Technical Report 3

The University Medical Center of Princeton

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Executive Summary

The University Medical Center of Princeton (UMCP) is a seven story, 92’ tall building that services the medical needs for Princeton students and the members of the surrounding community in Plainsboro, NJ. The superstructure is composed of a steel framing system with composite deck, and the lateral system is designed with a combination of braced frames and moment frames.

This technical report is meant to analyze the lateral system of the UMCP building to ensure it has satisfactory strength and serviceability requirements. To help aid with the investigation a 3D model was constructed in ETABS. The whole UMCP Building was not modeled because there is a separation point making the building act as two structures. The greater half of the building was modeled since it holds 68% of the mass and has a bigger length giving the worst case scenario for wind and seismic effects. Also, the structure is designed originally with a long curving rectangle in the East/West (X-direction), but the model was design without the curve making it a rectangle. It is assumed that in disregarding the curve there wouldn’t be a difference in the force load path, and since it is a curve we are not ignoring any force concentration effects.

To analyze the ETABS model there were eight load cases of wind and earthquake forces taken from ASCE7-10 that were applied to the rigid diaphragms on each floor. The diaphragms account for the floor mass of each level. Story drift, member strength, and member stiffness were verified by hand calculation checks of the models outputs. Story drift was the worst for wind case 2 in the North/South (Y-direction) and failed by 0.05 inches, and this was assumed to be negligible since it is so minor and the model isn’t completely accurate. Member strengths were adequate for all load cases applied verified by the hand calculation provided in the appendix. The frame stiffness was determined by applying a 100 kip load to the top of each lateral frame, and dividing the force applied by the lateral displacement of that frame.

Torsional effects were taken into account for the lateral system because the location center of rigidity, the center of mass, and the center of pressure were in different locations. Overturning moment controlled with the wind affect in the North/South (Y-direction). The building resistive moment was greater than the controlling overturning moment; this means the building will not “topple over.”

The controlling Load combination was the Load case two, and is more detailed later in the report. Overall, the UMCP Building lateral system was determined to be suitable for any wind or seismic effects.

Contents

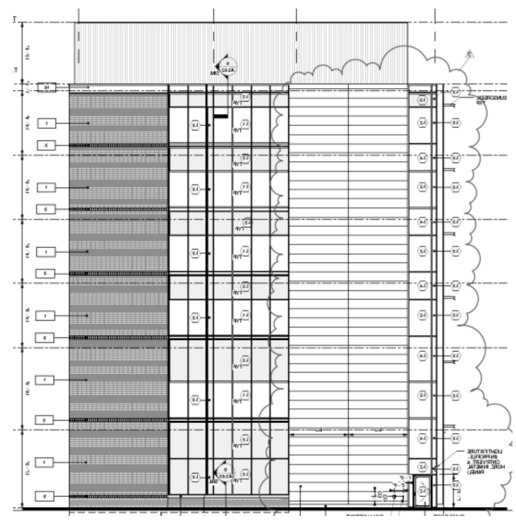
Building Introduction

Princeton University Medical Center was in a big need of change. The rapid growth of people plus the outdated building design and equipment were the main reasons to upgrade their old medical center.

FIGURE 1: UMCP SITE LOCATION SHOWN IN BLUE
SATELLITE PHOTO COURTESY OF GOOGLE MAPS

The University Medical Center at Princeton (UMCP) will also be joining the Pebble Project. Pebble Project is a research effort between The Center for Health Design and selected healthcare providers to measure the layout and design of a hospital and how it can increase quality care and economic performance. The design of this building is not just for looks, but to help operate a hospital in a healthy and efficient manner.

This six story tall building has a long and curving body that encases the parking lot to draw people into the building. Lighting is not going to be an issue during the day as the glass curtain wall is used on the south face of the building. Furthermore, it will provide a view to the outside for all the patients and workers in the building. The curtain wall is framed with aluminum reliefs and metal panels. The West and East elevations have a CMU ground face with a brick façade on the top floors, and there are very few windows since these walls are framed with steel bracing. The mechanical equipment is encased in 13.5’ parapets. Floors two through six almost mimic each other in framing and room layout. The entrance of the building has a wide atrium open to the second floor with interior wood shading panels. The overall design of the building is simple, sleek, and efficient.



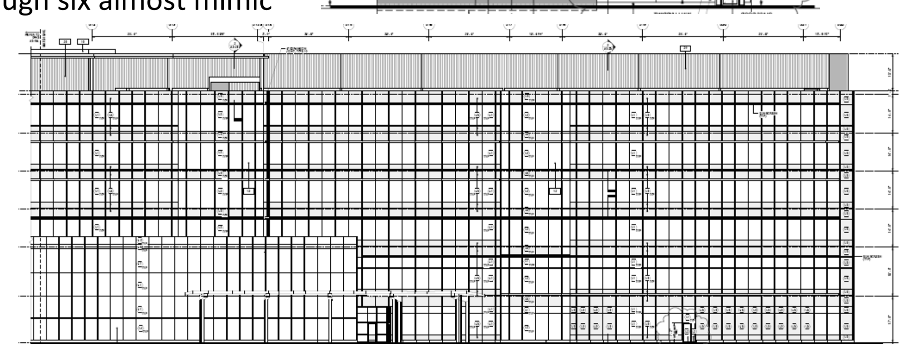


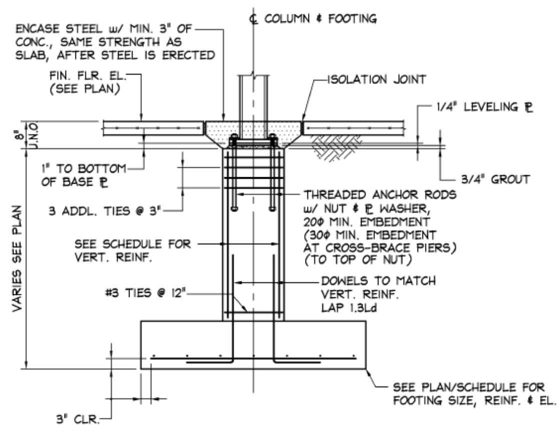
FIGURE 2: EAST AND SOUTH BUILDING ELEVATIONS
DRAWINGS COURTESY OF TURNER CONSTRUCTION

Structural Overview

The foundation plan for the University Medical Center is built on 4” to 5” Slab-On-Grade basement floor with interior concrete piers stabilizing wide flange columns, and an exterior 2’ thick foundation wall partially incasing mini tension piles. The design of the superstructure is primarily steel framing. The framed floors consist of a 3 span 3 ¼” lightweight concrete composite decking system with composite steel framing. Roof decking is type B 1 ½” galvanized metal deck, and 6 ½” normal weight concrete composite metal deck for the roof Penthouse area. There is also a massive curtain wall spanning the South end of the curving building, but this will not be analyzed in this technical report.

Foundations

According to drawing S3.01 all the subgrade footings were poured under the supervision of a registered Soils Engineer. The capacity of the soils, shown in the boring test specifications, came out to be 4,000psf and 8,000psf for the compacted/native soils (medium-dense/stiff) and decomposed bedrock respectively. The spread footings support wide flange columns, varying from W10x54 to W14x311, to anchor the superstructure (Refer to Figure 3 for more detail). The spacing for the foundation columns is not consistent throughout the basement, which that is the reason for the varying column sizes. Figure 3 shows a typical spread footing supporting a steel column. Outlying the basement is a 2’ thick foundation wall with mini tension piles that relives up to 150kips of tension from the concrete bearing wall.



TYPICAL COLUMN FOOTING WITH PIER

FIGURE 3: TYPICAL COLUMN FOOTING WITH PIER
DRAWING COURTESY OF TURNER CONSTRUCTION

Concrete Strengths:

- 3,000psi- Spread Footings, Wall Footings, Foundation Wall, & Retaining Walls
- Minimum of 3,000psi- Piers-match wall strength
- 3,500psi- Slab-On-Grade and Slab-On-Deck

Rebar Design:

- ASTM A615- Deformed Bars Grade 60
- ASTM A185- Welded Wire Fabric

Floor & Framing Systems

A typical beam spanning in the North/South direction, consists of a 26’ span then a 15’ span, and finally back to a 26’ span. The East/West girders span 29 ½’ typically and Appendix 1 helps better understand the layout of the building. Floors two through six do not change in design other than the column thickness, all of the floors use a 3 span 3 ¼" lightweight concrete composite decking. This creates a one-way composite flooring system connected to composite beams. Even though the first floor has an additional atrium, the decking is still consistent to the floors above. Figure 4 shows the wide flange beams used in each span.

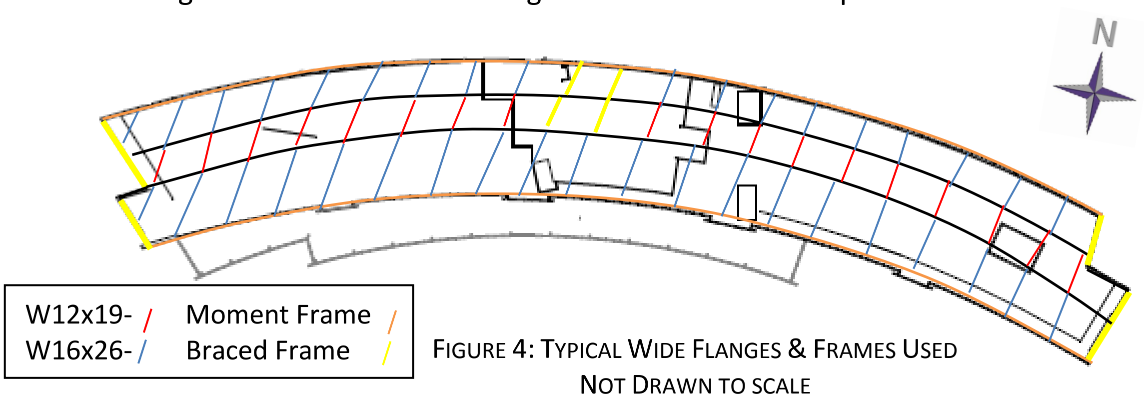


FIGURE 4: TYPICAL WIDE FLANGES & FRAMES USED
NOT DRAWN TO SCALE

The infill beams are usually at a spacing of 9.8' and they range from W16x26 for the 26' spans or W12x19 for the 15' spans. The girders typically span 29.5' and vary from W24x55 on the exterior girders to W21x44 on the interior girders. These composite beams use ¾" bolts to help anchor the decking. The typical bays then come out to be either 29.5'x26' or 29.5'x15'. There are also two transfer beams on the on column lines N2 and S3 to account for columns that do not line up on the first to second floor.

