The Commonwealth Medical College

Scranton, PA

Technical Report Three 2012

Xiao Ye Zheng

Structural Option

Advisor: Heather Sustersic

11/12/2012

Table of Contents

Executive Summary

This technical report discusses the lateral system analysis of The Commonwealth Medical College. The primary objective is to find the adequacy of the current system resistive strength. Relative stiffness, building torsion, shear strength, lateral displacement, story drift, serviceability, and overturning will be discussed throughout this report. Spot Checks were performed on one of the moment frame to determine the adequacy of the column and beam.

The lateral system used in TCMC consists of moment frames in the West wing, East wing, and the Link that connects them. They are located around the exterior perimeter of the building for maximum resistance. The frames in the Link start from the foundations to the ceiling of the second floor. The frames in the West wing and the East wing also start from the foundations but terminate at the ceiling of the fourth floor. However, for part of the West wing, moment frames were added to the penthouse, starting from the roof of the fourth floor to the roof of the penthouse.

A total of thirteen different load cases were found in ASCE 7-05 and were used to model TCMC in ETABS under lateral loads. Each case was run and the amount of shear at each level for each load case was documented. Comparing the shear forces, seismic forces controlled in both North-South and East-West Direction. TCMC is a building that is relatively heavy and short so it was expected that seismic forces would control.

There are 15 moment frames throughout TCMC. The stiffness of each frame was used to distribute direct shear and torsional shear for the controlling forces. Frame D, located in the West wing, was found to have the highest relative stiffness. Since seismic controlled for every floor in both directions, the seismic forces were used to calculate direct and torsional shear.

Building torsion was also calculated for both wind and seismic forces. The moments were obtained from ETABS due to incidental and accidental torsion. Moments due to an eccentricity between the center of mass and center of rigidity was also found. Total building torsion by found by the sum of all the moments.

Serviceability requirements were checked to see if story drifts were adequate. Because seismic is the controlling force, the story drifts caused by seismic forces were check with code. It was found that all drifts are less than 0.015h, allowable drift limit by code, so all serviceability requirements were met.

Determining the overturning moment in the foundations is crucial for a building. Again, using seismic forces, the overturning moment was found. TCMC's resistive moment was also found and it is more than 12 times of the overturning moment.

Lastly, spot checks were performed on a column and a beam at frame D and found to be adequate. The column was checked for combined axial and bending. The beam was check for its moment capacity.

Building Introduction

The Commonwealth Medical College (TCMC), also known as The Medical Sciences Building (MSB), is a medical school located in the heart of Scranton, PA. Costing over $120 million, this four story building, with an additional penthouse on the roof, was completed in April, 2011. The architecture was intended to complement the existing schools and hospitals in the surrounding area. Shown in Figure 1 is the building footprint of TCMC, highlighted in yellow, and the surrounding site.

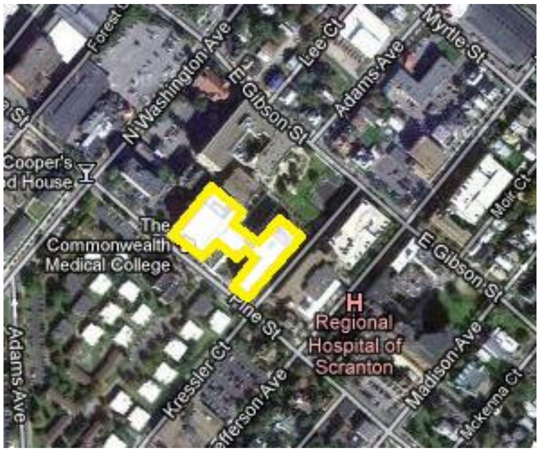


Figure 1 Aerial map from Google.com showing the location of the building site

TCMC is clad in brick, stone, and glass curtain wall. The building is separated into two individual wings, west wing and east wing. The link is the lobby area that connects the two wings and it is clad largely in insulated glass units to let natural sunlight in. An additional feature is the tower which is also clad largely in glass, as shown in Figure 2. The tower, located in the East wing, is considered the main focal point of the building. The interior space of the tower is mainly corridors and small meeting rooms so the students can enjoy the view.

Figure 2 Picture of the exterior showing the glass and brick facade on the TCMC. The Tower is shown, made will all glass walls. http://www.hok.com

TCMC is a multi-use building, using all modern technology. It has a library where students go for information, Clinical Skills and Simulation Center where students learn from beyond classrooms, lecture halls that can seat up to 160 students, classrooms with Wi-Fi connections, small group meeting rooms where a team of students can work together, and a luxurious student lounge for study or relaxation. Figure 3 shows the interior lobby of TCMC. TCMC also has a garden around the link that allows the occupants to enjoy the nice green views that the city cannot offer. The building is 93 feet tall, 185,000 square feet of space, and is a composite steel framed building that utilizes moment frames for its lateral system.

Figure 3 Interior picture of the TCMC lobby. http://www.hok.com

Structural Overview

Design Codes

According to Sheet LS100, the building was designed to comply with:

❖ Building Code	2006 International Building Code (IBC)
❖ Mechanical	2006 International Mechanical Code
❖ Electrical	2005 NFPA 70/ Nation Electrical Code
❖ Plumbing	2006 International Plumbing Code
	2006 International Fuel Gas Code
❖ Fire Protection	2006 International fire Code

All concrete work conforms to the requirements of the American Concrete Institute ACI-318-05.

Additional Code Reference from American Concrete Institute:

- ACI-211
- ACI-301
- ACI-302
- ACI-304
- ACI-305
- ACI-306
- ACI-315
- ACI-347

Regulatory Guidelines and Standards

❖ Accessibility	ICC/ANSI A117.1 1998

Material Properties

Concrete		
Usage	Weight	Strength (psi)
MAT Slab	Normal	4000psi
Columns	Normal	4000psi
Slab on Grade	Normal	3000psi
Caisson	Normal	4000psi
Wall	Normal	4000psi
Grade Beam	Normal	4000psi
Floor Slab	Normal	4000psi
Floor Slab	Lightweight	3500psi
Floor Slab	Normal	3500psi
Lean Concrete Fill	Normal	2000psi

Steel		
Type	Standard	Grade
Reinforcing Bars	ASTM A615	60
Composite Floor Deck	ASTM A992	20 gauge
Roof Deck	ASTM A992	B
Galvanized Plate	ASTM A992	50
W shape Steel	ASTM A992	50
Angles	ASTM A992	50
Bolts	ASTM A325	N/A
Anchor Rods	ASTM F1554	N/A
HSS	ASTM A992	50
Welded Wire Fabric	ASTM A185	70,000psi

Masonry		
Type	Standard	Strength (psi)
Grout	ASTM C476	5000psi
Concrete Masonry Units	ASTM C90	2100psi
Mortar	ASTM C270	N/A

Miscellaneous	
Type	Strength (psi)
Non-Shrink Grout	10,000psi

Figure 4 Tables showing materials that are used in the TCMC project

Foundations

The West wing of the TCMC is built with a mat slab foundation that is 4'-0" thick. The mat slab is designed for a soil bearing pressure of 3000psf. It is on top of a 2'-0" thick structural fill and a 4" mud slab. Figure 5 shows a typical section of the mat slab. After the mat slab, over 4' of compacted AASHTO # 57 stone typical was placed in followed by a 5" slab on grade. Due to the confidentially of the geotechnical report, the actual bearing capacity of the soil and the recommended type of foundations were never released.

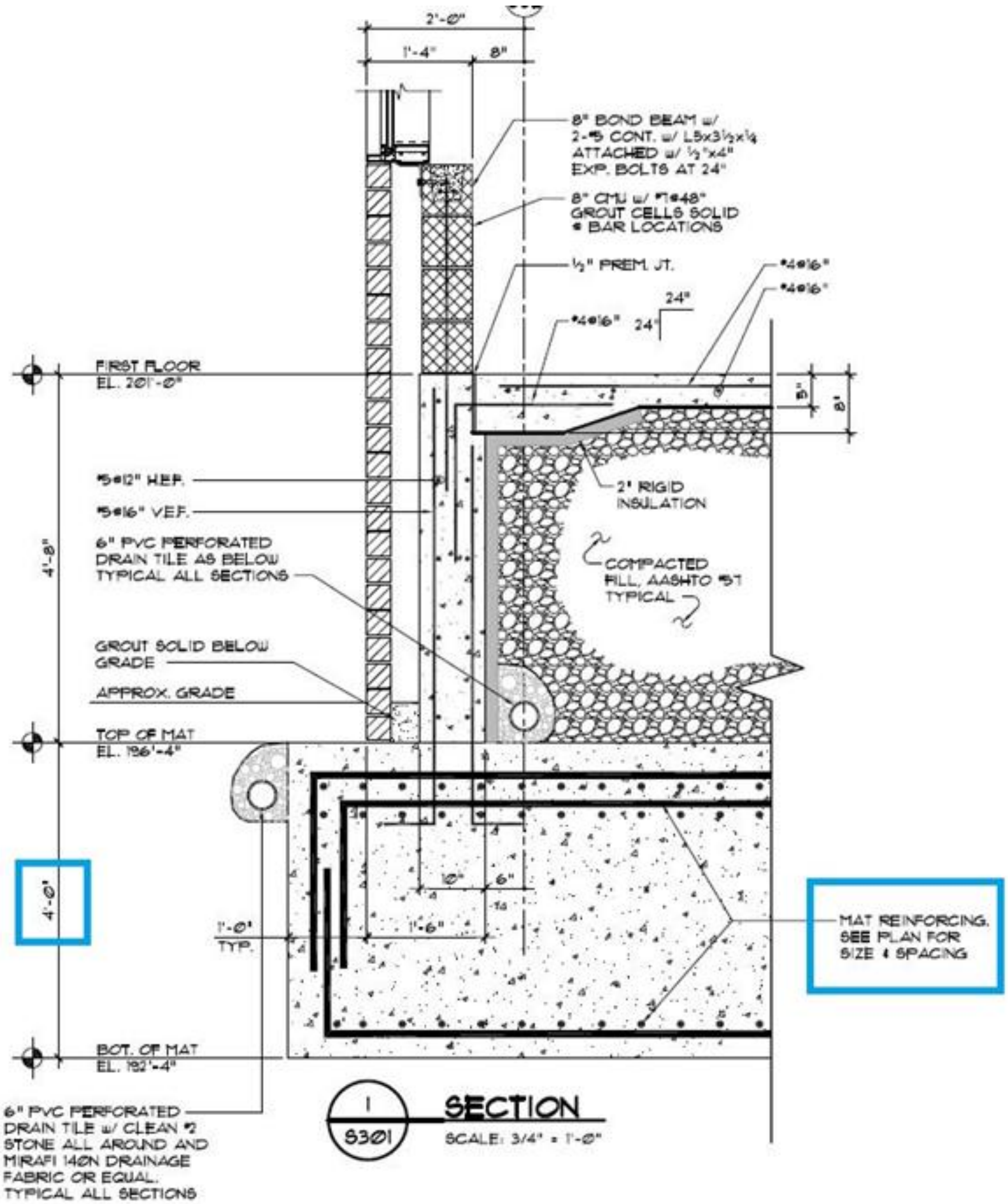


Figure 5 A typical Section cut showing the mat slab foundation. Courtesy of Highland Associates

The East wing of the TCMC has drilled caissons ranging from 36” to 60” in diameter and is used to carry loads from grade beams to bedrock below. The typical floor slab in the east wing is 7.5” and it’s also on top of compacted AASHTO material. This can all be visualized by looking at a typical section cut from Figure 6 below.

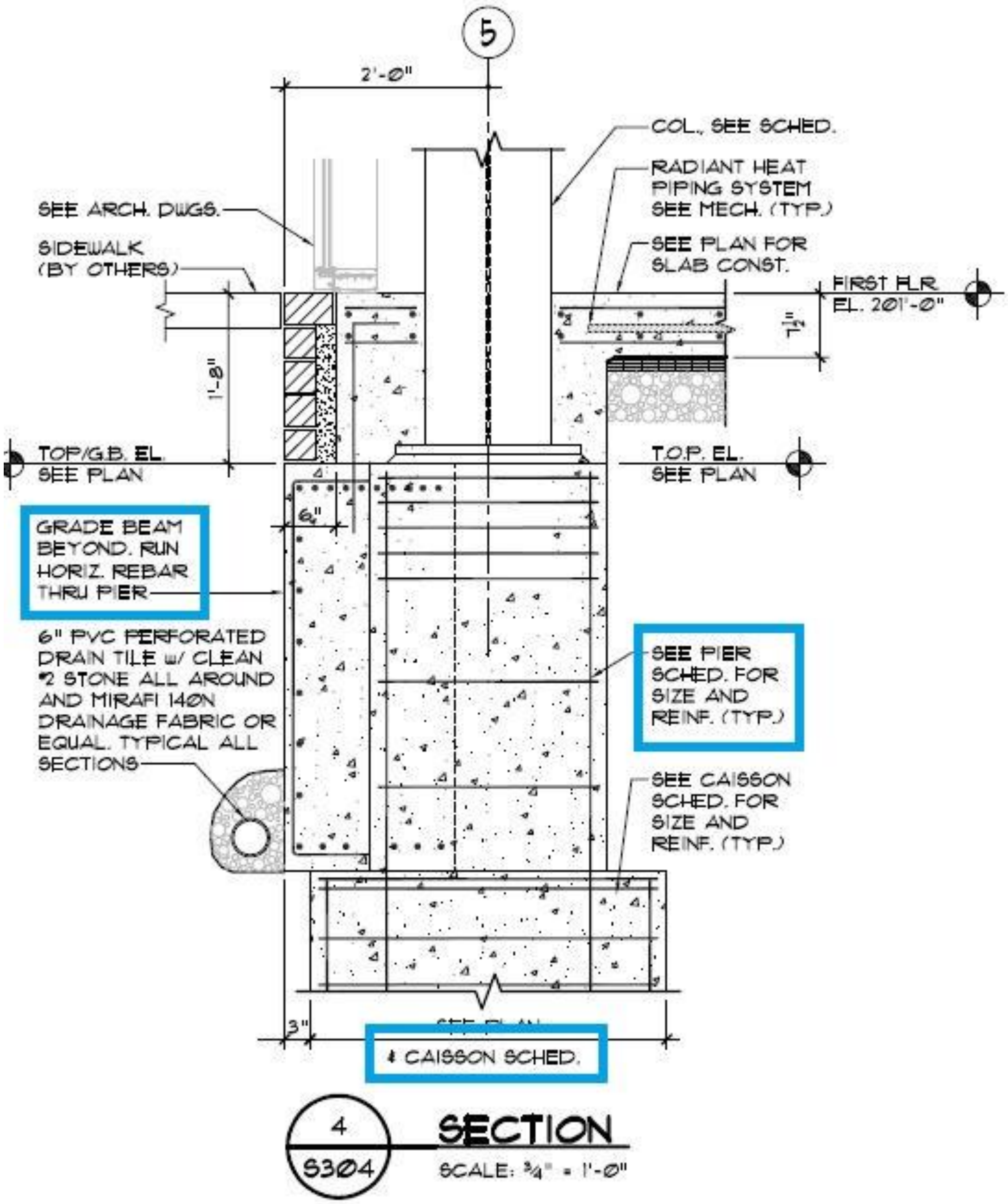


Figure 6 A section cut of a drilled caisson foundation. Courtesy of Highland Associates

Floor Systems

The existing floor system of the TCMC is held up by W-shaped steel columns and composite steel beams. Figure 7 shows the floor plan with different bay sizes in different colors. Bay sizes are shown along with the figure, with the span required for the slab first and the span required for the girder next, match with their colors. Small bays sizes are not shown in Figure 7.

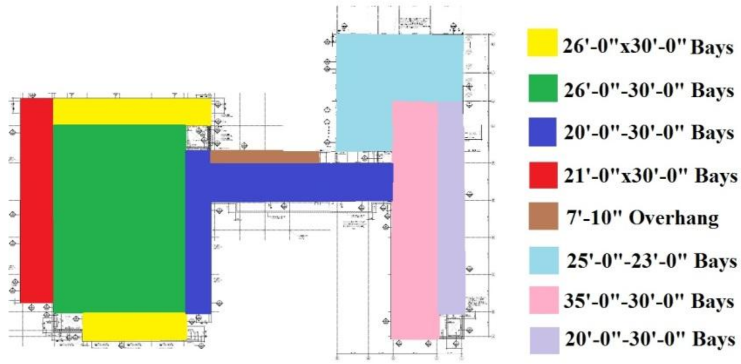


Figure 7 Different Bay sizes respective to their color

The floor is composite steel deck with concrete topping. The typical floor plan in the west wing is shown in Figure 8 along with two section cuts, Figures 9 and 10. It is a 4.5" normal weight concrete topping on a 3" lok-floor 20 gauge galvanized composite floor deck, giving it a total slab construction of 7.5". The east wing, and the link, has different slab thickness than the west wing. They are 3.25" lightweight concrete topping on U.S.D. 2" lok-floor 20 gauge galvanized composite floor deck, making the total thickness of 5.25".

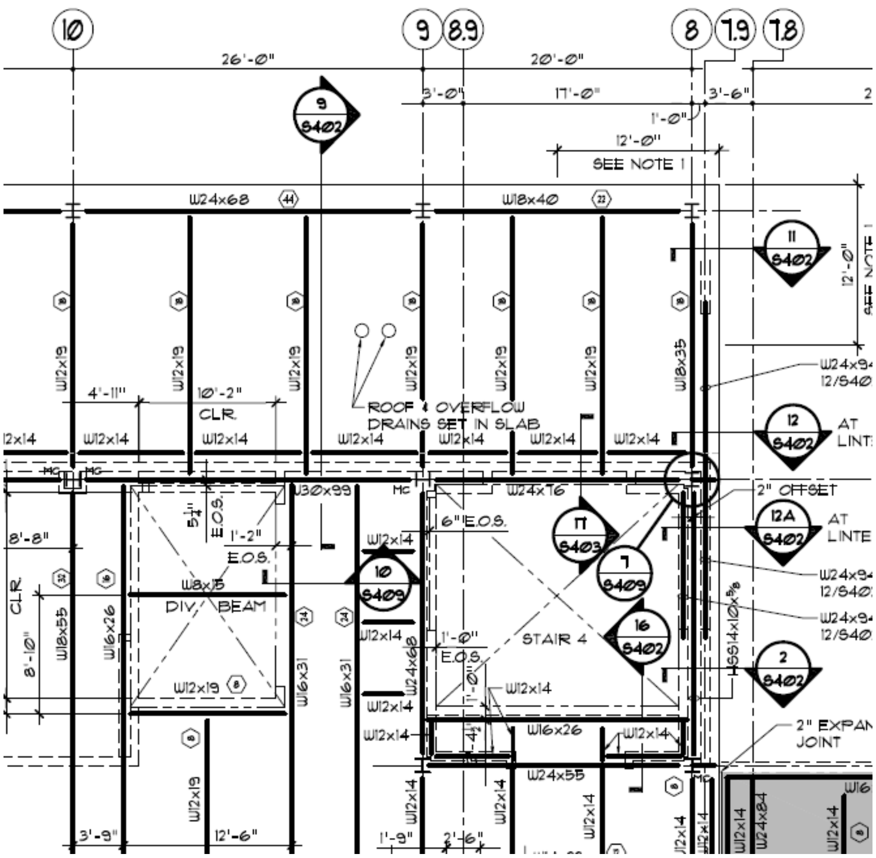


Figure 8 Partial plan showing the second floor, northeast corner of the west wing

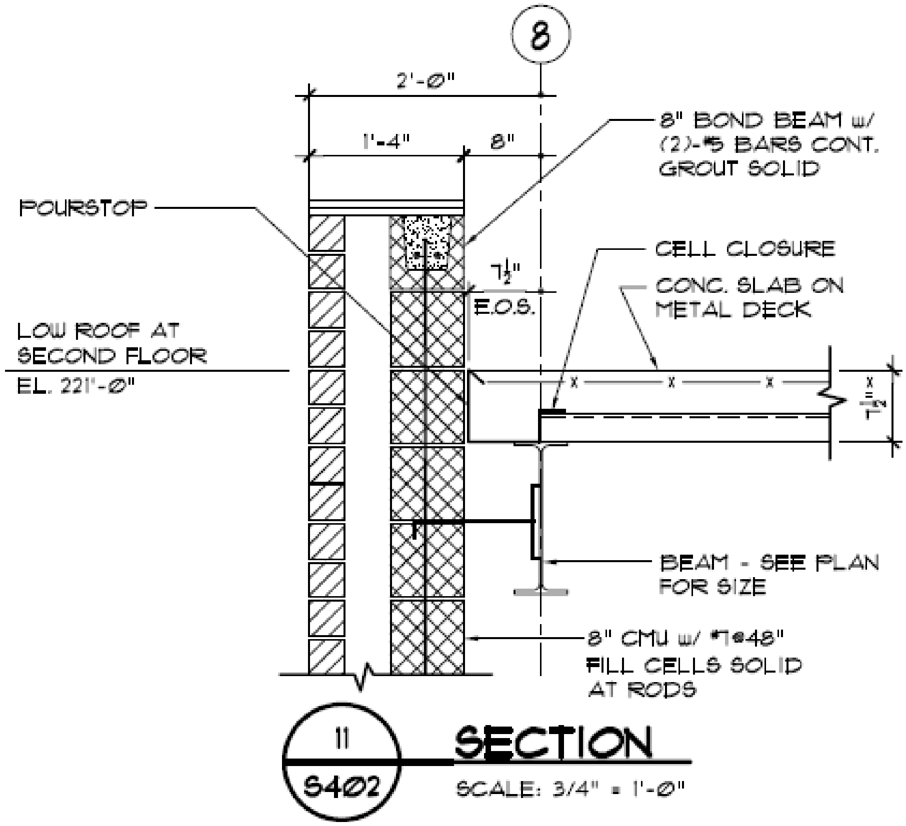


Figure 9 Section cut 11 from Figure 8

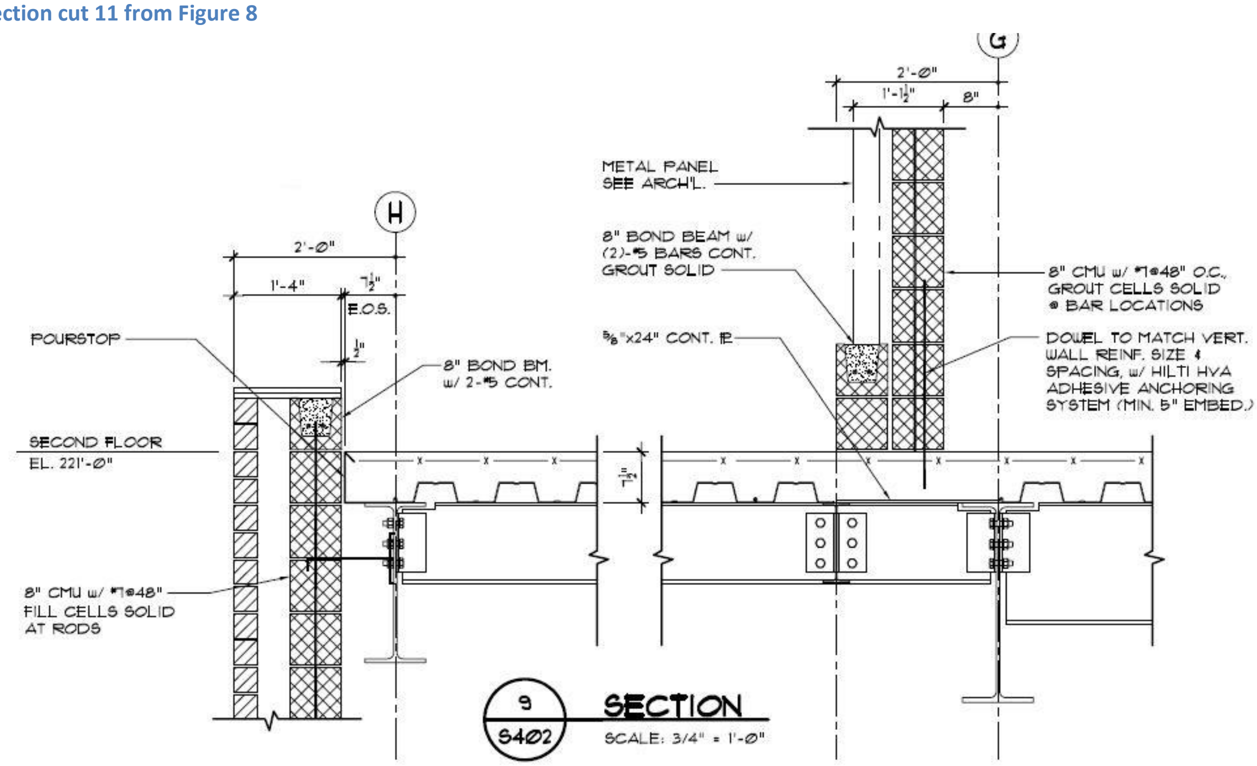


Figure 10 Section cut 9 from Figure 8

Roof Systems

TCMC has over 9 different roof heights, as shown in Figure 11, with the ground referenced at 0’-0”. The link between two wings has an average roof height of 36’. The west wing goes up to 92’. The Tower, shaded in red, in the east wing goes up to 89’-4”. The rest of the east wing goes up to 81’-4” while the east wing penthouse goes up to 102’.

