University Health Building

Located in the Mid-Atlantic Region

Technical Report III

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Executive Summary

Technical Report III was prepared in order to analyze and determine distribution of forces for the existing lateral force resisting system in the University Health Building located in the Mid-Atlantic. The report begins with an introduction and a study of existing building conditions followed by viable building codes and materials in the building. The reader can also find a summary of the hand calculations conducted to determine seismic and wind loads for the UHB.

A three-dimensional ETABS model was created in order to simplify the calculations of the UHB's irregular geometries. ASCE 7-05 load cases where then applied to the model to determine the controlling load factor of the UHB. A study of the building's relative stiffness, torsional moments, story drifts, and overturning moment was then conducted using the load cases. It was determined that seismic loading was controlling over wind loading, and that the build meets code and industry requirements regarding drift limitations. The building's overturning moment was determined to be 3707k-ft. Two strength checks were also conducted for critical members in the structure, which were also acceptable.

Finally, at the end of the report the reader can find appendices which contain back up calculations for items found in the report.

Building Introduction

This new 9 story 161000 square foot building will be a great addition to the university's campus. It is being built to house leaders in the public and private health policy sectors. The building is a mesh between office space and student classrooms nestled around a central sky lit atrium. The architect hopes that this mesh will help to bridge the gap between faculty and students. The classroom area appears as if the classrooms are floating on clouds in a glass enclosure. The concrete structure is enclosed by a curtain wall which is the building's main architectural feature. The curved saw blade-like curtain wall system encompasses one quarter of the building's façade and gives the building an edgy appearance.

The building façade is constructed of many different types of materials, ranging from stone to metal. The building's first floor is covered by a stone veneer giving the building a very stereotomic base. The rest of the building is clad in a mixture of glazing, metal panels, and terracotta. The West and Southeast facades are relatively similar to one another. They both have a pattern of terracotta, metal paneling, and glazing above the first floor with the majority material being covered with the terracotta. The south and north facades are also very similar except the south facade has an aluminum sunscreen system in place. Otherwise, these ends of the building are almost fully glazed. Lastly, the curved curtain wall with reveals located on the northeast side of the building is composed of mainly glazing with the reveals clad in terracotta. Some of these features can be seen in Figure 1.

Figure 1: Photo of Northwest corner of building showing façade materials. Rendering by Payette Architecture.

The majority of the roof is a garden roofing system. The system used on this project is the Sika Sarnafil Extensive Greenroof system. It uses 3in. of growing medium as well as pavers for maintenance. The rooftop penthouse will be covered with a fully adhered white, 60mm thick PVC membrane with a layer of 8in. thick tapered polyisocyanurate insulation boards underneath.

Lastly, the University Health Building is registered as a LEED – NC 2.2 Silver building. This rating includes many different LEED credits involving the façade, roof, and internal systems. The main points came from the heat island effect roof system, the building's proximity to transit, and use of efficient plumbing and lighting fixtures.

Structural Overview

Foundation

The foundation of University Health Building (UHB) consists of spread footings at the base of each column. On the western block of the building, the engineers utilized a grade beam and spread footing combination to help with the bracing of the basement wall shown in the Figure 2 below. This was not used on the east side of the building due to the absence of any underground levels. The spread footings are to be set on soils suitable to hold about 5000psf according to the Geotechnical report.

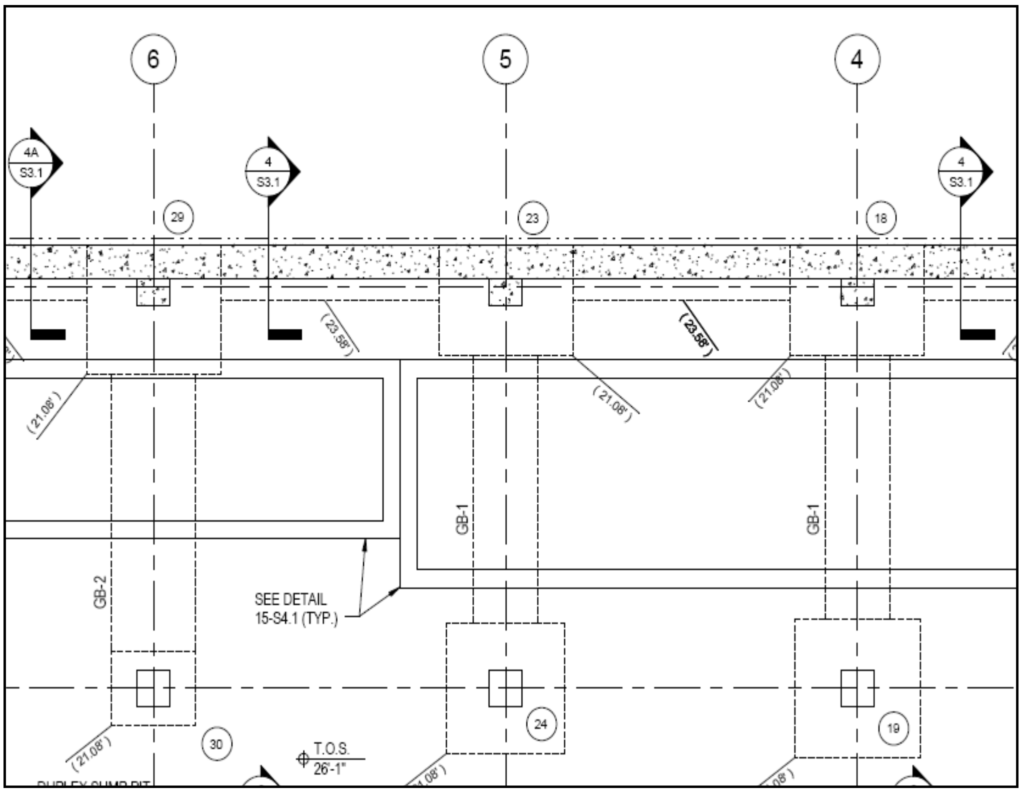

Figure 2: Grade beam and spread footing combination, taken from drawing S1.1

Floor Slabs

The basement level and ground level floor slabs are similar in the fact that they both have a relatively thick floor slab and drop panels comprised of high strength concrete in order to minimize the amount of beams necessary to handle the 21 ft. spans. Once you leave the ground floor, you will find that the slabs change from what was mentioned above to a post tensioned slab system. Also, above the ground floor on the east half of the building, the slabs have large continuous drop panels running between select columns. This type of system extends all the way to the penthouse slab with variations in slab and drop panel thicknesses.

Lateral System

Since the walls of the UHB building are non-load bearing, the lateral loads, due to wind and seismic, must be resolved by the columns and slabs of the building. The dominant lateral system of the UHB is concrete moment frames consisting of the post-tensioned slab and interior/exterior column system. In the case of wind, the load is transferred from the cladding to the exterior columns and slab edge. Then, it is distributed to the interior columns through the slab, and finally, its transferred to the foundation through the columns. The lateral system also utilizes one shear wall located beside the elevator shaft. The shear wall is called out in Figure 2.1.

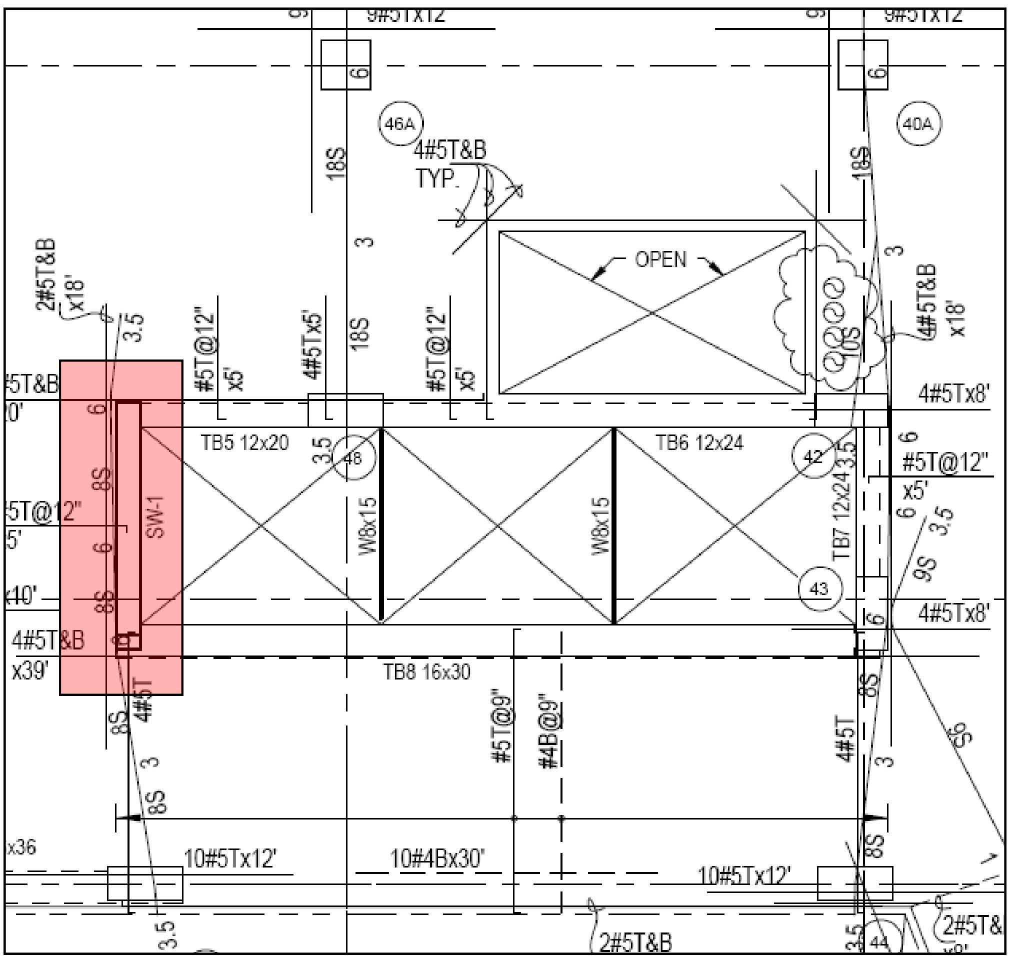

Figure 2.1: Location of shear wall, taken from S1.8

Roof System

The roof system is comprised of two different levels. The first being the lower roof where the green roof is located, and the second is the upper roof that covers the penthouse. The lower roof is a 12-14in. thick post tensioned slab and topped with a green roof system where exposed to the outside. The upper roof is supported by an 8in. post tensioned slab. Also, a portion of the penthouse roof is spanned with steel beams with a glazing system overtop to serve are the skylight for the central stair tower. Figure 3 below shows a partial roof plan showing the integration of the post tensioned concrete slab and central skylight area.