Steel Design:

- ASTM A992- Wide Flanges
- ASTM A500- Rectangular/Square Hollow Structural Sections Grade B, Fy=46ksi
- ASTM A500 or ASTM A53- Steel Pipe Type E or S Grade B
- ASTM F1554- Anchor Rods Grade 55

LATERAL SYSTEMS

The UMCP lateral systems design was comprised of typical steel moment frames in the East/West direction and steel concentrically braced frames in the North and South direction. Those framing systems only occurred on the perimeter of the building. Around the elevator shaft is another place where the design is concentrically braced. The lateral forces will travel into the composite deck, and then through the wide flange beams or HSS braces into the columns to the piers to then dissipate into the ground.

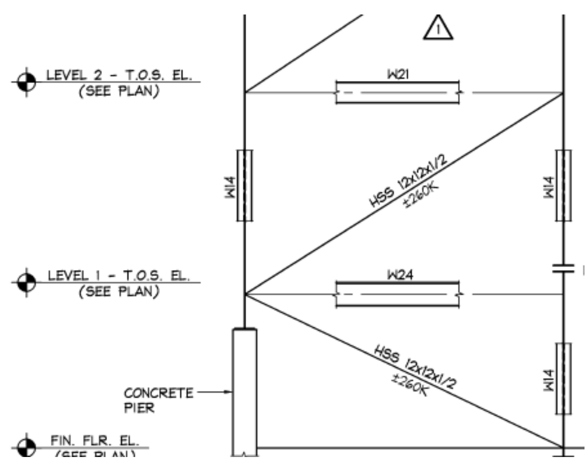


FIGURE 5: TYPICAL BRACED FRAME
COURTESY OF TURNER CONSTRUCTION

CODES/MEANS USED

This building fit into an Occupancy Category III. Any Hospital/Medical Center needs to be designed with an Occupancy Category III as a safety factor.

Original design codes used on this building were:

- 2006 International Building Code (IBC) with New Jersey Uniform Construction Code
- 2006 International Mechanical Code (IMC)
- 2005 National Electric Code (NEC) with local amendments
- 2006 International Energy Conservation Code with other local amendments
- 2006 International Fuel Gas Code with local amendments
- New Jersey Department of Health and Senior Services - "Licensing Standards for Hospitals, N.J.A.C 8.43G" and the 2006 Edition - "Guidelines for Design and Construction of Hospital and Health Care Facilities."

Design codes used for Thesis Calculations:

- ASCE 7-10 Minimum Design Loads for Buildings and other Structures
- American Institute of Steel Construction, 14th Edition AISC Steel Construction Manual
- 2008 Vulcraft Steel Roof & Floor Deck Manual

Gravity Loads

The UMCP structure was designed by O'Donnel & Naccarato, Inc. using the 2006 International Building Code with New Jersey Amendments. For the thesis calculations performed, ASCE7-10 was used to determine the snow, dead, and live loads. Every calculation was performed by using the LRFD method, and in later tech reports these checks will be analyzed on a computer modeling system.

Snow Loads

All the snow load calculations were taken from chapter 7 of ASCE7-10. The only places that needed to be designed for drift were the 13.5' parapets, and the two story tall atrium extension from the South face of the building. Since the parapets are so tall, only one direction was taken into account for the atrium drift because no snow will blow over top of a 13.5' parapet. The drift calculations for the parapet were only taken for the longer direction, East/West, since the snow load would be greater. The flat roof snow load, P_f, came out to be 19.5psf.

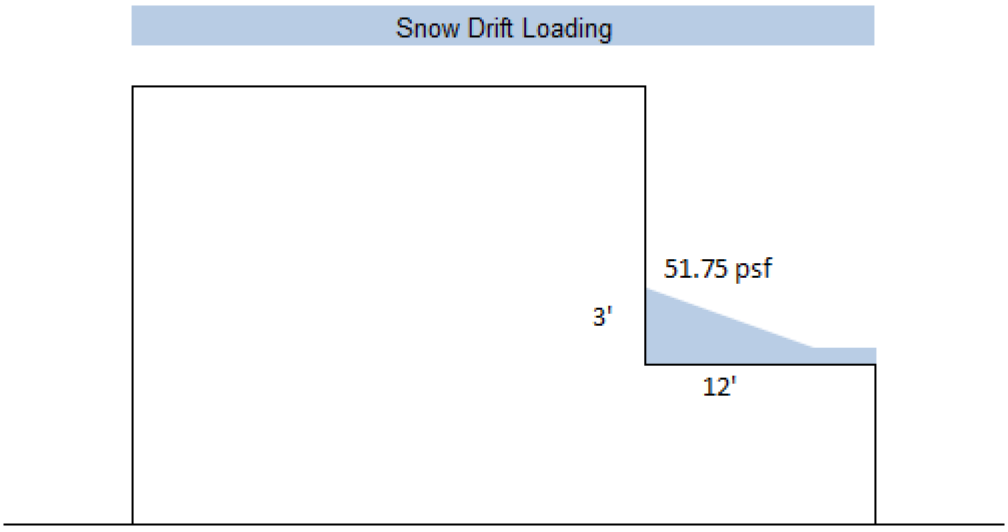


FIGURE 6: SNOW DRAFT LOAD ON ATRIUM ROOF
NOT DRAWN TO SCALE

Dead Loads

The roof dead loads for the mechanical equipment were assumed to be 150psf since there were multiple pieces of equipment weighing more than 15,000 pounds. The metal decking used for the roof did not add too much weight to the roof, only about 1.27psf. A framing allowance for the steel system was assumed to be 10psf for the roof and every other floor. Decking weight for the roof and the composite decking

FIGURE 7: TYPICAL BAY USED FOR SPOT CHECKS
COURTESY OF TURNER CONSTRUCTION

weight for the floors were taken out of the Vulcraft Steel and Roof Decking manual. Though, the decking for UMCP was manufactured by United Steel Inc. The decking was the same for all six floors, and it weighed 39.5psf. The composite beam check turned out to be the same that was designed to. The check for the girder and columns turned out to be a little different, which could be from the assumed weights or also using the newest codes and standards. The girder came out to be a W21x62, but was designed at a W24x62. This difference could be from different design practices and different loads assumed.

LIVE LOADS

Chapter 4 of ASCE7-10 provided the live loads for operating rooms, patient rooms, and corridors above first floor as 60psf, 40psf, and 80psf respectively. For the spot checks the spans crossed to different occupant rooms, so whichever occupancy had the higher live load is the one load that controlled. None of the tributary areas are big enough to use live load reduction factors.

Floor Live loads

Area	ASCE7-10 Loads
Lobby/Corridor 1st Floor	100psf
Corridors above1st Floor	80psf
Operating Rooms	60psf
Patient Rooms	40psf

Wind Loads

For the wind load calculations the MWFRS directional procedure was used to determine the lateral loads and the equations used to perform this method were taken from ASCE7-10 chapter 27. It turned out to be that the UMCP structure is flexible. Since UMCP has such a large area, with a wind speed of 120mph, the wind ended up controlling over the seismic loads. All supporting calculations can be found in Appendix 3.

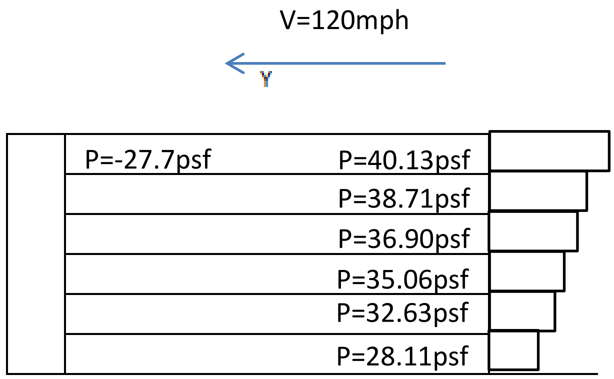
A diagram showing the wind pressure coming from East/West and North/South for those facades is shown below in figure 7 and figure 8. According to ASCE7-10 the parapets also needed to be taken as a separate practice, and are not included in the figures below. Since the UMCP building is curved the structure will catch more wind, but this discrepancy will be better evaluated during the next technical report because it was assumed that the curving face will act like a perfectly horizontal face. Through these calculations, the base shear for the East/West and North/South came out to be 1372kips and 2034kips, respectively. It was proven that the greater the area the more base shear will occur in the building.