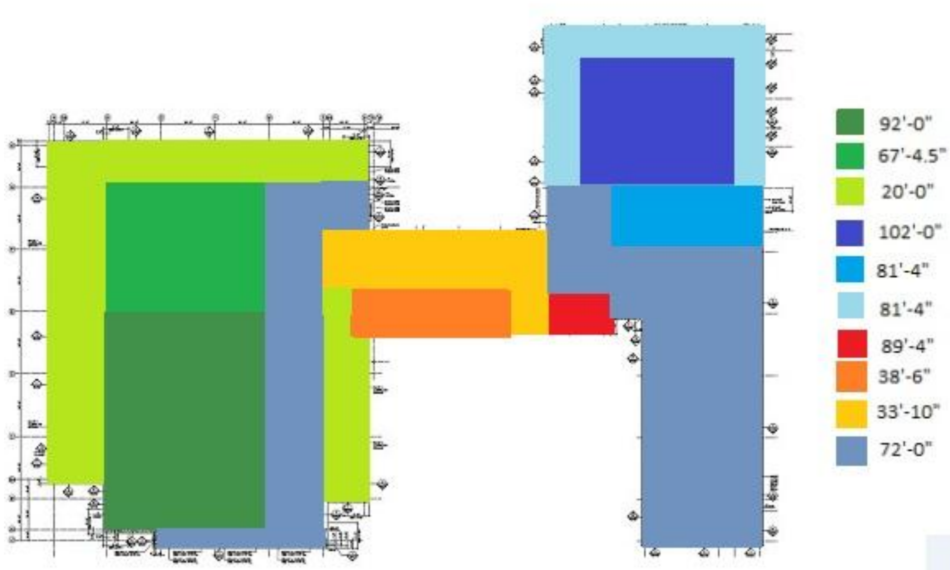


Figure 11 Plan showing the different roof heights; the darker, the higher.

The main roof is constructed of 1.5” type B wide rib, 22 gauge, painted roof deck supported by W-shape framing. A typical roof section cut is shown on Figure 12. The typical roofing system has two layers of 2” rigid roof insulation. The walls around the roof extend 4’ higher than the steel deck so that it can be used as railings.

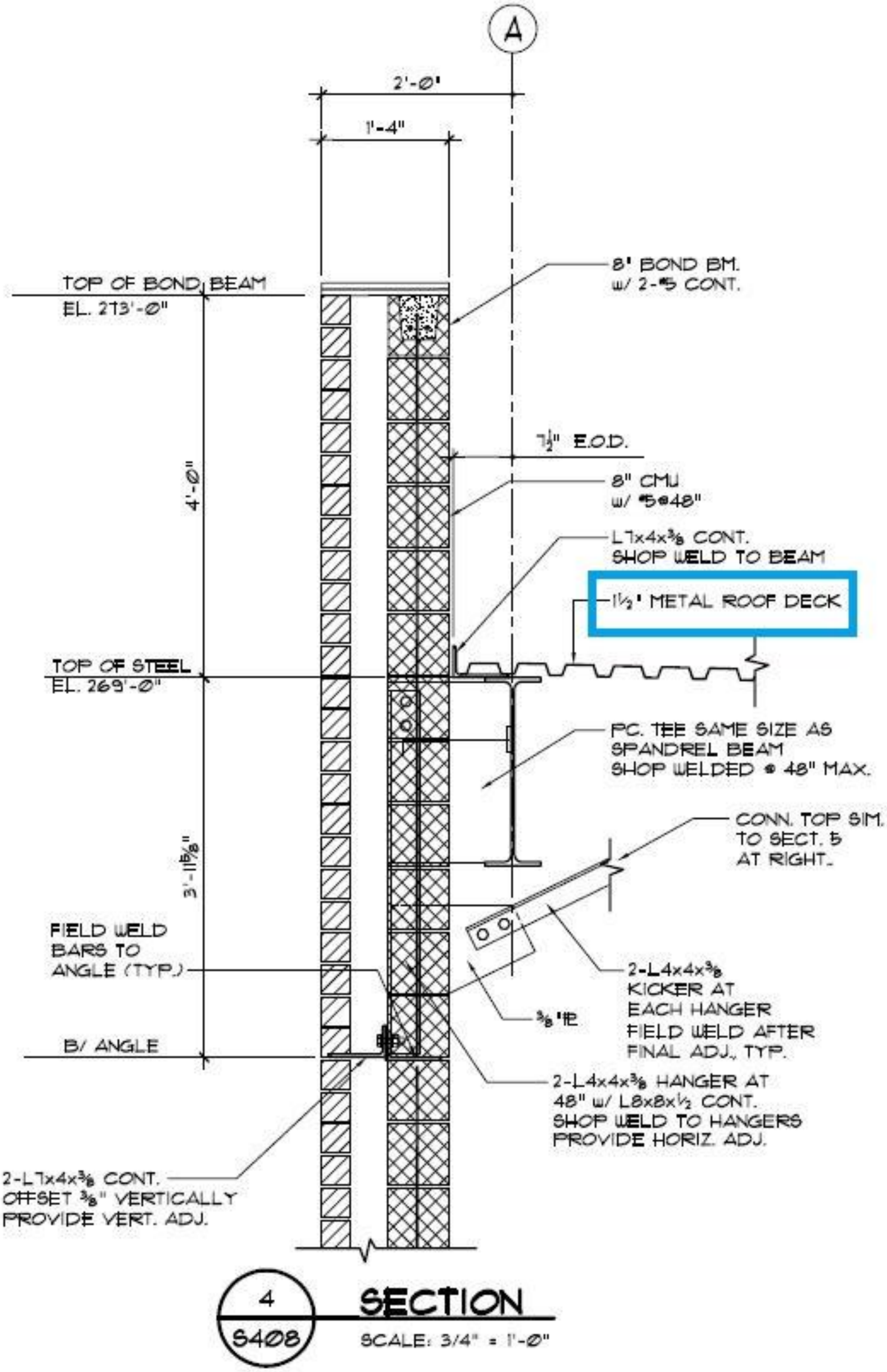


Figure 12 Typical roof section cut showing the roof deck. Courtesy of Highland Associates

Framing System

TCMC has a composite steel framed system. The sizes of the beams and columns ranged from W8x24, being the lightest, to W14x257, being the heaviest. The longest column is 44’-7” and it stopped between the third and fourth floor. An additional 48’-0” of lighter steel column is connected to this column, extending it all the way up to the penthouse.

Lateral System

The main lateral system used in TCMC consists of multiple moment frames. They are present in the west wing, east wing, and also in the link, as shown in Figure 13.1. Most frames are near the exterior wall to maximize the lateral force it can resist. The moment frames span across the entire building, from north to south and from east to west. This provides lateral resistance in each direction. The frames in the link begin on the first floor and extend to the roof, the third floor. The frames in the two wings begin on the first floor and extend to the floor of the penthouse. Figure 13.2 shows the only four frames that extend to the roof of the penthouse.

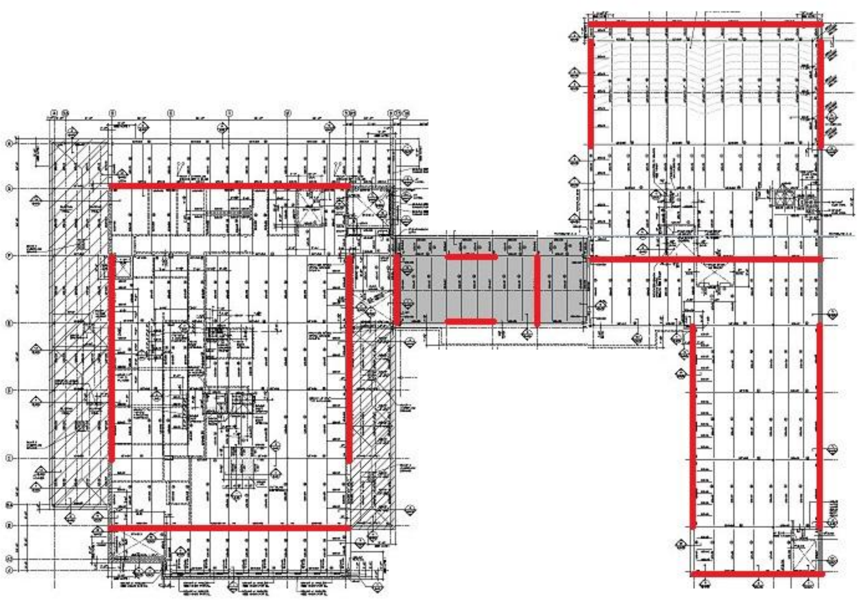


Figure 13.1 Locations of Moment Frames at TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

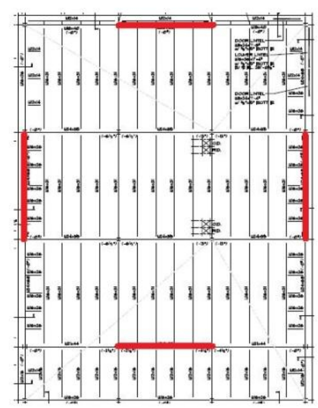


Figure 13.2 Locations of Moment Frames at the Penthouse of TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

Gravity Loads

The dead, live, and snow loads were calculated under this section for TCMC using IBC 2006, ASCE 7-05, and estimation.

Dead and Live Loads

For the dead load calculations, the materials that have the most impact on the dead weight of the building were found and then calculated. The west wing primarily uses composite 3" steel deck with concrete slab that weighs 75 psf according to Vulcraft Steel Deck catalog. The east wing and the hallway use 2" steel deck, lightweight concrete, so it only weights 42 psf. Then W-shape Steel Beams and Columns are assumed as 15 psf that covers that whole entire building. The heaviest exterior wall is chosen and is assumed throughout the building at 1000plf. Then these weights are multiplied by the area or the length that they occupied in to get the weight in pounds. A sample of this calculation is shown for the 2nd floor of the TCMC in Figure 14 below. Doing this for every level, a weight in psf and lbs are both obtained. Then the total dead weight is found to be around 22,378 kips and will be used later in seismic calculations. A breakdown of the weight per Level is shown in Figure 15.

Weight for 2nd Floor			
Material	Weight (psf)	Area or Length	Total Weight (lb)
Normal Weight Conc Slab with Deck	75 (psf)	20408 sf	1,530,600
Light Weight Conc Slab with Deck	42 (psf)	24952 sf	1,047,984
W-Shape Steel	15 (psf)	45360 sf	680,400
Exterior Walls	1000 (plf)	1418 lf	1,418,000
Total Weight			4,676,984
Total Weight per sf (close to design average dead load of 93 psf)			103.11

Figure 14 Total Weight per square foot of TCMC

Weight Per Level			
Level	Area (ft^2)	Weight (psf)	Weight (k)
1st	51,348.00	99.3	5099
2nd	45,360.00	103.1	4677
3rd	40,425.00	106.0	4286
4th	40,422.00	106.0	4286
Penthouse	10,337.00	209.2	2163
Roof (all level)	40,455.00	46.0	1867
Total	228,347.00		22378

Figure 15 Total Weights per Level of TCMC

The design live load for the TCMC can be found in the drawings on sheet S201A and S201B. A comparison of it to the minimum live load requirement from ASCE 7-05 can be seen on Figure 16. Notice that most design load are the same as the minimum required live load. However, some design live loads for several locations are higher because more live loads are expected.

Design Live Loads for West Wing			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Offices	50	50	
Lobbies/ Corridors	100	100	
Corridors above 1st	80	80	
Stairs	100	100	
Classrooms	40	40	
Laboratories	100	60	Larger equipment needed in TCMC Labs
Storage Rooms	125	125	Light warehouse
Restrooms	60	N/A	
Mechanical Room	150	N/A	
Mechanical Roof	30	N/A	
Roof	20	20	ordinary flat
Partitions	15	15	

Design Live Loads for Rest of Building			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Offices above 1st	65	50	Partitions and some heavier office equipment
Lobbies/ Corridors	100	100	
Corridors above 1st	80	80	
Stairs	100	100	
Classrooms	50	40	
Sorage above 1st	125	125	
Restrooms above 1st	75	N/A	
Auditorium	100	100	if seats are fixed, then only 60psf
Bookstore	150	N/A	
Lecture Halls	60	N/A	
Mechanical Room	150	N/A	
Library	75	N/A	
1st floor offices	65	50	
1st floor restrooms	75	N/A	
Roof	30	20	
Mechanical Roof	30	N/A	
1st floor storage	125	100	

Figure 16 Design live load is compared to ASCE 7-05, required live load

Snow Loads

The variables needed for snow load calculations are found on sheet S201B of the drawings. Figure 17 shows all the loads and variables that are from Sheet S201B of the structural drawing. Also, because of the many different roof heights, snow drifts can happen in over 10 different areas of the building. One of these areas is calculated and shown under Appendix A, snow load calculations. The result of that area is that the snow acuminated in the corner reached over 73 psf, more than double the amount compared to the regular flat roof amount of 30 psf. Snow drift is an important factor when designing TCMC.

Flat Roof Snow Load Calculations	
Variable	Value
Ground Snow Load (P_G)	35 psf
Flat Roof Snow Load (P_F)	30 psf
Snow Exposure Factor (C_E)	1.0
Importance Factor (I_S)	1.1
Thermal Factor (C_T)	1.0

Figure 17 Variable for snow load obtained from S201B

Lateral Loads

Wind Loads

A wind study was performed on TCMC using ASCE 7-05, MWFRS Analytical Procedure, as guide. Because TCMC is complex, for calculations, the building was modeled as two individual buildings, West wing, and East wing. A simplified building shape was used for both wings. This full calculation can be found under Appendix B. The structural drawing, sheet S201B, provided the basic wind load variables needed; see Figure 18. A factored base shear of 201.9k was found for the West wing in the North-South direction. A factored base shear of 106.6k was found for the East wing in the North-South direction. The two base shears were added together to get the total factored base shear for TCMC in the North-South Direction, which is 308.5k. As for the East-West direction, a factored base shear of 263.2k was found for the West and a factored base shear of 347.1k was found for the East wing. Base shear in the East Wing is the controlling factor for the East-West direction. The base shear in the East-West direction was found to be larger than the North-South direction. It was expected since the area of TCMC's east wall is slightly larger than the area of its south or north wall, hence, would have more forces acting upon it. The resistance to wind loads will be distributed to each moment frames based on their stiffness. This will be further discussed in later sections. Figure 19 gives the summary of the wind loads. Figure 20 to 27 on the next couple pages shows the wind pressures and wind forces acting on the West and East wing of TCMC, along with an elevation view.

WIND LOAD

BASIC WIND SPEED (V_{3S}) = 90 MPH
IMPORTANCE FACTOR (I_W) = 1.15
EXPOSURE CATEGORY = B

Figure 18 Wind Load from sheet S201B

Summary: Wind Loads on TCMC		
NS Base Shear	308.5	k
NS Overturning Moment	15110.7	k-ft
ES Base Shear	347.1	k
ES Overturning Moment	17014.2	k-ft

Figure 19 Summary of Wind Loads on TCMC

West Wing Wind Pressures N-S Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
Windward Walls	Ground	0	9.41	3.62	-3.62	13.02	5.79
	2nd	21	9.41	3.62	-3.62	13.02	5.79
	3th	37	9.94	3.62	-3.62	13.55	6.32
	4th	53	11.19	3.62	-3.62	14.81	7.58
	Penthouse	69.5	11.99	3.62	-3.62	15.61	8.37
	Roof	93	13.31	3.62	-3.62	16.93	9.70
Leeward Walls	All	All	-6.66	3.62	-3.62	-3.04	-10.28
Side Walls	All	All	-11.65	3.62	-3.62	-8.03	-15.27
Roof	N/A	0-46.5	-18.31	3.62	-3.62	-14.69	-21.93
	N/A	46.5-186	-9.99	3.62	-3.62	-6.37	-13.60

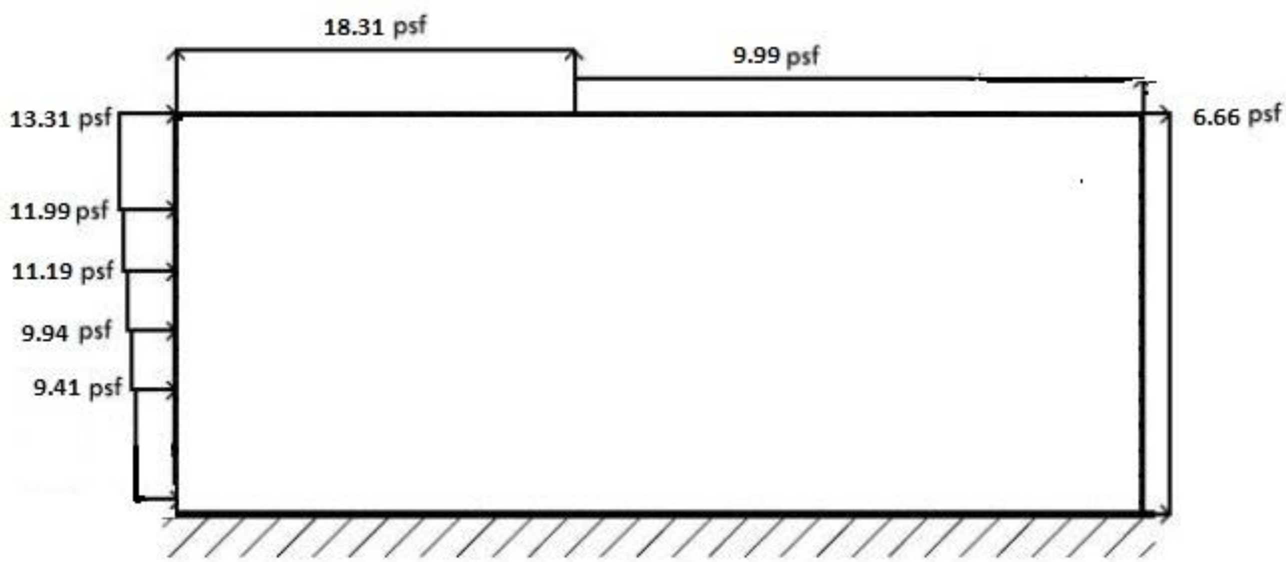


Figure 20 Wind Pressures acting on the West Wing, North and South facades

West Wing Wind Forces N-S Direction								
Floor	Height	Trib Below		Trib Above		Story Force	Story Shear	Overturning
	(ft)	height (ft)	area (sf)	height (ft)	area (sf)	(k)	(k)	Moment (k-ft)
Ground	0	0	0	10	1500	19.5	201.9	0.0
2nd	20	10	1500	8	1200	35.2	182.3	703.3
3th	36	8	1200	8	1200	32.5	147.2	1171.1
4th	52	8	1200	10	1500	40.0	114.7	2079.7
Penthouse	72	10	1500	10.5	1575	48.0	74.7	3455.5
Roof	93	10.5	1575	0	0	26.7	26.7	2480.1
Total						201.9	N/A	9889.7

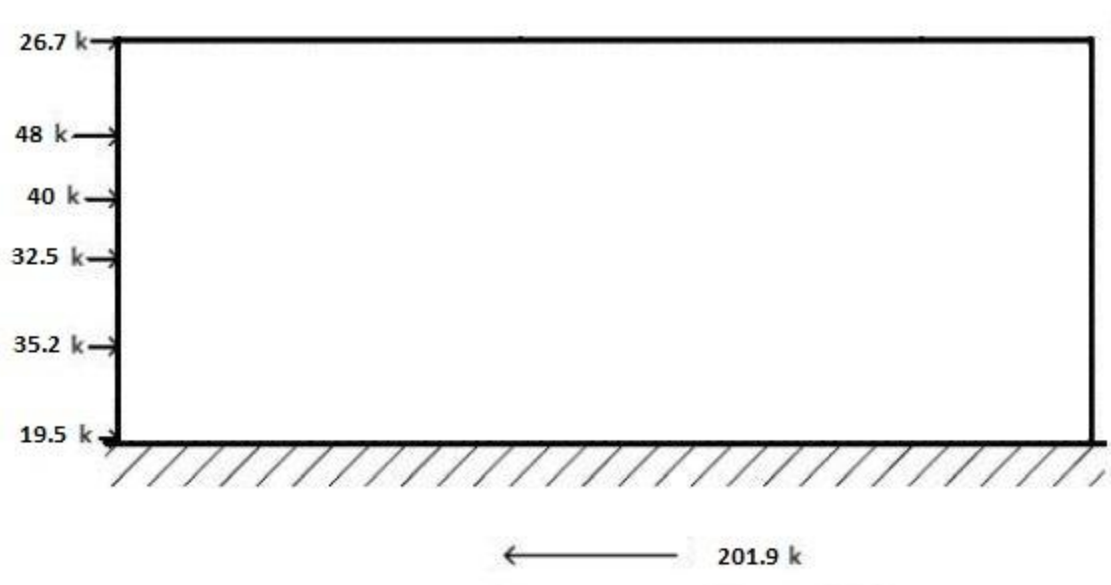


Figure 21 Wind Forces acting at each floor level on the West Wing, North and South facades

West Wing Wind Pressures E-W Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
	Ground	0	9.51	3.62	-3.62	13.13	5.89
	2nd	21	9.51	3.62	-3.62	13.13	5.89
Windward	3th	37	10.04	3.62	-3.62	13.66	6.43
Walls	4th	53	11.32	3.62	-3.62	14.93	7.70
	Penthouse	69.5	12.12	3.62	-3.62	15.74	8.50
	Roof	93	13.46	3.62	-3.62	17.08	9.84
Leeward Walls	All	All	-7.57	3.62	-3.62	-3.95	-11.19
Side Walls	All	All	-11.78	3.62	-3.62	-8.16	-15.39
Roof	N/A	0-93	-15.14	3.62	-3.62	-11.52	-18.76
	N/A	93-186	-8.41	3.62	-3.62	-4.79	-12.03
	N/A	>186	-5.05	3.62	-3.62	-1.43	-8.67

