.08	32.65	2047.50	793.80
1	14.08	30.97	2583.00	1757.70

CASE 4 EAST-WEST e_x neg e_y pos				
Story	F_x	Fy	xccor	yccor
11	32.09	74.32	1846.90	1304.10
10	27.59	57.99	1675.80	1540.40
9	30.40	70.94	1675.80	1540.40
8	29.28	68.12	1553.00	1474.20
7	28.15	65.31	1553.00	1474.20
6	25.34	61.37	1102.50	1474.20
5	20.27	51.80	1102.50	1474.20
4	16.33	37.72	1102.50	1474.20
3	15.20	35.47	1102.50	1474.20
2	14.08	32.65	1102.50	1474.20
1	14.08	30.97	1638.00	1757.70

Figures 24 – 28: the effective coordinates for wind case 4

WIND DRIFTS

Wind loads determined in technical report 1 were used in the ETABS model to determine the story drifts. The load case that controlled for wind was load case 1. This load case is 100% of the wind load is applied in the North-South or East-West direction. The controlling load combination for story drift in both directions was load combination 4:

1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)

The story and total drifts were checked with accordance to ASCE 7-05 to determine whether or not the deflections were acceptable. The drift limit using the story height can be calculated using this equation: Δ_{wind} = H/400 (from ASCE 7-05)

Torsion was taken into account when adjusting the coordinates to the eccentricity found when looking at wind load cases 2 and 4. The results for wind drift in the X and Y-directions were tabulated after running the ETABS model for unfactored wind loads. Below are the wind drifts which all checked out to be acceptable when compared to the drift limitation.

Controlling Wind Drift: North-South Direction									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δwind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δwind = H/400 (in)		
11	140.17	0.00461	<	0.35043	acceptable	0.03944	<	1.97125	acceptable
10	125.5	0.00425	<	0.31375	acceptable	0.03483	<	1.62083	acceptable
9	110.83	0.00470	<	0.27708	acceptable	0.03058	<	1.30708	acceptable
8	96.17	0.00480	<	0.24043	acceptable	0.02588	<	1.03000	acceptable
7	81.5	0.00477	<	0.20375	acceptable	0.02108	<	0.78958	acceptable
6	66.83	0.00301	<	0.16708	acceptable	0.01632	<	0.58583	acceptable
5	53.5	0.00286	<	0.13375	acceptable	0.01331	<	0.41875	acceptable
4	43.5	0.00307	<	0.10875	acceptable	0.01045	<	0.28500	acceptable
3	33.5	0.00267	<	0.08375	acceptable	0.00738	<	0.17625	acceptable
2	23.5	0.00211	<	0.05875	acceptable	0.00471	<	0.09250	acceptable
1	13.5	0.00260	<	0.03375	acceptable	0.00260	<	0.03375	acceptable

Controlling Wind Drift: East-West Direction									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δwind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δwind = H/400 (in)		
11	140.17	0.00497	<	0.35043	acceptable	0.01559	<	1.97125	acceptable
10	125.5	0.00094	<	0.31375	acceptable	0.01062	<	1.62083	acceptable
9	110.83	0.00103	<	0.27708	acceptable	0.00968	<	1.30708	acceptable
8	96.17	0.00109	<	0.24043	acceptable	0.00865	<	1.03000	acceptable
7	81.5	0.00111	<	0.20375	acceptable	0.00756	<	0.78958	acceptable
6	66.83	0.00101	<	0.16708	acceptable	0.00645	<	0.58583	acceptable
5	53.5	0.00106	<	0.13375	acceptable	0.00544	<	0.41875	acceptable
4	43.5	0.00114	<	0.10875	acceptable	0.00438	<	0.28500	acceptable
3	33.5	0.00111	<	0.08375	acceptable	0.00324	<	0.17625	acceptable
2	23.5	0.00099	<	0.05875	acceptable	0.00213	<	0.09250	acceptable
1	13.5	0.00114	<	0.03375	acceptable	0.00114	<	0.03375	acceptable

Figures 20 & 30: Allowable wind drifts in the North-South and East-West Directions

SEISMIC DRIFTS

Seismic loads determined in technical report 1 were used in the ETABS model to determine the story drifts. Seismic drift protects against building failure/collapse unlike wind drift which is a serviceability requirement. The drift limitation for seismic drift can be calculated using this equation: $\Delta_{seismic} = 0.015h_{sx}$ (from ASCE 7-05)

Below are the seismic drifts which all checked out to be acceptable when compared to the drift limitation.

Controlling Seismic Drift: North-South Direction									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δwind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δwind = H/400 (in)		
11	140.17	0.00051	<	0.35043	acceptable	0.00125	<	1.97125	acceptable
10	125.5	0.00011	<	0.31375	acceptable	0.00074	<	1.62083	acceptable
9	110.83	0.00012	<	0.27708	acceptable	0.00063	<	1.30708	acceptable
8	96.17	0.00013	<	0.24043	acceptable	0.00051	<	1.03000	acceptable
7	81.5	0.00016	<	0.20375	acceptable	0.00038	<	0.78958	acceptable
6	66.83	0.00003	<	0.16708	acceptable	0.00022	<	0.58583	acceptable
5	53.5	0.00002	<	0.13375	acceptable	0.00019	<	0.41875	acceptable
4	43.5	0.00001	<	0.10875	acceptable	0.00017	<	0.28500	acceptable
3	33.5	0.00001	<	0.08375	acceptable	0.00016	<	0.17625	acceptable
2	23.5	0.00005	<	0.05875	acceptable	0.00015	<	0.09250	acceptable
1	13.5	0.00010	<	0.03375	acceptable	0.00010	<	0.03375	acceptable