Windward Pressure East/West										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	78	0.85	26.63	22.67 (+/-) 4.79	27.46	18.21	27.70	18.37	0
2	17	78	0.87	27.26	23.20 (+/-) 4.91	28.11	225.06	27.70	37.81	17
3	35	78	1.01	31.65	26.94 (+/-) 5.70	32.63	238.88	27.70	34.57	18
4	49	78	1.085	34.00	28.94 (+/-) 6.12	35.06	224.54	27.70	30.25	14
5	63	78	1.142	35.78	30.46 (+/-) 6.44	36.90	236.33	27.70	30.25	14
6	77	78	1.198	37.54	31.95 (+/-) 6.76	38.71	247.92	27.70	30.25	14
roof	91	78	1.242	38.92	33.12 (+/-) 7.01	40.13	21.91	27.71	15.13	14
						Σ	1194.64		178.25	

B	78.00
g_q	3.40
gv	3.40
c	0.20
z(bar)	54.60
L_z	552.98
b(bar)	0.65
ά	0.15
Vz(bar)	123.61
l	500.00
ε	0.20
h	91
L	402
V	120.00

G	1.06
n_s	0.60
I_z	0.18
Q	0.88
N_1	2.69
R_n	0.07
β	0.01
g_R	4.07
$R_{h,n}$	2.04
$R_{L,n}$	30.12
$R_{B,n}$	1.75
R_h	0.37
R_L	0.03
R_B	0.41
R	0.79

Base Shear	Over Turning Moment
1372.89 k	59162.01 k-ft



Windward Pressure North/South										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	78	0.85	26.63	18.85 (+/-) 4.79	23.64	91.94	24.22	94.19	0
2	17	78	0.87	27.26	19.29 (+/-) 4.91	24.20	193.75	24.22	193.91	17
3	35	78	1.01	31.65	22.40 (+/-) 5.70	28.09	205.65	24.22	177.29	18
4	49	78	1.085	34.00	24.06 (+/-) 6.12	30.18	193.31	24.22	155.13	14
5	63	78	1.142	35.78	25.32 (+/-) 6.44	31.77	203.46	24.22	155.13	14
6	77	78	1.198	37.54	26.57 (+/-) 6.76	33.32	213.44	24.22	155.13	14
roof	91	78	1.242	38.92	27.54 (+/-) 7.01	34.55	110.64	24.22	77.56	14
						Σ	1120.24		914.15	

B	457.50
g_q	3.40
gv	3.40
c	0.20
z(bar)	54.60
L_z	552.98
b(bar)	0.65
ά	0.15
Vz(bar)	123.61
l	500.00
ε	0.20
h	91
L	78
V	120.00

G	0.88
n_a	0.60
I_z	0.18
Q	0.78
N_1	2.69
R_n	0.07
β	0.01
g_R	4.07
$R_{h,n}$	2.04
$R_{L,n}$	5.84
$R_{B,n}$	10.24
R_h	0.37
R_L	0.16
R_B	0.09
R	0.39

Base Shear	Over Turning Moment
2034.39 k	59284.17 k-ft

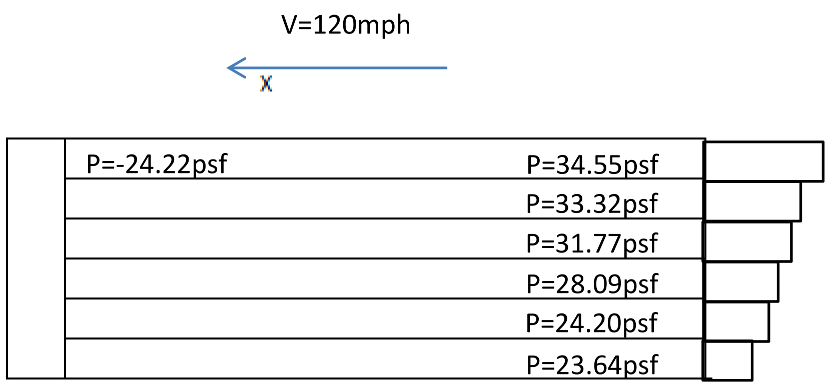


FIGURE 9: EAST/WEST WIND LOAD VARIABLES, LOADS, & PRESSURE DIAGRAM

Seismic Loads

For the seismic design process, ASCE7-10 chapter 12 was applied. The USGS Earthquake Ground Motion Parameter Application was used to find the seismic response coefficients (S_1 and S_s) for Princeton, New Jersey. Since all of the floors have the same floor plans and use the same decking, each floor weighs the same. The roof weighs more due to the fact that the mechanical equipment is so heavy. Also, the response modification factor value, R, changes from 3.25 to 3.5 in the North/South and East/West direction since the framing is moment resisting in the one direction and braced in the other. Figure 9 shows the story shear forces in each direction and the calculations for determining these values are located in Appendix 4.

North-South Direction Loading

T= 0.590 s
k= 1.250
V_b= 461.0377 kips

Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	4270.58	1200297	0.548	253
6	14	77	1510.18	344463	0.157	73
5	14	63	1510.18	268043	0.122	56
4	14	49	1510.18	195782	0.089	41
3	18	35	1510.18	128562	0.059	27
2	17	17	1510.18	52130	0.024	11
		Σ	11821.48	2189277		461

East-West Direction Loading

T= 1.034 s
k= 1.630
V_b= 248.2511 kips

Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	4270.58	1200297	0.548	136
6	14	77	1510.18	344463	0.157	39
5	14	63	1510.18	268043	0.122	30
4	14	49	1510.18	195782	0.089	22
3	18	35	1510.18	128562	0.059	15
2	17	17	1510.18	52130	0.024	6
		Σ	11821.48	2189277		248

19006.73

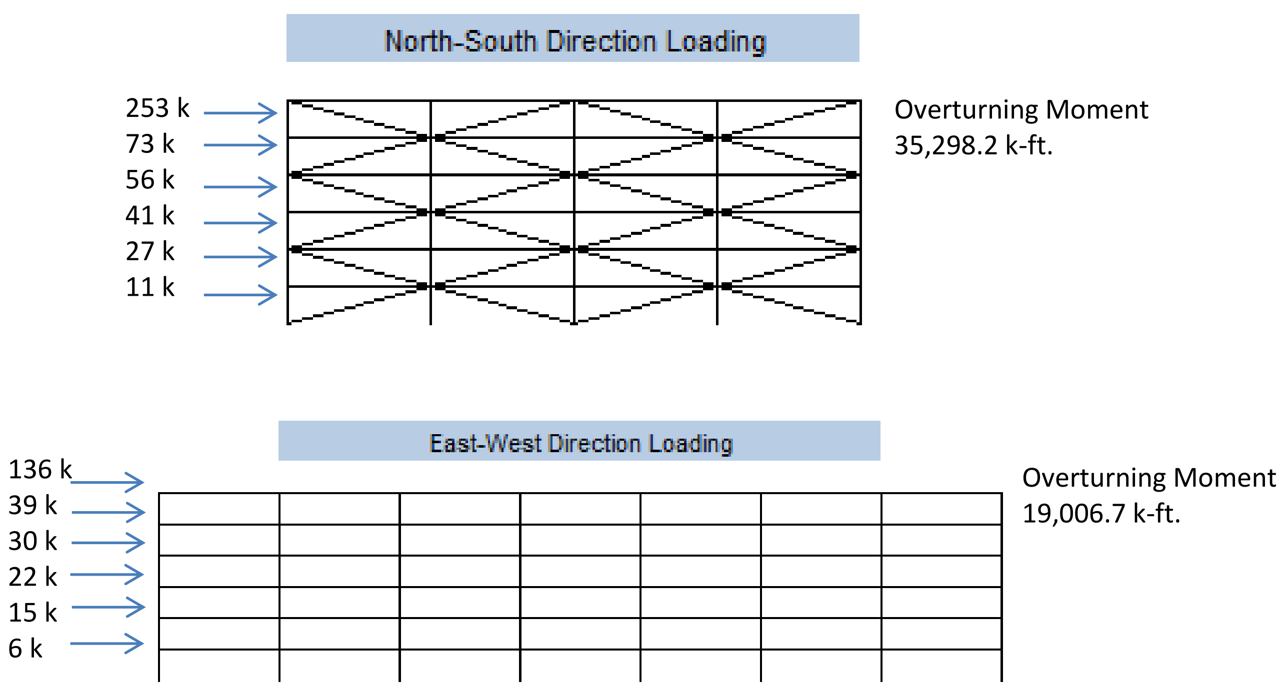


FIGURE 10: NORTH/SOUTH SEISMIC LOAD VARIABLES, LOADS, & FORCE DIAGRAM
NOT DRAWN TO SCALE

In the next section of the report we will be analyzing the later system of the UMCP Building in depth. First stating how the force is transferred through the building and dissipated to the ground, then going through how the ETABS model was constructed, and finally, presenting the analysis of the output forces and drifts taken from ETABS for the lateral system in the UMCP Building accompanied by hand calculations.

Lateral Load Path

There are moment frames spanning the whole length of the outside face of the East and West sides of the building. These moment frames take most of the lateral load in that direction besides the four braced frames in the same direction. The North and South sides of the building are all braced framing, with an additional ten other braced frames sporadically placed throughout the building. The layout of the lateral framing system is shown later in the report in better detail.

The force from the earthquake starts in the ground then shifts the weight of the dead loads on each floor back and forth. The force from the wind starts from the pressure on the façade then disperses into the beams. Since the UMCP Building is very long the building acts as two structures split by a separation joint. In a moment frame the beam takes the load from the wind or earthquake affects and transfers the force into the column and to the next moment frame and eventually is carried down into the ground. In a braced frame the beam and the cross brace share the load, acting like a triangular figure dispersing the force into the column, and from the column the force travels down to the ground. Since the building has a simple geometry there are not many weak links or areas of concern except at the corners of the building.

ETABS Model

To help analyze the lateral systems for the UMCP Building a computer model was constructed Using ETABS. The structure as a whole was examined with a 3D model to determine how the structure would react to eight different load combinations of wind and earthquake forces. The relative stiffness of each lateral framing system was also analyzed.

Only the lateral systems were modeled, so no gravity columns or beams were imported. Also only the bigger part of the building separated by the expansion joint was modeled because this holds the most weight and a greater façade area, which means greater force due to wind and seismic loads. Furthermore, the original shape of the building is a curved rectangular shape, but to simplify the model without changing the structural integrity of the building, the model was constructed without the curve, displayed as a rectangle.

The lateral systems included steel braced frames and moment frames with pin connections at the base of each column. For the steel braced frames, the start and end of the beams and braces were released from their moments to only take axial forces. All of the steel sections, wide flanges and HSS, used in this model were imported from AISCE13. Lastly, rigid diaphragms were drawn to account for the additional masses of each floor.

Hand calculations are accompanied with this model to help provide the accuracy of the computer model, and to check critical members with high axial force and large moments. The image below shows the 3D model used for the analysis.