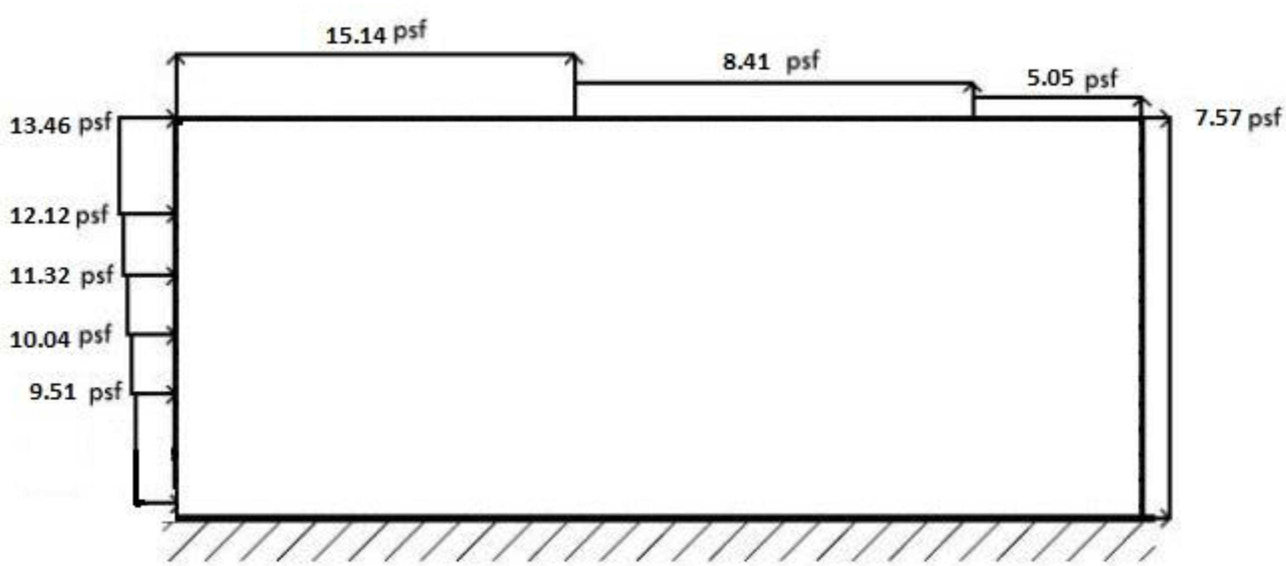


Figure 22 Wind Pressures acting on the West Wing, East and West facades

West Wing Wind Forces E-W Direction								
Floor	Height (ft)	Trib Below height (ft)	Trib Below area (sf)	Trib Above height (ft)	Trib Above area (sf)	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	0	0	0	10	1940	25.5	263.2	0.0
2nd	20	10	1940	8	1552	45.8	237.8	916.7
3th	36	8	1552	8	1552	42.4	191.9	1526.6
4th	52	8	1552	10	1940	52.2	149.5	2711.8
Penthouse	72	10	1940	10.5	2037	62.6	97.4	4506.4
Roof	93	10.5	2037	0	0	34.8	34.8	3235.1
Total						263.2	N/A	12896.7

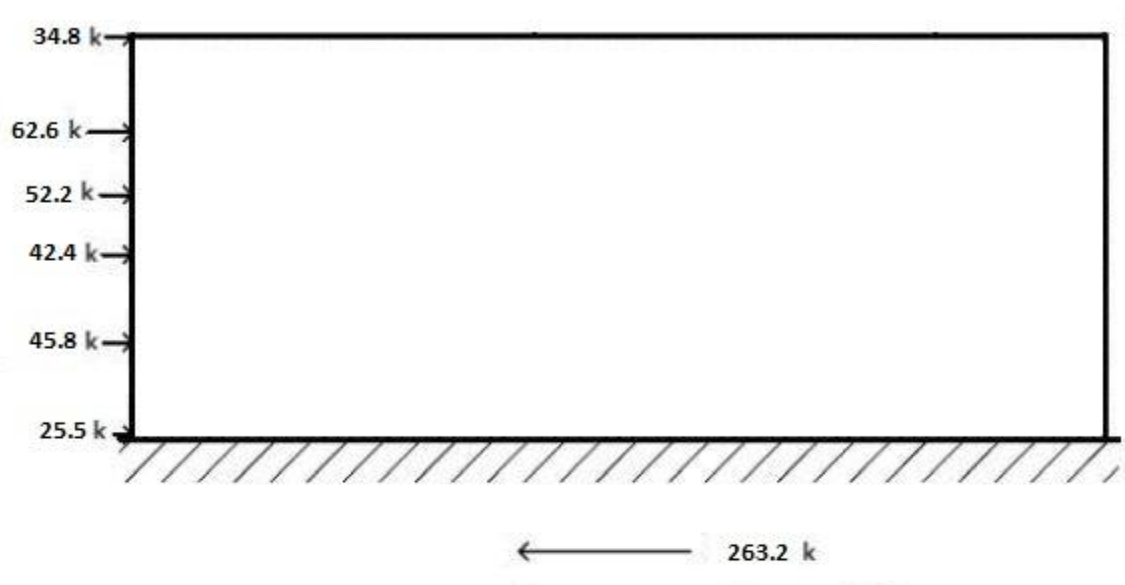


Figure 23 Wind Forces acting at each floor level on the West Wing, East and West facades

East Wing Wind Pressures N-S Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
Windward Walls	Ground	0	9.28	3.62	-3.62	12.90	5.66
	2nd	21	9.28	3.62	-3.62	12.90	5.66
	3th	37	9.80	3.62	-3.62	13.42	6.19
	4th	53	11.05	3.62	-3.62	14.66	7.43
	Penthouse	69.5	11.83	3.62	-3.62	15.45	8.21
	Roof	93	13.14	3.62	-3.62	16.76	9.52
Leeward Walls	All	All	-8.21	3.62	-3.62	-4.59	-11.83
Side Walls	All	All	-11.50	3.62	-3.62	-7.88	-15.11
Roof	N/A	0-46.5	-21.35	3.62	-3.62	-17.73	-24.97
	N/A	46.5-186	-11.50	3.62	-3.62	-7.88	-15.11

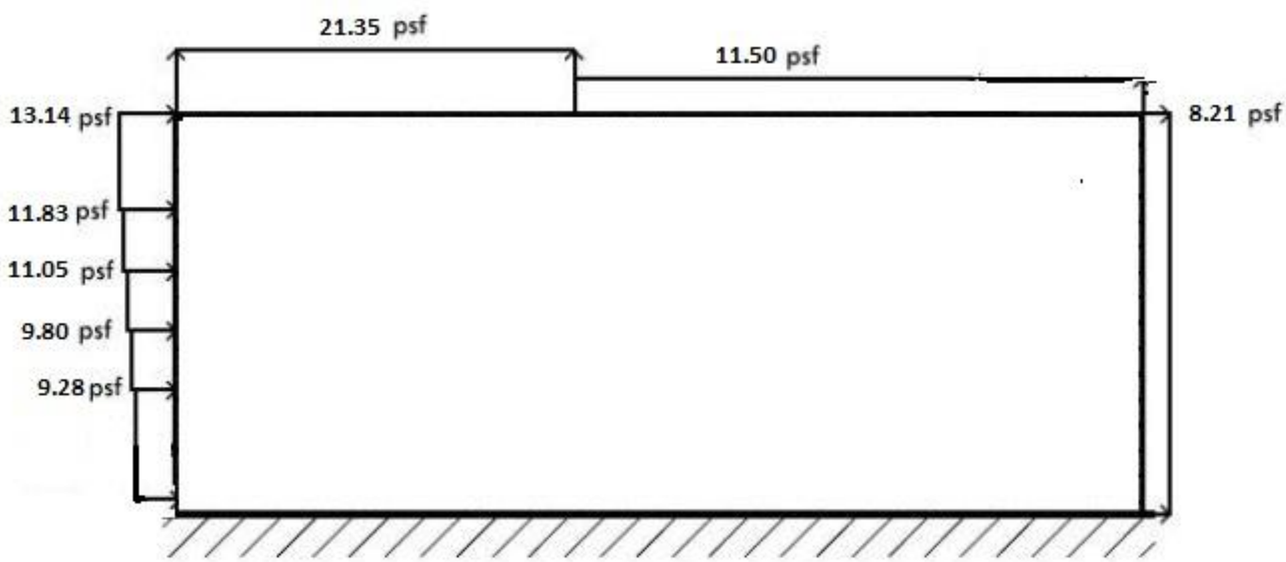


Figure 24 Wind Pressures acting on the East Wing, North and South facades

East Wing Wind Forces N-S Direction								
Floor	Height	Trib Below		Trib Above		Story Force	Story Shear	Overturning
	(ft)	height (ft)	area (sf)	height (ft)	area (sf)	(k)	(k)	Moment (k-ft)
Ground	0	0	0	10	800	10.3	106.6	0.0
2nd	20	10	800	8	640	18.6	96.3	371.5
3th	36	8	640	8	640	17.2	77.7	618.5
4th	52	8	640	10	800	21.1	60.5	1098.0
Penthouse	72	10	800	10.5	840	25.3	39.4	1824.1
Roof	93	10.5	840	0	0	14.1	14.1	1308.9
Total						106.6	N/A	5221.1

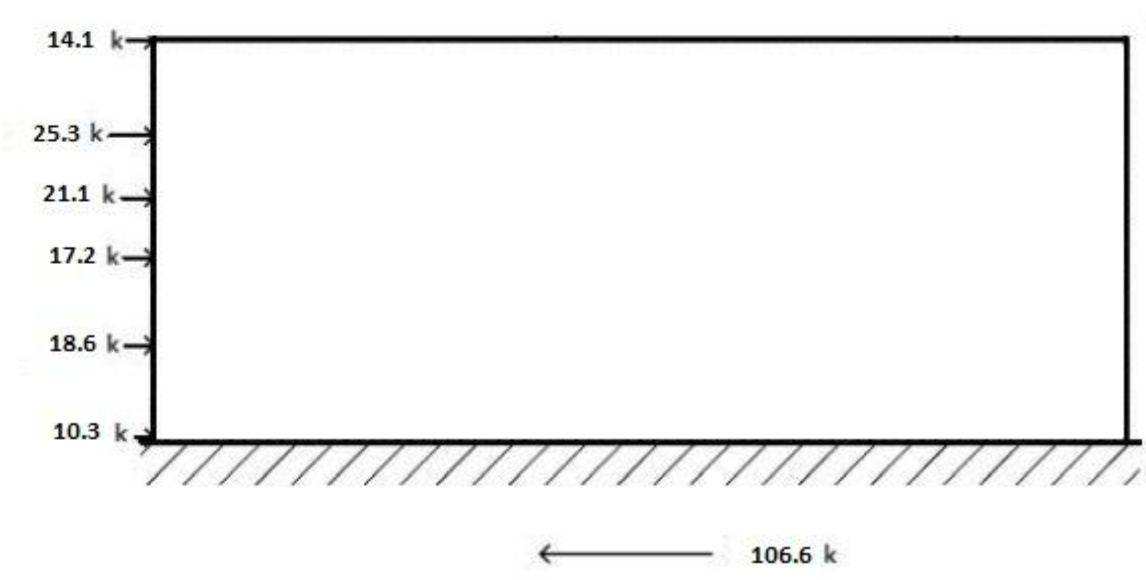


Figure 25 Wind Forces acting at each floor level on the East Wing, North and South facades

East Wing Wind Pressures E-W Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
	Ground	0	9.80	3.62	-3.62	13.42	6.19
	2nd	21	9.80	3.62	-3.62	13.42	6.19
Windward	3th	37	10.36	3.62	-3.62	13.97	6.74
Walls	4th	53	11.67	3.62	-3.62	15.29	8.05
	Penthouse	69.5	12.50	3.62	-3.62	16.11	8.88
	Roof	93	13.88	3.62	-3.62	17.50	10.26
Leeward Walls	All	All	-6.94	3.62	-3.62	-3.32	-10.56
Side Walls	All	All	-12.14	3.62	-3.62	-8.52	-15.76
Roof	N/A	0-93	-15.61	3.62	-3.62	-11.99	-19.23
	N/A	93-186	-8.67	3.62	-3.62	-5.06	-12.29
	N/A	>186	-5.20	3.62	-3.62	-1.59	-8.82

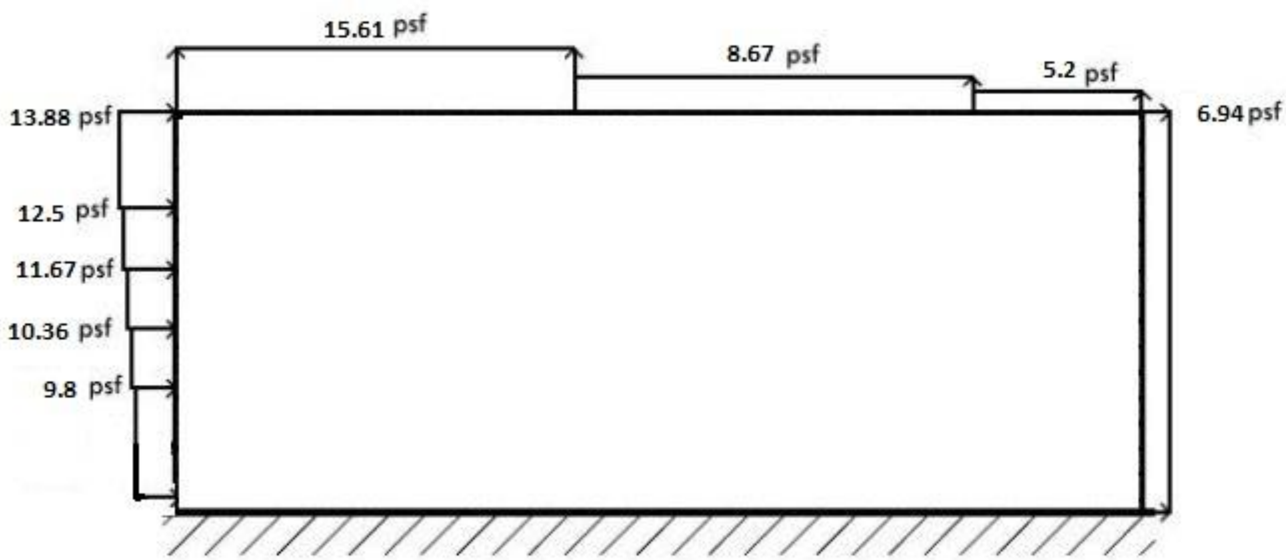


Figure 26 Wind Pressures acting on the East Wing, East and West facades

East Wing Wind Forces E-W Direction								
Floor	Height	Trib Below		Trib Above		Story Force	Story Shear	Overturning
	(ft)	height (ft)	area (sf)	height (ft)	area (sf)	(k)	(k)	Moment (k-ft)
Ground	0	0	0	10	2500	33.6	347.1	0.0
2nd	20	10	2500	8	2000	60.4	313.6	1208.0
3th	36	8	2000	8	2000	55.9	253.2	2012.3
4th	52	8	2000	10	2500	68.8	197.3	3576.9
Penthouse	72	10	2500	10.5	2625	82.6	128.5	5946.2
Roof	93	10.5	2625	0	0	45.9	45.9	4271.0
Total						347.1	N/A	17014.2

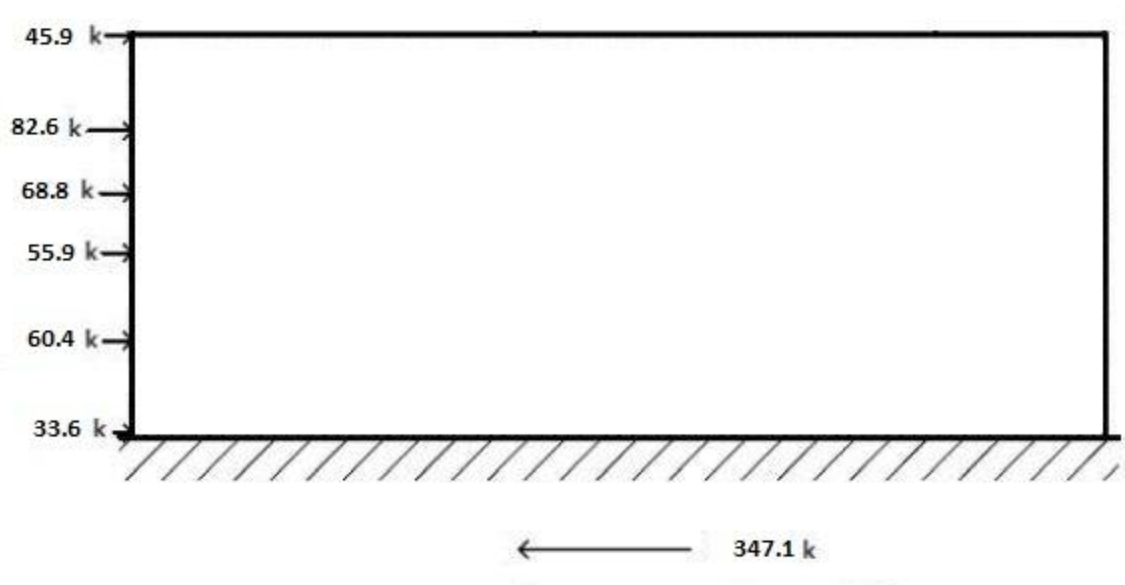


Figure 27 Wind Forces acting at each floor level on the East Wing, East and West facades

Seismic Loads

Seismic loads were calculated using ASCE 7-05, chapters 11 and 12. Sheet S201B in the structural drawings had a table with the seismic design data and from that, the other variables were easily calculated. Figure 28 is from S201B, showing the variables used. Figure 29 shows the excel chart of the calculated variables.

Through this analysis, the base shear was found to be 745k in both the North-South and East-West direction. The effective weight of the whole building was estimated based on the loads given. Each story force was found and was added together to determine the total base shear due to seismic. The forces will then be distributed to each moment frame based on stiffness. Figure 30, on the next page, shows that table with the distribution of forces, along with an elevation view.

SEISMIC DESIGN DATA

SEISMIC USE GROUP = III
SPECTRAL RESPONSE COEFFICIENTS
S_S = .199 S_1 = .058
SITE CLASS = B
SEISMIC IMPORTANCE FACTOR (Ie) = 1.25
SEISMIC DESIGN CATEGORY = A
BASIC SEISMIC FORCE RESISTING SYSTEM
ORDINARY STEEL MOMENT FRAMES
R = 3

Figure 28 Variables from structural drawings S201 B. Courtesy of Highland Associates.

Calculated Variables	
Fa	1
Fv	1
Sms	0.199
Sm_1	0.058
S_{DS}	0.133
S_{D1}	0.039
R	3.5
T	1.05
T_L	6
Cs	0.0333

Figure 29 Calculated Variables for Seismic

Vertical Distribution of Seismic Forces							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	1867	603893	0.252	187.78	187.78	17463.3
Penthouse	72	2163	504842	0.211	156.98	344.75	11302.4
4th	52	4286	660627	0.276	205.42	550.17	10681.7
3th	36	4286	413369	0.173	128.53	678.71	4627.2
2nd	20	4677	213197	0.089	66.29	745.00	1325.8
1st	0	5099	0	0.000	0.00	745.00	0.0
Total			2395927.97	1.000	745.00	N/A	45400.5

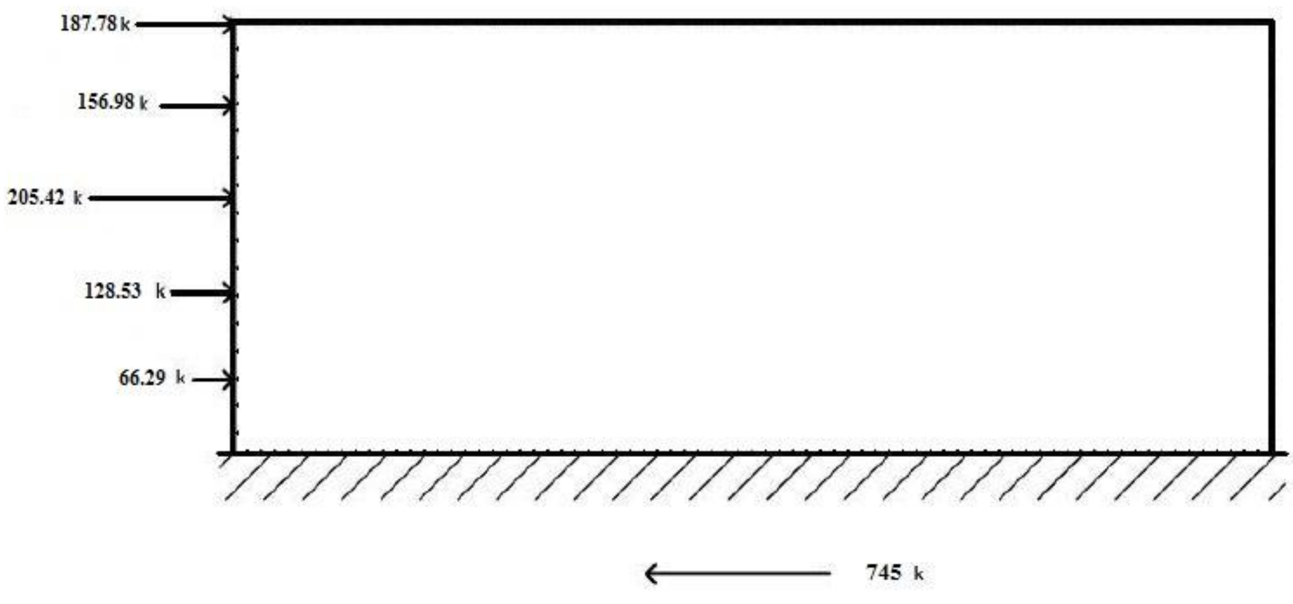


Figure 30 Table showing the vertical distribution of seismic forces with an elevation view. The same forces apply to both N-S and E-W direction

Comparison of Wind and Seismic Forces

By comparing the lateral loads produced by wind and seismic forces, it is clear that seismic forces controlled over wind forces in both North-South and East-West direction, as shown in Figure 31. The shear values have been factored by 1.6 for wind loads to allow for LRFD comparison between the two loads. TCMC is relatively a heavy and a short building so it was expected that seismic forces would controlled over wind.

Comparison of Seismic and Wind Forces			
	Wind, N-S	Wind, E-W	Seismic
Base Shear (k)	308	347	745
Overturning Moment (k-ft)	15111	17014	45400

Figure 31 Comparison of Seismic and Wind Forces

Lateral Load Path

As lateral forces from wind are applied to TCMC, they are transferred from the façade to the composite floor system through the connections. From there, the loads are transferred to the 15 main moment frames. These moment frames starts at the foundation and ends at the roof height for maximum effect. The loads are then transferred from the frames to the foundation.

Lateral forces for seismic loads are resisted by the foundations, and the 15 moment frames that run the height of the building. When each floor is seismically loaded, it transfers the load to the moment frames and then goes back to the foundation.

Lateral System Analysis

ETABS Models

A model of the lateral system for TCMC was designed using ETABS, shown in Figure 32. Line elements were used to model the moment frames, which are the columns and the beams. These were given the exact steel section according to the structural drawings. All columns and beams are W-flange members. Area elements were used to model the floor and the roof, and also, were given the exact materials, weight, and properties as shown on the structural plans. Because TCMC uses a steel deck with concrete topping, the diaphragm was assumed to be rigid in ETABS.

The moment frames on the penthouse was a special case. It was modeled to have moment connections only at the roof. For the rest of the beams and columns below, moment release were assigned to them. Figure 33 and 34 shows the location of the moment frames and the rigid floor diaphragms on the 2nd and 3rd story level.

Because TCMC is a complex building, it was also designed on ETABS as 3 individual buildings, the West wing, the East Wing, and the Link. Moment frames for each of these are shown in Figure 35.1 to 35.3. The outputs of these small models were then compared to the main model and the results were very similar.