Controlling Seismic Drift: East-West Direction									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δwind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δwind = H/400 (in)		
11	140.17	0.00114	<	0.35043	acceptable	0.00540	<	1.97125	acceptable
10	125.5	0.00035	<	0.31375	acceptable	0.00426	<	1.62083	acceptable
9	110.83	0.00040	<	0.27708	acceptable	0.00391	<	1.30708	acceptable
8	96.17	0.00043	<	0.24043	acceptable	0.00351	<	1.03000	acceptable
7	81.5	0.00045	<	0.20375	acceptable	0.00307	<	0.78958	acceptable
6	66.83	0.00041	<	0.16708	acceptable	0.00263	<	0.58583	acceptable
5	53.5	0.00042	<	0.13375	acceptable	0.00222	<	0.41875	acceptable
4	43.5	0.00046	<	0.10875	acceptable	0.00179	<	0.28500	acceptable
3	33.5	0.00045	<	0.08375	acceptable	0.00133	<	0.17625	acceptable
2	23.5	0.00042	<	0.05875	acceptable	0.00088	<	0.09250	acceptable
1	13.5	0.00046	<	0.03375	acceptable	0.00046	<	0.03375	acceptable

Figures 31 & 32: Allowable seismic drifts in the North-South and East-West Directions

Torsion

Torsion for the building can result from a difference in the center of mass and the center of rigidity. The difference between the points is an eccentricity that the loads are applied at. The eccentricity multiplied by the force results in a moment on the building. As stated before the wind load cases 2 and 4 include eccentricities on the wind loads that create torsion on the building. The eccentricity for both these cases is 15 percent of the building's width. ASCE 7-05 describes torsion that is produced by accidental eccentricities for seismic loads.

Overturning Moment

Overturning happens the moment created by the building's self weight does not offset lateral forces on the building. If the building's self weight does not compensate for the moment, the foundation can be designed to counteract the overturning moment. In the design of the foundation, friction from the soil can used to assist the foundation counteract the overturning moment.

Below are the overturning moments determined using the ETABS model. Critical story shears applied at the center of mass at each level were used to tabulate the moments at each level, and the moments were totaled to determine the building's overturning moment.

Seismic Overturning Moment			
Story	**Height**	**Story Shear (k)**	**Overturning Moment (k-ft)**
11	140.17	11.87	0
10	125.5	42.86	174
9	110.83	86.01	803
8	96.17	123.69	2064
7	81.5	159.45	3878
6	66.83	189.3	6217
5	53.5	217.12	8741
4	43.5	244.59	10912
3	33.5	265.82	13358
2	23.5	280.77	16016
1	13.5	286.82	18824
		Total Moment:	80987

Figure 33: Seismic Overturning Moment

Wind Overturning Moment (X / N-S)			
Story	**Height**	**Story Shear (k)**	**Overturning Moment (k-ft)**
11	140.17	91.2	1772
10	125.5	169.6	9418
9	110.83	256	19892
8	96.17	339.2	31876
7	81.5	419.2	46345
6	66.83	491.2	62358
5	53.5	548.8	79825
4	43.5	595.2	94770
3	33.5	638.4	109790
2	23.5	678.4	127521
1	13.5	718.4	144516
		Total Moment:	728084

Wind Overturning Moment (Y / E-W)			
Story	**Height**	**Story Shear (k)**	**Overturning Moment (k-ft)**
11	140.17	211.2	4403
10	125.5	376	14471
9	110.83	577.6	27276
8	96.17	771.2	43726
7	81.5	956.8	45349
6	66.83	1131.2	84201
5	53.5	1278.4	102291
4	43.5	1385.6	121167
3	33.5	1486.4	142456
2	23.5	1579.2	164683
1	13.5	1667.2	194045
		Total Moment:	944069

Figures 34 & 35: Wind Overturning Moment in N-S & E-W Directions

In result of the calculation of the moment resulting from the building's self weight it is found that there will be no uplift in the foundation. The building self weight compensates for the overturning moment caused by lateral forces. The calculations for dead load moments can be found in Appendix D.

Also, spot checks were done on two lateral braces located at the intersection of grid line A and 6. Both members were found to be an acceptable design.

CONCLUSION

The lateral forces including wind and seismic loads were calculated in technical report 1 and used in this report for the analysis of the lateral system. The controlling load combination was found to be: 1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)

The calculation of drifts included the unfactored wind loads for case 1 and the unfactored seismic loads. The controlling load combination was also used in the member spot checks. All the drift values were found to be less than the drift limitations stated in ASCE 7-05, which makes the drift displacements acceptable. The drift limitations used are:
$\Delta_{wind} = H/400$ and $\Delta_{seismic} = 0.015h_{sx}$

The overturning moment was found not to cause any uplift in the foundation. The self weight of the building compensates for the overturning moment cause by lateral forces.

Member spot checks were perform on two HSS steel shape braces and two W14 steel shape columns where grid lines A and 6 intersect, and on the first and 5th floors. The design of the members was found to hold the controlling load combination, which means the design is acceptable.