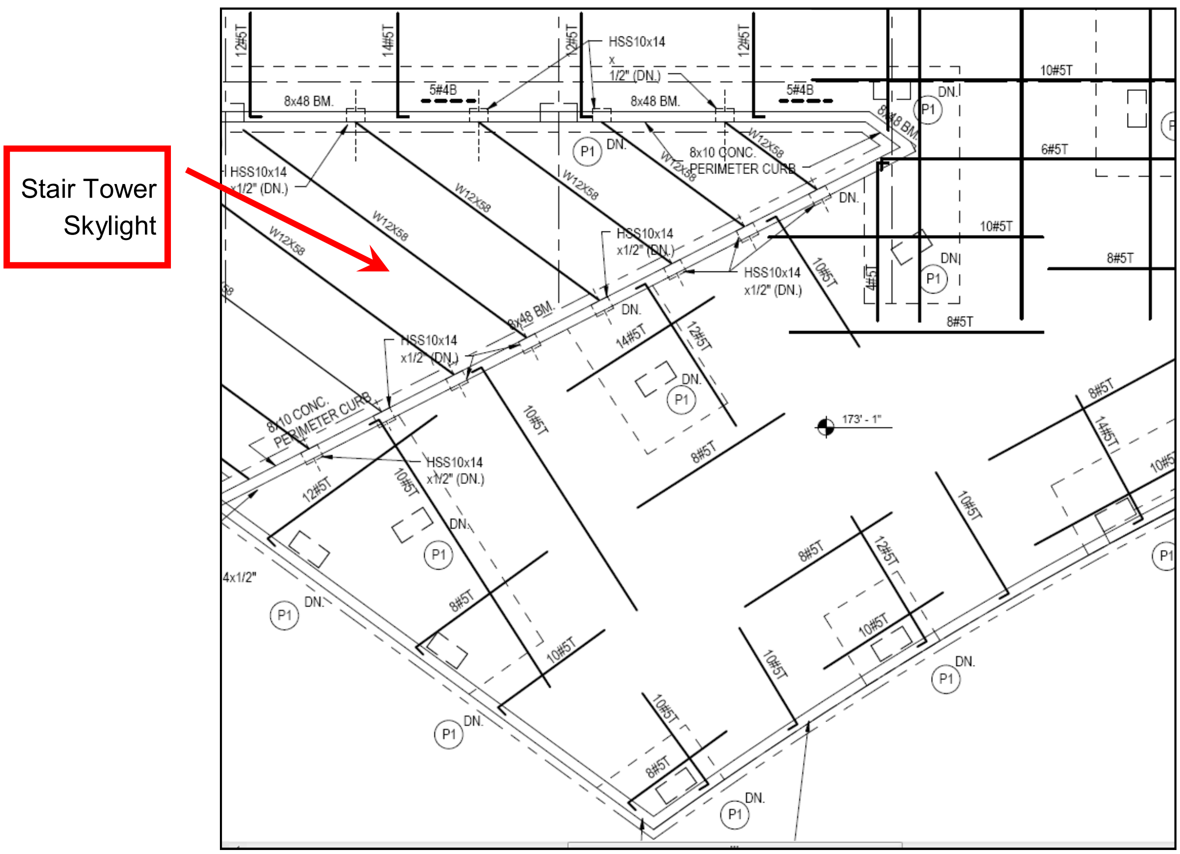

Figure 3: Integrations of both steel and concrete systems on roof, taken from drawing S1.11

Codes & References

Design Codes

- Building Code
  - International Building Code - IBC 2006 system
- Reference Codes
  - American Society of Civil Engineers - ASCE 7-05
  - American Concrete Institute Building Code - ACI 318-05, ACI 530-05, ACI 530.1-05
  - American Institute of Steel Construction - AISC 360-05

Thesis Codes

- Building Code
  - International Building Code - IBC 2009
- Reference Codes
  - American Society of Civil Engineers - ASCE 7-05
  - American Concrete Institute Building Code - ACI 318-08
  - American Institute of Steel Construction - AISC 14th Edition

Design Loads

This thesis project will be conducted using the Load and Resistance Factor Design (LRFD) method as it is quickly becoming the industry standard. Thesis loads were determined using ASCE 7-05 unless a category were not listed specifically. Then, design loads were used in its place. At the time this report was written, it was undetermined what the design engineer used for dead loads. See Figure 4 below to see the comparison between design and thesis loads.

	(psf)	
Live Loads	Design	Thesis
Roof	30	20
Mechanical Penthouse	150	150
Green Roof	35	35
Stairways	100	100
Corridors	100	100
Loading Dock	450	450
Light Storage	125	125
Retail	100	100
Office	80	80
Partitions	20	20

	(psf)	
Snow	Design	Thesis
Ground Snow	30	30
Flat Roof	21	21
Snow Exposure Factor	0.7	0.7
Snow Importance Factor	1	1

	(psf)	
Dead Load	Design	Thesis
MEP Allowance	-	5
Roof material	-	5
Green Roof	-	50
	(pcf)	
NW Concrete	150	150

Figure 4: Summary of Live Snow and Dead loads

Material Strengths

General material strengths were found on S4.9 and are displayed in Figure 5. The general types and strengths can be overridden per special callouts on the floor plans. On many floors, slab strengths are a combination of 6000psi and 8000psi. See Figure 6 and 7 for good examples of the drawings superseding the general strengths. The figures show variations in concrete strength as the building elevation increases and slab thickness increases.

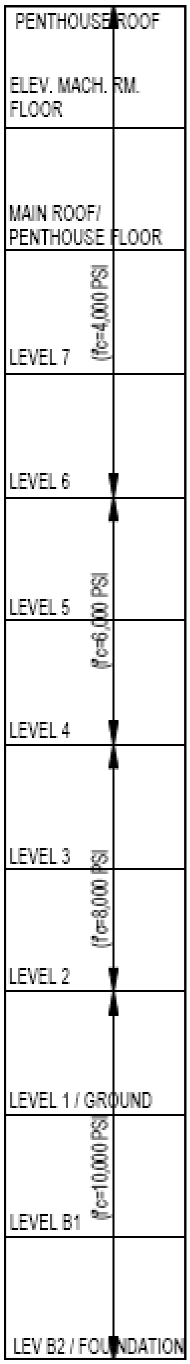

Figure 6: Variations in column concrete strengths per level

Item	Type	Strength
Steel Beams	ASTM-A992	Fy= 50
Post tensioning Tendons	ASTM A-416	Fu= 270
Reinforcement	ASTM-A615	Fy= 60
Masonry	ASTM C-90	f'c=1.5
Grade Beams	NW Conc.	f'c= 4
Column Footings	NW Conc.	f'c= 5
Slab on grade	NW Conc.	f'c= 5
Floor slabs	NW Conc.	f'c= 6
Columns	NW Conc.	See Fig.

Figure 5: Material strength table

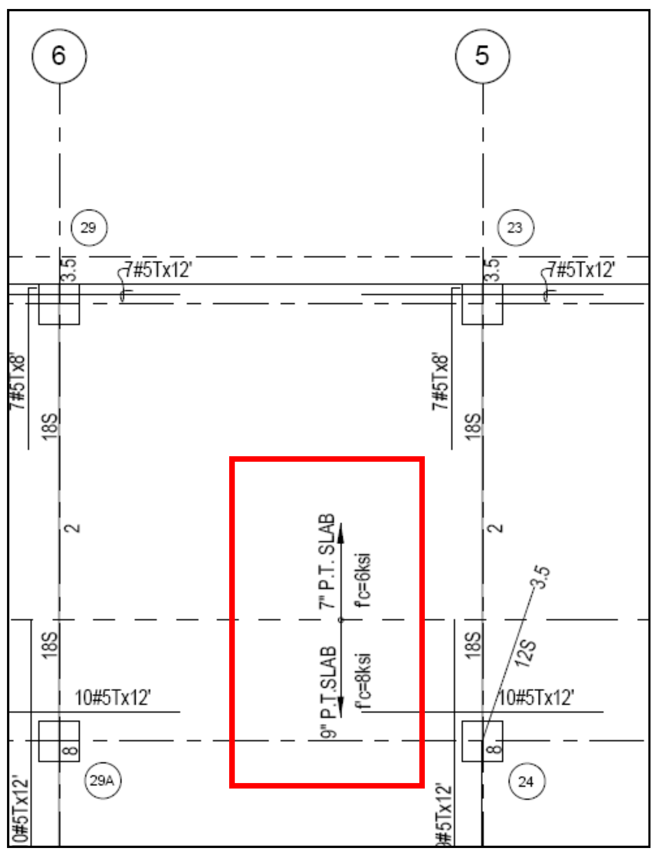

Figure 7: Variations in slab concrete strength

Lateral Loads

Wind Loading

Design wind loads were determined using the Analytical Procedure from Chapter 6 of ASCE 7-05. It was determined that the building should be designed as a Partially Enclosed building with Exposure Category B. The base shear and overturning moment due to wind were calculated to be 302k and 18071ft-k respectively. The base shear was broken down further into a force per story. The per story loading diagram can be seen in Figure 10. To be conservative, when calculating the External Pressure Coefficient (C_p) the Horizontal Distance of the Building parallel to the wind direction (L) was taken from the windward wall to the point on the building furthest from the windward wall. Also, non-linear walls were estimated as the elevation distance of that portion of the building, known as the Horizontal Distance of the Building, perpendicular to the wind direction (B). These assumptions are demonstrated in Figure 8 below. The results for 3 different winds are shown in the tables in Figure 9 below as well as additional calculations in Appendix A.