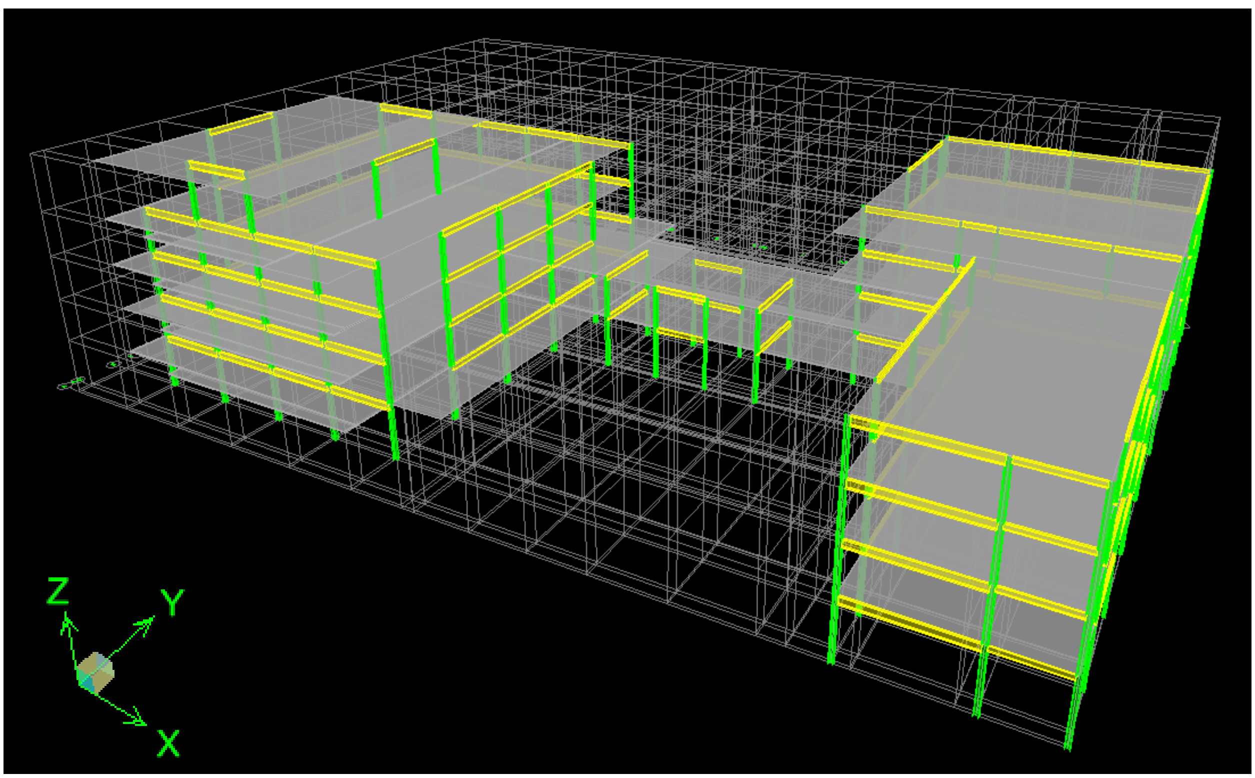


Figure 32 ETABS model of TCMC

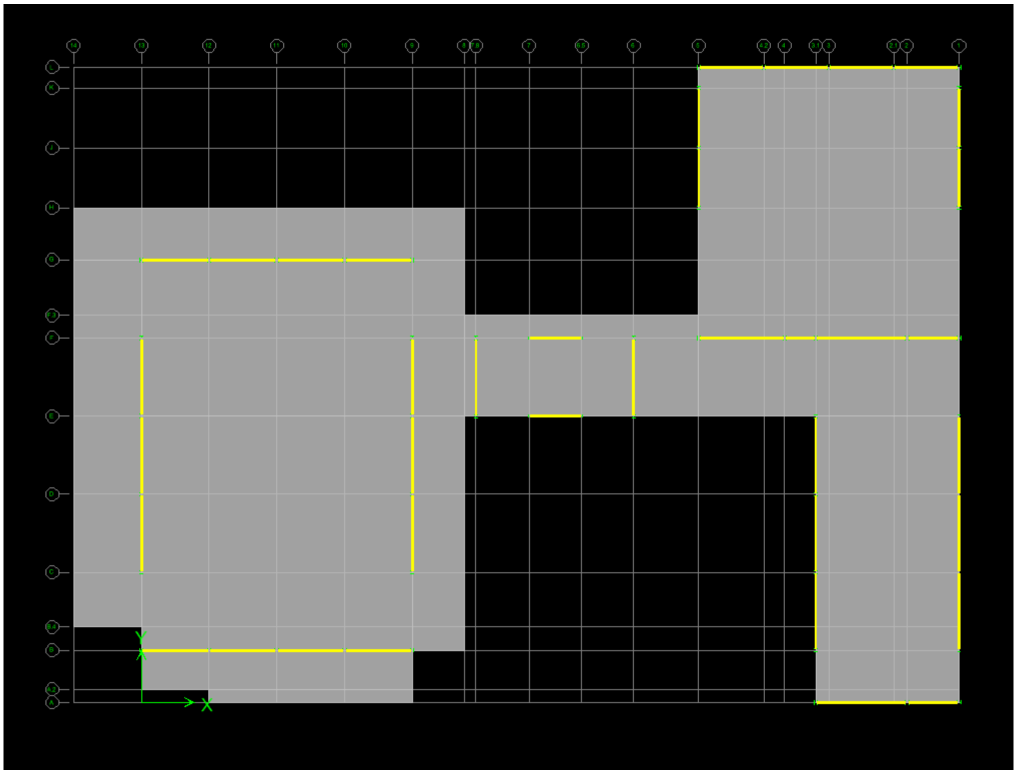


Figure 33 2nd Story Moment Frames with Rigid Diaphragm

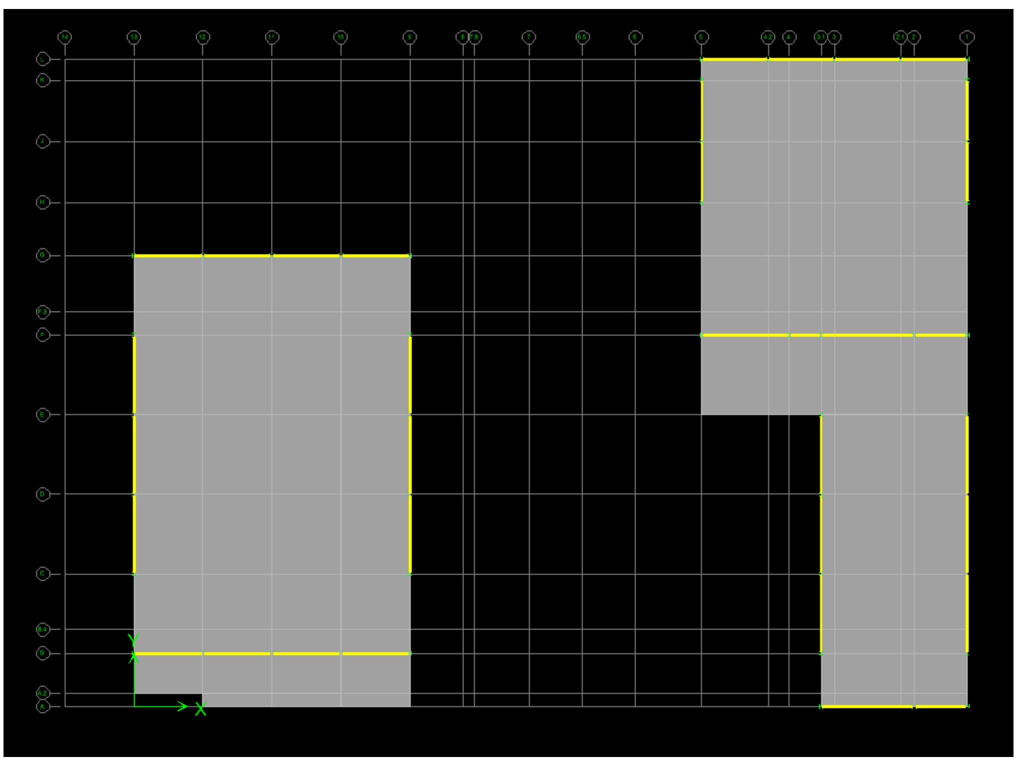


Figure 34 3rd Story Moment Frames with Rigid Diaphragm

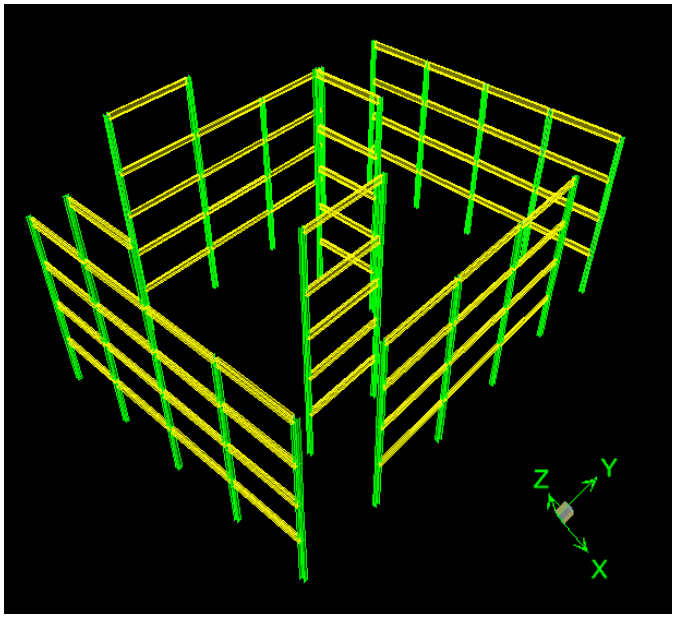


Figure 35.1 3-D view of West Wing Moment Frames

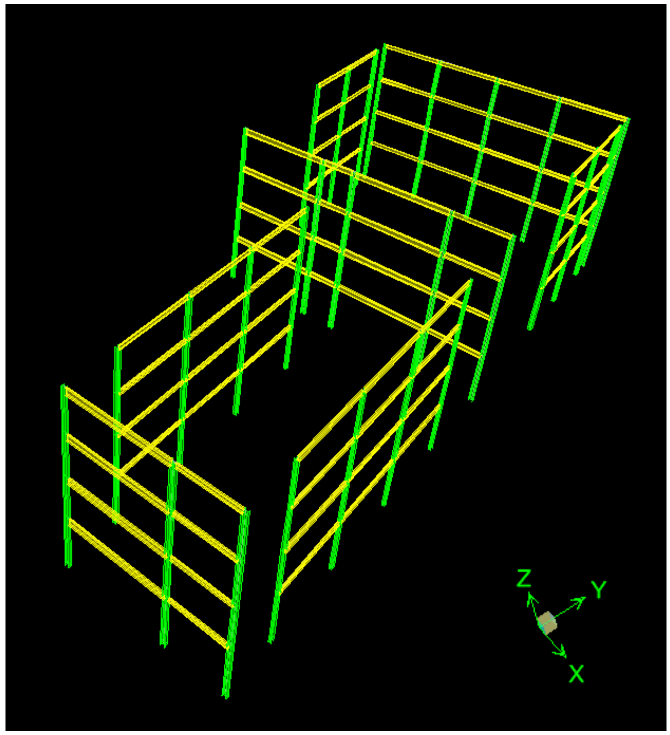


Figure 35.2 3-D view of East Wing Moment Frames

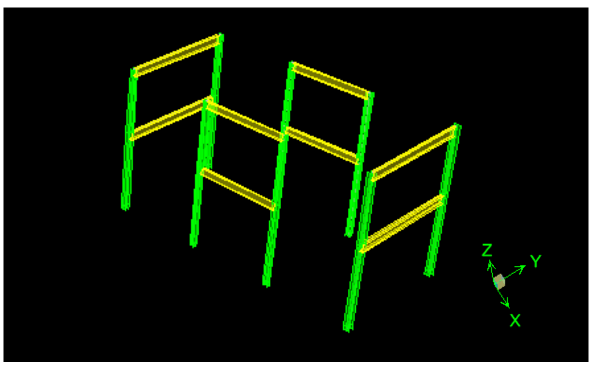


Figure 35.3 3-D view of the Link between the two Wings

Relative Stiffness and Rigidity

The transfer of load to the moment frames depend on the stiffness of that frame. The stiffer the frame, the more load it can transfer. Figure 36 shows the location of the 15 main frames in TCMC that were analyzed. This does not include the penthouse moment frames. The stiffness of each frame was found using the equation K=P/δ. P is the 1k horizontal load that was applied to each frame at the main roof level δ is the defection obtained at the main roof level from the 1k load. Frame D was found to be the stiffest while Frame M was found to be the least stiff. Relative stiffness was also calculated referencing Frame D. Obtaining the stiffness of each frame is important because this information is necessary to compute for direct shear and torsional shear. All values are shown in Figure 37, on the next page.

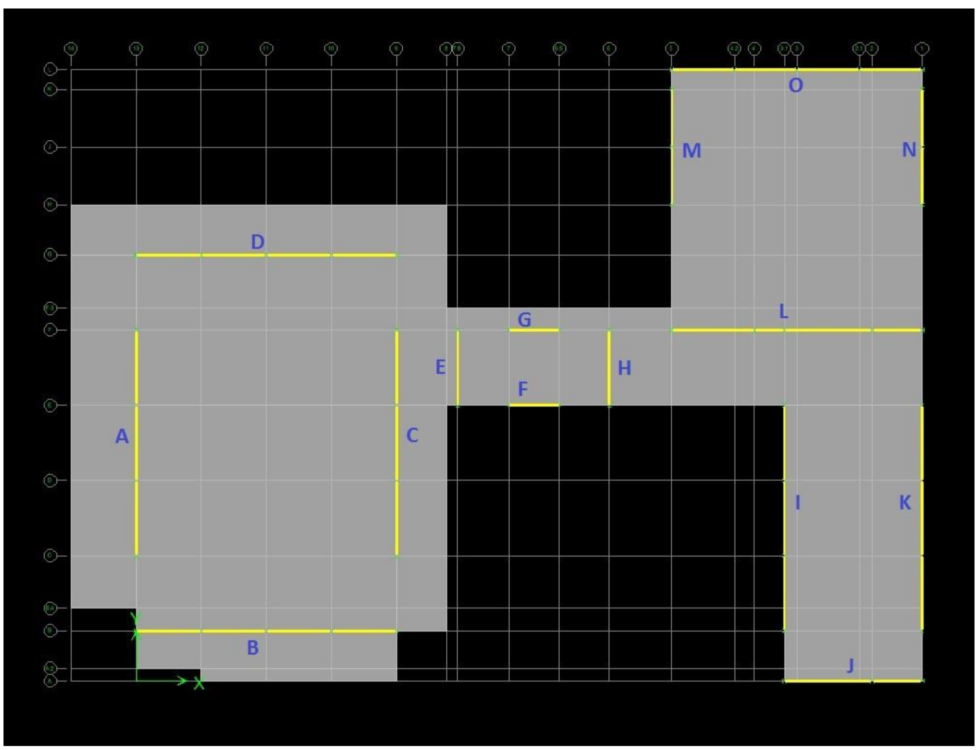


Figure 36 Moment Frame Location

Relative Stiffness of Moment Frames				
Frame	Force at Mainroof Level (k)	Displacement at Mainroof (in)	Stiffness of Frame, K (k/in)	Relative Stiffness of Frame, K_{rel} (k/in)
A	1	0.0146	68.46	0.409
B	1	0.0073	136.67	0.817
C	1	0.0138	72.73	0.435
D	1	0.0060	167.31	1.000
I	1	0.0174	57.47	0.344
J	1	0.0098	102.04	0.610
K	1	0.0172	58.14	0.348
L	1	0.0111	90.09	0.538
M	1	0.0176	56.82	0.340
N	1	0.0172	58.14	0.348
O	1	0.0121	82.64	0.494
Frame	Force at Mainroof Level (k)	Displacement at 3rd Level (in)	Stiffness of Frame, K (k/in)	Relative Stiffness of Frame, K_{rel} (k/in)
E	1	0.0113	88.56	0.529
F	1	0.0146	68.72	0.411
G	1	0.0159	62.75	0.375
H	1	0.0138	72.39	0.433

Figure 37 Relative Stiffness of Moment Frames

Load Combinations

Figure 38 below shows the load combinations from ASCE 7-05 that was considered when modeling TCMC on ETABS. However, only the load combinations that contained lateral load were considered, which removes load combination 1 and 2. The loads of 1.6W and 1.0E were also considered for the analysis.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Figure 38 ASCE 7-05 Basic Combinations

Figure 39 shows the four wind load cases from ASCE 7-05 that was also considered to find the controlling load. Overall, case 1 controlled the wind forces for TCMC. However, seismic still controlled the design in both directions.

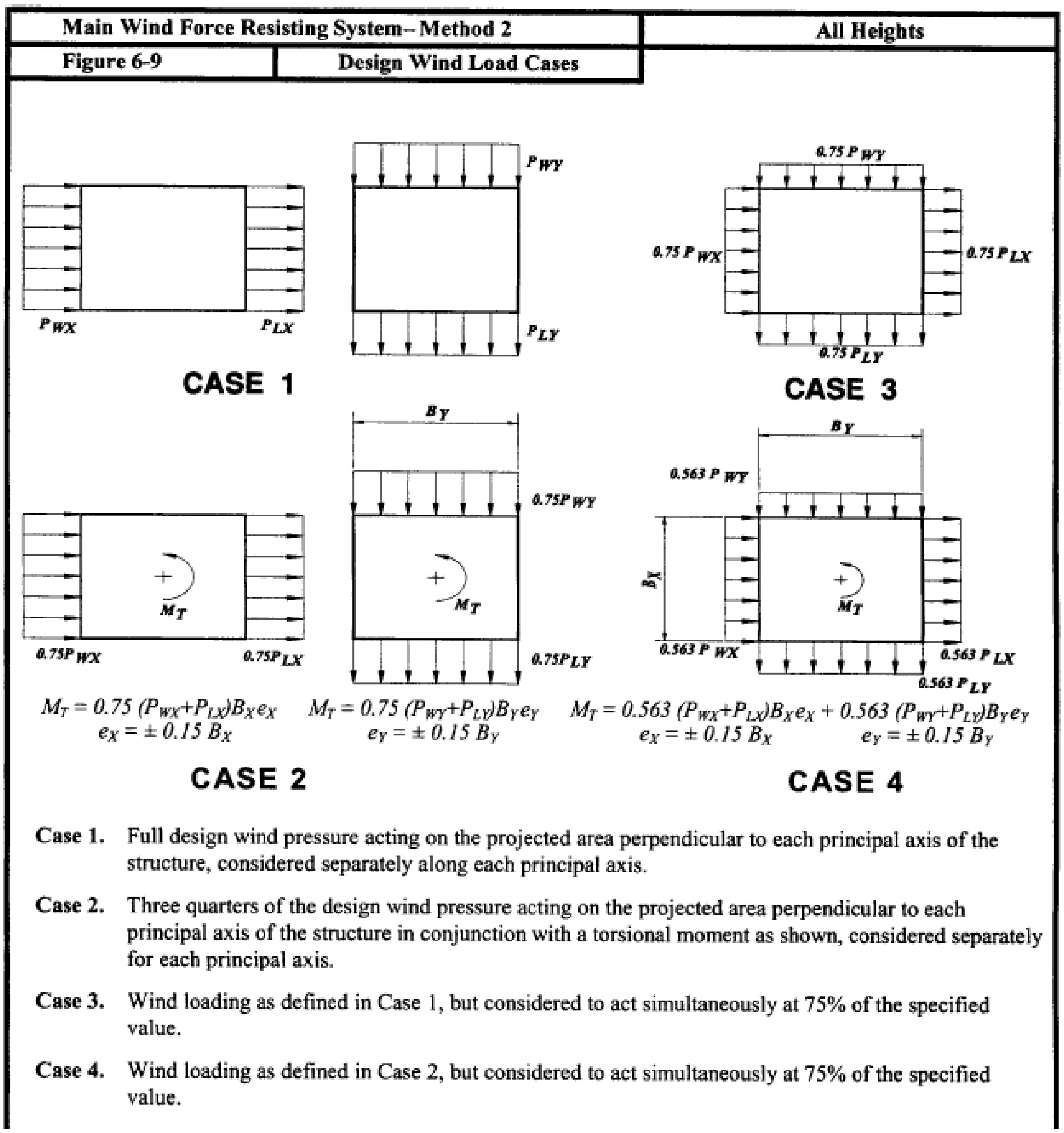


Figure 39 Wind Load Cases from ASCE 7-05

Building Torsion

Building torsion was found due to seismic force, North-South wind force, and East-West wind force. The total moment found is the sum of torsional and accidental moment. In ETABS incidental torsion were accounted for but the torsion due to the difference in the center of rigidity from the center of mass was not. To find the accidental moment in ETABS, an assumed 5% eccentricity was used in the model.

Figures 40 and 41 shows that seismic forces are the controlling factors for building torsion. This is due to larger story forces that an earthquake is assumed to produce near Scranton, PA. The moment in the North-South direction was found to be larger than in the East-West direction because of the longer building length of the North-South wall.

Building Torsion, N-S Direction (Seismic)							
Floor Level	Story Force (k)	Center of Rigidity	Center of Mass	e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment M_T (k-ft)
Pentroof	187.8	25.8	29.0	3.15	591.5	3229.8	3821.3
Mainroof	157.0	134.5	125.8	-8.7	-1365.7	2700.1	1334.3
4th	205.4	153.3	160.6	7.29	1497.5	3533.2	5030.7
3th	128.5	154.0	159.7	5.65	726.2	2210.7	2936.9
2nd	66.3	126.9	127.8	0.93	61.6	1140.2	1201.8
						Sum =	14325

Building Torsion, E-W Direction (Seismic)							
Floor Level	Story Force (k)	Center of Rigidity	Center of Mass	e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment M_T (k-ft)
Pentroof	187.8	44.6	47.6	2.97	557.7	2328.5	2886.2
Mainroof	157.0	99.5	100.3	0.78	122.4	1946.6	2069.0
4th	205.4	110.3	112.2	1.89	388.2	2547.2	2935.5
3th	128.5	111.4	114.0	2.62	336.7	1593.8	1930.5
2nd	66.3	111.4	109.9	-1.49	-98.8	822.0	723.2
						Sum =	10544

Figure 40 Building Torsion due to Seismic forces

Building Torsion, N-S Direction (Wind)							
Floor Level	Story Force (k)	Center of Rigidity	Center of Mass	e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment M_T (k-ft)
Pentroof	40.8	25.8	29.0	3.15	128.5	701.8	830.3
Mainroof	73.3	134.5	125.8	-8.70	-637.7	1260.8	623.0
4th	61.1	153.3	160.6	7.29	445.4	1050.9	1496.3
3th	49.7	154.0	159.7	5.65	280.8	854.8	1135.6
2nd	53.8	126.9	127.8	0.93	50.0	925.4	975.4
						Sum =	5061

Building Torsion, E-W Direction (Wind)							
Floor Level	Story Force (k)	Center of Rigidity	Center of Mass	e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment M_T (k-ft)
Pentroof	45.9	44.6	47.6	2.97	136.3	569.2	705.5
Mainroof	82.6	99.5	100.3	0.78	64.4	1024.2	1088.7
4th	68.8	110.3	112.2	1.89	130.0	853.1	983.2
3th	55.9	111.4	114.0	2.62	146.5	693.2	839.6
2nd	60.4	111.4	109.9	-1.49	-90.0	749.0	659.0
						Sum =	4276

Figure 41 Building Forces due to Wind forces

Lateral Load Distribution

Direct Shear

Direct Shear was calculated for all 15 of the main moment frames, shown again on Figure 42. Part of Table 43 and 44 shows the distribution of direct shear to each frame based on their stiffness in both directions and for both wind and seismic loads. Frames A, C, E, H, M, N, I, and K, resist direct shear in the North-South direction, and Frames B, D, B, G, O, L, J, resist direct shear in the East-West direction. Frame D resist the most shear because it has the highest relative stiffness. Seismic forces controlled over wind forces in both direction so the frames experience more direct shear under seismic forces. Direct shear calculation can be found on Appendix G.

Torsional Shear

Unlike direct shear, all frames experience torsional shear regardless of the direction it is in. The torsional shear from the difference between the center of rigidity and the center of mass, outputs from ETABS, was calculated. The values of torsional shear are also found on Figure 43 and 44. Both shear values were added together to find the total shear that the frames will experience. Torsional shear calculation can also be found on Appendix G.