APPENDIX A- EXISTING TYPICAL BAY AND DESIGN VALUES

DESIGN LOADS AND FACTORS

DESIGN CODE: INTERNATIONAL BUILDING CODE 2003 ED

LIVE LOAD DATA

FLOOR OR ROOF AREA	LOAD (psf)
LABS / OFFICES	100
CORRIDORS, LOBBIES & EXITS	100
GARAGE	40
MECHANICAL EQUIP ROOMS	150
ROOF	30

LIVE LOAD REDUCTION APPLIED TO:

- ☒ COLUMNS
- ☒ GIRDERS
- ☐ BEAMS
- ☐ 2-WAY SLABS

SPECIAL LOADING:

SNOW LOAD DATA

ROOF AREA	LOAD (psf)
GROUND SNOW LOAD (P_g)	30
FLAT ROOF SNOW LOAD (P_f)	23
DRIFT	VARIES

FACTOR	VALUE
SNOW EXPOSURE (C_e)	1.0
SNOW LOAD IMPORTANCE (I_s)	1.0
THERMAL FACTOR (C_t)	1.0

SPECIAL SNOW CONSIDERATIONS:

☒ GOVERNS ROOF DESIGN

WIND LOAD DATA

FACTOR	VALUE
BASIC WIND SPEED (V_{3S}) (MPH)	90
WIND IMPORTANCE (I_w)	1.0
OCCUPANCY CATEGORY	II
WIND EXPOSURE	B
INTERNAL PRESSURE COEFFICIENT	±0.18
COMPONENTS AND CLADDING WIND PRESSURE (PSF)	*VARIES

* CALCULATED PRESSURES TO BE DETERMINED BY COMPONENT AND CLADDING PROVIDER.

SPECIAL WIND CONSIDERATIONS:

☐ GOVERNS LATERAL DESIGN

EARTHQUAKE DESIGN DATA

FACTOR	VALUE
SEISMIC IMPORTANCE FACTOR (I_E)	1.0
SEISMIC USE GROUP	I
SPECTRAL RESPONSE ACCELERATION 0.2 SEC (S_S)	0.33
SPECTRAL RESPONSE ACCELERATION 1.0 SEC (S_1)	0.082
SITE CLASS	C
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{DS})	0.27
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{D1})	0.09
SEISMIC DESIGN CATEGORY	B

ANALYSIS PROCEDURE – EQUIVALENT LATERAL FORCE

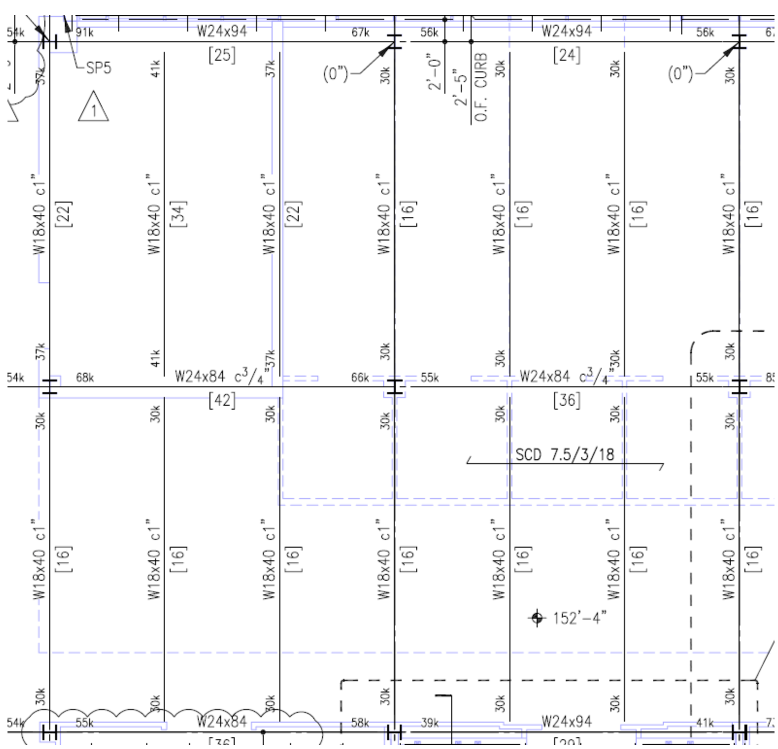
BASIC SEISMIC-FORCE-RESISTING SYSTEM	ORDINARY STEEL CONCENTRIC BRACED FRAMES Cs=0.021 R=3
DESIGN BASE SHEAR (kips)	

SPECIAL SEISMIC CONSIDERATIONS:

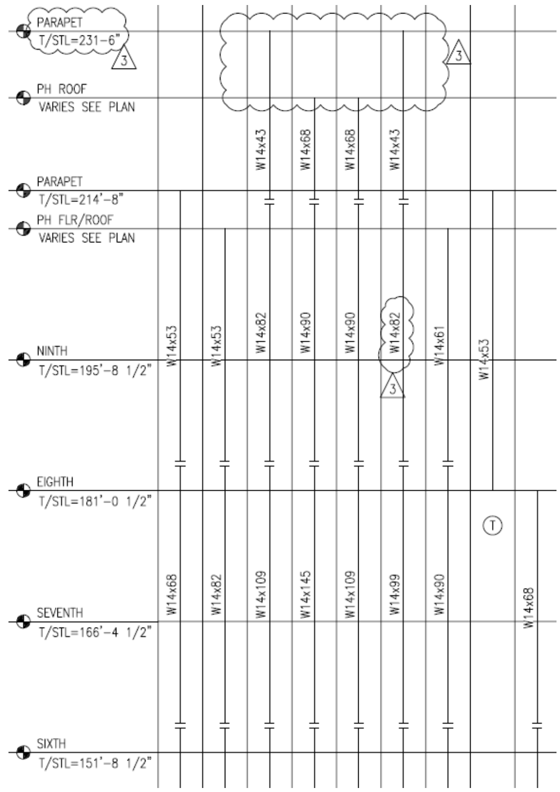
☒ GOVERNS LATERAL DESIGN

2

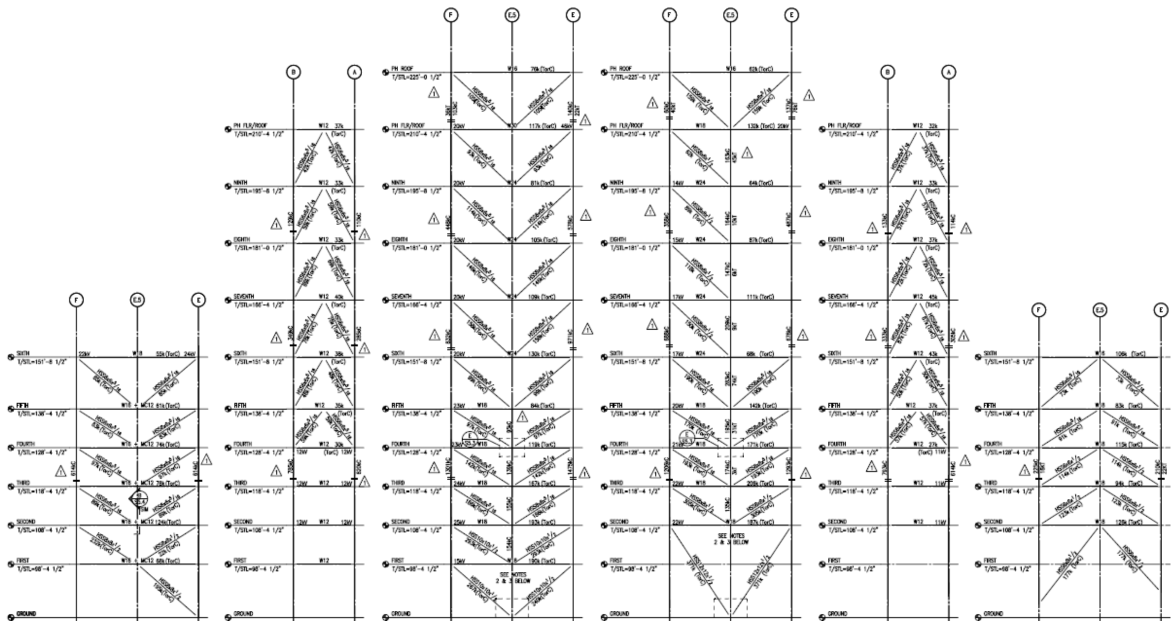
Existing Typical Bay (6th Floor)