Wind Loading Summary

All analyses resulted in the same conclusion due to the assumptions made for distances (L) and (B). These calculations are conservative, so designing for these pressures will be sufficient. Further testing in a wind tunnel could provide more information to help reduce these pressures for a less conservative design.

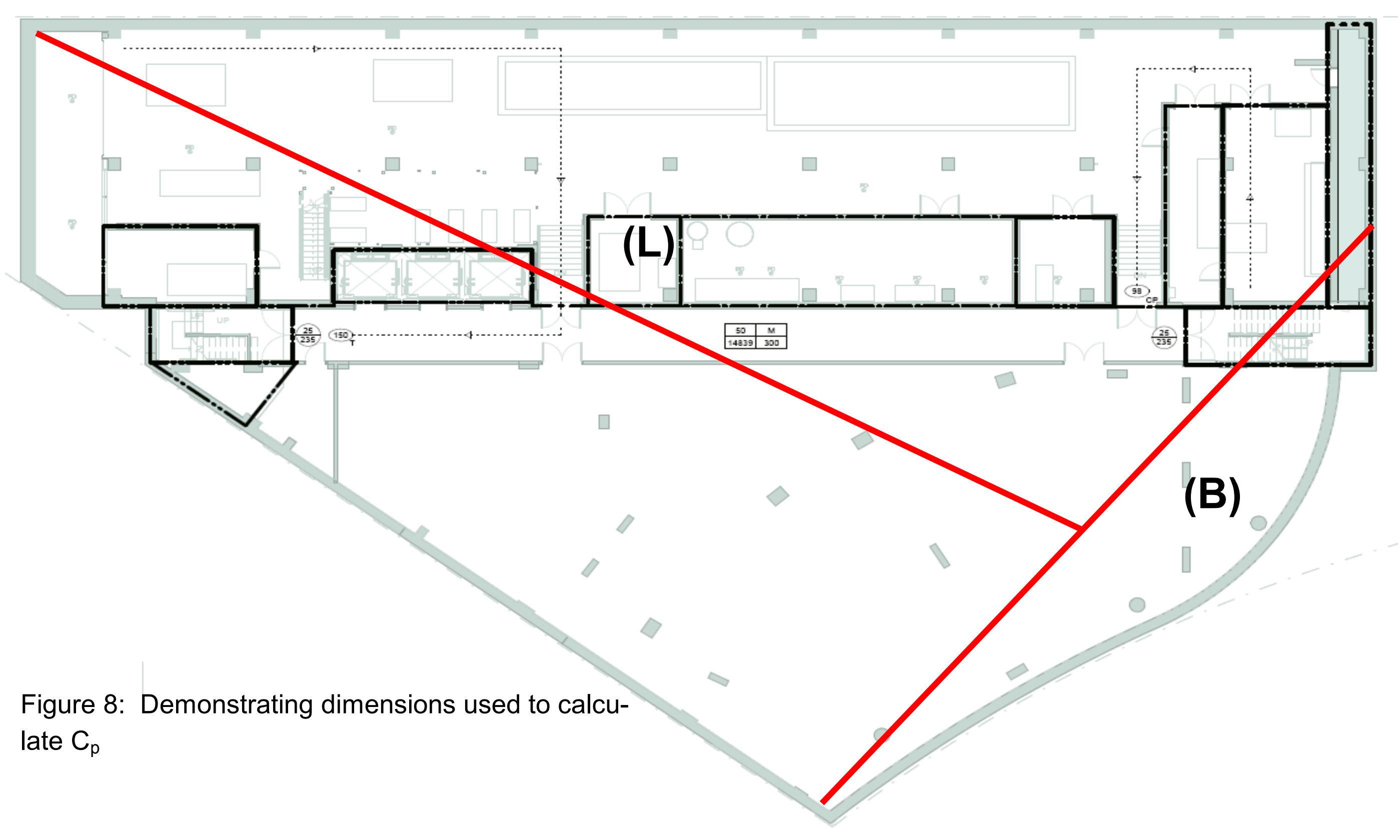

Figure 8: Demonstrating dimensions used to calculate C_p

West Wall							
Story	Height	K_z	q_z	P_w	P_i	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					$\sum$	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Southeast Wall							
Story	Height	K_z	q_z	P_w	P_i	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					$\sum$	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Northeast Wall							
Story	Height	K_z	q_z	P_w	P_i	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					$\sum$	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Figure 9: Windward and Leeward wind force calculation table

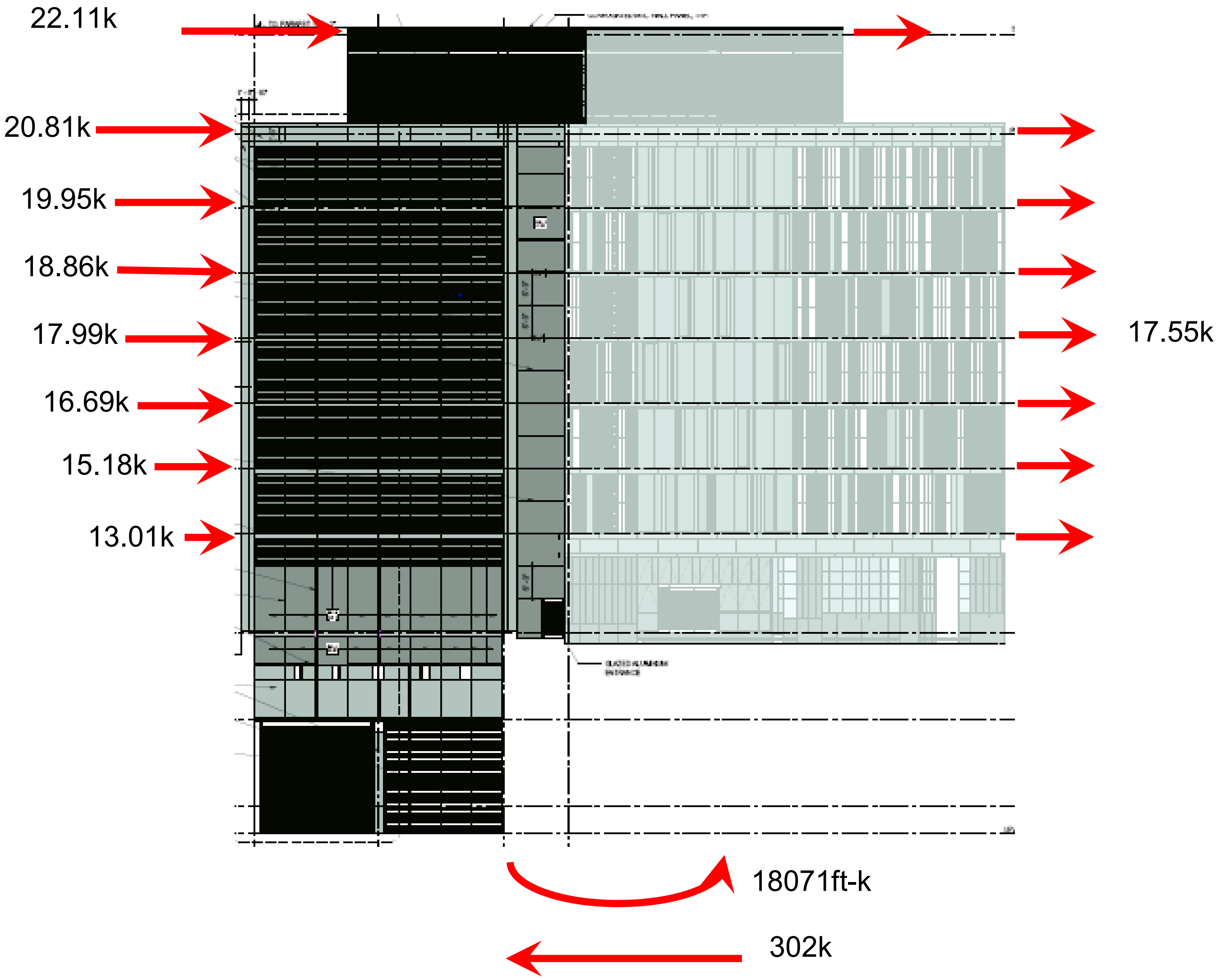

Figure 10: Wind loading diagram

Note: Southeast and Northeast loading diagrams not shown due to similarity to Figure 10 above.

Seismic Loading

Design seismic loads were determined using the Equivalent Lateral Force Procedure from Chapter 12 of ASCE 7-05. Seismic base shear was calculated by first determining building period of vibration and building weight. The base shear was then broken down and shown in Figure 11 on a per floor basis. Figure 12 shows the same per story force in diagram form. The seismic base shear was determined to be 606k and the overturning moment to be 3707ft-k. The large overturning moment is largely due to the thick, heavy green roof and penthouse slab making the building somewhat top heavy. Additional calculations can be found in Appendix B.