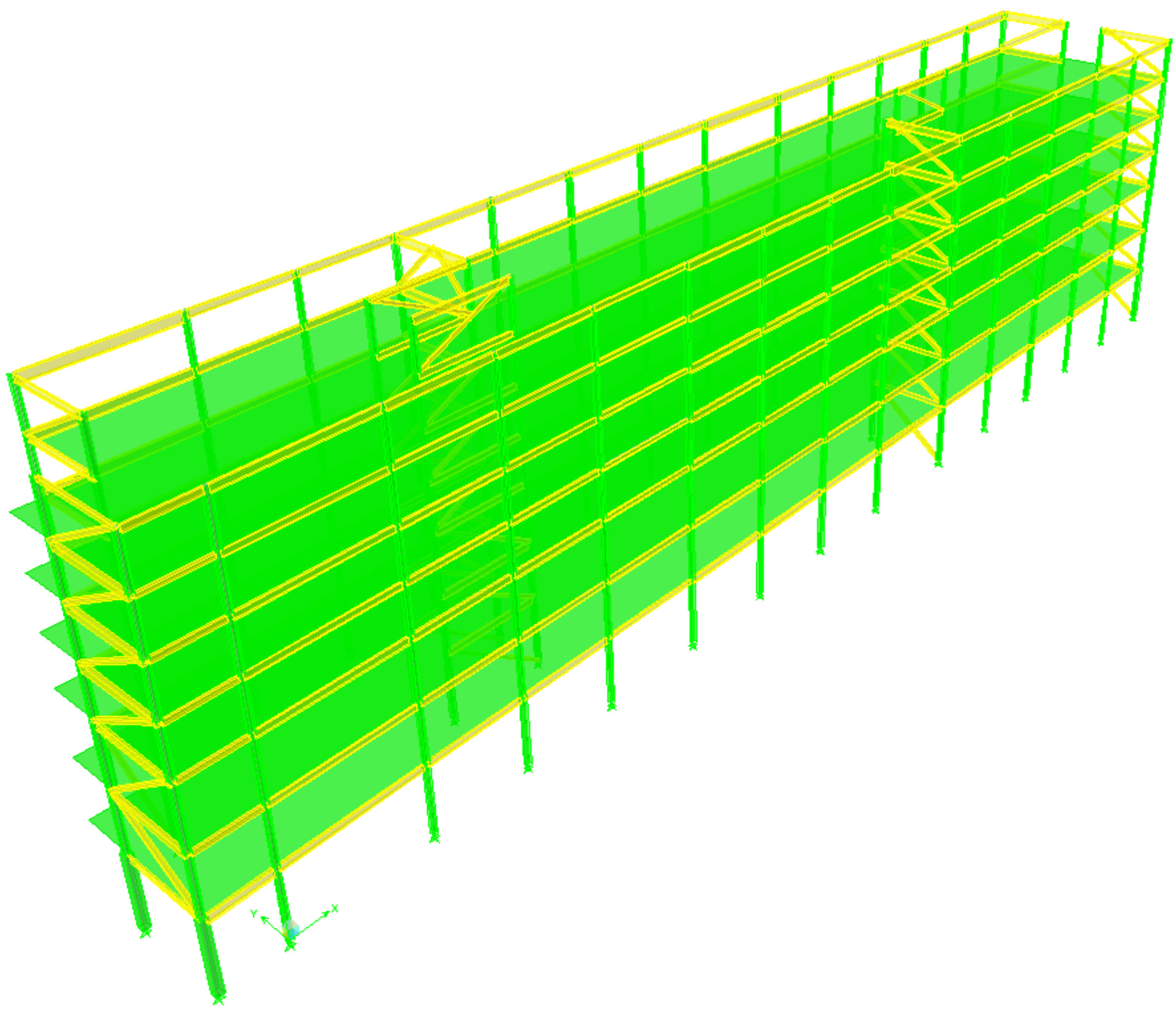


FIGURE 11: 3D ETABS MODEL WITH EXPOSED SIXTH FLOOR SHOWING BRACED & MOMENT FRAME SYSTEMS

Lateral Systems Analysis

LOAD CASES

There were seven different load combinations used from ASCE 7-10 that were applied to the building model in ETABS. The two earthquake load cases are the forces found in the earthquake calculations found preciously in the report in the X and Y direction. In the figure shown below, the four wind cases were taken into account, with two subcases in case 1 and case 2 for the X and Y direction. No gravity loads, besides dead weight of the floor diaphragms, were taken into account since that is irrelevant for a lateral analysis.

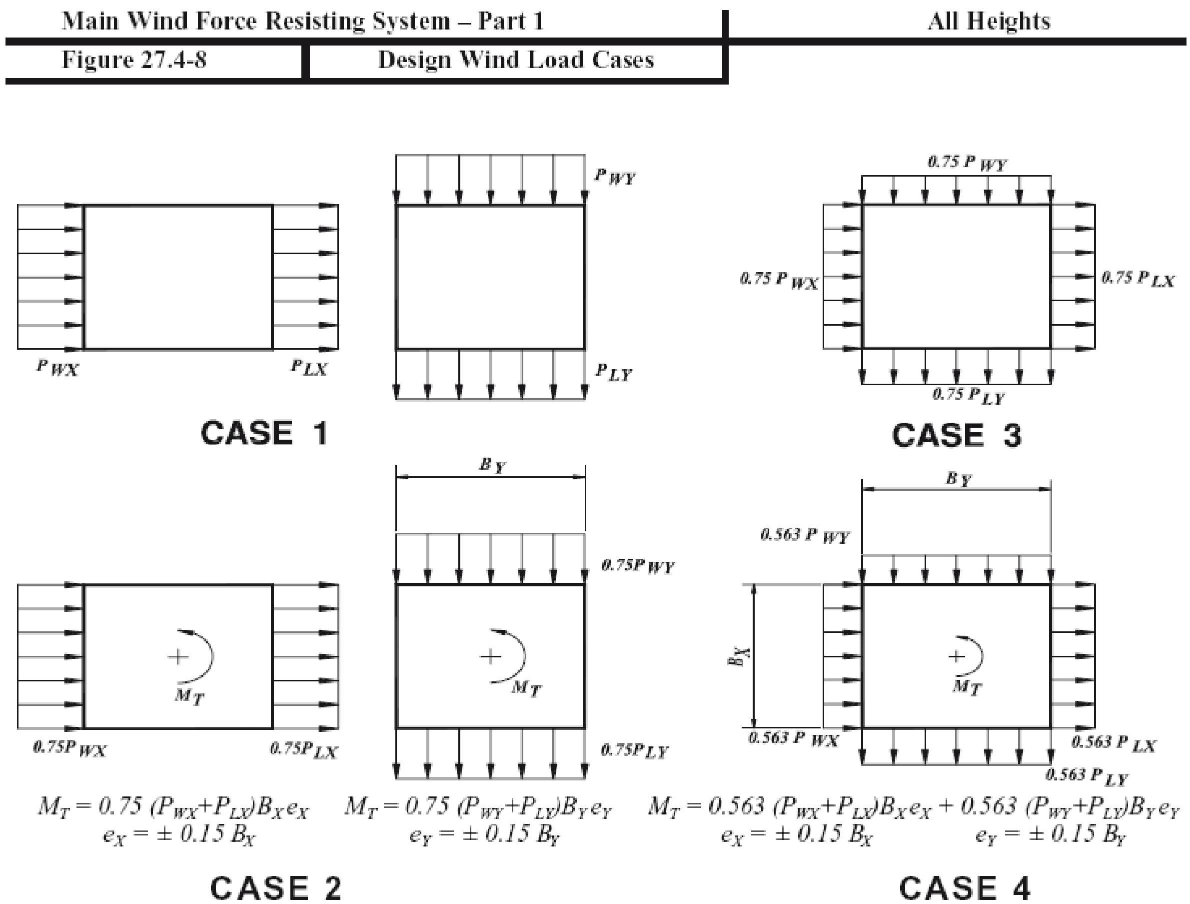


FIGURE 12: MWFRS ASCE7 LOAD CASES

RELATIVE STIFFNESS

The table below shows the relative stiffness calculations of each frame in the North/South (modeled Y-direction) and East/West (modeled X-direction). The relative stiffness is the percent distribution of lateral loads throughout the building. The stiffness was determined by applying a 100 kip load to the top of each frame, and finding the lateral displacement in the direction the load was applied. The formula for relative stiffness is k=P/Δ. Where k is the stiffness, P is the force, in this case 100 kips, and Δ is the displacement. The relative stiffness of a frame is defined by the stiffness of the frame divided by the total stiffness of all the lateral frames, Relative k=ki/Σki. Figure 13 references what relative stiffness applies to what frame.

Relative Stiffness

X-direction	P/d	ki (F/in)	Relative ki
Moment Frame 1:	(100/3.532)*13 bays	368.06	0.2998
Braced Frame 1:	100/2.351	42.54	0.0346
Braced Frame 2:	100/2.256	44.32	0.0361
Braced Frame 3:	100/1.985	50.38	0.0410
Moment Frame 2:	(100/2.7868)*13 bays	466.62	0.3800
	Sum of kix:	971.92	0.7915
Y-direction	**P/d**	**ki (k/in)**	**Relative ki**
Braced Frame 4:	100/8.374	11.94	0.0097
Braced Frame 5:	100/2.067	48.38	0.0394
Braced Frame 6:	100/2.401	41.65	0.0339
Braced Frame 7:	100/1.990	50.25	0.0409
Braced Frame 8:	100/1.894	52.80	0.0430
Braced Frame 9:	100/1.963	50.94	0.0415
	Sum of kiy:	255.96	0.2085
	Sum of All ki:	1227.88	1.00 √