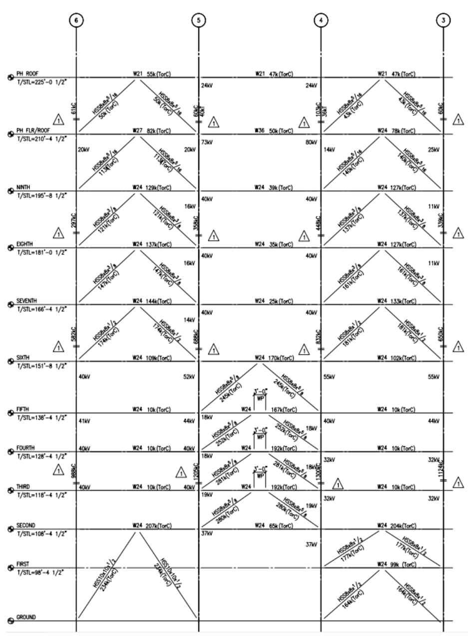
Typical Column Schedule



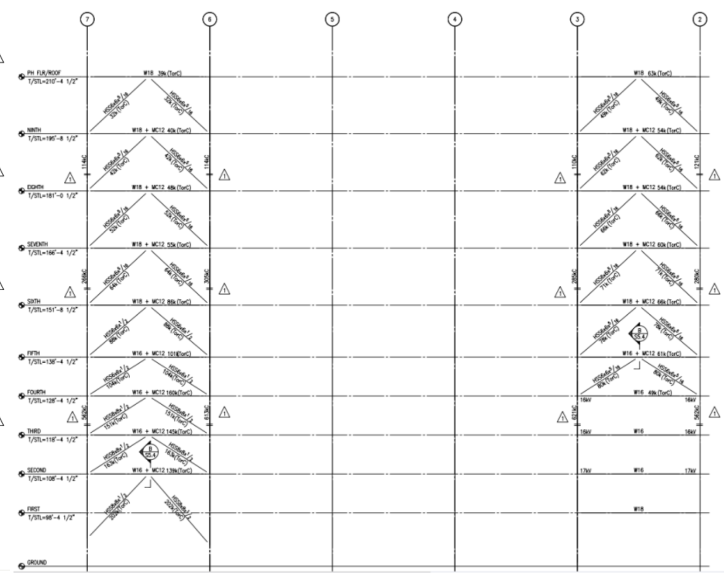
Braced Frame Schedule



ELEVATION AT LINE 1 ELEVATION AT LINE 3 ELEVATION AT LINE 4 ELEVATION AT LINE 5 ELEVATION AT LINE 6 ELEVATION AT LINE 10

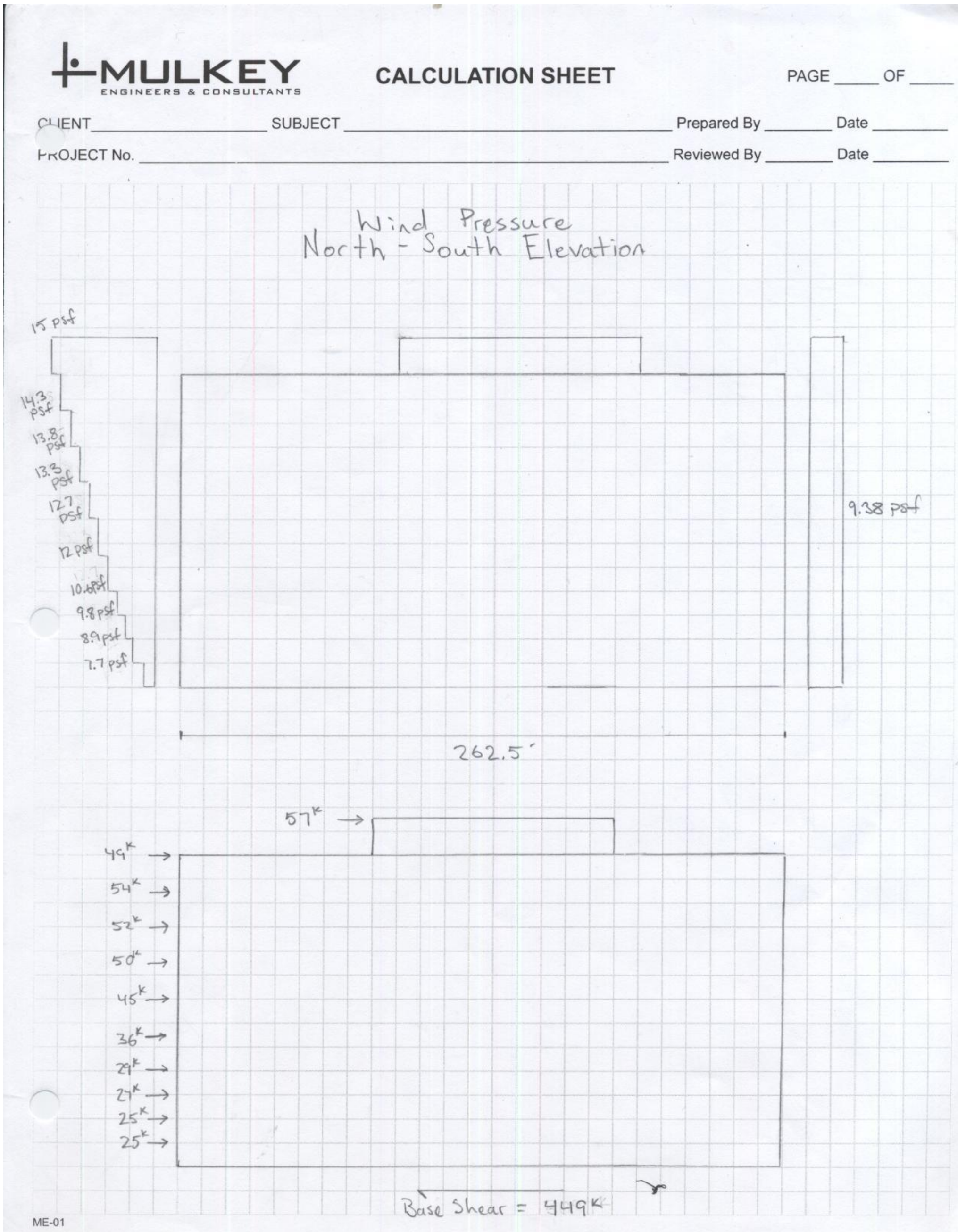


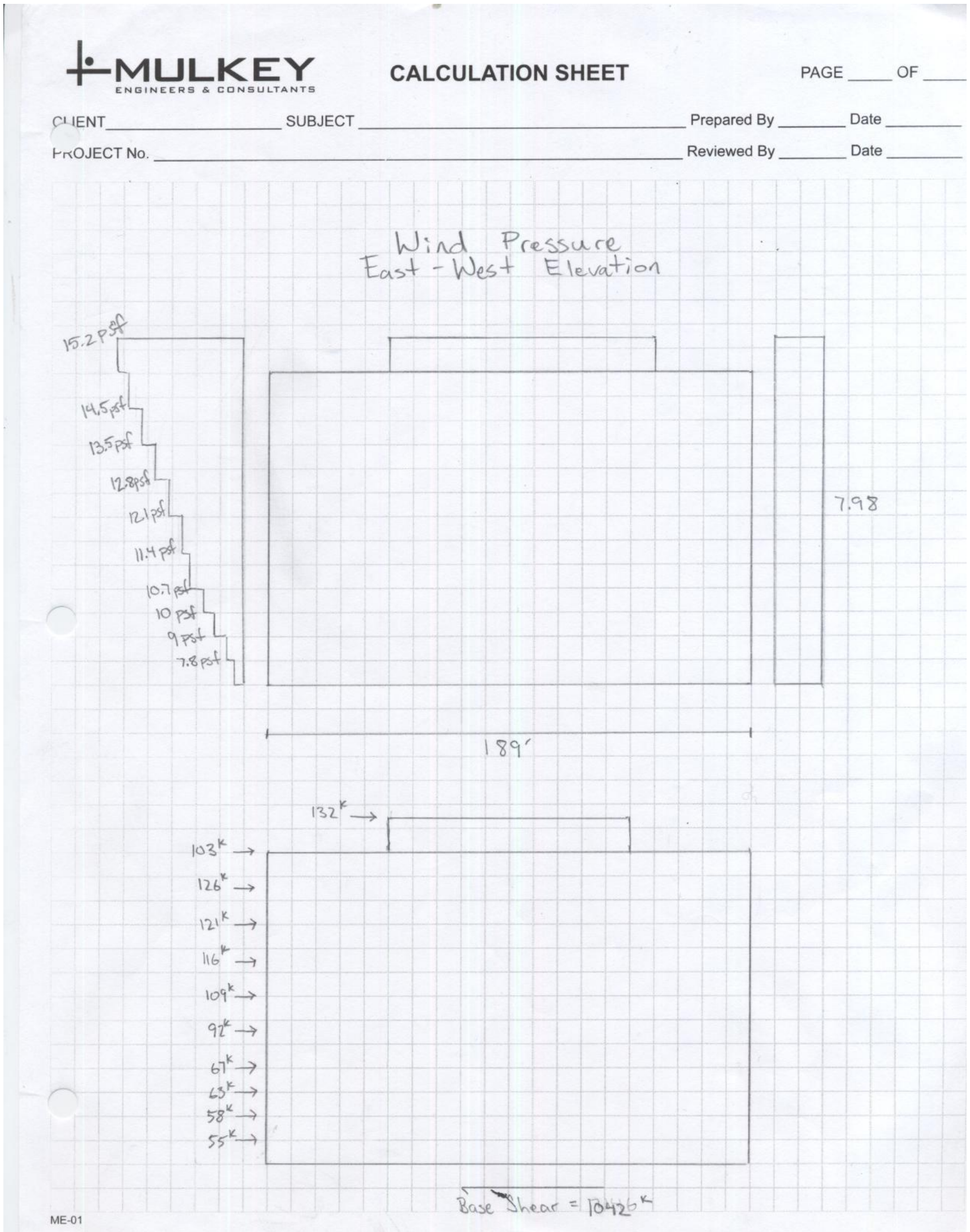
ELEVATION AT LINE A



ELEVATION AT LINE F

APPENDIX B-WIND LOADS




MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE ____ OF ____
CLIENT ____ SUBJECT ____ Prepared By ____ Date ____
PROJECT No. ____ Reviewed By ____ Date ____
Wind Pressure
East - West Elevation
15.2 psf
14.5 psf
13.5 psf
12.8 psf
12.1 psf
11.4 psf
10.7 psf
10 psf
9 psf
7.8 psf
7.98
189'
132k
103k
126k
121k
116k
109k
92k
67k
63k
58k
55k
Base Shear = 1042k
ME-01

Zachary Yarnall

MULKEY ENGINEERS & CONSULTANTS

CALCULATION SHEET

PAGE ____ OF ____

CLIENT ________ SUBJECT Senior Thesis Prepared By ______ Date 09/29/09

PROJECT No. Method 2 - Wind Analysis Reviewed By ______ Date ______

Velocity Pressures, q_z and q_h

- From ASCE 7-05 Fig. 6-1

  Basic wind speed, V = 90 mph / 40 m/s

- From ASCE 7-05 Fig. 6-4

  Wind directionality factor, $K_d = 0.85$ (for buildings)

- From ASCE 7-05 Fig. 7-4

  Importance factor, $I = 1.1$ (Category III)