Story Forces Calcualtion								
Story	Height (ft)	W (kips)	k	$\sum w_i h_i$	C_{vx}	V (k)	Total Story Force (k)	Overturning Moment (ft-k)
1	0	0	0	1264056.00	0	645	0.00	0.00
2	18	1671	1.00	1264056.00	0.02	645	15.35	276.26
3	30	1707	1.00	1264056.00	0.04	645	26.13	783.92
4	42	1639	1.00	1264056.00	0.05	645	35.13	1475.27
5	54	1640	1.00	1264056.00	0.07	645	45.19	2440.20
6	66	1640	1.00	1264056.00	0.09	645	55.23	3645.23
7	78	1870	1.00	1264056.00	0.12	645	74.43	5805.29
Penthouse	91	3686	1.00	1264056.00	0.27	645	171.16	15575.12
T.O.C. Roof	110	1145	1.00	1264056.00	0.10	645	64.27	7069.43
				$\sum$	0.75		486.87	37070.72

Figure 11: Story Forces Calculation

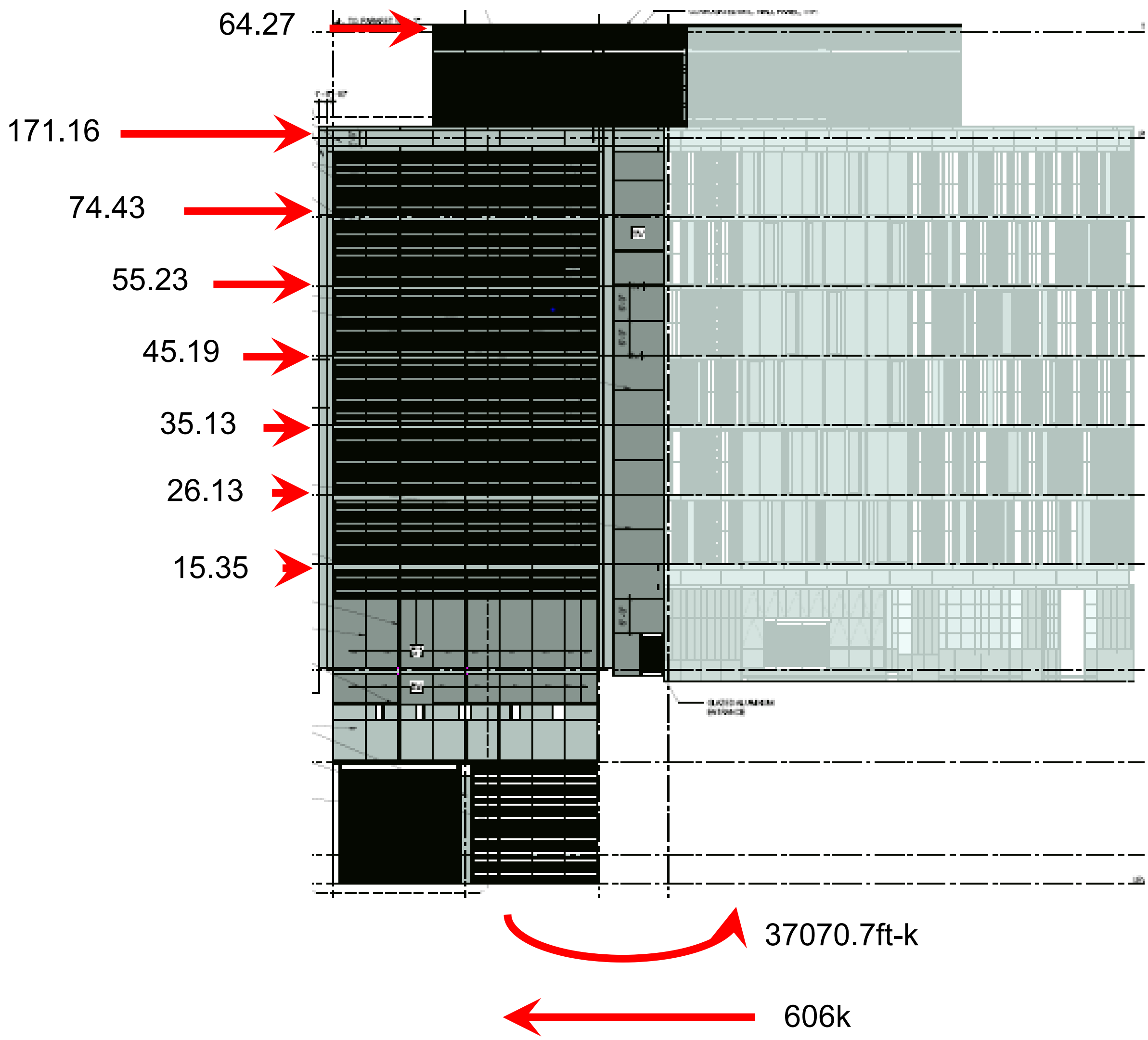

Figure 12: Seismic Loading Diagram

ETABS Model

A model of the UHB was constructed in ETABS in order to analyze the effect of lateral forces, both seismic and wind, on the building. Due to the complexity and shape of the building, hand calculations would be lengthy and very conservative due to the simplifying assumptions that would have to be made. The computer model allows one to analyze a complex structure with ease. A photo of the model can be seen below in Figure 13.

All columns, floor slabs, drop panels, and beams were modeled due to the writers uncertainty of what would be the main lateral force resisting system. The modulus of elasticity for all concrete types was cut in half to account for cracked section properties in the model. This is allowed by ACI 318-08 section 8.8.2. All members were modeled with mass so that the program would be able to calculate the center of mass. This center of mass will later be used as the point at which seismic loads will be applied. The floor slabs were modeled as rigid diaphragms. This means that all points on the slab will move as one. A more accurate representation would have been to use shell elements to model the slab but given the complexity and variations in slab depths of the slab system a ridged diaphragm was a viable simplification.

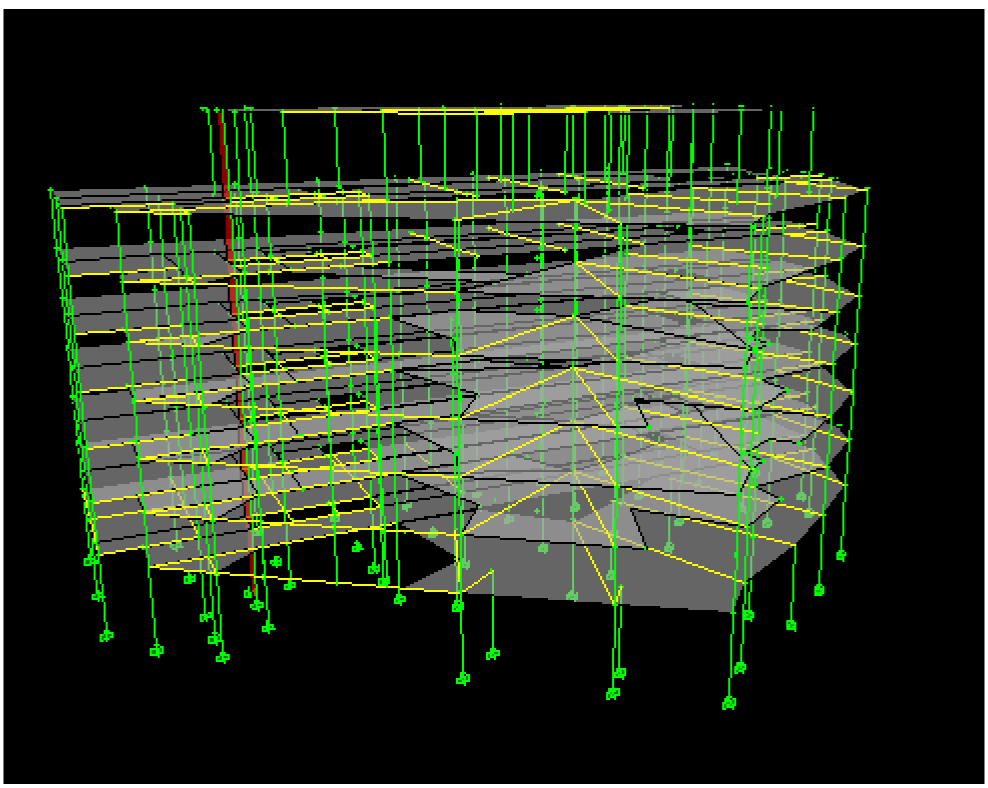

Figure 13: Photo of ETABS model

Load Cases

Strength and design load cases were used from ASCE 7-05. These load cases take into consideration both factored gravity loads and factored lateral loads. The load cases used in this report are as follows:

1.4D

1.2D+1.6L+0.5(Lr or S or R)

1.2D+1.6(Lr or S or R)+(L or 0.5W)

1.2D+1.0W+1.0L+0.5(Lr or S or R) ← Controlling load combination for wind

1.2D+1.0E+L+0.2S ← Controlling load combination for seismic

0.9D+1.0W

0.9D+1.0E

Because this report's basis is to evaluate the lateral system, only pure wind and seismic loads will be applied to the structure. This was deemed viable due to the model not using P-delta effects. If P-delta effects were applied, the gravity loads would have a greater effect on the displacements of the model, but P-delta effects are out of the scope of this report. The wind and seismic cases considered are shown below in Figure 14. The wind cases were derived from the cases specified in ASCE 7-05. Where WMX/Y is the wind force acting at an eccentricity of 15% the length perpendicular to the load.