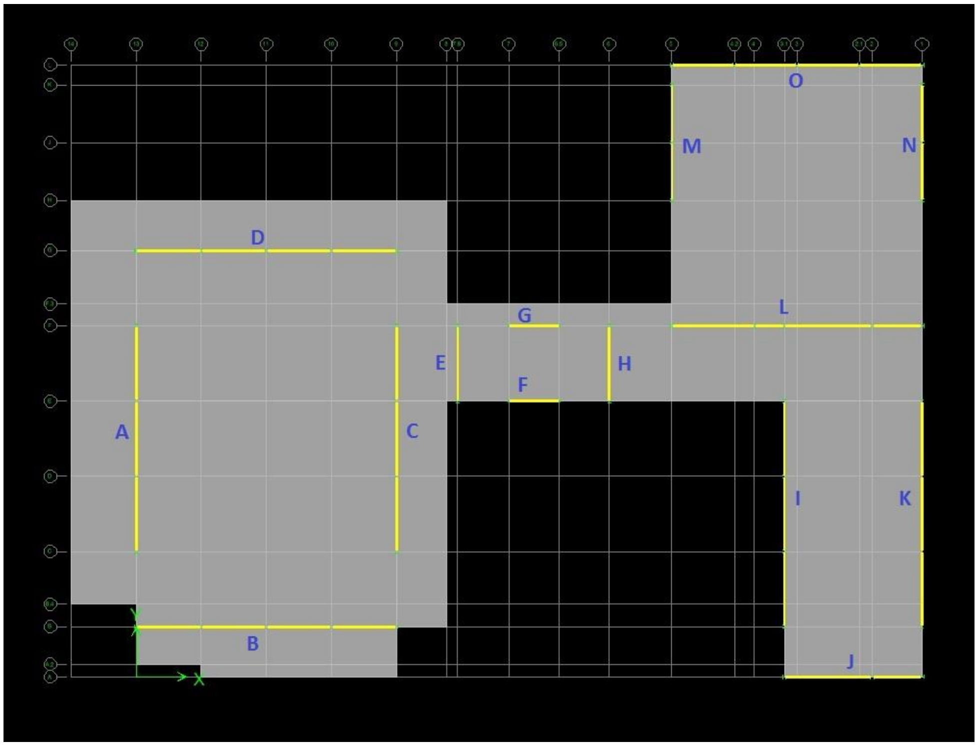


Figure 42 Moment Frame Locations

North-South Direction Wind									
Frame	Stiffness (K)	Lateral Force (kips)	e_x	e_y	d	$K*d^2$	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
A	68.46	308	8.70	2.98	-134.5	1238459	39.58	-2.20	37.38
C	72.73	308	8.70	2.98	-30.5	67657	42.05	-0.53	41.52
E	88.56	308	8.70	2.98	-6.0	3188	51.20	-0.13	51.08
H	72.39	308	8.70	2.98	54.5	215016	41.85	0.94	42.80
M	56.82	308	8.70	2.98	79.5	359117	32.85	1.08	33.93
N	58.14	308	8.70	2.98	179.5	1873285	33.62	2.49	36.11
I	57.47	308	8.70	2.98	124.5	890799	33.23	1.71	34.94
K	58.14	308	8.70	2.98	179.5	1873285	33.62	2.49	36.11
B	136.67	308	8.70	2.98	79.5	863789	0.00	2.60	2.60
D	167.31	308	8.70	2.98	-70.5	831573	0.00	-2.82	-2.82
F	68.72	308	8.70	2.98	-10.5	7576	0.00	-0.17	-0.17
G	62.75	308	8.70	2.98	-40.5	102926	0.00	-0.61	-0.61
O	82.64	308	8.70	2.98	-144.5	1725544	0.00	-2.85	-2.85
L	90.09	308	8.70	2.98	-40.5	147770	0.00	-0.87	-0.87
J	102.04	308	8.70	2.98	99.5	1010222	0.00	2.43	2.43
Total K_{N-S}	532.71				Sum=	11210205	308.0		311.56

East-West Direction Wind									
Frame	Stiffness (K)	Lateral Force (kips)	e_x	e_y	d	$K*d^2$	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
A	68.46	347	8.70	2.98	-134.5	1238459	0.00	-2.48	-2.48
C	72.73	347	8.70	2.98	-30.5	67657	0.00	-0.60	-0.60
E	88.56	347	8.70	2.98	-6.0	3188	0.00	-0.14	-0.14
H	72.39	347	8.70	2.98	54.5	215016	0.00	1.06	1.06
M	56.82	347	8.70	2.98	79.5	359117	0.00	1.22	1.22
N	58.14	347	8.70	2.98	179.5	1873285	0.00	2.81	2.81
I	57.47	347	8.70	2.98	124.5	890799	0.00	1.93	1.93
K	58.14	347	8.70	2.98	179.5	1873285	0.00	2.81	2.81
B	136.67	347	8.70	2.98	79.5	863789	66.77	2.93	69.70
D	167.31	347	8.70	2.98	-70.5	831573	81.74	-3.18	78.57
F	68.72	347	8.70	2.98	-10.5	7576	33.58	-0.19	33.38
G	62.75	347	8.70	2.98	-40.5	102926	30.66	-0.68	29.97
O	82.64	347	8.70	2.98	-144.5	1725544	40.38	-3.22	37.16
L	90.09	347	8.70	2.98	-40.5	147770	44.02	-0.98	43.03
J	102.04	347	8.70	2.98	99.5	1010222	49.85	2.73	52.59
Total K_{E-W}	710.22				Sum=	11210205	347.0		351.01

Figure 43 Direct and Torsional Shear produced by wind forces.

North-South Direction Seismic									
Frame	Stiffness (K)	Lateral Force (kips)	e_x	e_y	d	$K*d^2$	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
A	68.46	745	8.70	2.98	-134.5	1238459	95.74	-5.32	90.42
C	72.73	745	8.70	2.98	-30.5	67657	101.71	-1.28	100.43
E	88.56	745	8.70	2.98	-6.0	3188	123.85	-0.31	123.54
H	72.39	745	8.70	2.98	54.5	215016	101.24	2.28	103.52
M	56.82	745	8.70	2.98	79.5	359117	79.46	2.61	82.08
N	58.14	745	8.70	2.98	179.5	1873285	81.31	6.03	87.34
I	57.47	745	8.70	2.98	124.5	890799	80.37	4.14	84.51
K	58.14	745	8.70	2.98	179.5	1873285	81.31	6.03	87.34
B	136.67	745	8.70	2.98	79.5	863789	0.00	6.28	6.28
D	167.31	745	8.70	2.98	-70.5	831573	0.00	-6.82	-6.82
F	68.72	745	8.70	2.98	-10.5	7576	0.00	-0.42	-0.42
G	62.75	745	8.70	2.98	-40.5	102926	0.00	-1.47	-1.47
O	82.64	745	8.70	2.98	-144.5	1725544	0.00	-6.90	-6.90
L	90.09	745	8.70	2.98	-40.5	147770	0.00	-2.11	-2.11
J	102.04	745	8.70	2.98	99.5	1010222	0.00	5.87	5.87
Total K_{N-S}	532.71				Sum=	11210205	745.0		753.62

East-West Direction Wind									
Frame	Stiffness (K)	Lateral Force (kips)	e_x	e_y	d	$K*d^2$	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
A	68.46	745	8.70	2.98	-134.5	1238459	0.00	-5.32	-5.32
C	72.73	745	8.70	2.98	-30.5	67657	0.00	-1.28	-1.28
E	88.56	745	8.70	2.98	-6.0	3188	0.00	-0.31	-0.31
H	72.39	745	8.70	2.98	54.5	215016	0.00	2.28	2.28
M	56.82	745	8.70	2.98	79.5	359117	0.00	2.61	2.61
N	58.14	745	8.70	2.98	179.5	1873285	0.00	6.03	6.03
I	57.47	745	8.70	2.98	124.5	890799	0.00	4.14	4.14
K	58.14	745	8.70	2.98	179.5	1873285	0.00	6.03	6.03
B	136.67	745	8.70	2.98	79.5	863789	143.36	6.28	149.64
D	167.31	745	8.70	2.98	-70.5	831573	175.50	-6.82	168.68
F	68.72	745	8.70	2.98	-10.5	7576	72.09	-0.42	71.67
G	62.75	745	8.70	2.98	-40.5	102926	65.82	-1.47	64.35
O	82.64	745	8.70	2.98	-144.5	1725544	86.69	-6.90	79.78
L	90.09	745	8.70	2.98	-40.5	147770	94.50	-2.11	92.39
J	102.04	745	8.70	2.98	99.5	1010222	107.04	5.87	112.91
Total K_{E-W}	710.22				Sum=	11210205	745.0		753.62

Figure 44 Direct and Torsional Shear produced by seismic forces.

Lateral Displacements and Story Drifts

The lateral displacements and story drifts for each frame was found from ETABS outputs. This was done by using the controlling loads, seismic, in each direction. The largest displacement found is at the penthouse roof level, which is 0.347 in. The largest story drift is 0.0055at frame B, penthouse roof level.

The story drift found was compared to the allowable story drift by code. Table 45 shows the formula for determining the allowable story drift. TCMC is an occupancy category III building so it requires the formula 0.015h. It was found that all floors levels in each direction met the serviceability requirements for seismic.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

Figure 45 From ASCE 7-05, Allowable Story Drift limit by code

Table 46 and 47 shows the total floor displacement at each level, and the inter-story drift value for each level. Also, the allowable drift value by code is listed. All story levels have met the drift requirement.

Story Drift (Seismic Controlling Force), N-S Direction					
	Floor	Displacement (in)	Story Drift (in)	Allowable Drift (in)	Met Code?
A	Pentroof	0.346	0.0059	1.40	Yes
	Mainroof	0.226	0.0026	1.08	Yes
	4th	0.183	0.0034	0.78	Yes
	3th	0.129	0.0039	0.54	Yes
	2nd	0.065	0.0034	0.30	Yes
C	Mainroof	0.201	0.0022	1.08	Yes
	4th	0.165	0.0030	0.78	Yes
	3th	0.116	0.0035	0.54	Yes
	2nd	0.059	0.0031	0.30	Yes
E	3th	0.120	0.0036	0.54	Yes
	2nd	0.061	0.0032	0.30	Yes
H	3th	0.120	0.0036	0.54	Yes
	2nd	0.061	0.0032	0.30	Yes
M	Mainroof	0.187	0.0020	1.08	Yes
	4th	0.154	0.0027	0.78	Yes
	3th	0.110	0.0033	0.54	Yes
	2nd	0.056	0.0029	0.30	Yes
N	Mainroof	0.187	0.0020	1.08	Yes
	4th	0.154	0.0028	0.78	Yes
	3th	0.110	0.0033	0.54	Yes
	2nd	0.056	0.0029	0.30	Yes
I	Mainroof	0.235	0.0028	1.08	Yes
	4th	0.189	0.0035	0.78	Yes
	3th	0.133	0.0041	0.54	Yes
	2nd	0.067	0.0035	0.30	Yes
K	Mainroof	0.210	0.0024	1.08	Yes
	4th	0.171	0.0031	0.78	Yes
	3th	0.120	0.0036	0.54	Yes
	2nd	0.061	0.0032	0.30	Yes

Figure 46 Displacement and Story Drift caused by seismic forces in the N-S direction.

Story Drift (Seismic Controlling Force), E-W Direction					
	Floor	Displacement (in)	Story Drift (in)	Allowable Drift (in)	Met Code?
B	Pentroof	0.347	0.0055	1.40	Yes
	Mainroof	0.235	0.0028	1.08	Yes
	4th	0.190	0.0035	0.78	Yes
	3th	0.133	0.0040	0.54	Yes
	2nd	0.067	0.0035	0.30	Yes
D	Mainroof	0.193	0.0021	1.08	Yes
	4th	0.158	0.0028	0.78	Yes
	3th	0.112	0.0034	0.54	Yes
	2nd	0.057	0.0030	0.30	Yes
F	3th	0.121	0.0036	0.54	Yes
	2nd	0.061	0.0032	0.30	Yes
G	3th	0.116	0.0035	0.54	Yes
	2nd	0.059	0.0031	0.30	Yes
O	Mainroof	0.172	0.0018	1.08	Yes
	4th	0.143	0.0025	0.78	Yes
	3th	0.102	0.0031	0.54	Yes
	2nd	0.052	0.0027	0.30	Yes
L	Mainroof	0.201	0.0022	1.08	Yes
	4th	0.165	0.0030	0.78	Yes
	3th	0.116	0.0035	0.54	Yes
	2nd	0.059	0.0031	0.30	Yes
J	Mainroof	0.240	0.0029	1.08	Yes
	4th	0.190	0.0036	0.78	Yes
	3th	0.135	0.0041	0.54	Yes
	2nd	0.069	0.0036	0.30	Yes

Figure 47 Displacement and Story Drift caused by seismic forces in the E-W direction.

Overturning and Foundation Stability

Determining the effects of overturning moment on the foundation system is crucial when designing for the foundations and the lateral systems. The foundations must be strong enough to resist both the gravity load of the building and the moment caused by the lateral loads. Table 48 below shows the overturning moment that the lateral forces had cause. The largest moment was found to be from seismic in both the North-South and East-West direction, which is 45,401k-ft. However, the building's resisting moment for the North-South direction was found to be 1,241,979k-ft and for the East-West direction, was found to be 643,368k-ft. These resisting moments are far greater, more than 12 times that of the overturning moments, which is acceptable. Foundations are designed with a high safety factor because the whole building depends on it to work properly.

Overturning and Resisting Moments							
Floor	Height (ft)	Seismic		N-S Wind		E-W Wind	
		Lateral Force (k)	Moment (k-ft)	Lateral Force (k)	Moment (k-ft)	Lateral Force (k)	Moment (k-ft)
Pentroof	93	187.78	17463.54	40.8	3794.4	45.9	4268.7
Mainroof	72	156.98	11302.56	73.3	5277.6	82.6	5947.2
4th	52	205.42	10681.84	61.1	3177.2	68.8	3577.6
3th	36	128.53	4627.08	49.7	1789.2	55.9	2012.4
2nd	20	66.29	1325.8	53.8	1076	60.4	1208
Overturning Moment		Sum=	45401	Sum=	15114	Sum=	17014
Resisting Moment =			643368		1241979		643368

Figure 48 Overturning moment caused by seismic lateral force and the resisting moment of TCMC.

Moment Frame Capacity Check

Spot checks were performed on frame D in TCMC. It proved that the structural elements of the lateral system had a much greater capacity than required to resist both the gravity loads and the lateral loads. The existing beams and columns for TCMC were found to be oversized. For column G12, it was found that a W14x 90 was sufficient for the given loads while TCMC uses W14x 257. For the beam, a W18x97 was sufficient but TCMC uses W 30x99, which is close in terms of which is more economical. The frame with the column and beam section is shown in Figure 49 below. Because the columns and beams are oversized the cost of the building is increased. Using oversized elements may be one of the reasons why TCMC cost over $600 per square foot.

Figure 49 Frame D: The column and beam sections that TCMC uses

Conclusion

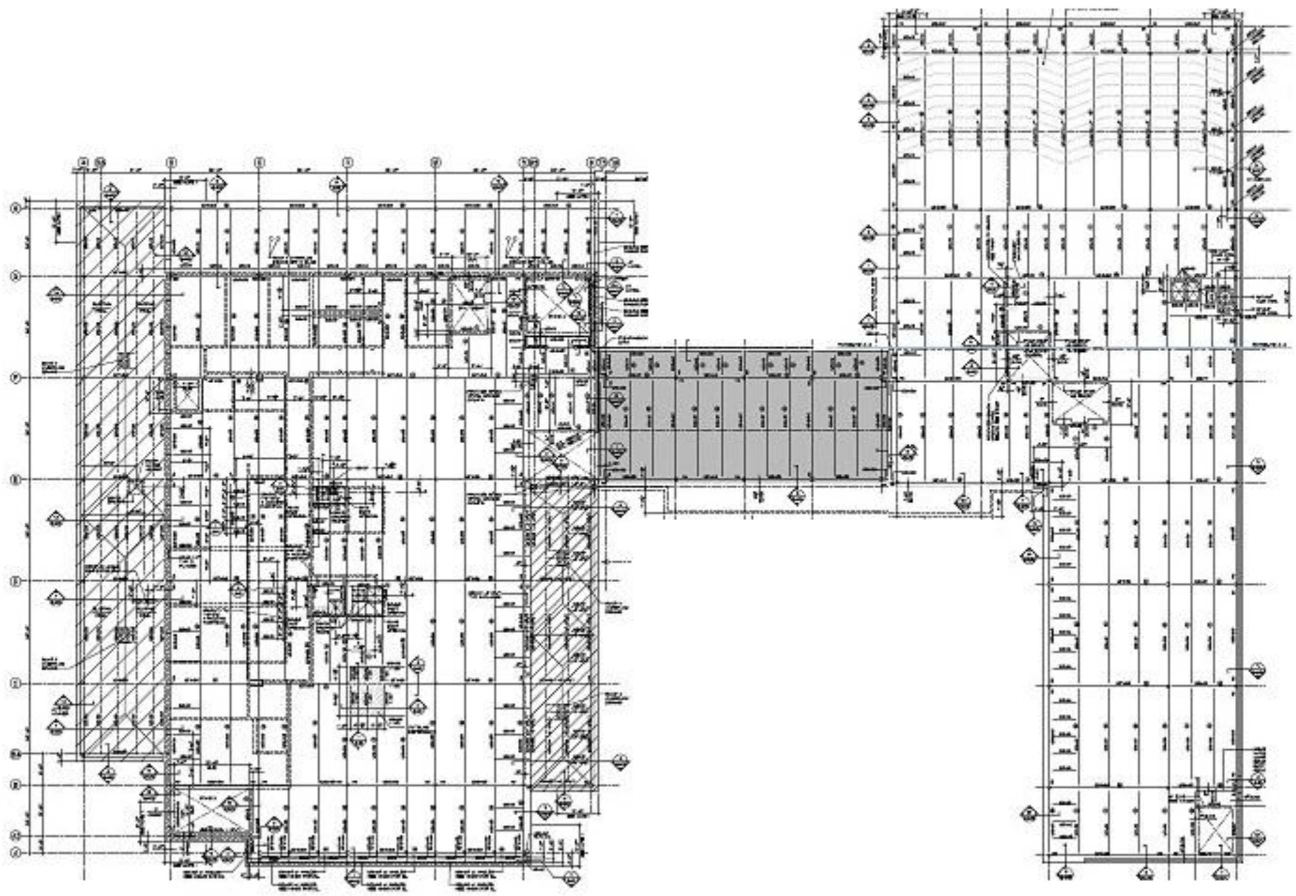
Through analysis, the lateral system of The Commonwealth Medical College (TCMC) was found to be sufficient to carry both the seismic and wind forces in each direction and met serviceability requirements set forth by ASCE 7-05. Hand calculations, Excel spreadsheets, and an ETABS model were used to complete this analysis. Hand calculations were done to confirm the outputs of the ETABS model to determine that the model was designed properly.

Using ETABS, TCMC's 15 main moment frames, and the 4 additional penthouse moment frames, were modeled, along with rigid diaphragms for each story level. The outputs were then analyzed and some verified with hand calculations. The outputs obtained was used to review for stiffness, controlling load combinations, direct and torsional shear, building torsion, lateral displacement, story drift, serviceability, overturning moments, and the strength of the framing elements. It was found that the building as whole, performed very effectively. The foundation and the lateral system were sufficient to carry the loads.

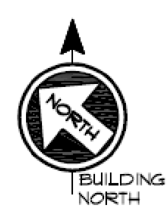
Seismic forces were found to be the controlling factor in both the North-South and East-West direction. This is important because seismic forces also caused the greatest overturning moment in the foundations. However, TCMC could resist more than 12 times of that force, which makes the design acceptable. The distribution of shears from seismic forces to the frames showed that frame D took the largest load because it has the highest stiffness, as determined earlier in the report.

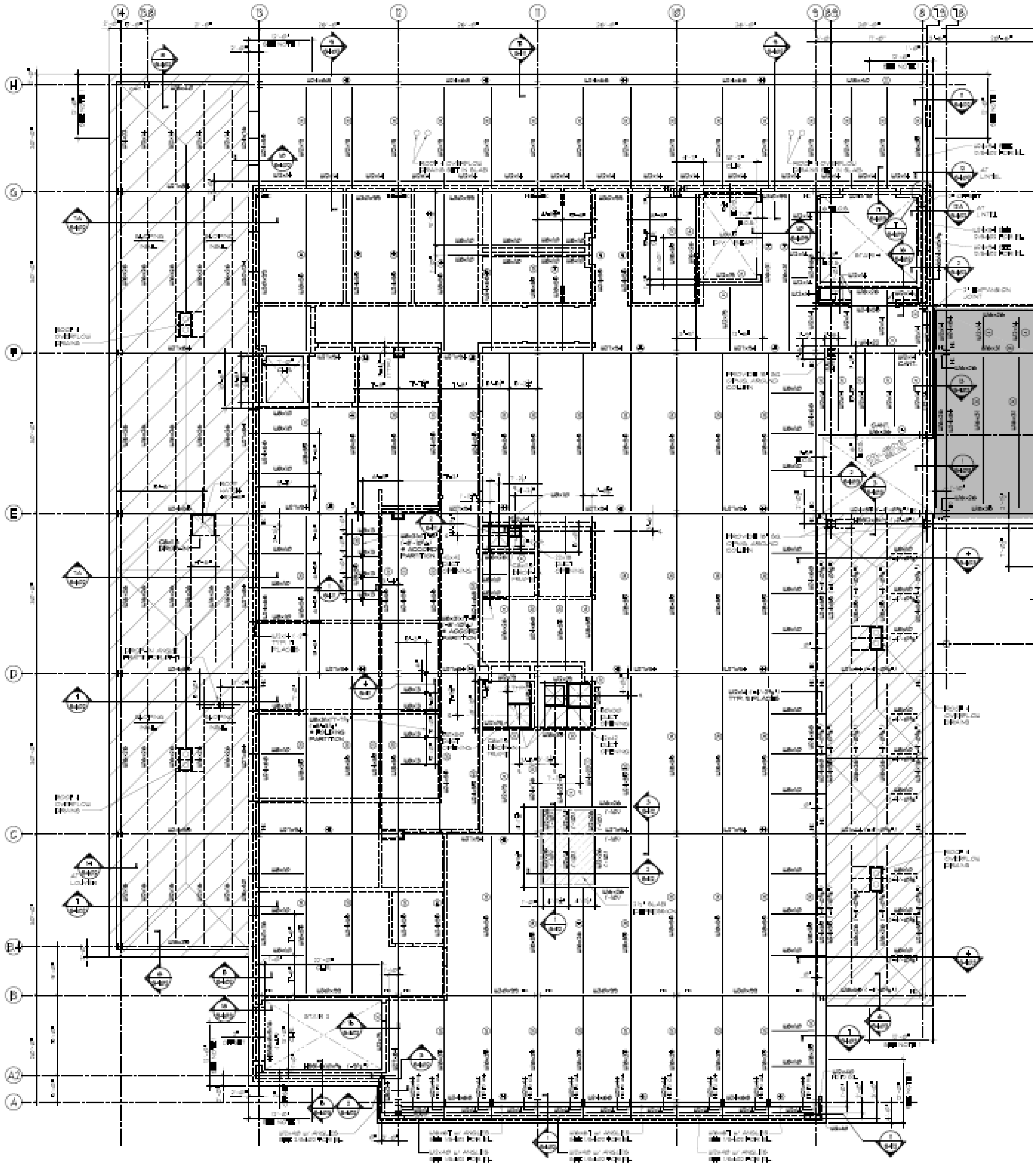
Lastly, spot checks where done on a typical frame column and beam and it proved that the structural elements of the lateral system had a much greater capacity than required to resist both the gravity loads and the lateral loads. The existing beams and columns for TCMC were found to be oversized. This may be one of the reasons why TCMC cost over $600 per square foot.