- From ASCE 7-05 6.5.6

  Exposure Category B (located in urban area)

- Are all 5 conditions of 6.5.7.1 met? No

- From ASCE 7-05

  Topographic factor, $K_{zt} = 1.0$

- From ASCE 7-05 Table 6-3 and Table 6-2

  Velocity pressure exposure coefficients

  $K_z = 1.10$ ($z_g = 1200$, $\alpha = 7.0$)
  $K_h = 1.10$

  $K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}$ (sample calculation)

  $= 2.01\left(\frac{147.5}{1200}\right)^{\frac{2}{7}} = 1.10$

- From ASCE 7-05 Eq. 6-15

  Velocity pressure at height z and h (refer to spread sheet)

  $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (sample calculation)

  $= 0.00256(1.10)(1.0)(0.85)(90)^2(1.10)$

  $= 21.47$

ME-01

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CALCULATION SHEET PAGE ____ OF ____

CLIENT ____________ SUBJECT ____________ Prepared By ______ Date ______

PROJECT No. ____________ Reviewed By ______ Date ______

Gust Effect Factors, G and G_f

- Building natural frequency, n_1 (ASCE 7-05, C6.5.8, Eq. C6-17)

  $n_1 = 100/H = 100/147.5 = 0.68$ (average value)

- Damping ratio, β (ASCE 7-05, C6.5.8)

  $\beta_1 = 1.0\%$ per ISO

- Structure Dimensions

  $h = 147.5'$
  $B = 262.5'$ (N/S Elevation)
  $L = 189'$ (E/W Elevation)