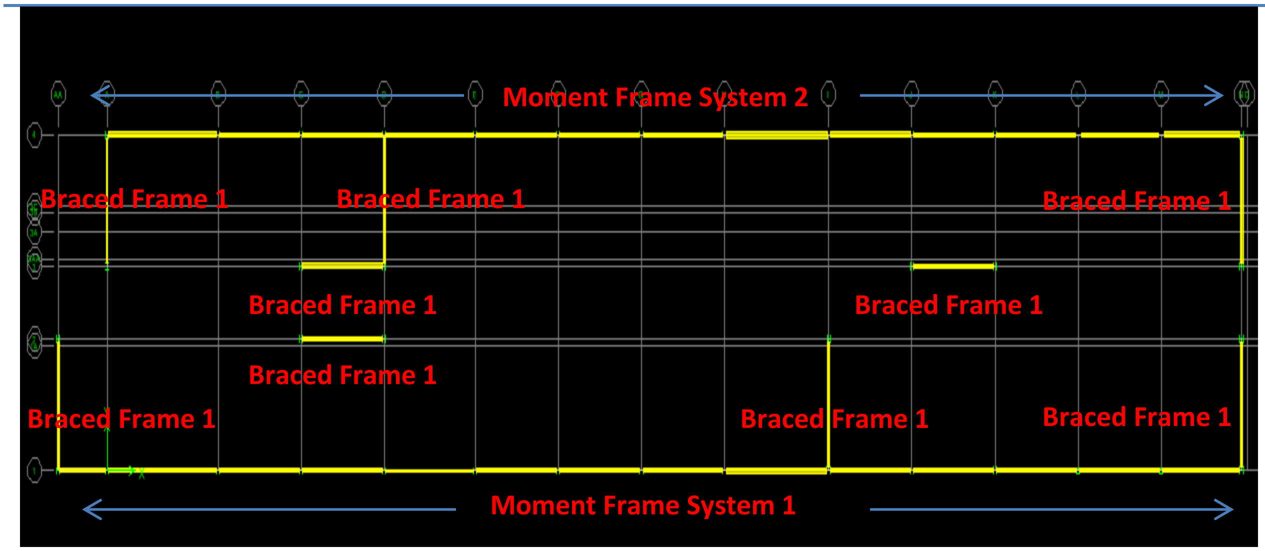


FIGURE 13: FRAME DESIGNATION

Load Distributions

Both the center of mass and the center of rigidity were taken from the ETABS model, and are shown in the table below. Since the locations of the center of mass, center of rigidity, and center of pressure all differ in location causes an eccentric load case, torsional forces must be considered when checking lateral systems. The hand calculations in Appendix 5 check the values in load distribution tables below. The hand calculations were taken for the center of rigidity on story 4. The load distributions found in the table below is the frames percentage of force that it will carry when a direct force or eccentric force is applied.

Center of Mass & Center of Rigidity

	Center of Mass		Center of Rigidity	
Story	Xcm	Ycm	Xcr	Ycr
7	2351.4	409.9	2358.6	553.8
6	2351.4	409.9	2367	541.1
5	2351.4	409.9	2362.9	530
4	2351.4	409.9	2375.1	514.8
3	2351.4	409.9	2404.3	498.9
2	2351.4	409.9	2516.8	470.4
1	2351.4	409.9	2734	444.5

Load Distributions in the X-Directions

	ex	Xi	Yi	dj	Kd^2	Kd	Direct loads	Torsion Loads
MF1	23.7	2415.6	0.00	514.80	97543814.270	189479.049	0.378	0.0268
BF 1	23.7	1003.8	324.00	190.80	1548474.692	8115.695	0.043	0.0011
BF 2	23.7	1003.8	504.00	10.80	5169.525	478.660	0.045	0.0001
BF 3	23.7	3604.2	504.00	10.80	5876.071	544.081	0.051	0.0001
MF2	23.7	2415.6	828.00	313.20	45772617.373	146145.011	0.480	0.0206
BF 4	23.7	207.60	162.00	2582.70	368305.369	30841.892	-	0.0044
BF 5	23.7	0.00	666.00	2375.10	5559054.687	114905.660	-	0.0162
BF 6	23.7	1180.8	666.00	1194.30	2071710.715	49741.774	-	0.0070
BF 7	23.7	3073.2	162.00	698.10	1762834.272	35080.402	-	0.0050
BF 8	23.7	4831.2	162.00	2456.10	6846775.623	129677.930	-	0.0183
BF 9	23.7	4831.2	666.00	2456.10	6373902.941	125119.715	-	0.0177
MF: Moment Frame								
BF: Braced Frame								

Load Distributions in the Y-Directions								
	ey	Xi	Yi	dj	Kd^2	Kd	Direct loads	Torsion Loads
MF1	105	2415.6	0.00	514.80	97543814.270	189479.049	-	0.1184
BF 1	105	1003.8	324.00	190.80	1548474.692	8115.695	-	0.0051
BF 2	105	1003.8	504.00	10.80	5169.525	478.660	-	0.0003
BF 3	105	3604.2	504.00	10.80	5876.071	544.081	-	0.0003
MF2	105	2415.6	828.00	313.20	45772617.373	146145.011	-	0.0913
BF 4	105	207.60	162.00	2582.70	368305.369	30841.892	0.047	0.0193
BF 5	105	0.00	666.00	2375.10	5559054.687	114905.660	0.189	0.0718
BF 6	105	1180.8	666.00	1194.30	2071710.715	49741.774	0.163	0.0311
BF 7	105	3073.2	162.00	698.10	1762834.272	35080.402	0.196	0.0219
BF 8	105	4831.2	162.00	2456.10	6846775.623	129677.930	0.206	0.0810
BF 9	105	4831.2	666.00	2456.10	6373902.941	125119.715	0.199	0.0782
MF: Moment Frame								
BF: Braced Frame								

DRIFT ANALYSIS

All the story drifts in the table below were determined for all the load cases for the wind and seismic forces in the model. Checking wind drift is more for the comfort of the occupants and so no damage occurs on the curtain wall. A seismic drift check is for the safety of the occupants so the building doesn't collapse. All of the drifts for wind loads were compared to H/400, where H is the height of the story in inches. The drifts for the seismic cases were compared to .01*H. Drift is the difference in displacement between a single story. The greatest drift for earthquake load was in the Y direction between story 2 and 3, and the greatest drift for wind was case 2 in the Y direction. The last table checks if the drift meet to code, and the wind is over by 0.05 of an inch. This could be negligible since the building was not perfectly modeled as the original curved shape of the building.

Story Displacements for Each Load Combination

	Wind Analysis						Seismic	
Story	**Case 1:x**	**Case 1:y**	**Case 2:x**	**Case 2:y**	**Case 3**	**Case 4**	**X-dir.**	**Y-dir.**
7	2.54	1.76	1.95	1.13	1.82	1.14	1.185	1.51
6	2.37	1.61	1.81	1.04	1.71	1.03	1.05	1.3
5	2.14	1.42	1.63	0.92	1.54	0.90	0.89	1.08
4	1.84	1.21	1.40	0.78	1.33	0.76	0.71	0.86
3	1.49	0.97	1.34	0.63	1.08	0.60	0.54	0.65
2	1.02	0.68	0.78	0.44	0.75	0.39	0.34	0.39

Story Drift for Each Load Combo

Wind Analysis						Seismic	
Case 1:x	**Case 1:y**	**Case 2:x**	**Case 2:y**	**Case 3**	**Case 4**	**X-dir.**	**Y-dir.**
0.17	0.15	0.14	0.09	0.11	0.11	0.135	0.21
0.23	0.19	0.18	0.12	0.17	0.13	0.16	0.22
0.3	0.21	0.23	0.14	0.21	0.14	0.18	0.22
0.35	0.24	0.06	0.15	0.25	0.16	0.17	0.21
0.47	0.29	0.56	0.19	0.33	0.21	0.2	0.26

Max Wind Drift for Story 2	Max Seismic Drift for Story 2
H/400 (in.)	.01*H (in.)
0.51	2.04
Fails by .05 in.	**Works Fine**

OVERTURNING ANALYSIS

Overturning moments are a result of too great of a wind or seismic forces that would cause the building to flip. The overturning moments were calculated previously for both wind and seismic. If either one of those moments are greater than the moment due from the weight of UMCP building tipping over in the shortest direction, North/South (Y-direction), then the building will overturn. The tables below compare those moments and it shows that the building will not "topple over."

Floor	Floor Weight (pounds)
1	4645835
2	4546532
3	4546532
4	4546532
5	4546532
6	4546532
7	27378496
Total (kips):	**27378**

Resisting Moment				
	Building Weight	(k)		27378
x	Building Width/2	(ft)	x	76/2
=	resisting moment	(k-ft)	=	1,067,761 k-ft
Wind Moment:	**59,284.2 k-ft**	**Less Than**	**1,067,761 k-ft**	**No Overturn**
Seismic Moment:	**35,298.2 k-ft**	**Less Than**	**1,067,761 k-ft**	**No Overturn**

Member Checks

Spot checks were analyzed at a critical location for a cross brace member and for a column. The HSS cross brace in brace frame 5 was chosen because it took on the biggest axial force in the North/South (Y-direction) from wind case 1 obtained by the ETABS model. Wind case 1 in the East/West (X-direction) also produced the biggest force in the bottom column of brace frame 3. A detailed hand calculation for these spot checks can be viewed in Appendix 6.

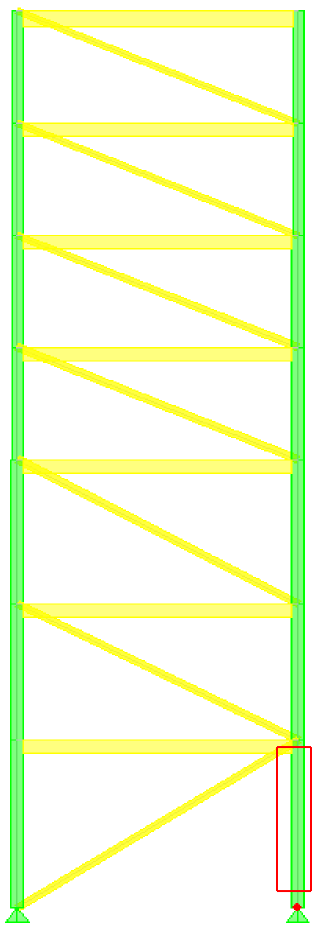
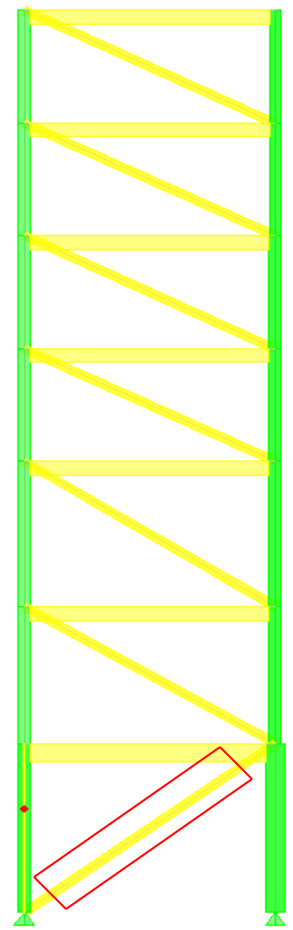
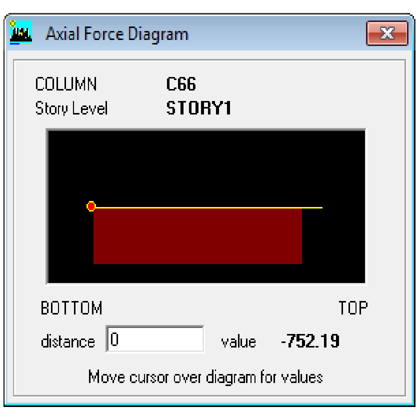
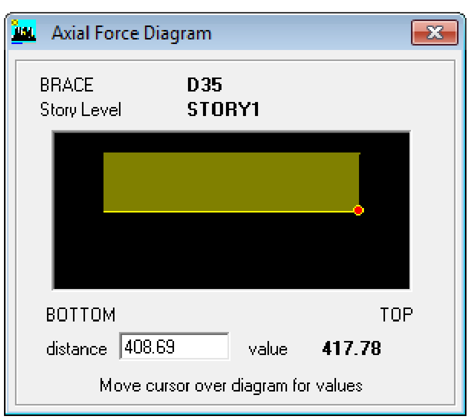