Appendix A

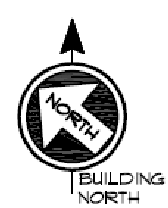


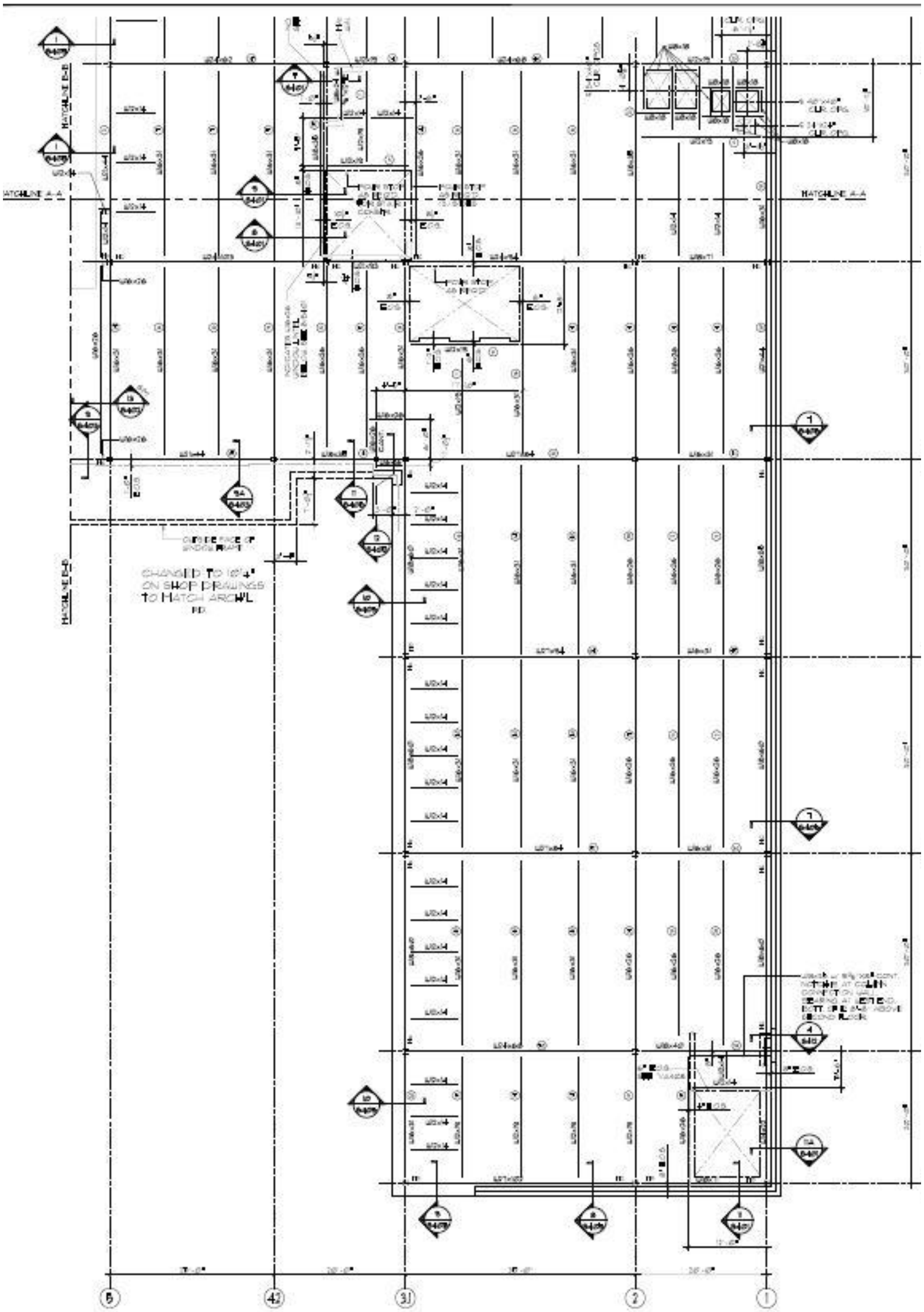
Framing Plan of the 2nd Floor, Courtesy of Highland Associates



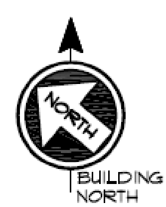


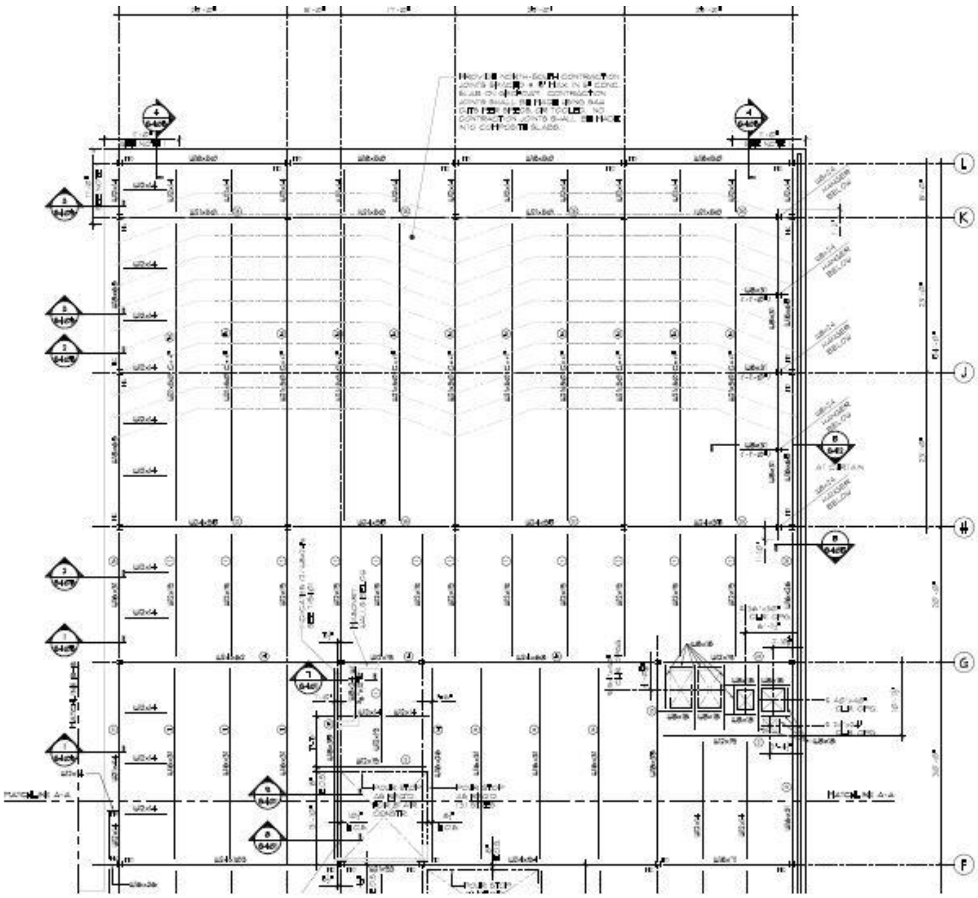
2nd Story frame, west wing, Courtesy of Highland Associates



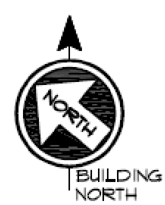


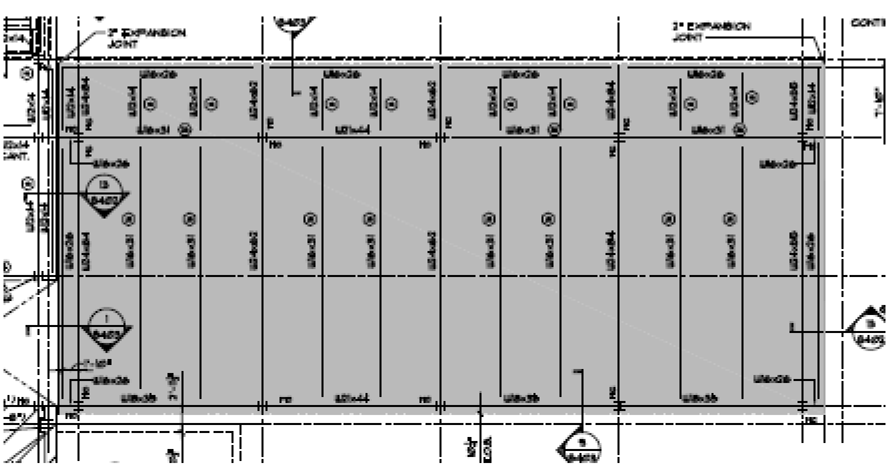
2nd Story frame, east wing (south), Courtesy of Highland Associates



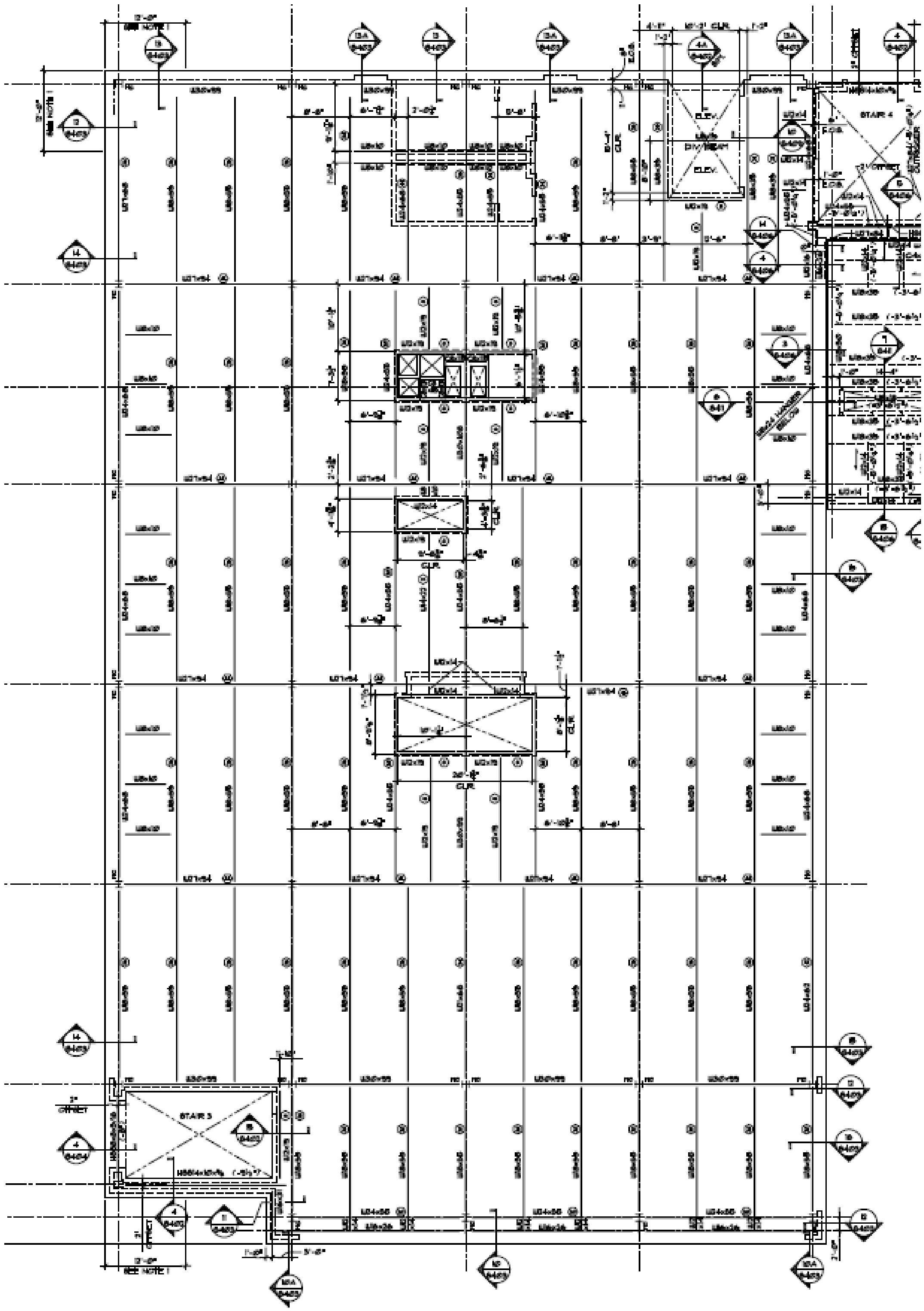


2nd Story frame, east wing (north), Courtesy of Highland Associates

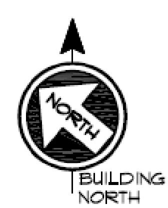


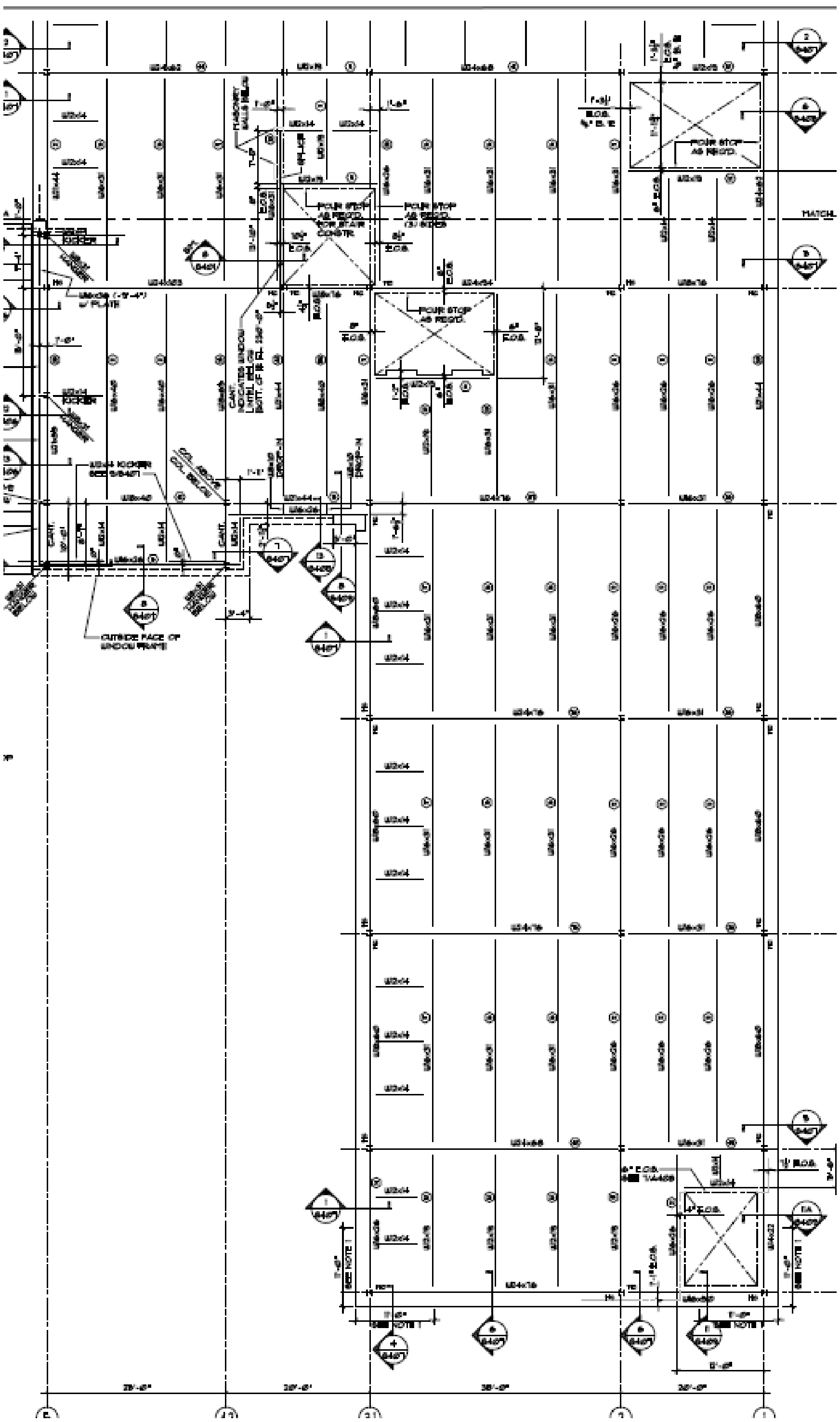


2nd Story frame, Link, Courtesy of Highland Associates

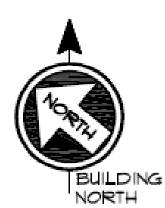


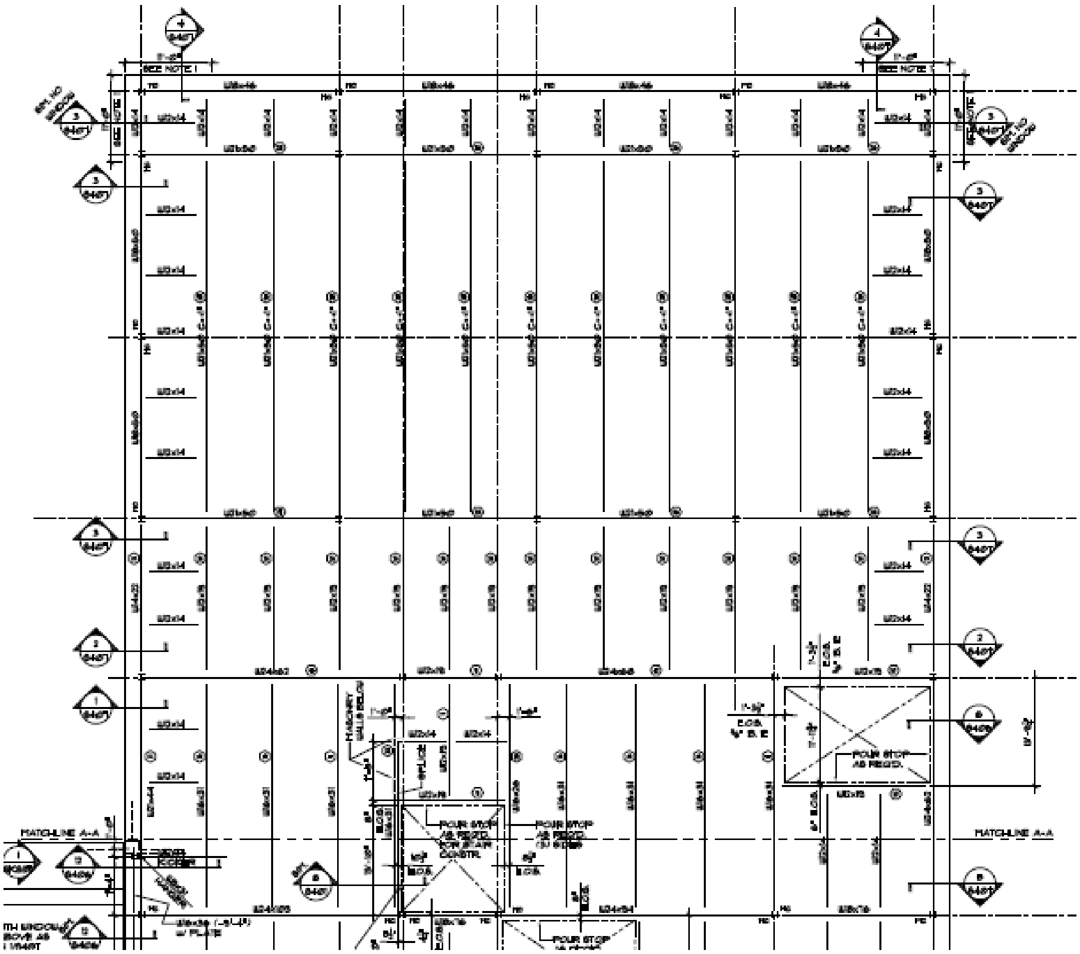
3rd Story frame, west wing, Courtesy of Highland Associates



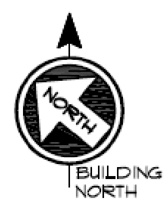


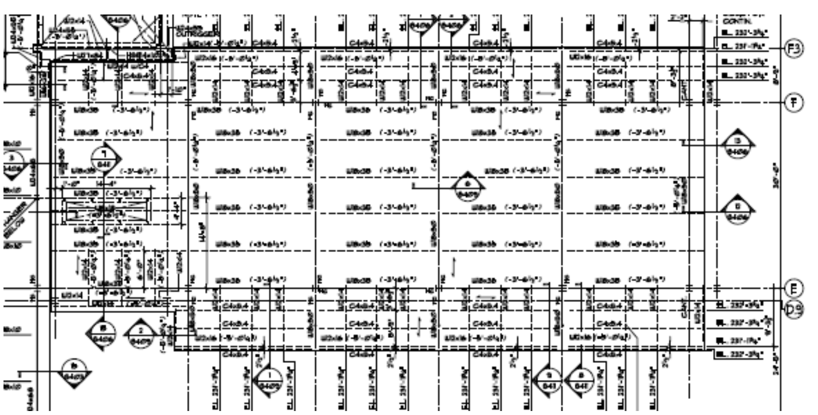
3rd Story frame, east wing (south), Courtesy of Highland Associates