  $n_1 < 1\ Hz$

- $\therefore$ Structure is flexible

  $g_Q = g_v = 3.4$

  $$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad \text{(Eq. 6-9)}$$

  $$= \sqrt{2\ln(3600(0.68))} + \frac{0.577}{\sqrt{2\ln(3600(0.68))}} = 4.097$$

  $\bar{z} = 0.6h = 0.6(147.5) = 88.5^{ft} > 30^{ft} = z_{min}$ (from ASCE 7-05 Fig. 6-2)

  $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{\frac{1}{6}} = 0.30\left(\frac{33}{88.5}\right)^{\frac{1}{6}} = 0.255$ ($c = 0.30$ from ASCE 7-05 Fig. 6-2)

  $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{88.5}{33}\right)^{\frac{1}{3}} = 444.6$ ($\ell = 320$ from ASCE 7-05 Fig. 6-2)

  $$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{(Eq. 6-6)}$$

  $$Q_{N/S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{262.5+147.5}{444.6}\right)^{0.63}}} = 0.791$$

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Gust Effect Factors, G and G_f - continued

$$Q_{E/W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{189 + 147.5}{444.6}\right)^{0.63}}} = 0.809$$

- Basic wind speed, V

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad (\text{Eq. } 6\text{-}14) \quad (\bar{b} = 0.45,\ \bar{\alpha} = \tfrac{1}{4} \text{ from ASCE 7-05 Fig. 6-2})$$

$$= (0.45)\left(\frac{88.5}{33}\right)^{\frac{1}{4}}(90)\left(\frac{88}{60}\right) = 76.01$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.68(444.6)}{76.01} = 3.98 \quad (\text{Eq. } 6\text{-}12)$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.98)}{(1 + 10.3(3.98))^{5/3}} = 0.059$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{6.07} - \frac{1}{2(6.07)^2}\left(1 - e^{-2(6.07)}\right) = 0.151$$

$$\eta = 4.6 n_1 \frac{h}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{147.5}{76.01} = 6.07$$

$$R_{B_1} = \frac{1}{10.8} - \frac{1}{2(10.8)^2}\left(1 - e^{-2(10.8)}\right) = 0.088 \quad (N/S)$$

$$\eta = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{262.5}{76.01} = 10.80$$

$$R_{B_2} = \frac{1}{7.78} - \frac{1}{2(7.78)^2}\left(1 - e^{-2(7.78)}\right) = 0.12 \quad (E/W)$$

$$\eta = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{189}{76.01} = 7.78$$

$$R_{L_1} = \frac{1}{36.17} - \frac{1}{2(36.17)^2}\left(1 - e^{-2(36.17)}\right) = 0.027 \quad (N/S)$$

$$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{262.5}{76.01} = 36.17$$

$$R_{L_2} = \frac{1}{26.04} - \frac{1}{2(26.04)^2}\left(1 - e^{-2(26.04)}\right) = 0.038 \quad (E/W)$$

$$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{189}{76.01} = 26.04$$

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<u>Gust Effect Factors, G and Gf</u> - continued

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \qquad \text{(Eq. 6-10)}$$

$$R_1 = \sqrt{\frac{1}{1.0}(0.059)(0.151)(0.088)(0.53 + 0.47(0.027))} = 0.021 \qquad (N/S)$$

$$R_2 = \sqrt{\frac{1}{1.0}(0.059)(0.151)(0.12)(0.53 + 0.47(0.038))} = 0.024 \qquad (E/W)$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) \qquad \text{(Eq. 6-8)}$$

$$G_{f_1} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.791)^2 + (4.097)^2(0.021)^2}}{1 + 1.7(3.4)(0.255)}\right) \qquad (N/S)$$

$$= 0.876$$

$$G_{f_2} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.809)^2 + (4.097)^2(0.024)^2}}{1 + 1.7(3.4)(0.255)}\right) \qquad (E/W)$$

$$= 0.887$$

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Building Main Wind-force Resisting Systems

- The building is enclosed
- The building has a parapet
- Velocity Pressure $q_p = 21.41$ mph
- Combined net pressure coefficient, GC_{pn}

  $GC_{pn} = +1.5$ windward

  $GC_{pn} = -1.0$ leeward
- Combined net design pressure on the parapet

  $P_p = q_p\, GC_{pn}$ (Eq. 6-20)

  $= (21.41)(1.5) = 32.12$ (windward)
  $= (21.41)(-1.0) = -21.41$ (leeward)
- The building is not rigid
- Determine velocity pressure q_z for windward walls along the height of the building and q_h for leeward walls, side walls, and roof
- Pressure coefficient, C_p for the walls and roof (Fig. 6-6 or 6-8)

  $\frac{L}{B} = \frac{262.5}{189} = 1.39$ (N/S) $\Rightarrow C_p = -0.5$ ($L/B = 0$-1)

  $= \frac{189}{262.5} = 0.72$ (E/W) $\Rightarrow C_p = -0.42$ (interpolated)
  $\{L/B = 0$-$1,\ L/B = 2\}$
  $\{C_p = -0.5,\ C_p = -0.3\}$

C_p	(N/S)	(E/W)
Windward wall	0.8	0.8
Leeward wall	-0.5	-0.42
Side wall	-0.7	-0.7

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Building, Main Wind - force Resisting Systems - Continued

- Determine design wind pressures, p_z

$p_z = q_z G_z C_p$ (Eq. 6-19)

Windward sample calculation (N/S)

$p_z = 21.47(0.876)(0.8) = 15.0 \text{ psf}$

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APPENDIX C-SEISMIC LOADS

Zachary Yarnall

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Consideration of Seismic Design Requirements

- Not a detached one- or two-family dwelling.
- Not an agricultural storage structure intend for incidental human occupancy
- Does the structure require special consideration with respect to response characteristics and evironment that are not addressed in Chapter 15 and for which other regulations provide criteria?