Figure 13: Critical Members







Conclusion

Technical report three confirmed that the UMCP building lateral system was adequate with respect to strength and serviceability. This is proven by the analysis of hand calculations and the 3D ETABS model constructed.

The ETABS model was drawn with the steel sections imported from AISCE13. Also the model was simplified to the larger structure separated from the expansion joint, and the building does not curve although it is assumed it would act the same way as if it was. The braced frames were modeled to have their joints released so that it would only account for axial force.

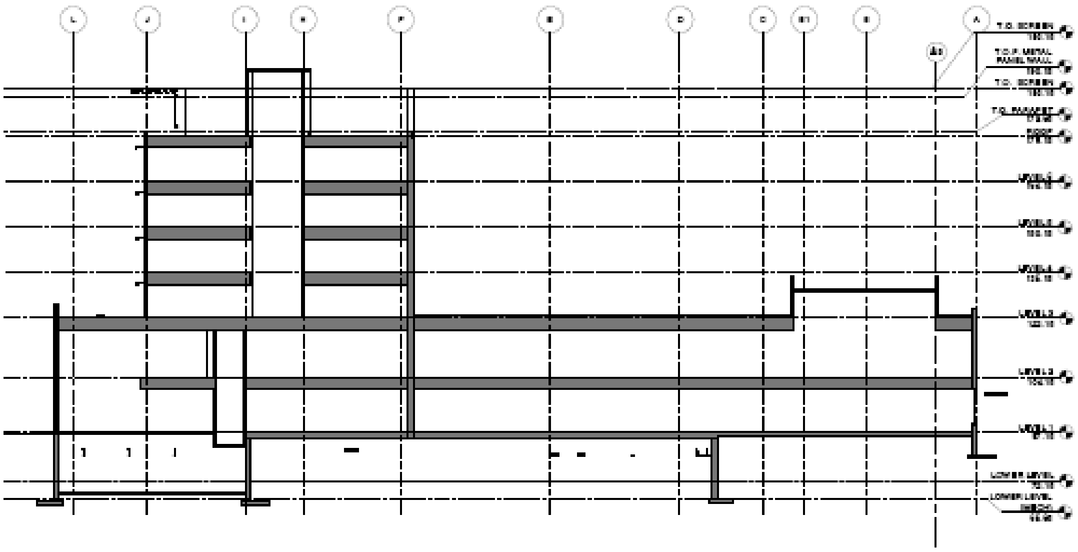
There were 8 load cases assigned to the lateral system of the ETABS model taken from ASCE7-10. Seismic had two load cases in North/South (X) and East/West (Y) directions, and wind had six load cases from figure 27.4-8 in ASCE7-10. After inputting these 8 load cases the computer model output was used to determine relative stiffness, drift of the entire structure, and the loads used to spot check critical members. In the wind load case of the Y-direction, drift failed by 0.05 inches. This could be from simplifying the model too much, and can be safely ignored due to how small it fails by.

The building resistive moment was also found to be adequate for the controlling wind load in the North/South (Y-direction). The critical members chosen from ETABS were determined to be sufficient to carry the loads applied.

The UMCP Building will be later studied with a more in depth model of the actual curved shape, in addition with the smaller part of the whole structure separated by the expansion joint. Overall the lateral system can resist any lateral load force that will act on it.

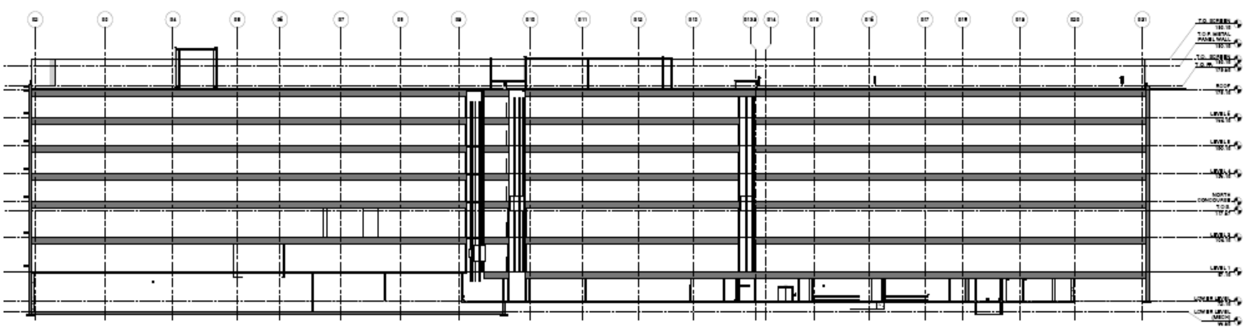
Appendices

Appendix 1: Architectural Sections & Plans



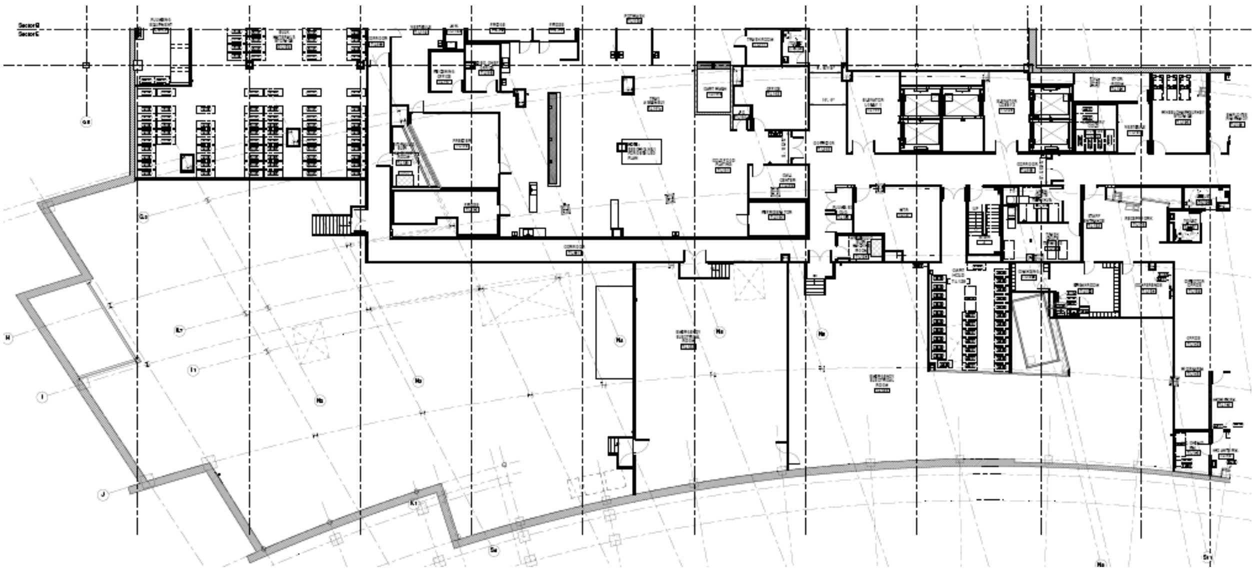
EAST/WEST SECTION

COURTESY OF TURNER CONSTRUCTION



NORTH/SOUTH SECTION

COURTESY OF TURNER CONSTRUCTION



TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

Appendix 2: Snow Load

Alex Burg | Tech. Report 1 | Snow load | 1/2

Flat Roof Snow Load: p_f

$p_f = 0.7\,C_e C_t I_s p_g$

$I_s = 1.10$ -> Table 1.5-2

$C_e = 1.0$ -> Table 7-2 (Exposure C, Partially Exposed Roof)

$C_t = 1.0$ -> Table 7-3

$p_g = 25\ \text{lb/ft}^2$

$p_f = 0.7(1.0)(1.0)(1.10)(25)$

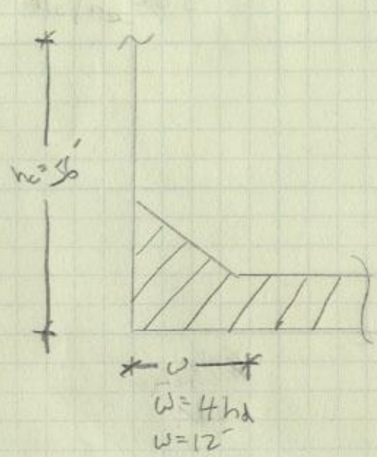
$\boxed{p_f = 19.25\ \text{psf}}$

Building steps back @ floor 3 -> Find Drift

$l_u = 80\ \text{ft}$

$h_d = 0.43(80)^{1/3}(25+10)^{1/4} - 1.5$

$h_d = 3'$ fig 7.9



$\gamma = 0.13 p_g + 14$

$= 0.13(25) + 14$

$= 17.25$

$h_b = p_s/\gamma$

$= 19.25/17.25$

$= 1.1$

$p_d = h_d \gamma$

$= 3(17.25)$

$= \boxed{51.75\ \text{psf}}$

$h_c/h_b = 56/1.1$

$= 51 > 0.2$

∴ Calculate Drift

Alex Burg | Tech. Report 1 | Snow load | 2/2

<u>Drifts for a Parapet</u>

l_u shall be taken as the length of the roof E-W since this is the longest length