Wind Load Cases
1.0WX
1.0WY
.75WX + .75WMX
.75WX - .75WMX
.75WY + .75WMY
.75WY - .75WMY
.75WX + .75WY
.563WX + .563WY + .563WMX + .563WMY
.563WX + .563WY + .563WMX - .563WMY
.563WX + .563WY - .563WMX + .563WMY
.563WX + .563WY - .563WMX - .563WMY

Seismic Load Cases
1.0EX
1.0EY
1.0EX + 5% Accidental Torsion
1.0EY + 5% Accidental Torsion

Figure 14: Wind and Seismic load cases used

Relative Stiffness

The lateral loads are distributed throughout the UHB with respect to each element's stiffness relative to all the other element's stiffness. This means that the stiffest element will be loaded with the most force and the second stiffest element will hold the second most load and so forth. Most connections in the UHB are moment resisting, due to the monolithic concrete construction. The designer did not call out the lateral system, the major lateral components needed to be determined. A 1000k point force was placed on the UHB model first in the X-direction and then in the Y-direction. Then, the shear forces in all moment resisting frames were measured. The frames with the greatest shear forces were placed in Figure 15. The location of these frames and shear wall is displayed in Figure 16. The relative stiffness was then calculated for these elements, and the leftover was lumped into the "other" category which represents all other moment resisting elements in the structure.

These relative stiffness percentages were used for each floor of the UHB due to the frames being continuous until the roof level. The sizes of the columns, beams, and slab did not change making this a good representation for all floors. The only changes were that the column's f'c and area of steel decreased with height, but all decreases were relative in all columns. This can also be confirmed when looking at Figure 18.1 where the centers of rigidity are tabulated. There are only slight variations in the center of rigidity on each floor. Note the compass on the next page as these directions will be important when reading the rest of this report.

Relative Stiffness					
X-Direction			Y-Direction		
Element	Force (k)	Relative K (%)	Element	Force (k)	Relative K (%)
MF1	61.57	6.16	MF7	52.57	5.26
MF2	57.23	5.72	MF8	72.54	7.25
MF3	90.47	9.05	MF9	76.78	7.68
MF4	86.54	8.65	MF10	61.47	6.15
MF5	158.24	15.82	MF11	49.20	4.92
MF6	133.58	13.36	SW1	93.24	9.32
Other	412.37	41.24	Other	594.20	59.42
Σ	1000	100	Σ	1000	100

Figure15: Forces and relative stiffness for the main lateral resisting elements in the UHB

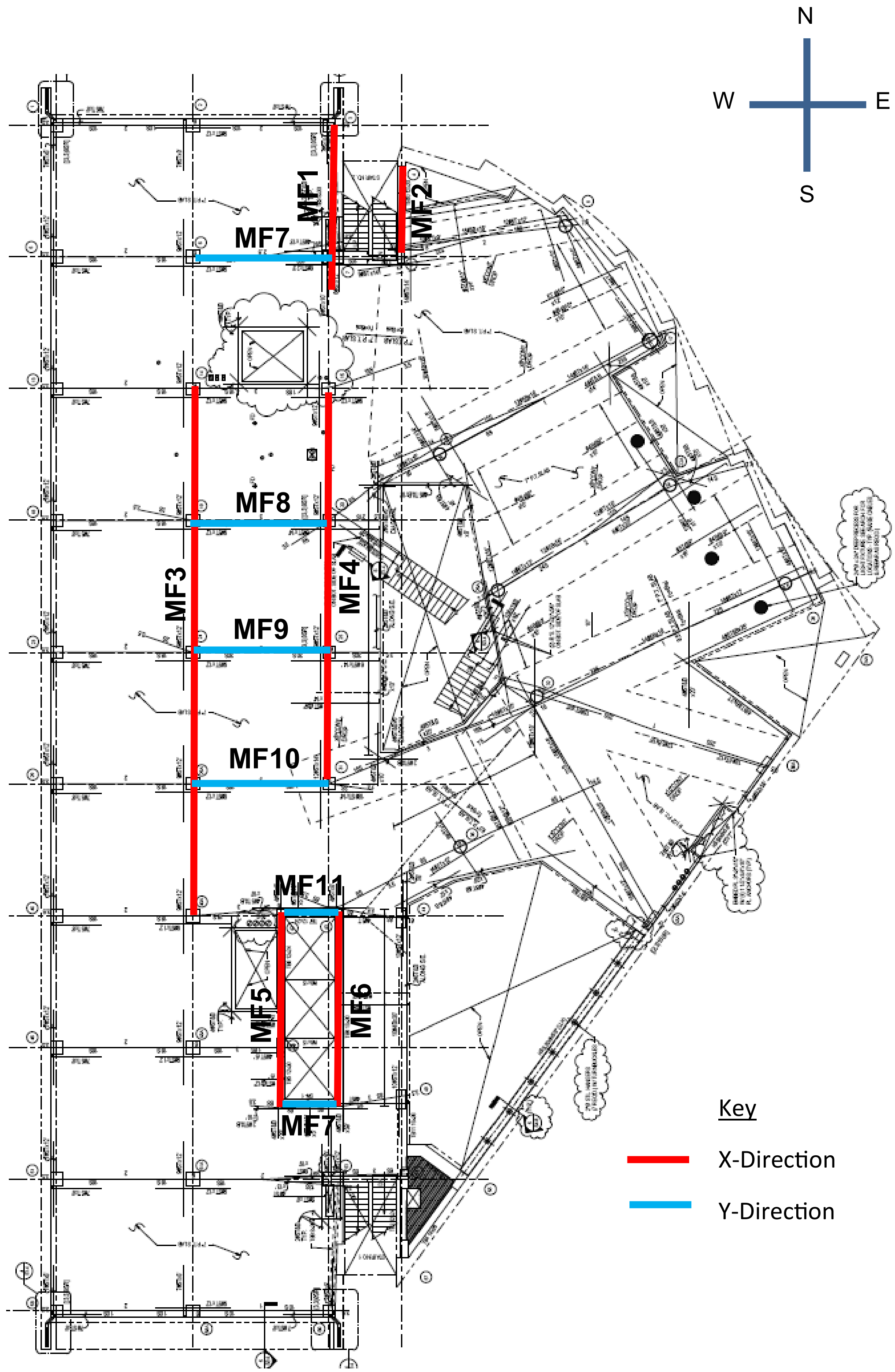

Figure 16: Location of main Lateral elements in the UHB

Building Drift

The amount a building can drift due to seismic is governed by ASCE 7-05 and for wind there is no allowable code limit, but most designers use the rule of thumb of H/400 due to serviceability issues. All previously stated load cases were applied and maximum drifts in two major directions were calculated in Figures 17-18 below.

Story drift ratios were obtained from the ETABS model. Seismic drifts were then to be multiplied by an amplification factor (Cd/I) as noted in ASCE 7-05 section 12.8.6. Ordinary concrete moment frames have a Cd equal to 1.0 and the importance factor for the UHB is 1.25. This generates a number less than one and will not be used but is still shown below. Allowable seismic drift is limited to $0.015h_{sx}$ for other structures with a risk category III.

Seismic Drift: North-South

Floor	Story Height (ft)	Story Drift Ratio (in/in)	Drift Amplification	Story Drift (in)	Allowable Story Drift (in)	Total Drift (in)	Allowable Total Drift (in)	Acceptable
8	18.5	0.003325	0.8	0.738150	3.33	3.1556	19.80	Yes
7	13.5	0.002332	0.8	0.377784	2.43	2.4175	16.47	Yes
6	12	0.002679	0.8	0.385776	2.16	2.0397	14.04	Yes
5	12	0.002801	0.8	0.403344	2.16	1.6539	11.88	Yes
4	12	0.002799	0.8	0.403056	2.16	1.2506	9.72	Yes
3	12	0.002601	0.8	0.374544	2.16	0.8475	7.56	Yes
2	12	0.002089	0.8	0.300816	2.16	0.4730	5.40	Yes
1	18	0.000797	0.8	0.172152	3.24	0.1722	3.24	Yes

Seismic Drift: East-West

Floor	Story Height (ft)	Story Drift Ratio (in/in)	Drift Amplification	Story Drift (in)	Allowable Story Drift (in)	Total Drift	Allowable Total Drift	Acceptable
8	18.5	0.005119	0.8	1.136418	3.33	7.0767	19.80	Yes
7	13.5	0.004674	0.8	0.757188	2.43	5.9403	16.47	Yes
6	12	0.006155	0.8	0.88632	2.16	5.1831	14.04	Yes
5	12	0.007056	0.8	1.016064	2.16	4.2968	11.88	Yes
4	12	0.007261	0.8	1.045584	2.16	3.2808	9.72	Yes
3	12	0.006981	0.8	1.005264	2.16	2.2352	7.56	Yes
2	12	0.005316	0.8	0.765504	2.16	1.2299	5.40	Yes
1	18	0.002150	0.8	0.4644	3.24	0.4644	3.24	Yes

Figure 17: Drift due to seismic loads

Wind Drift: North-South							
Floor	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)	Total Drift	Allowable Total Drift	Acceptable
8	18.5	0.000280	0.06216	0.555	0.843054	3.30	Yes
7	13.5	0.000523	0.08473	0.405	0.780894	2.75	Yes
6	12	0.000740	0.10656	0.36	0.696168	2.34	Yes
5	12	0.000864	0.12442	0.36	0.589608	1.98	Yes
4	12	0.000912	0.13133	0.36	0.465192	1.62	Yes
3	12	0.000928	0.13363	0.36	0.333864	1.26	Yes
2	12	0.000855	0.12312	0.36	0.200232	0.90	Yes
1	18	0.000357	0.07711	0.54	0.077112	0.54	Yes