No

∴ Seismic requirements of ASCE 7-05 must be considered

Seismic Ground Motion Values

- Determine S_s and S_1 from Figs. 22-1 through 22-14
  Site Classification C
  Seismic Design Category B (from Structural Plans)

  Occupancy Category II => I = 1.0

  $S_s = 0.3$ (from Fig. 22-1) > 0.15
  ∴ No
  $S_1 = 0.06$ (from Fig. 22-2) > 0.04

- Is the structure seismically isolated or does it have damping systems on site with $S_1 \geq 0.6$? No
- Determine S_{MS} and S_{M1} by Eqs. 11.4-1 and 11.4-2

  $S_{MS} = F_a S_s = (1.2)(0.3) = 0.36$ $F_a = 1.2$ (from Fig. 11.4-1)

  $S_{M1} = F_v S_1 = (1.7)(0.06) = 0.102$ $F_v = 1.7$ (from Fig 11.4-2)

- Determine S_{DS} and S_{D1} by Eqs. 11.4-3 and 11.4-4

  $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.36) = 0.24$
  $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.102) = 0.068$

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Permitted Analytical Procedures

- Equivalent lateral force procedure

response modification coefficient, $R = 3$

importance factor, $I = 1.0$

approximate fundamental period of the structure, T_a

$$T_a = C_t h_n^x = 0.03(149.5)^{0.75} = 1.27$$

$C_t = 0.03$

$x = 0.75$ (from Table 12.8-2)

$T_L = 6$ (from Fig. 22-15) $> T_a$

Determine C_s by Eqs. 12.8-3 and 12.8-2

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \le \frac{S_{DS}}{\left(\frac{R}{I}\right)} \quad \checkmark$$

$$\frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.068}{1.27\left(\frac{3}{1}\right)} = 0.0178$$

$$\frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.24}{3} = 0.08$$

Determine effective seismic weight W in accordance with 12.7.2

$W = 40{,}378$ kips (calculation on excel sheet)

$V = C_s W = 0.0178(40{,}378) = 719$

$T < 0.5$ sec $\therefore$ $k = 1$ (12.8.3)

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- Equivalent lateral force procedure - continued

Determine lateral seismic force f_x at level x by Eqs. 12.8-11 and 12.8-12

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$ (Calculations in Excel Sheet)

$$\sum_{i=1}^{n} w_i h_i^k = (40{,}378)(147.5) = 5{,}955{,}755$$

Determine seismic design story shear V_x by Eq. 12.8-13

$$V_x = \sum_{i=x}^{n} F$$

Determine inherent torsional moment, M_t

Determine accidental torsional moment, M_{ta}

Determine the deflection δ_x at levels x by Eq. 12.8-15

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

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APPENDIX D-OVERTURNING AND SPOT CHECKS

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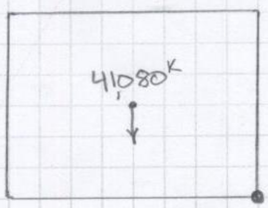
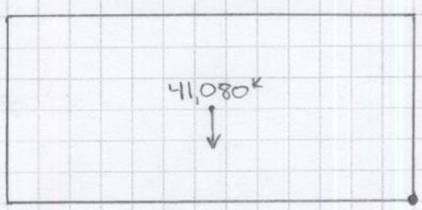
North - South Dead Load Moment



$$M_{N\text{-}S} = 1.2(41{,}080)\left(\frac{189}{2}\right)$$
$$= 4{,}658{,}000^{\,kft}$$
$$> 728{,}084^{\,kft} \quad \therefore \text{Okay}$$

East - West Dead Load Moment



$$M_{E\text{-}W} = 1.2(41{,}080)\left(\frac{262.5}{2}\right)$$
$$= 6{,}470{,}100^{\,k\text{-}ft}$$
$$> 944{,}069^{\,kft} \quad \therefore \text{Okay}$$

∴ No uplift in foundations in either direction

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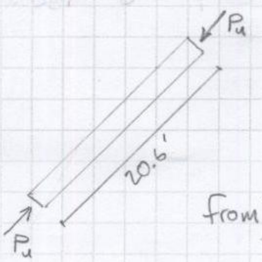
Brace Member 5th floor A-6 $(1.2D + 1.6W + 1.0L + 0.5L_r)$



HSS 6×6×½

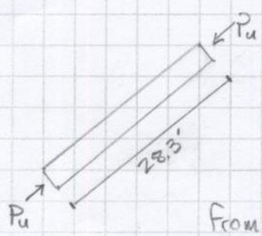
$P_u = 65^k$

$F_y = 42^{ksi}$

from Table 4-3 $\phi P_n = 90^k > P_u$ ∴ Okay

$\frac{P_u}{\phi P_n} = \frac{65}{90} = 0.722 < 1.0$ ∴ Okay

Brace Member 1st floor A-6 $(1.2D + 1.6W + 1.0L + 0.5L_r)$



HSS 8×8×½

$P_u = 183^k$

$F_y = 42^{ksi}$

From Table 4-3 $\phi P_n = 351^k > P_u$ ∴ Okay

$\frac{P_u}{\phi P_n} = \frac{183}{351} = 0.521 < 1.0$ ∴ Okay

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