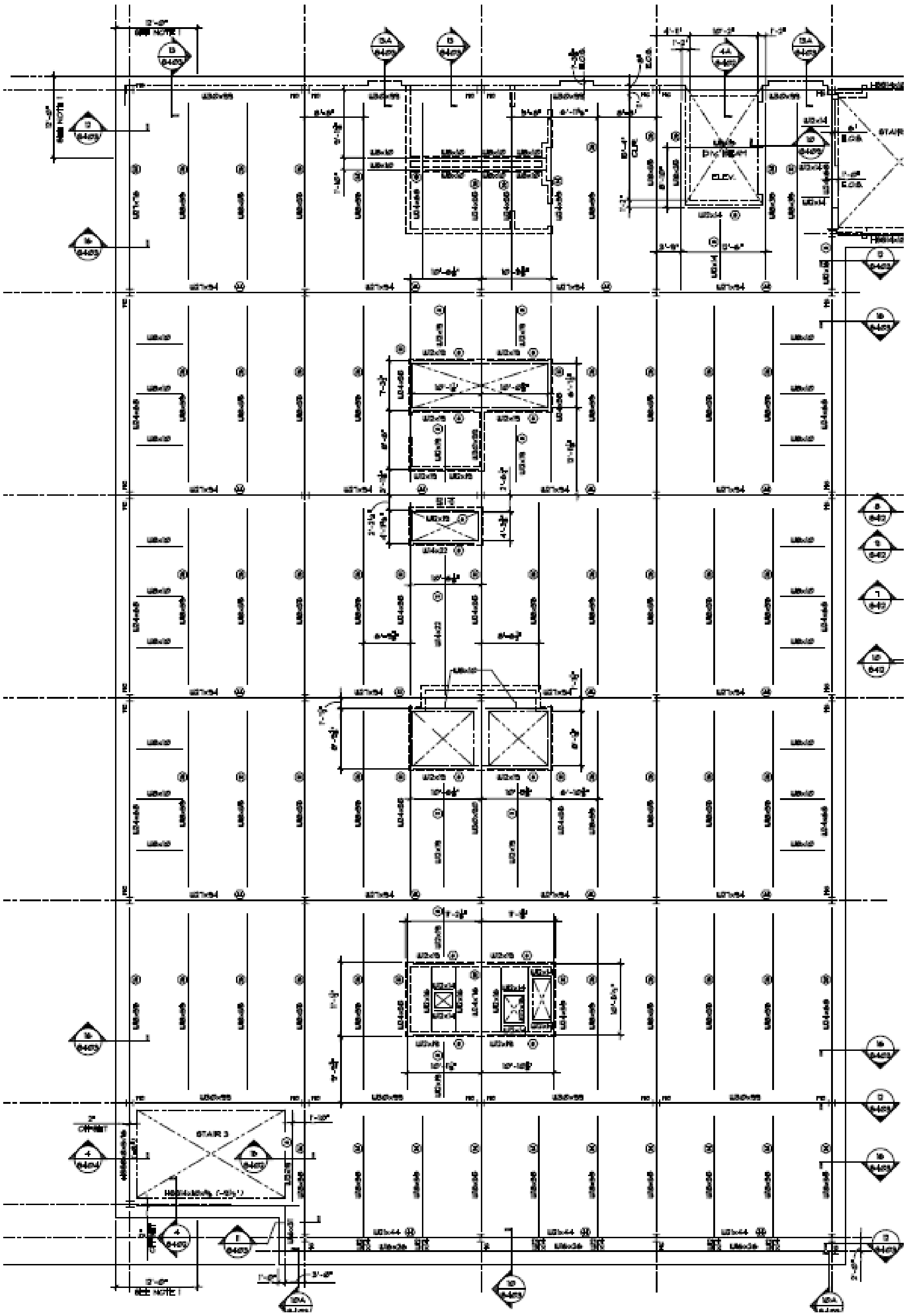


3rd Story frame, east wing (north), Courtesy of Highland Associates

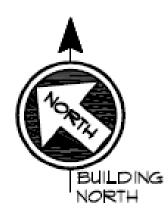


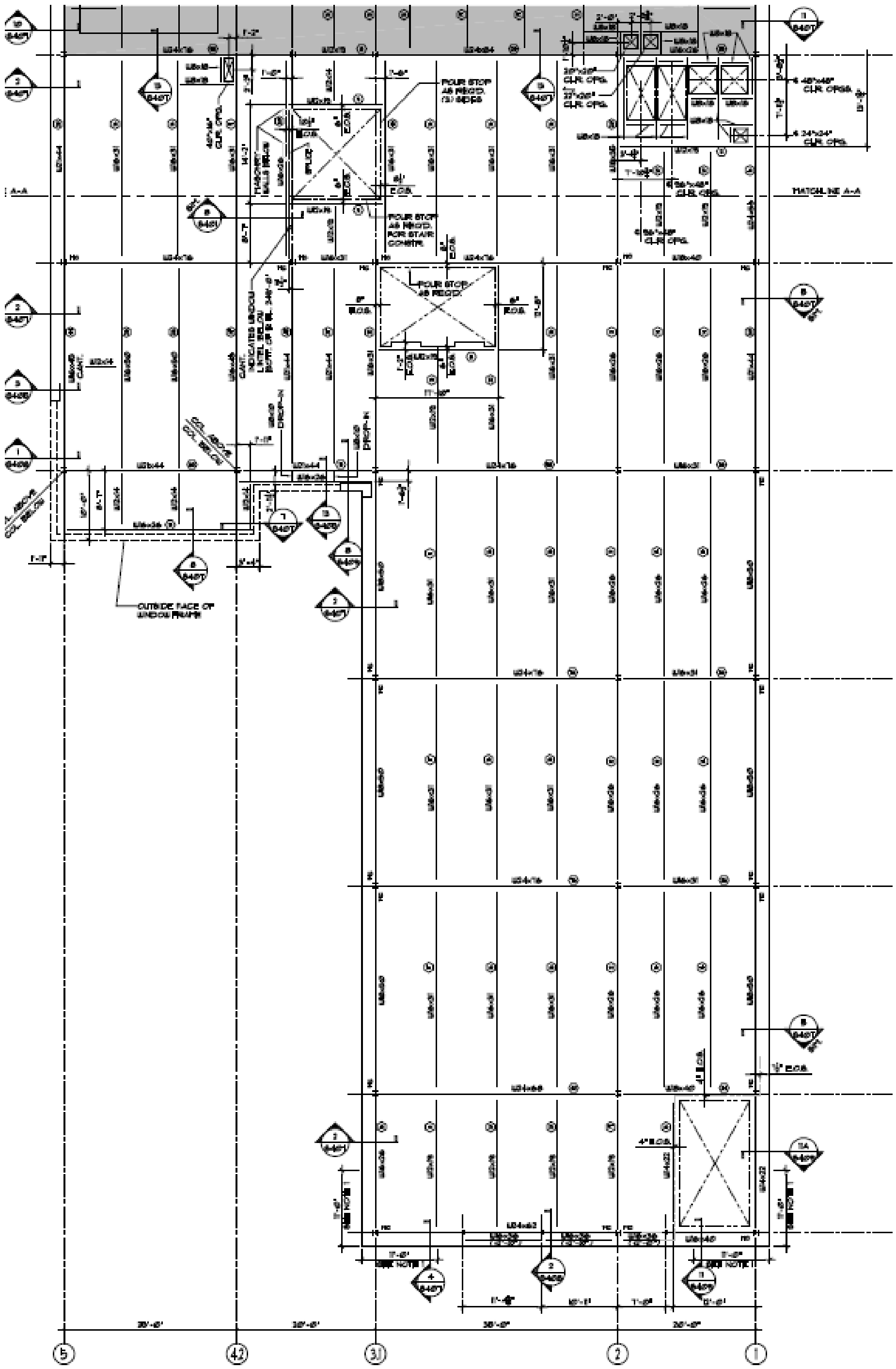


3rd Story frame, Link, Courtesy of Highland Associates

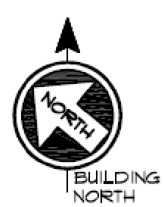


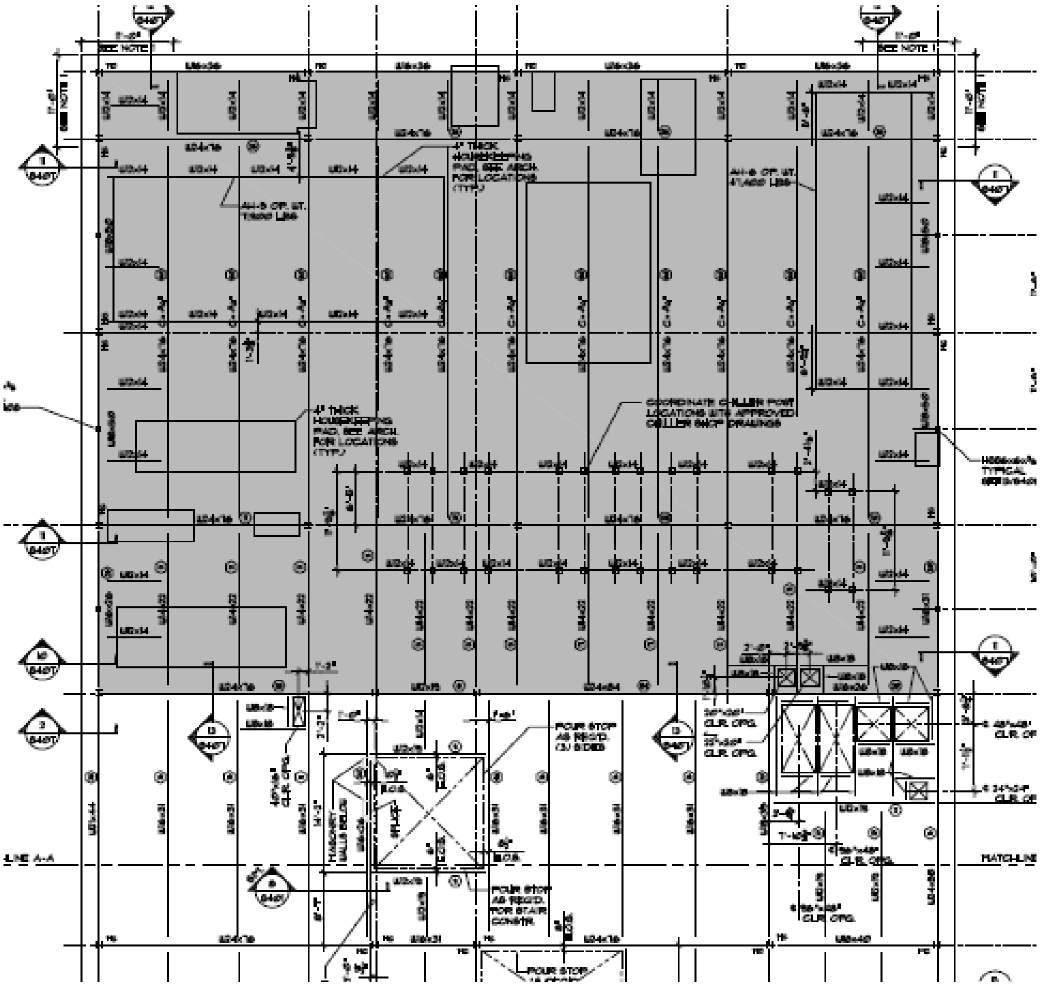
4th Story frame, west wing, Courtesy of Highland Associates



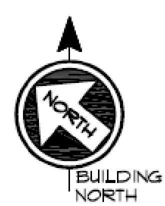


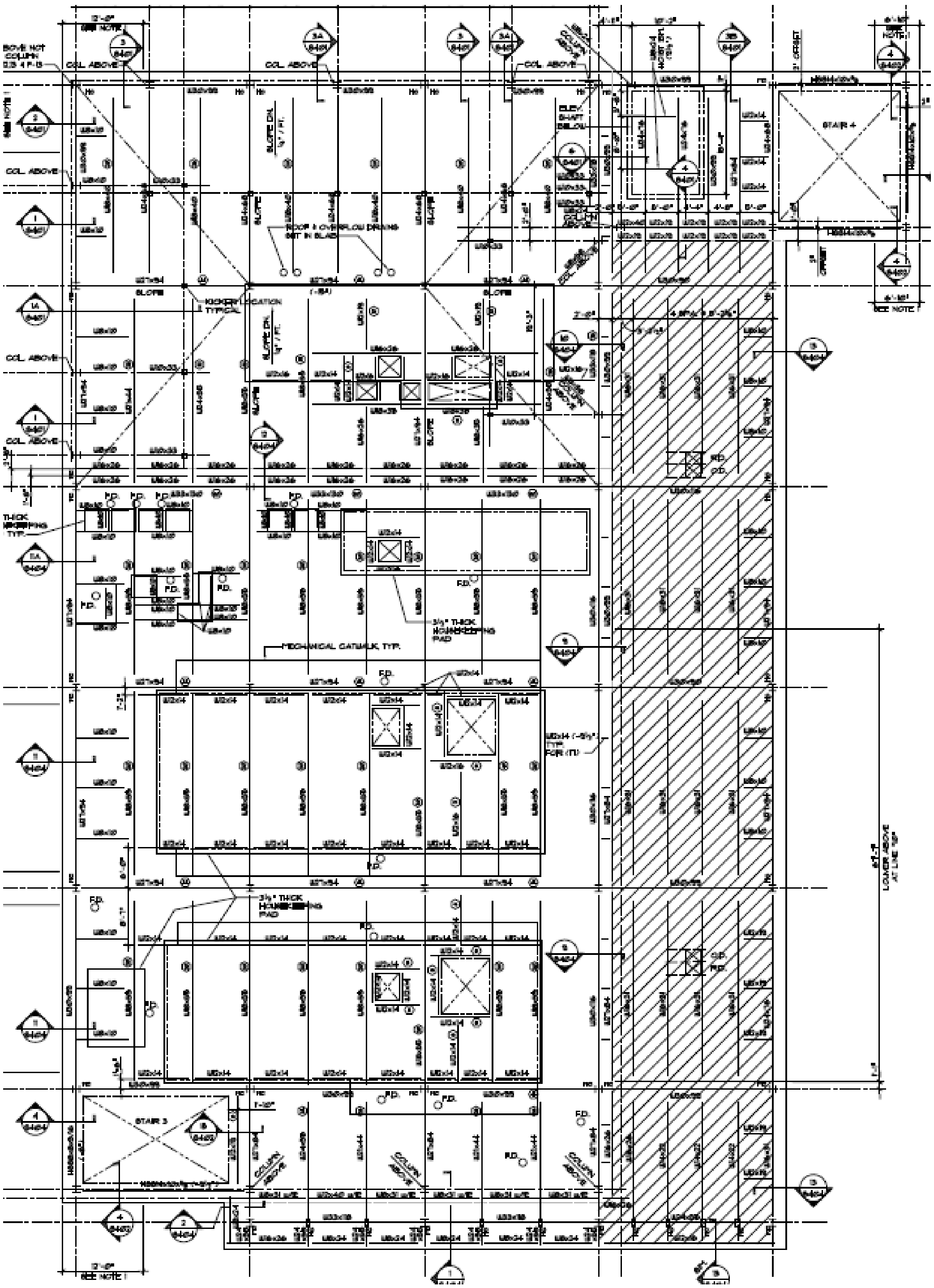
4th Story frame, east wing (south), Courtesy of Highland Associates



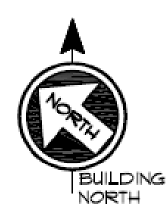


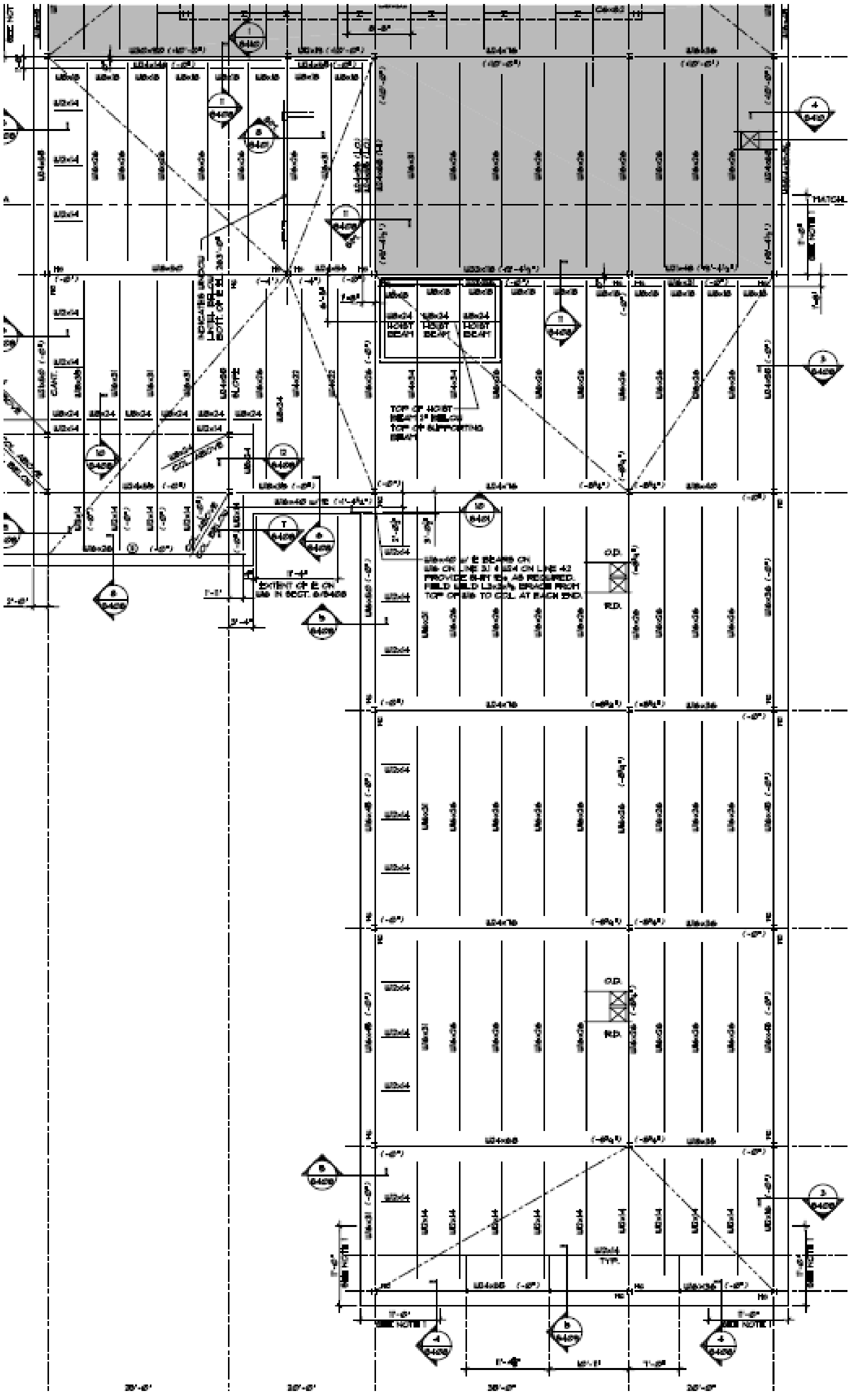
4th Story frame, east wing (north), Courtesy of Highland Associates



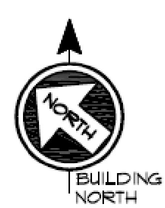


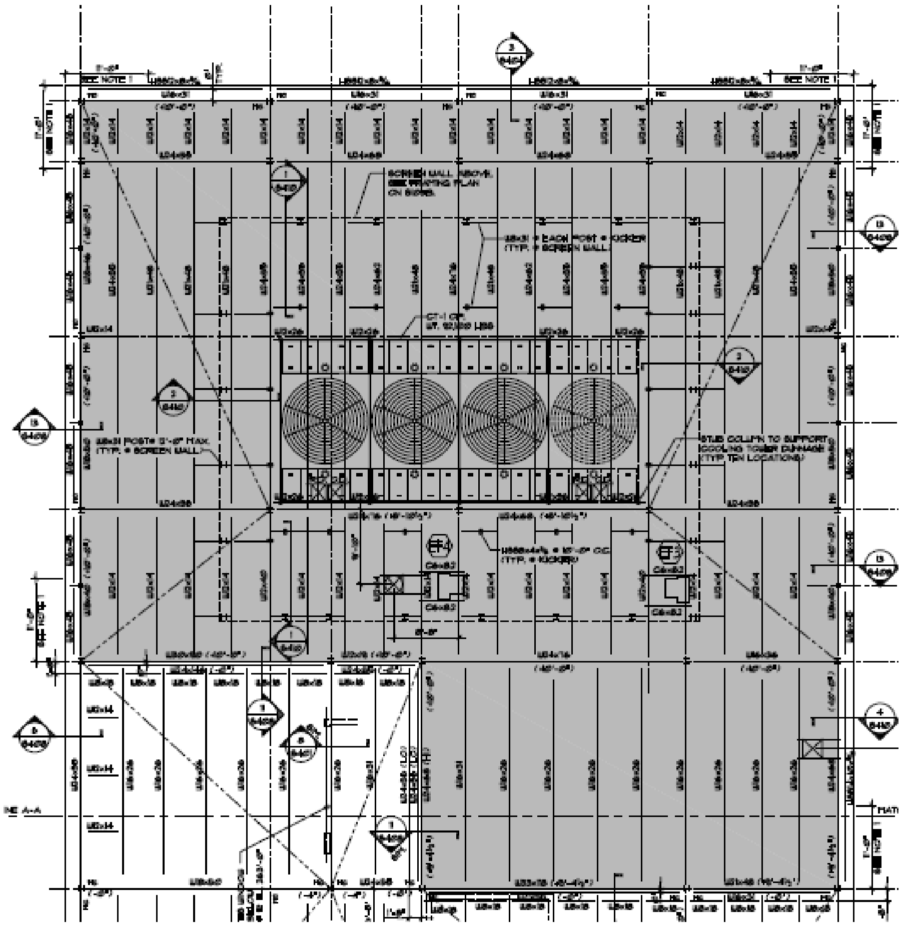
Main Roof Story frame, west wing, Courtesy of Highland Associates



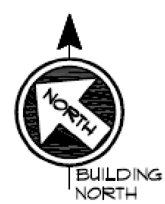


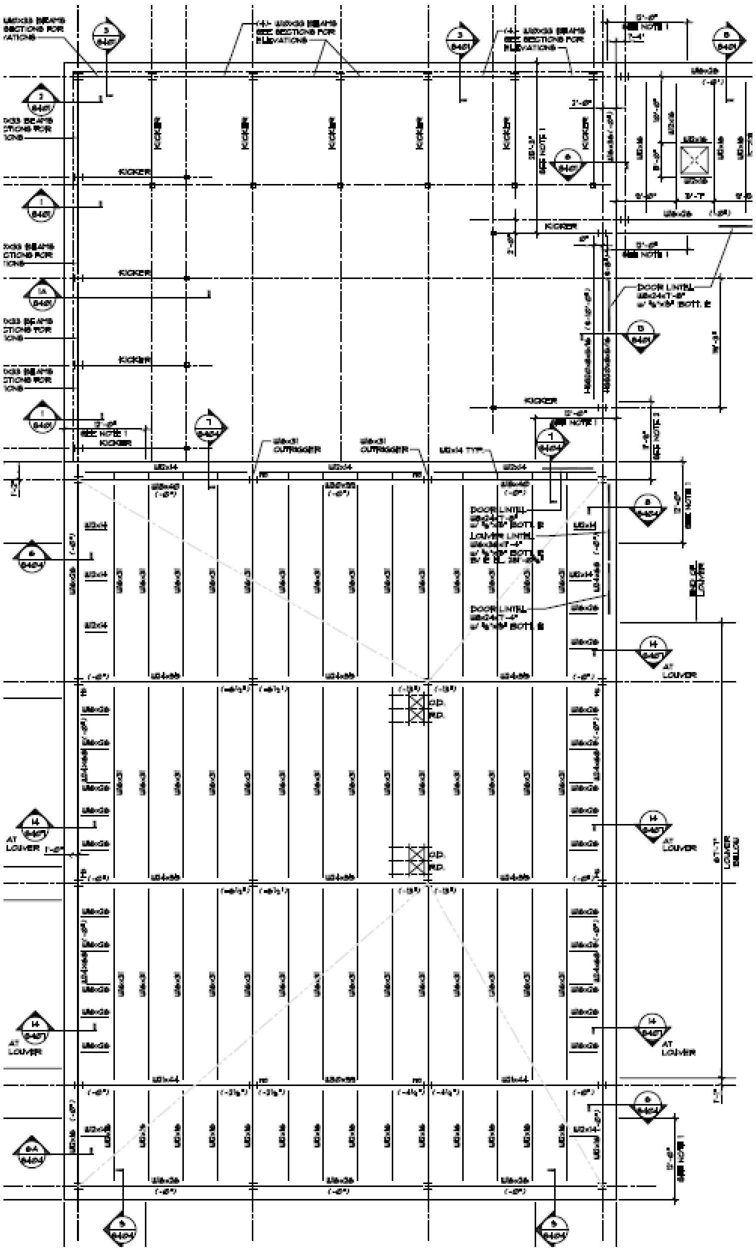
Main Roof Story frame, east wing (south), Courtesy of Highland Associates



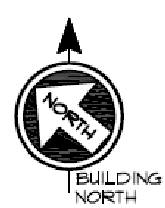


Main Roof Story frame, east wing (north), Courtesy of Highland Associates

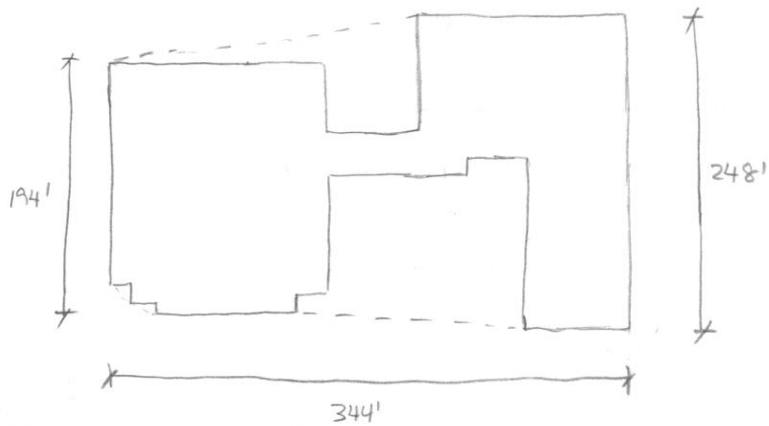




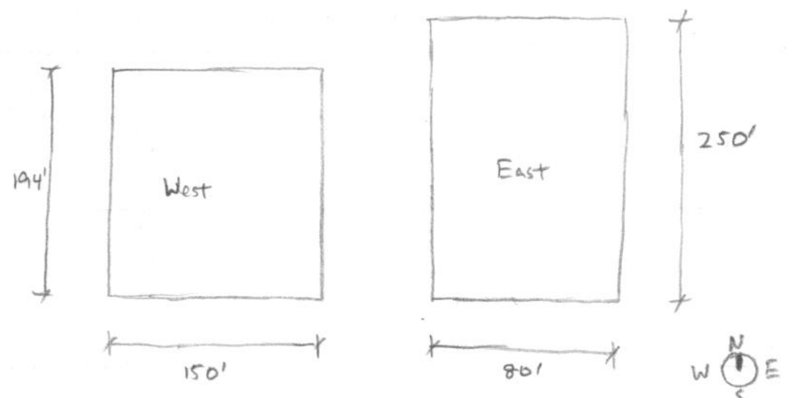
Penthouse Roof Story frame, west wing, Courtesy of Highland Associates



Wind Design | Tech 3 | pg 1 of 7



Simplified Building



The building is changed to two rectangular buildings for easier calculation. Keeping their area relatively the same, the West wing changed to 150' x 194' and the East Wing changed to 80' x 250' in the calculation.