Wind Drift: East-West							
Floor	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)	Total Drift	Allowable Total Drift	Acceptable
8	18.5	0.001095	0.24309	0.56	2.49525	3.30	Yes
7	13.5	0.001324	0.21449	0.41	2.25216	2.75	Yes
6	12	0.001991	0.28670	0.36	2.03767	2.34	Yes
5	12	0.002513	0.36187	0.36	1.75097	1.98	Yes
4	12	0.002773	0.39931	0.36	1.38910	1.62	Yes
3	12	0.002888	0.41587	0.36	0.98978	1.26	Yes
2	12	0.002388	0.34387	0.36	0.57391	0.90	Yes
1	18	0.001065	0.23004	0.54	0.23004	0.54	Yes

Figure 18: Drifts due to wind loads

Seismic Torsion

The center of mass (COM) and center of rigidity (COR) of the UHB do not coincide. This creates an eccentricity between the seismic load and resisting elements causing a torque on the building. Due to the complexity of building geometries, ETABS was used to calculate the COM and COR of the UBH building. The results and location on a typical floor can be seen in Figures 18.1-18.2. The placement of these points was then checked mentally to make sure it seemed to be in a logical position in the building. This was done in lieu hand calculations due to their complexity. The effects of accidental torsion due to seismic loads must also be considered. This is done by applying the seismic load at an eccentricity equal to 5% of the transverse direction of the loading. The moment due to COM/COR difference (Mt) and moment due to accidental torsion (Ma) are then added to acquire total moment. This calculation can be seen below in Figure 19 for two different directions.

Story	X-COM	Y-COM	X-COR	Y-COR
8	1210.49	872.24	878.65	880.56
7	1204.35	873.86	951.47	853.61
6	1223.38	893.84	899.21	863.20
5	1212.09	872.71	858.29	873.11
4	1228.67	876.33	822.09	879.23
3	1215.32	852.41	777.92	890.79
2	1208.94	869.98	738.17	900.44
1	1072.99	882.57	739.40	904.16

Figure 18.1: Locations of COM and COR per floor

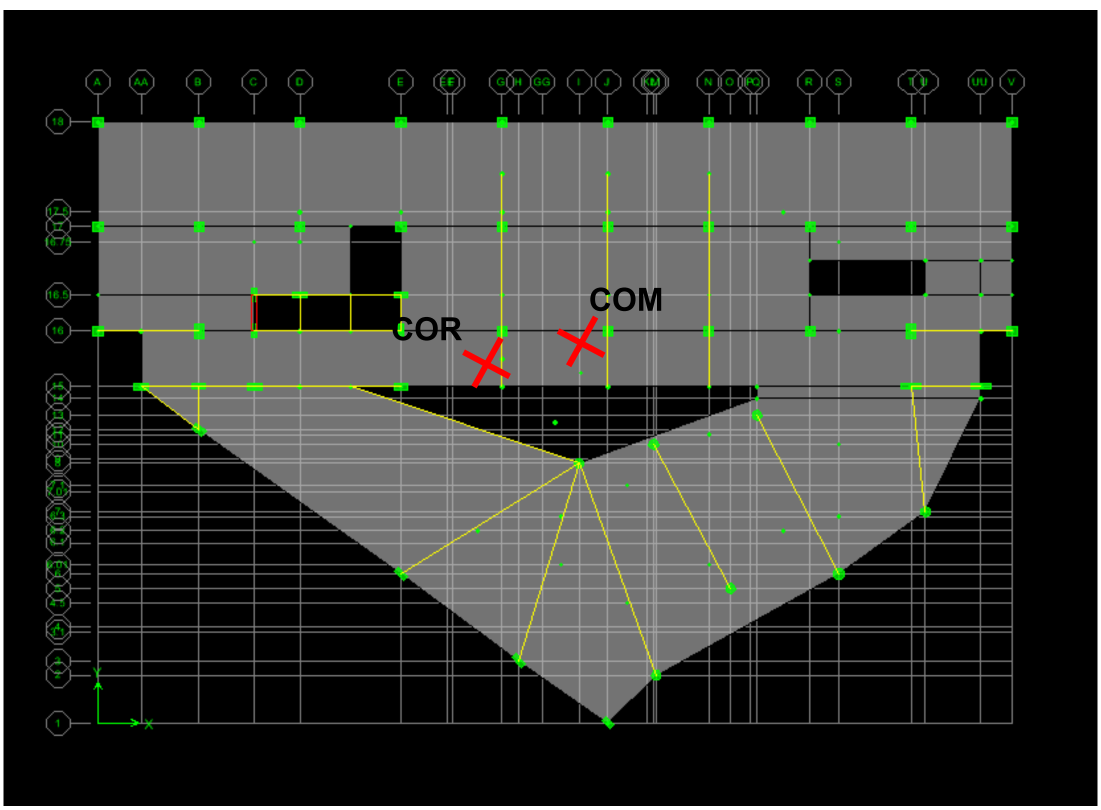

Figure 18.1: Photo from ETABS model story 7 showing COM and COR

Seismic Torsion: North-South							
Floor	Story Force (k)	COM	COR	ey (in)	Mt (k-in)	Ma (k-in)	Mtotal (k-in)
8	46.62	872.248	880.564	8.32	387.69	291.38	679.07
7	186.67	873.456	853.614	19.84	3703.91	1166.69	4870.59
6	92.14	893.219	863.201	30.02	2765.86	575.88	3341.73
5	61.92	871.802	873.113	1.31	81.18	387.00	468.18
4	45.09	875.288	879.227	3.94	177.61	281.81	459.42
3	28.72	851.308	890.786	39.48	1133.81	179.50	1313.31
2	15.44	868.819	900.441	31.62	488.24	96.50	584.74
1	10.39	882.142	904.158	22.02	228.75	64.94	293.68
						∑	**12010.73**

Seismic Torsion: East-West							
Floor	Story Force (k)	COM	COR	ey (in)	Mt (k-in)	Ma (k-in)	Mtotal (k-in)
8	46.62	1210.496	878.658	331.84	15470.29	291.38	15761.66
7	186.67	1204.394	915.471	288.92	53933.26	1166.69	55099.94
6	92.14	1223.378	899.214	324.16	29868.47	575.88	30444.35
5	61.92	1212.085	858.286	353.80	21907.23	387.00	22294.23
4	45.09	1228.673	822.087	406.59	18332.96	281.81	18614.78
3	28.72	1215.316	777.915	437.40	12562.16	179.50	12741.66
2	15.44	1208.935	738.174	470.76	7268.55	96.50	7365.05
1	10.39	1072.993	739.402	333.59	3466.01	64.94	3530.95
						∑	**165852.62**

Figure 19: Calculation of torsion due to seismic loading

Overturning Moment and Foundation

Overturning moment must be checked by designers when considering lateral loads. If the overturning moment is greater than the resisting moment created by the building weight, this situation will create uplift in gravity members and must be considered in design. The greatest overturning moment was determined to be applied when considering seismic loading. The comparison between wind and seismic can be seen in Figure 20 below. A design rule of thumb is that two-thirds of the resisting moment created by the building weight should be greater than the overturning moment for a gravity only foundation to be acceptable. This comparison resulted in the resisting moment to be 842000k-ft which is far greater than the maximum overturning moment of 37773k-ft. Therefore, the gravity only foundation is suitable for the design lateral loads. See Appendix C for calculations.

The overturning moments calculated using ETABS are very similar to the moments that were calculated by hand. This validates using the loads calculated by ETABS. The comparison can be seen in Figure 21.

Overturning Moment						
Story Height	E-W Wind (k)	Moment (ft-k)	N-S Wind (k)	Moment (k-ft)	Seismic (k)	Moment (k-ft)
110	21.57	2372.70	14.87	1635.70	46.62	5128.20
91.5	58.39	5342.69	38.42	3515.43	186.67	17080.31
78	45.20	3525.60	29.73	2318.94	92.14	7186.92
66	41.31	2726.46	27.18	1793.88	61.92	4086.72
54	39.97	2158.38	24.19	1306.26	45.09	2434.86
42	38.39	1612.38	25.26	1060.92	28.72	1206.24
30	36.44	1093.20	22.05	661.50	15.44	463.20
18	42.30	761.40	27.83	500.94	10.39	187.02
	∑	**19592.81**		**12793.57**		**37773.47**

Figure 20: Comparison of overturning moments

	ETABS	Hand Calc.	% Error
Wind OTM	19593	18071	8.42
Seismic OTM	37773	37071	1.90

Figure 21: ETABS vs. Hand Calculations for OTM

Strength Check

In Technical Report I, a spot check was done for an interior column loaded with gravity loads only. This same column was checked again for combined gravity and lateral loads due to what has been determined from the controlling seismic loading. The column was modeled in SPcolumn and an interaction diagram was calculated and shown in Figure 22. The combined loading was then plotted on the diagram confirming the columns design as satisfactory. A strength check was also complete for the single shear wall in the UHB. It was also determined to be satisfactory. See Appendix D for complete calculations.