The roof is curved as well, but just take the whole length of the roof

$l_u = 451.5'$

$h_d = 0.43\,(451.5)^{1/3}\,(25+10)^{1/4} - 1.5$

$h_d = 6.5' \times 0.75 = 4.9$

$P_d = 4.9\,(17.25)$

$= \boxed{84.53 \text{ psf}}$

Appendix 3: Lateral Wind Loads

Alex Burg | Tech. Report 1 | Wind loads | 1/2

Constants

$V = 120$ mph (Figure 26.5-1B)

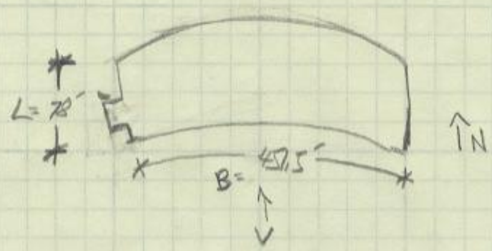
$I = 1.0$ (Table 1.5-2)

$K_d = 0.85$ (Table 26.6-1)

$K_{zt} = 1.0$ (Table 26.8-1)

$GC_{pi} = \pm 0.18$ (Enclosed Building)

Exposure C



Approximate Natural Frequency

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{7098}{91} = 78$$

$h = 91 < 4(78) = 312$ ✓ approximate frequency Applies

$$n_a = \frac{22.2}{h^{0.8}} \quad (26.9\text{-}2)$$

$$n_a = \frac{22.2}{91^{0.8}} = 0.6 < 1 \Rightarrow \text{Flexible}$$

$R_h = 0.37$
$R_L = 0.16$
$R_B = 0.09$

$R_{h,n} = 4.6 n_1 h / \bar{V}_{\bar{z}}$
$= 4.04$

$R_{L,n} = 15.4 n_1 L / \bar{V}_{\bar{z}}$
$= 5.84$

$R_{B,n} = 4.6 n_1 B / \bar{V}_{\bar{z}}$
$= 10.24$

Gust Factor

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$\boxed{G = 0.88}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{54.6}\right)^{1/6} = 0.18$$

Constants

$g_Q = 3.4$
$g_v = 3.4$
$c = .2$ Table (26.9-1)
$\bar{z} = .6h = .6(91) = 54.6$

$$g_R = \sqrt{2 \ln 3{,}600 n_1} + \frac{0.577}{\sqrt{2 \ln (3600 n_1)}}, \quad n_1 = n_a$$

$g_R = 4.07$

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

$Q = 0.8$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$\bar{\epsilon} = 1/3$ $\bar{b} = 0.65$
$l = 500$ $\bar{\alpha} = 1/6.5$
$L_{\bar{z}} = 553$

$$R = \sqrt{\tfrac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$R = 0.39$

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

$\beta = 1\%$ $R_n = 0.07$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.69$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right) V$$

$\bar{V}_{\bar{z}} = 123.61$

Alex Burg | Tech. Report 1 | Wind loads | 2/2

Velocity pressure exposure, K_z ⇒ 1st Floor

$z = 1.242$ (interpolated from table 27.3-1)

$Z_g = 900$

$K_z = 2.01 (15/900)^{2/9.5}$

$\boxed{K_z = 0.85}$

Velocity Pressure, q_z → 1st floor only

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

$= 0.00256 (0.85)(1)(0.85)(120)^2$

$\boxed{q_z = 26.6 \frac{lb}{ft^2}}$

Wind Pressure, p

$p = q G C_p - q_i (GC_{pi})$

$= 26.6 (0.68)(0.8) - 26.6(\pm 0.18)$

$= 14.5 \pm 4.8$

$C_p = .8$ windward 27.4-1
$C_p = -0.5$ leeward wall (long wall)
$C_p = -0.2$ leeward wall (short wall)

See spreadsheet for additional work

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Appendix 4: Lateral Seismic Loads

Alex Burg | Tech. Report 1 | Seismic Load for lateral System | 1/6

$S_s = 0.310$ (USGS) $F_a = 1.552$ 11.4-1 Site Class D

$S_1 = 0.064$ (USGS) $F_v = 2.4$ 11.4-2

$S_{ms} = F_a S_s = 1.552(0.31) = \underline{0.481}$

$S_{m1} = F_v S_1 = 2.4(0.064) = \underline{0.154}$

$S_{DS} = (2/3) S_{ms} = 2/3(0.481) = \underline{0.321}$

$S_{D1} = 2/3\, S_{m1} = 2/3(0.154) = \underline{0.1024}$

Design Category, 11.6-1 => $\underline{B}$

I => 1.25 (Table 1.5-2)

R = 3.25 (Table 12.2-1) Steel ordinary concentrically braced frames N-S Direction

$C_t = 0.02$ $x = .75$ (Table 12.8-2)

$h_n = 91'$

$T_a = C_t h_n^x = 0.02(91)^{.75} = 0.59$

$T_L = 6$ sec (Fig 22-12) $C_u = 1.7$ (Table 12.8-1)

$T = C_u T_a = 1.7(0.59) = 1.003$

$T_L > T$ use $C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.1024}{1.003\left(\frac{3.25}{1.25}\right)}$

$C_s = \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{0.321}{\left(\frac{3.25}{1.25}\right)} = 0.123$

$C_s = 0.039 > 0.01$ ✓

$C_s = 0.039 < 0.123$

So

$C_s = 0.039$

Alex Burg | Tech Report 1 | Seismic Design for lateral System | 2/6

Base Shear

$V = C_s \cdot W$

Assuming That Floors 2 - 6 are equal weight
+ 1st Floor is its own weight

Constants Floor Weights

Slab + Decking = 39 psf   MEP = 15 psf

Steel Beam + Column Allowence = 10 psf

Total Dead Load = 64 psf

Constant Roof Weights

Decking = 1.78 psf   Snow = 19.25

Steel Beam + Column Allowence = 10 psf

MEP Equipment = 150 psf

Total, W_r = 181 psf

Floor + Roof Area

Floors 2 - 6 + Roof are all equal areas

Area = 451.5' × 78' = 35,217 ft^2

$W_{floor} = 35{,}217\ ft^2 \times 64\ \frac{lbs}{ft^2}$

$= 2{,}254^k$

$W_{roof} = 35{,}217\ ft^2 \times 181\ \frac{lb}{ft^2}$

$= 6{,}374^k$

$W_{total} = 6{,}374 + 2{,}254 \times 5$

$= 17{,}644^k$

Alex Burg | Tech. Report 1 | Seismic Design for Lateral Load | 7/6

Base Shear

$V = C_s W$

$V = 0.039(17{,}644)$

$= 600^k$

Vertical Distribution

$F_{roof} = C_{vr} V$

$= 0.55(600)$

$= 330$

$F_6 = 0.16(600)$

$= 96$

$F_5 = 0.12(600)$

$= 72^k$

$F_4 = 0.09(600)$

$= 54^k$

$F_3 = 0.06(600)$

$= 36^k$

$F_2 = 0.02(600)$

$= 12^k$

$C_{vr} = \dfrac{W_{roof}\, h_r^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$