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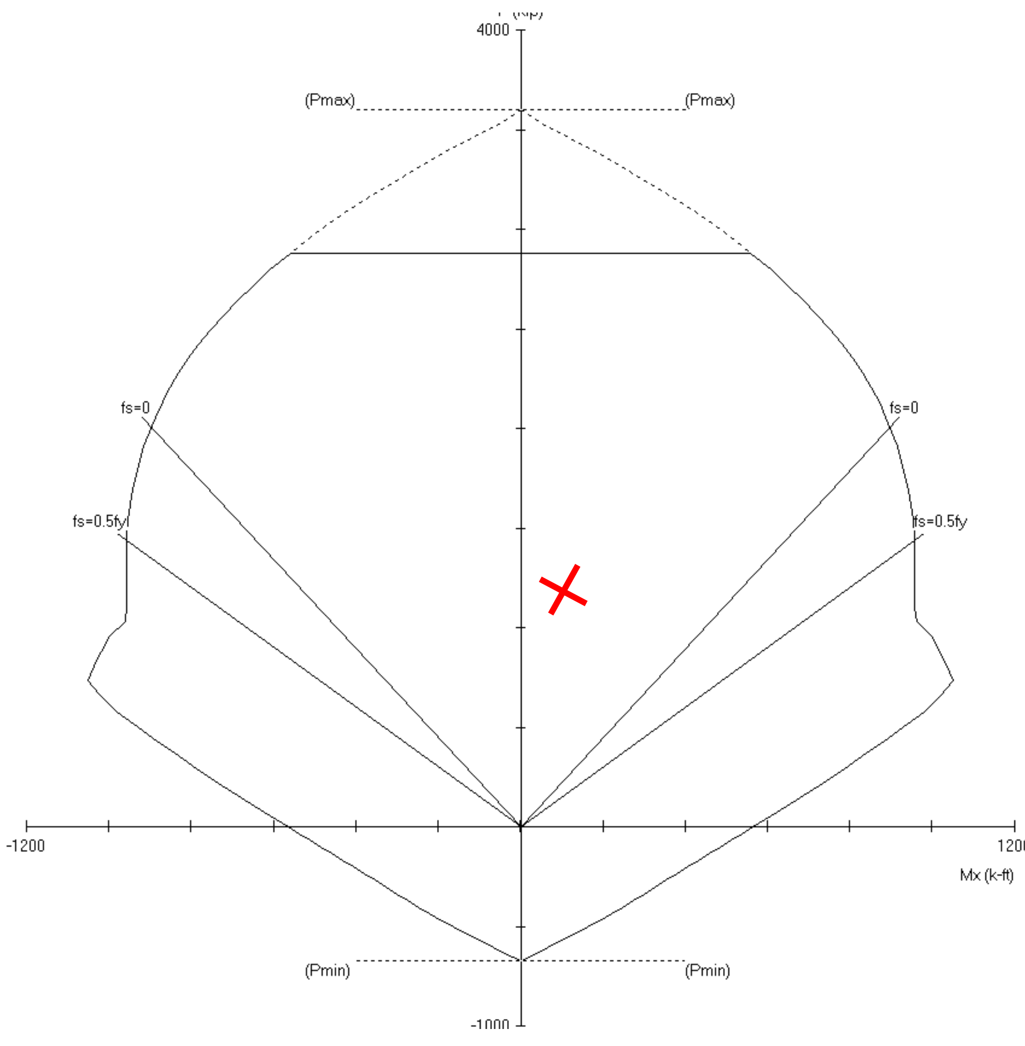


Figure 22: SPcolumn interaction diagram

Conclusion

The scope of this report was to analyze the existing lateral system of the UHB. It was determined that the main lateral resisting system in the UHB are moment resisting concrete frames. A three-dimensional model was then created of the UHB's structural components. In attempt to accurately depict the structure, fixed connections were used at column bases, floor diaphragms were models as ridged, and the concrete's modulus of elasticity was reduced by half to induce cracked sections. The model was also used to determine the UHB's COM and COR. The use of the program made this calculation a lot simpler than trying to manipulate the building's complex geometries by hand.

Load paths and distribution patterns were determined using a 1000k load to calculate relative stiffness of the building's components. Both seismic and wind loads were then applied to the lateral system in various cases to determine maximum drift for each story. Drift was then compared to allowable drifts. The drift comparison found that the UHB met all code and industry standard requirements for drift. It was also determined that seismic is the controlling lateral load, and the building should be designed for this loading. Building torsion due to seismic and overturning moment for the structure was also calculated in the report. The controlling overturning moment was found to be less than two-thirds of the resisting moment. This dismisses any issues with the foundation due to lateral loadings. Lastly, a strength check was conducted on an interior column. It was found that this column meets the loads required.

Appendix A

E. LANDIS | TECH 1 | WIND LOAD

WIND WEST WALL

BUILDING OCCUPANCY = III

$V = 90 mph$

IMPORTANCE FACTOR = 1.0

$GC_{PI} = +0.55, -0.55$

EXPOSURE CATEGORY = B

$K_z = K_h$ = FOR 110ft = 1.02

GUST FACTOR (G) = 0.85

ENCLOSURE CLASS = PARTIALLY ENCLOSED

C_p WINDWARD WALL = 0.8

LEEWARD WALL = $L/B = \frac{130}{204} = .64$

$C_p = -0.5$

$K_{zt} = 1.0$

$K_d = 0.85$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256(1.02)(1.0)(0.85)(90)^2(1.0) = 18 \text{ lb/ft}^2$

DESIGN WIND WIND PRESSURE (MWFRS)

$p = qGC_p - q_z(GC_{PI})$

$P_w = 18(0.85)(0.8) - 18(\pm.55) = 22.14 \text{ psf}$

$P_L = 18(0.85)(-0.5) - 18(\pm.55) = -17.55 \text{ psf}$

SOUTHEAST WALL

C_p WINDWARD = 0.8

LEEWARD = $L/B = \frac{149}{130} = 1.14$

USE $C_p = -0.5$

NORTHEAST WALL

C_p WINDWARD = 0.8

LEEWARD = $L/B = \frac{160}{140} = 1.14$

USE $C_p = -0.5$

Appendix B

EVAN LANDIS | EARTHQUAKE | TECH 1

AMPAD

BUILDING INFORMATION

S_{MS} 0.243
S_{M1} = 0.121

OCCUPANCY = III

SEISMIC IMPORTANCE FACTOR = I = 1.25 (TABLE 1.5-2)

DESIGN SPECTRAL RESPONSE SHORT PERIOD = S_S = 0.15 S_{DS} = 0.16

1s PERIOD = S_1 = 0.050 S_{D1} = 0.08

SEISMIC DESIGN CATEGORY = B

SITE CLASS = D

RESPONSE FACTOR = 3 ORDINARY REIN. CONCRETE MOMENT FRAMES

S_S, S_{DS}, S_1, S_{D1} WERE OBTAINED USING: LAT. 38.8° LONG. -77°

GEOHAZARDS. USGS. GOV/DESIGN MAPS

Pg-89

* SEISMIC DESIGN INFORMATION WILL BE CALCULATED USING ASCE 7-05 EQUIVALENT LATERAL FORCE PROCEDURE

* SEISMIC DESIGN CATEGORY → "B" (TABLE 11.6-1 & 11.6-2)

FUNDAMENTAL PERIOD

$T = C_T h_N^{x}$

$C_T = 0.016$ $x = 0.9$] CONCRETE MOMENT FRAMES TAKING 100% OF LATERAL LOADS

BUILDING HEIGHT = h = 110 ft

$T = 0.016(110)^{0.9} = 1.1s$

$T \le T_L$

$T_L = 8s$ (FIGURE 22-15)

E. LANDIS | EARTHQUAKE | TECH 1

SEISMIC RESPONSE COEFFICIENT

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.16}{(3/1.25)} = .067$$

FOR $T \leq T_L$ $\quad C_s < \frac{S_{D1}}{(R/I)T}$ & $> .01$

$$= \frac{.08}{(3/1.25)1.1} = .03 \checkmark$$

USE .03

BUILDING WEIGHT

2ND FLOOR:

7IN SLAB = 12500 SF   8IN SLAB: 2260 SF

SLAB WEIGHT = $150\left(12500(7/12) + 2260(8/12)\right) = 1319750$ lbs

10" DROP PANEL BEAMS = 2218

$150(2218(10/12)) = 277250$ lbs

MEP ALLOWANCE: $(5\ lb/SF)(12500 + 2260) = 73800$ lbs

2ND FLOOR TOTAL = 1670800 lbs

3RD FLOOR:

7IN SLAB: 13600 SF   9IN SLAB: 1600 SF

10" DROP PANEL BEAMS: 1170 SF

8" DROP PANEL BEAMS: 1060 SF

SLAB/BEAM WEIGHT = $150\left(13600(7/12) + 1600(9/12) + 1170(10/12) + 1060(8/12)\right)$

$= 1631450$ lbs

E. LANDIS | EARTHQUAKE | TECH 1

MEP ALLOWANCE: $(5\ ^{lb}/_{SF})(13600 + 1600) = 76000$ lbs

TOTAL 3RD FLOOR WEIGHT = 1707450 lbs

4TH FLOOR:

7in SLAB = 13000 SF 9in SLAB: 2200 SF

10in DROP = 1100 SF 8in DROP: 1040 SF

MEP ALLOWANCE: $(5\ ^{lb}/_{SF})(13000 + 2200) = 76000$ lbs

SLAB WEIGHT = $150\left(13000\left(\tfrac{7}{12}\right) + 2200\left(\tfrac{9}{12}\right) + 1100\left(\tfrac{10}{12}\right) + 1040\left(\tfrac{8}{12}\right)\right)$

$= 1639150$ lbs

TOTAL 4th FLOOR WEIGHT = 1715150

5TH FLOOR: COMPARABLE TO FLOORS 3 & 4

TOTAL = 1640000 lbs

6TH FLOOR: COMPARABLE TO FLOOR 5

TOTAL = 1640000 lbs

7TH FLOOR:

7" SLAB = 8800 SF 8.5" SLAB = 3600 SF

10" SLAB = 2700 SF 8" DROP = 1500 SF

9.5" DROP = 1300 SF

SLAB WEIGHT = 1794000 lb

MEP ALLOWANE = 76000 lb (TYP)

TOTAL = 1870000 lbs

E. LANDIS | EARTHQUAKE | TECH 1

8TH FLOOR / LOWER ROOF:

14 in SLAB = 2000 SF   12'' SLAB = 13200 SF

10 in DROP = 2450 SF

SLAB WEIGHT = 2636000 lb

MEP = 50 lb/SF (13200 + 2000) = 750000 lb

GREEN ROOF = 50 lb/SF (6000) = 300,000 lb

TOTAL = 3686000 lb

PENTHOUSE ROOF:

8'' SLAB = 9500 SF   8'' DROP = 1000 SF

SLAB = 1050000 lb

MEP = 10 lb/SF (9500) = 95,000 lb

TOTAL = 1145000

464357
464000
134357
336000

BUILDING ENVELOP:

$$(15\ \text{lb/SF})(\underbrace{570\ \text{ft}}_{\text{PERIMETER}} \times \underbrace{110\ \text{ft}}_{\text{BUILDING HEIGHT}}) = 940500\ \text{lbs}$$

COLUMNS:

40 COLUMNS w/ ≈ 576 IN²

h = 110 ft

WEIGHT = 110 ft × 576/144 × 150 × 40 = 2640000 lb

W ≈ 20210 kips

E. LANDIS | EARTHQUAKE | TECH 1

BASE SHEAR

$$V = C_s W$$

$$V = .03(20210) = 606^k$$

BASE SHEAD DISTRIBUTION

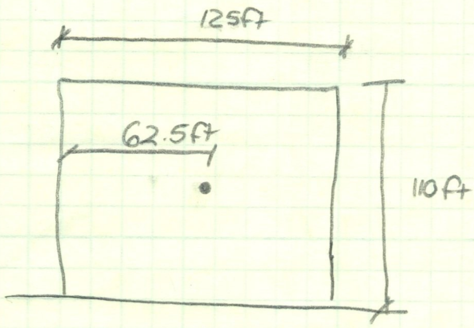
$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$$

$T = .03 < .5 \therefore K = 1$

Appendix C

E. LANDIS | TECH 3 | OVERTURNING



BUILDING W = 20210^k (SEE SEISMIC CALCS)

$$OTM = W(e) = 20210^k(62.5ft) = 1263125\ k\cdot ft$$

$$\tfrac{2}{3}(1263125) = \underline{842083\ k\cdot ft} \geq 37070\ k\cdot ft$$

∴ FOUNDATION NEED NOT CONSIDER LATERAL LOADS

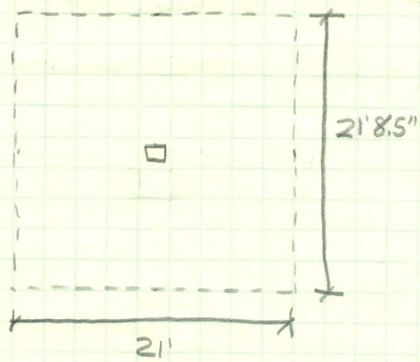
Appendix D

E. LANDIS | TECH 1 | COLUMN SPOT CHECK

COLUMN AT INTERSECTING LINES B & 5

BASEMENT LEVEL B1 COL #25

21
21'8.5"



TRIB AREA CONSTANT FOR ALL FLOORS

= 456 SF

LOADING

FLOOR B1: D_L = 8" SLAB = $150 \times (456 \times 8/12) = 45.6^k$

3½" DROP = $150 \times (8 \times 8 \times 3½/12) = 2.8^k$

24×26 BEAMS = $150 \times (\frac{24}{12} \times \frac{18}{12} \times 21) = 9.6^k$ $\boxed{DL = 65.8^k}$

14×26 BEAMS = $150 \times (\frac{14}{12} \times \frac{18}{12} \times 21) = 5.5^k$

SDL = 5 psf (456) = 2.3^k

LL = 80 psf (456) = $\boxed{36.5^k}$

FLOOR 1: D_L = 8' SLAB = $150(456 \times 8/12) = 45.6^k$

3½" DROP = 2.8^k

24×36 BEAM = $150(24/12 \times 28/12 \times 10.5) = 7.4^k$ $\boxed{DL = 58.1^k}$

SDL = 2.3^k

$\boxed{LL = 36.5^k}$

E. LANDIS | TECH 1 | COLUMN SPOT CHECK

FLOOR 2: DL = 7" SLAB = $150 \times (456 \times 7/12) = 39.9^k$

SDL = 2.3^k

$\boxed{DL = 42.2}$

$\boxed{LL = 36.5^k}$

FLOORS 3, 4, 5, 6 SAME AS 2

FLOOR 7: SAME + 8" DROP DL = $42.2 + 150(15.5 \times 7 \times 8/12) = \boxed{53^k = DL}$

LL = 36.5^k

PENTHOUS FLOOR: DL: 12" SLAB = $150(456 \times 12/12) = 68.4^k$

10" DROP = $150(15.5 \times 9 \times 10/12) = 17.4^k$

GREEN ROOF = $50psf(6 \times 21) = 6.3^k$

SNOW = $30psf(6 \times 21) = 3.8^k$

$\boxed{DL = 95.9^k}$

LL: MECH ROOM = $150psf(15 \times 21) = 47.2^k$

GREEN ROOF LL = $50(6 \times 21) = 6.3^k$

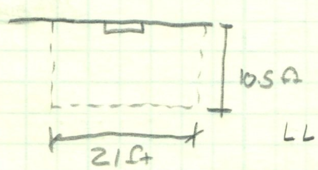
$\boxed{LL = 53.5^k}$

PENTHOUSE ROOF: DL = 8" SLAB = $150(10.5 \times 21 \times 8/12) = 22.1^k$

ROOFING MAT = $5^{psf}(10.5 \times 21) = 1.1^k$

SNOW = $30psf(10.5 \times 21) = 6.6^k$

$\boxed{DL = 29.8^k}$



LL = $20psf(10.5 \times 21) = \boxed{4.4^k}$

LIVE LOAD REDUCTION

$K_{LL} A_T = (4)(456) = 1824 > 400$ ✓

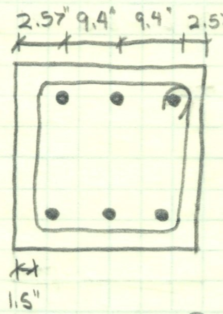
$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 349^k\left(0.25 + \frac{15}{\sqrt{1824}}\right) = \boxed{209^k} \geq .5(349) = 174^k$$

EXCLUDES ROOF LL

E. Landis | Tech 1 | Column Spot Check

Load on column $= 1.6(209.8^{k}) + 1.2(513.6) + .5(10.4)$

$= 957^{k}$



$f'_c = 10{,}000\,psi$

24" x 24"

(6) #11's #3 Ties

Pure Axial

$\phi P_o = \phi\, 0.85 f'_c (A_c) + A_s f_y$

$= (0.65)0.85(10)(576) + (1.56/144)(6)(60) = 3185^{k}$

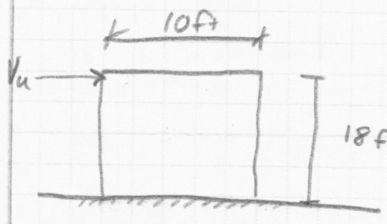
Moment

97 ft·k from ETABS model

EVAN LANDIS | SHEAR WALL

SHEAR WALL STR CHECK

1^{ST} STORY SHEAR WALL



12" THICK
$f'_c = 10{,}000\,psi$
$F_y = 60{,}000$
#6 BARS @ 8"

$V_u = 48^k$ FROM ETABS

MAX PERMITTED STR

$V_u \leq \phi V_n = \phi 10\sqrt{f'_c}\,hd$ $\qquad d = 0.8(10 \times 12) = 96"$

$= .75(10)\sqrt{10{,}000}\,(12")(96")$

$= 864^k$

SHEAR STR

$V_c = 2\sqrt{f'_c}\,hd = 2\sqrt{10{,}000}\,(12)(96) = 230^k$

$V_c \leq 3.3\sqrt{f'_c}\,hd + \cancel{\frac{N_u d}{l_w}}^{\,0} = 3.3\sqrt{10{,}000}\,(12)(96) + 0 = 380^k$

$V_c \leq \left[0.6\sqrt{f'_c} + \left[l_w(1.25\sqrt{f'_c}\right] \Big/ \left[\left(M_u/V_u\right) - l_w/2\right]\right] hd$

$\left[0.6\sqrt{10000} + \left[10 \times 12(1.25\sqrt{10000}\right] \Big/ \left[\frac{10368}{48} - 10 \times 12/2\right]\right](12")(96")$

$M_u = 48 \times 12 \times 18 = 10368^{k \cdot in}$

$= 179^k(0.75) = \phi V_c = 134^k \geq 48^k$ ✓

EVAN LANDIS SHEAR WALL

DEFLECTION

$\Delta = V_u h^3 / 3E_m I + 1.2 V_u h / E_v A$

$I = 12(96)^3/12 = 8.8E5 \text{ in}^4$

$A = 12(12 \times 10) = 1440 \text{ in}^2$

$= 48(18 \times 12)^3 / 3(5700\sqrt{10{,}000})\, 8.8E5 + 1.2(48)(18 \times 12) / 0.4(5700)\sqrt{10000}\,(1440)$

$= 3.21 \times 10^{-5}$ in FOR CANTILEVER

$\Delta = V_u h^3 / 12E_m I + 1.2 V_u h / E_v A$

$48(18 \times 12)^3 / 12(5700\sqrt{10000})(8.8E^5) + 1.2(48)(18 \times 12) / .4\,5700\sqrt{1000}\,(1440)$

$= 4.39E^{-5} \le l/400 = 18(12)/400 = .54 \checkmark$ OK