$C_{vr} = \dfrac{6{,}374(91)^{1.25}}{3{,}267{,}578}$

$= 0.55$

$C_6 = \dfrac{2{,}254(77)^{1.25}}{3{,}267{,}578}$

$= 0.16$

$C_5 = \dfrac{2{,}254(63)^{1.25}}{3{,}267{,}578}$

$= 0.12$

$C_4 = \dfrac{2{,}254(49)^{1.25}}{3{,}267{,}578}$

$= 0.09$

$C_3 = \dfrac{2{,}254(35)^{1.25}}{3{,}267{,}578}$

$= 0.06$

$C_2 = \dfrac{2{,}254(17)^{1.25}}{3{,}267{,}578}$

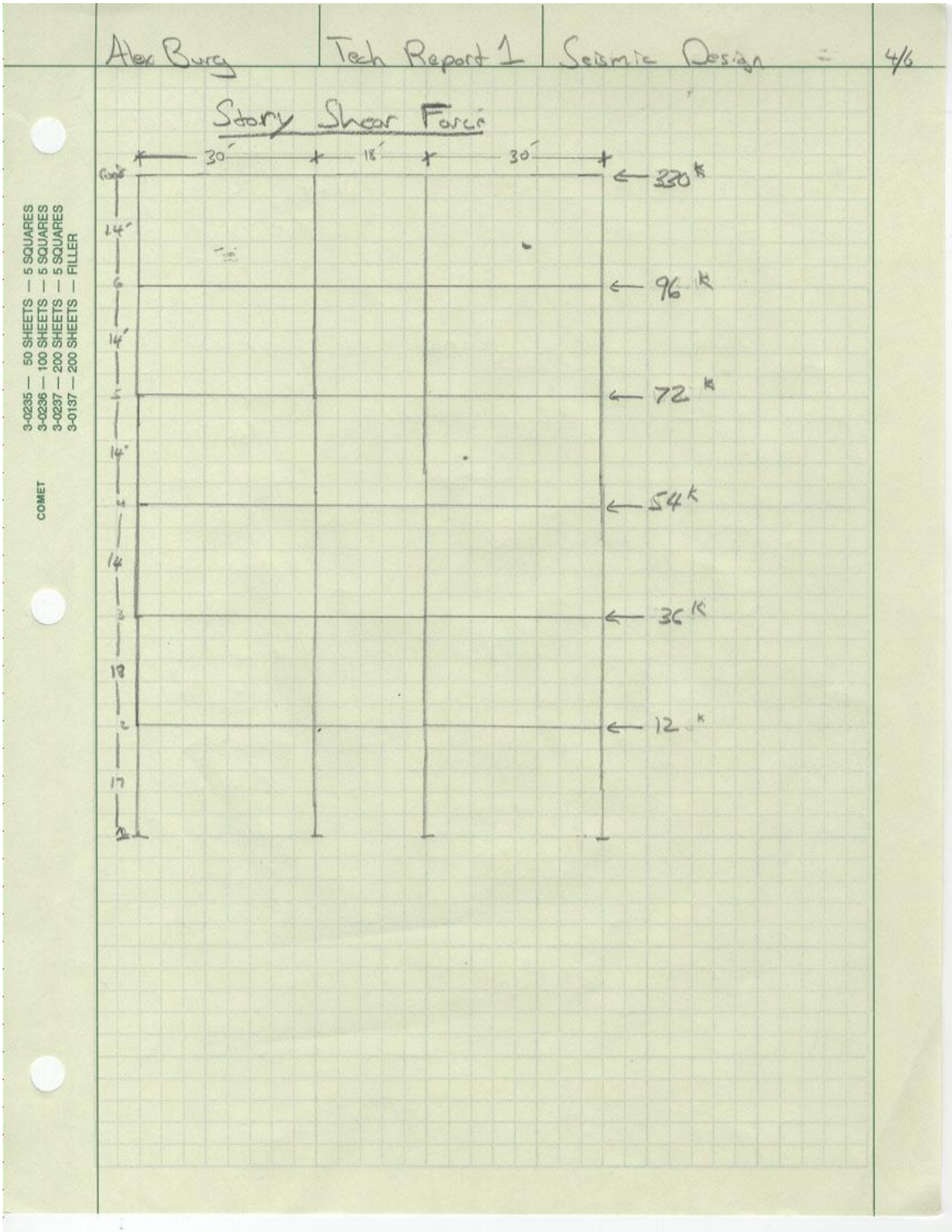
$= 0.02$

$\Sigma C = 1 \therefore$ OK

$k = 1.25$ interpolated from eq. 12.8-12

$\sum_{i=1}^{n} w_i h_i^{\,k} = 6{,}374(91)^{1.25}$
$+ 2{,}254(77)^{1.25}$
$+ 2{,}254(63)^{1.25}$
$+ 2{,}254(49)^{1.25}$
$+ 2{,}254(35)^{1.25}$
$+ 2{,}254(17)^{1.25}$

$3{,}267{,}578$


Alex Burg
Tech Report 1
Seismic Design
4/6
Story Shear Force
30'
18'
30'
14'
14'
14'
14
18
17
330 k
96 k
72 k
54 k
36 k
12 k
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Alex Burg | Tech Report 1 | Seismic Load | 5/6

The west to East Frames are Steel Ordinary moment Frames

Having an $R = 3.5$ (Table 12.2-1)

$$\left.\begin{array}{l} C_t = 0.028 \\ x = .8 \end{array}\right\} \text{(table 12.8-2)}$$

$T_a = C_t h_n^x$ $\qquad C_u = 1.7$ (table 12.8-1)

$= 0.028(91)^{.8}$

$= 1.034$

$T = C_u T_a$

$= 1.7(1.034)$

$= 1.76 < T_L = 6$ use $C_s = \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.1024}{1.76(3.5/1.25)} = 0.021 > 0.01$ ✓

$C_s = \dfrac{S_{DS}}{(R/I_e)} = \dfrac{0.321}{(3.5/1.25)} = 0.115 > 0.021$ ← controls

$C_s = 0.021$

<u>Base Shear</u>

$V = C_s \cdot W_{total}$

$= 0.021(17{,}644)$

$= 370.5^k$

Alex Burg | Tech Report 1 | Seismic Load | 6/6

Vertical Distribution

K = 1.63 interpolated from eq 12.8-12

$\sum_{i=1}^{n} w_i h_i^k = 6{,}374(91)^{1.63} + 2{,}254(77)^{1.63} + 2254(63)^{1.63} + 2{,}254(49)^{1.63} + 2{,}254(35)^{1.63} + 2{,}254(17)^{1.63}$

$= 16{,}807{,}752$

$C_{vr} = \frac{6{,}374(91)^{1.63}}{16{,}807{,}752} = 0.59$

$C_6 = \frac{2{,}254(77)^{1.63}}{16{,}807{,}752} = 0.16$

$C_5 = \frac{2254(63)^{1.63}}{16{,}807{,}752} = 0.12$

$C_4 = \frac{2254(49)^{1.63}}{16{,}807{,}752} = 0.08$

$C_3 = \frac{2254(35)^{1.63}}{16{,}807{,}752} = 0.04$

$C_2 = \frac{2254(17)^{1.63}}{16{,}807{,}752} = 0.01$

$\Sigma C = 1$ ✓

$F_{roof} = 0.59(370.5) = 218.7^K$

$F_6 = 0.16(370.5) = 59.3^K$

$F_5 = 0.12(370.5 = 44.5^K$

$F_4 = 0.08(370.5 = 29.6^K$

$F_3 = 0.04(370.5 = 14.8^K$

$F_2 = 0.01(370.5) = 3.7^K$

Appendix 5: Distributed Forces

Alex Burg | Tech. Report 3 | Distributed Forces | 1/1

Center of Rigidity: of Floor 4

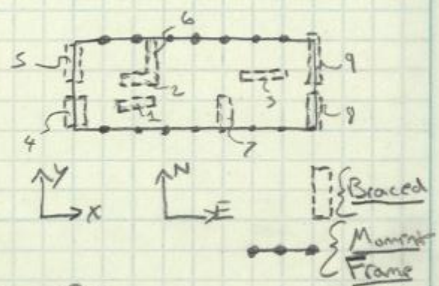
$\bar{x}_r = 2375.1'' = 197.9'$

$\bar{y}_r = 514.8'' = 42.9'$

Center of Mass: of Floor 4

$\bar{x}_m = 2351.4'' = 195.95'$

$\bar{y}_m = 409.9'' = 34.16'$



Direct Loads Floor 4

$$F_{ix} = \frac{K_{ix}}{\Sigma K_{ix}} P \leftarrow \qquad F_{iy} = \frac{K_{iy}}{\Sigma K_{iy}} P \downarrow$$

Moment Frame 1:

$$F_{m1} = \frac{368.06}{971.92} P_x$$

$$F_{m1} = 0.38 P_x \leftarrow$$

Braced Frame # 4

$$F_{B4} = \frac{11.94}{255.96} P_y$$

$$F_{B4} = 0.05 P_y \downarrow$$

Refer to Spread Sheet for the rest of the Forces & k values

Torsional Loads Floor 4

$$F_{ix} = \frac{K_i d_i P e_x}{\Sigma K_i d_i^2}$$

$e = |x_r - x_m|$

d_i = distance from the x_r to the center of frame i

Refer to Spread Sheet

Moment Frame 1, X-direction

$$F_{m1_{Tx}} = \frac{368.06(516.39)(23.7)}{\Sigma(98147530 + \ldots + \ldots)} P$$

$$= 0.011 P_x \rightarrow$$

Braced Frame #4: Y-dir.

$$F_{B4_{Ty}} = \frac{11.94(315.81)(104.9)}{408987129.6}$$

$$= 0.007984 P_y \downarrow$$

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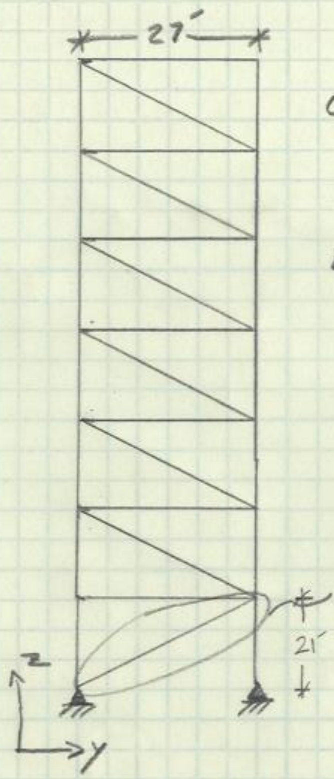

Check HSS cross bracing member in Brace Frame #5

ETAB output gives an Axial force of

$P_u = 417.78^k$

$L = 34'$

This output came from Case 1 wind load in the y-direction

Critical Member - HSS 12x12x½

AISC Table 4-4 $\left\{ \begin{array}{l} P_n = 519^k > 417.8^k \therefore \text{OKAY} \\ I = 457 \text{ in}^4 \\ A_g = 20.9 \text{ in}^2 \end{array} \right.$

$$P_{cr} = \frac{\pi^2 E I}{L^2}$$

$$= \frac{\pi^2 (29000)\, 457}{(34 \times 12)^2}$$

$$= 785.8$$

$$\phi P_n = 0.9(785.8)$$

$$\phi P_n = 707.19^k > 417.78^k \therefore \text{OKAY}$$

Appendix 6: Member Checks

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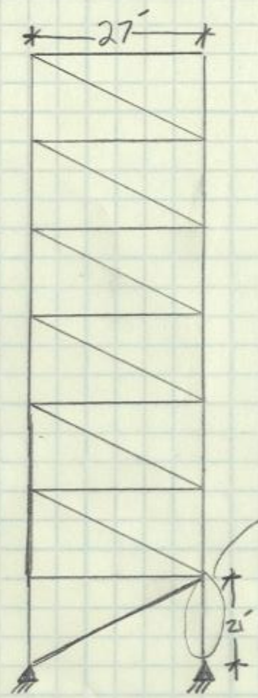

Check W14 column in brace frame # 3

ETAB output gives an Axial force of

$P_u = -752.19$
$L = 21'$

This output value came from case 1 wind load case in the x-direction with a momente of

$M_u = 327.6$

Critical Member - W14x211

AISC Table 6-1 $\begin{cases} \phi P_n = 0.488\ K^{-1} \\ b_x = 0.634\ (ft\text{-}k)^{-1} \end{cases}$

$$pP_r = \left(\frac{.488}{1000}\right)(752.19)$$

$$= 0.37 > 0.2$$

$$pP_r + b_x M_{rx} = 0.37 + \left(\frac{.634}{1000}\right) 372.6$$

$$= 0.6 \leq 1.0 \therefore \underline{\text{OKAY}}$$