* East Wing Controls Wind in the E-W Direction due to larger area.
* There are multiple roof heights with the TCMC but for this analysis, assume the highest roof is throughout the new rectangular building, which is 93'.

From design, $V_{3s} = 90MPH$, Scranton PA

$I_w = 1.15$ = Buildings with a capacity > 500 for college

Exposure Category = B Urban surroundings

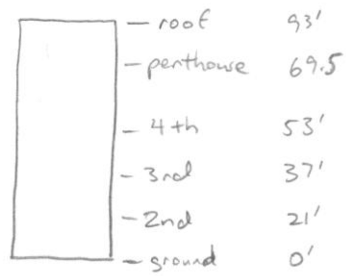
$K_d = 0.85$ for buildings main wind force resisting system.

$K_{zt} = 1.0$

K_z = varies depending on which story

$GC_{pi} = \pm 0.18$ Enclosed building; Fully

Wind Design | Tech 3 | pg 2 of 7



$$q_z = 0.00256 K_z K_{zt} K_d V^2 I_w$$
$$= 0.00256(.99)(1.0)(.85)(90)^2(1.15)$$

$q_{zr} = 20.1$ psf for roof

$$= 0.00256(.89)(1.0)(.85)(90)^2(1.15)$$

$q_{zp} = 18.1$ psf for penthouse floor

$$= 0.00256(.83)(1.0)(.85)(90)^2(1.15)$$

$q_{z4} = 16.9$ psf for 4th story floor

$$= 0.00256(.74)(1.0)(.85)(90)^2(1.15)$$

$q_{z3} = 15.0$ psf for 3th story floor

$$= 0.00256(.70)(1.0)(.85)(90)^2(1.15)$$

$q_{z2} = 14.2$ psf for 2nd story floor

$$= 0.00256(.70)(1.0)(.85)(90)^2(1.15)$$

$q_{zg} = 14.2$ psf for ground story floor

Finding Gust Effect Factor

$n_a = \frac{22.2}{h^{.8}} = \frac{22.2}{93^{(.8)}} = .59 < 1$ Hz so calculate in the event that building is flexible

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

g_Q and $g_v = 3.4$

$n_1 = \frac{100}{H} = \frac{100}{93} = 1.07$ average value C26.9-6 ASCE7-10

$n_1 = \frac{75}{H} = \frac{75}{93} = 0.81$ lower bound value C26.9-7 ASCE7-10

$$g_R = \sqrt{2\ln(3{,}600)(1.07)} + \frac{0.577}{\sqrt{2\ln(3{,}600)(1.07)}} = 4.32$$

$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ $\bar{z} = \max \begin{cases} .6(93) = 55.8 \text{ ft} ✓ \\ 30 \text{ ft} \end{cases}$

$I_z = .30\left(\frac{33}{55.8}\right)^{1/6} = .275$ $c = 0.30$

Wind Design | Tech 3 | Page 3 of 7

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

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$ β assumed to be

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

$$N_1 = \frac{n_1 L_z}{V_z}$$ Constants are from table 26.9-1 (ASCE 7-10)

$$L_z = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$ $\bar{\epsilon} = 1/3$, $l = 320$ ft

$$= 320\left(\frac{55.8}{33}\right)^{1/3}$$

$$= 381.2$$

$$V_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$ $\bar{b} = .45$, $\bar{\alpha} = 1/7$

$$= .45\left(\frac{55.8}{33}\right)^{1/7}\left(\frac{88}{60}\right)90$$

$$= 64.0$$

$$N_1 = \frac{1.07(381.2)}{64} = 6.37$$

$$R_n = \frac{7.47(6.37)}{1 + 10.3(6.37)^{5/3}} = .044$$

Wind Design	Tech 3	Page 4 of 7

West Wing

N-S Direction

h = 93 ft
L = 150 ft
B = 194 ft

Is not required, does not control design

E-W Direction

h = 93 ft
L = 194 ft
B = 150 ft

β = 1% recommended by ASCE 7-05
= .01

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{4.6(1.07)(93)}{64} = 7.15 \qquad \eta_h = 7.15$$

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6(1.07)(194)}{64} = 14.9 \qquad \eta_B = \frac{4.6(1.07)(150)}{64} = 11.5$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(1.07)(150)}{64} = 38.6 \qquad \eta_L = \frac{15.4(1.07)(194)}{64} = 49.9$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$R_h = \frac{1}{7.15} - \frac{1}{2(7.15)^2}\left(1 - e^{-2(7.15)}\right) = .130 \qquad R_h = .130$$

$$R_B = \frac{1}{14.9} - \frac{1}{2(14.9)^2}\left(1 - e^{-2(14.9)}\right) = .064 \qquad R_B = \frac{1}{11.5} - \frac{1}{2(11.5)^2}\left(1 - e^{-2(11.5)}\right) = .083$$

$$R_L = \frac{1}{38.6} - \frac{1}{2(38.6)^2}\left(1 - e^{-2(38.6)}\right) = .026 \qquad R_L = \frac{1}{49.9} - \frac{1}{2(49.9)^2}\left(1 - e^{-2(38.6)}\right) = .020$$

$$R = \sqrt{\frac{1}{0.01}(.044)(.13)(.064)(0.53 + 0.47(.026))} = .14 \qquad R = \sqrt{\frac{1}{0.01}(0.044)(.13)(.083)(0.53 + 0.47(0.020))} = .16$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{194+93}{381.2}\right)^{0.63}}} = .81 \qquad Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{150+93}{381.2}\right)^{0.63}}} = .82$$

$$G_f = 0.925\left(\frac{1 + 1.7(.275)\sqrt{(3.4)^2(.81)^2 + (4.32)^2(.140)^2}}{1 + 1.7(3.4)(.275)}\right) \qquad G_f = 0.925\left(\frac{1 + 1.7(.275)\sqrt{(3.4)^2(.82)^2 + (4.32)^2(.16)^2}}{1 + 1.7(3.4)(.275)}\right)$$

G_f = .828 $\qquad$ G_f = .837

Wind Design | Tech 3 | Page 5 of 7

West Wing Pressures

$$p = qGC_p - q_i(GC_{pi})$$

N-S.

Wall

Windward $C_p = 0.8$

Sidewall $C_p = -0.7$

Leeward $C_p = L/B = 150/194 = .77$

$C_p = -0.5$

Roof

$\theta = 0°$ $h/L = 93/150 = .62$

$C_p = -1.1$ for 0 to $h/2$ by interpolation

$C_p = -0.6$ for $> h/2$

E-W

$C_p = 0.8$

$C_p = -0.7$

$C_p = L/B = 194/150 = 1.29$

$C_p = -0.45$ by interpolation

Roof

$\theta = 0$ $h/L = 93/194 = .48$

$C_p = -0.9$ for 0 to h

$C_p = -0.5$ for h to 2h

$C_p = -0.3$ for $> 2h$

$p = 20.1(.828)(0.8) - 20.1(\pm .18) =$

$= 13.31 \pm 3.62$

$= 9.69$ or 16.93

at roof height, windward wall

* see excel for rest of calculations.

Wind Design | Tech 3 | Page 6 of 7

East Wing

$\beta = 1\%$ recommended by ASCE 7-05
$= .01$

N-S Direction

$h = 93$ ft
$L = 80$ ft
$B = 250$ ft

$$\eta_h = \frac{4.6(1.07)(9.3)}{64} = 7.15$$

$$\eta_B = \frac{4.6(1.07)(250)}{64} = 19.2$$

$$\eta_L = \frac{15.4(1.07)(80)}{64} = 20.6$$

$R_h = .130$

$$R_B = \frac{1}{19.2} - \frac{1}{2(19.2)^2}\left(1 - e^{-2(19.2)}\right) = .050$$

$$R_L = \frac{1}{20.6} - \frac{1}{2(20.6)^2}\left(1 - e^{-2(20.6)}\right) = .047$$

$$R = \sqrt{\frac{1}{0.01}(.044)(.13)(.05)(.53 + .47(.047))} = .13$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{250+93}{381.2}\right)^{0.63}}} = .793$$

$$G_f = 0.925\left(\frac{1 + 1.7(.275)\sqrt{(3.4)^2(.793)^2 + (4.32)^2(.13)^2}}{1 + 1.7(3.4)(.275)}\right)$$

$G_f = .817$

E-W Direction

$h = 93$ ft
$L = 250$ ft
$B = 80$ ft

$\eta_h = 7.15$

$$\eta_B = \frac{4.6(1.07)(80)}{64} = 6.15$$

$$\eta_L = \frac{15.4(1.07)(250)}{64} = 64.4$$

$R_h = .130$

$$R_B = \frac{1}{6.15} - \frac{1}{2(6.15)^2}\left(1 - e^{-2(6.15)}\right) = .150$$

$$R_L = \frac{1}{64.4} - \frac{1}{2(64.4)^2}\left(1 - e^{-2(6.15)}\right) = .015$$

$$R = \sqrt{\frac{1}{0.01}(0.044)(.13)(.15)(.53 + .47(.015))} = .21$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{80+93}{381.2}\right)^{0.63}}} = .850$$

$$G_f = 0.925\left(\frac{1 + 1.7(.275)\sqrt{(3.4)^2(.85)^2 + (4.32)^2(.21)^2}}{1 + 1.7(3.4)(.275)}\right)$$

$G_f = .863$

Wind Design | Tech 3 | Page 7 of 7

East Wing Pressures

$p = qGC_p - q_i(GC_{pi})$

Wall

	N-S.	E-W
Windward	$C_p = 0.8$	$C_p = 0.8$
Side wall	$C_p = -0.7$	$C_p = -0.7$
Leeward	$C_p \Rightarrow L/B = 80/250 = .32$	$C_p = 250/80 = 3.125$
	$C_p = -0.5$	$C_p = -0.25$ by interpolation

Roof

$\theta = 0°$ $\quad h/L = 93/80 = 1.162$ $\qquad h/L = 93/250 = .372$

$\frac{h}{2} = \frac{93}{2} = 46.5$

$2h = 186$

Roof area >> 1000 sf $\quad$ R.F = .8

$C_p = -1.3$ for 0 to $h/2$

$C_p = -0.7$ for $> h/2$

$C_p = -0.9$ for 0 to h

$C_p = -0.5$ for h to 2h

$C_p = -0.3$ for > 2h

$p = 20.1(.817)(0.8) - 20.1(\pm .18) = 9.51$

$= 13.14 \pm 3.62$

$= 9.51$ or 16.75

at roof height, windward wall

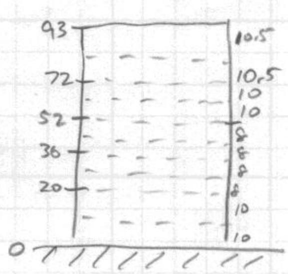
$p = 20.1(.863)(0.8) - 20.1(\pm .18)$

$= 13.88 \pm 3.62$

$= 10.26$ or 17.5

* See Excel for rest of calculations

Wind Forces Breakdown



Seismic Analysis	Tech 3	B 1 of 2

Given on Sheet S201B

Using ASCE 7-05 to do calculations.

Seismic Use Group = III

$S_s = .199$ $S_1 = .058$

Site Class = B

$I_e = 1.25$

Seismic Design Category = A

Ordinary Steel Moment Frames.

$F_a = 1.0$ $S_{ms} = 1.0(.199) = .199$

$F_v = 1.0$ $S_{m1} = 1.0(.058) = .058$

$S_{DS} = \frac{2}{3} S_{ms} = \frac{2}{3}(.199) = .133$

$S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(.058) = .039$

From Table 12.2-1 $R = 3.5$ for ordinary Steel Moment Frames

$\Omega = 3$

$C_d = 3$

Equivalent Lateral Force / Base Shear

$V = C_s W$ $W = 22,378$ kip

$C_t = .028$, $x = 0.8$ from table 12.8-2

$T = C_T h_N{}^x$ $h_N = 93'$

$T = (.028)(93)^{(.8)} = 1.05s$

$C_s = \frac{S_{DS}}{R/I} = \frac{.133}{(5/1.25)} = .0333$

For PA, fig 22-15 shows $T_L = 6$ sec

$T = 1.05 \leq 6$ so $C_s \leq \frac{S_{D1}}{T(R/I)} = \frac{.039}{1.05(5/1.25)} = .009 \approx .01$

$C_s > 0.01$

Use $C_s = .0333$

$V = .0333(22,378)$

$V = 745$ k

40

Seismic Analysis	Tech 3	pg 2 of 2

Vertical Distribution of Seismic Forces.

$$F_x = C_{ux} V$$

$$C_{ux} = \frac{w_x h_x^k}{\sum w_i k_i^k} \; ; \quad K = 1 \text{ or } 2 \text{ for } 0.5 < T < 2.5s \text{ on interpolation}$$

$$\frac{2.5-.5}{2-1} = \frac{1.05-.5}{K-1} \Rightarrow K = 1.275$$

$$C_{rf} = \frac{1867(93)^{1.275}}{\cancel{(5099)}(0)^{1.275} + (1867)(93)^{1.275} + (2163)(72)^{1.275} + (4286)(52)^{1.275} + (4286)(36)^{1.275} + (4677)(20)^{1.275}}$$

$$C_{rf} = 0.252$$

See rest on excel sheet in the report, under seismic analysis.

$$F_x = 0.252(745)$$

$$F_r = 187.74\,k$$

See rest on excel.

41

Snow Load Calcs	Tech 3	pg 1 of 2

Given in Sheet S201B

$P_g = 35\,psf$

$P_f = 30\,psf$

$C_e = 1.0$

$I_s = 1.1$

$C_T = 1.0$

$P_{f\,cal} = 0.7 C_e C_t I P_g$

$= 0.7(1.0)(1.0)(1.1)(35)$

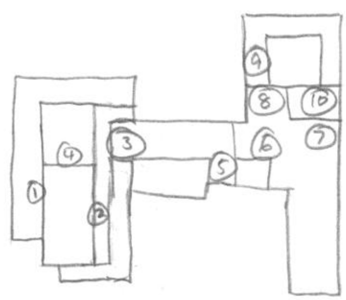
$= 26.95$, we will use 30psf as designed.

Snow Density

$\gamma = 0.13 p_g + 14 \leq 30pcf$

$= 0.13(35) + 14 = 18.55$

$h_b = P_f / \gamma = 30/18.55 = 1.62\ ft$

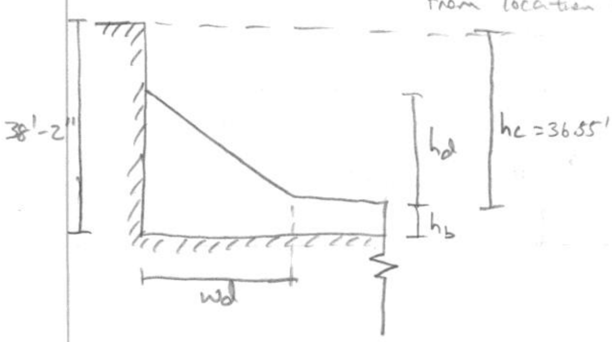


Since roof height varies, snow drift must be considered. Snow drifts can happen in many places. One of the case will be selected and calculated.

\# important drift locations.

Drift #3 will be calculated and it is most likely the worst possible scenario.

From location 3 to 2



$h_c = 38.17 - 1.62 = 36.55'$

$\frac{h_c}{h_b} = \frac{36.55}{1.62} = 22.56 > 0.2$

so drift will be considered

Snow Load Calcs | Tech 3 | pg 2 of 2

Windward Figure 7-8 in ASCE 7-05, If $l_u < 25$ ft, use $l_u = 25$
$l_u = 43$ ft in this case

$h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5$

$= 0.43\sqrt[3]{43}\sqrt[4]{35+10} - 1.5$

$= 2.29$ ft $< 36.55'$ ✓

$w_d = 4(2.29) = 9.15$ ft

Leeward $l_u = 26$ ft

$h_d = 0.43\sqrt[3]{26}\sqrt[4]{p_g+10} - 1.5$

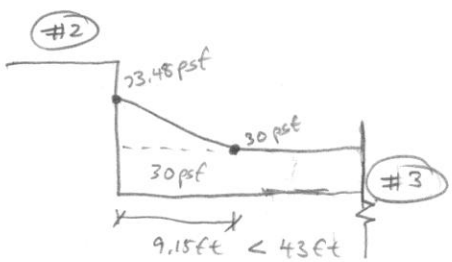
$= 1.70 < 36.55$

$w_d = 4(1.7) = 6.8$ ft

Windward case controls.

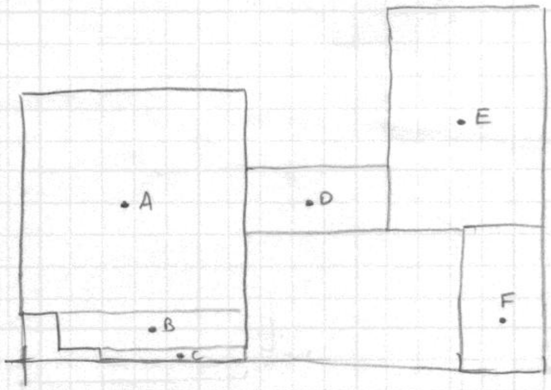
$p_d = 2.29(18.55) = 42.48$ psf

$43.48 + 30 = 73.48$ psf = highest snow load



COM Check | Tech 3 | Pg 1 of 1

COM check, for level 2



* See Excell spread sheets for rest of calculations

Areas:

	Total weight
A = (170)(152) = 25840 ft²	2403.1k
B = (20)(104) = 2080 ft²	193.4k
C = (5)(78) = 390 ft²	36.3k
D = (38.75)(81) = 3138 ft²	235.4k
E = (134)(100) = 13400 ft²	1005.0k
F = (110)(55) = 6050 ft²	453.8k
	4326.9k

Area		X_{com}		Y_{com}
A =	152/2 =	76 ft	170/2 + 24 =	110
B =	104/2 + 28	80 ft	20/2 + 5 =	15
C =	78/2 + 54	93 ft	5/2 =	2.5
D =	81/2 + 152	192.5 ft	38.75/2 + 110 =	129
E =	100/2 + 233	283 ft	134/2 + 100 =	169
F =	55/2 + 278	305 ft	110/2 =	55

Center Of Mass

$$COM_x = \frac{2403.1(76) + 193.4(80) + 36.3(93) + (235.4)(193) + (1005.0)(283) + 453.8(305)}{4326.9}$$

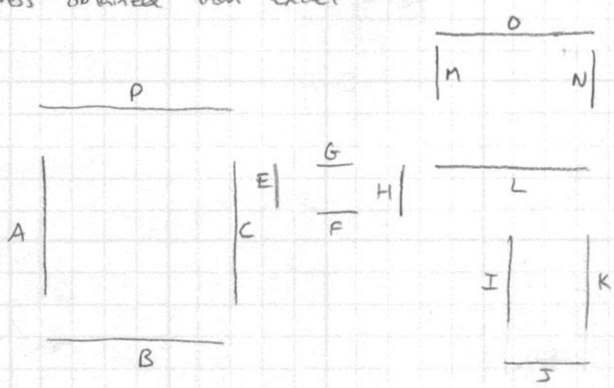
$$= 154.8 \text{ ft}$$

$$COM_y = \frac{2403.1(110) + 193.4(15) + 36.3(2.5) + 235.4(129) + 1005(177) + 453.8(55)}{4326.9}$$

$$= 113.8 \text{ ft}$$

COR check	Tech 1	pg 1 of 1

Use stiffness obtained from excel



E-W   * already referenced to ETABS

X:

$B \Rightarrow K = 136.67$	$X_r = 52$ ft	
$D \Rightarrow K = 167.31$	$X_r = 52$ ft	
$F \Rightarrow K = 68.72$	$X_r = 159$ ft	
$G \Rightarrow K = 62.75$	$X_r = 159$ ft	Total $K_{E-W} = 710.23$
$O \Rightarrow K = 82.64$	$X_r = 264$ ft	
$L \Rightarrow K = 90.09$	$X_r = 264$ ft	
$J \Rightarrow K = 102.04$	$X_r = 291$ ft	

N-S

y:

$A \Rightarrow K = 68.46$	$Y_r = 95$ ft	
$C \Rightarrow K = 72.73$	$Y_r = 95$ ft	
$E \Rightarrow K = 88.56$	$Y_r = 125$ ft	
$H \Rightarrow K = 72.39$	$Y_r = 125$ ft	
$M \Rightarrow K = 56.82$	$Y_r = 213$ ft	Total $K_{N-S} = 532.7$
$N \Rightarrow K = 58.14$	$Y_r = 213$ ft	
$I \Rightarrow K = 54.47$	$Y_r = 65$ ft	
$K \Rightarrow K = 58.14$	$Y_r = 65$ ft	

$$X_{COR} = \frac{136.67(52) + 167.31(52) + (68.72)(159) + (62.75)(159) + (82.64)(264) + 90.09(264) + (102.04)(291)}{710.23} =$$

$X_{COR} = 132.7$ ft

$$Y_{COR} = \frac{68.46(95) + 72.73(95) + 88.56(125) + (72.39)(125) + (56.82)(213) + (58.14)(213) + 54.47(65) + 58.14(65)}{532.7}$$

$Y_{COR} = 118.02$ ft

Direct/Torsional Shear | Tech 3 | pg 1 of 1

$COR_x = 134.5$ $COM_x = 125.8$ $e_x = 8.70$

$COR_y = 99.5$ $COM_y = 100.3$ $e_y = 2.98$

Using Stiffness value from COR check,

Direct Shear

E-W

d_y

$d = 79.5$ $F_{DB} = \frac{(136.67)(F)}{710.23} = .192F$ ←

$d = -70.5$ $F_{DD} = 0.236F$ ←

$d = -10.5$ $F_{DF} = 0.097F$ ←

$d = -40.5$ $F_{DG} = 0.088F$ ←

$d = -144.5$ $F_{DO} = 0.116F$ ←

$d = -40.5$ $F_{DL} = 0.127F$ ←

$d = 99.5$ $F_{DJ} = 0.144F$ ←

d_x N-S

$d = -134.5$ $F_{DA} = 0.129F$ ↓

$d = -30.5$ $F_{DC} = 0.137F$ ↓

$d = -6$ $F_{DE} = 0.166F$ ↓

$d = 54.5$ $F_{DH} = 0.136F$ ↓

$d = 79.5$ $F_{DM} = 0.107F$ ↓

$d = 179.5$ $F_{DN} = 0.109F$ ↓

$d = 124.5$ $F_{DI} = 0.108F$ ↓

$d = 179.5$ $F_{DK} = 0.109F$ ↓

See Excel sheet.

Direct / Torsional Shear | Tech 3 | page 2 of 2

Torsional Shear

$$\Sigma K d_i^2 = 11{,}210{,}205$$

E-W

$$F_{TB} = \frac{F_{ey}(d_B K_B)}{\Sigma K d_i^2} = \frac{F(2.98)(79.5)(136.67)}{11{,}210{,}205} = .$$

Done for all 15 frames

N-S

$$F_{TA} = \frac{F_{ex}(d_A K_A)}{\Sigma K d_i^2} = \frac{F(8.7)(-134.5)(68.46)}{11{,}210{,}205} = .00715$$

Done for all 15 frames.

See Excel for rest of calculations.

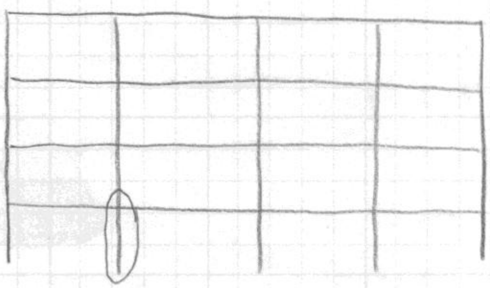
Spot Checks	Tech 3	Page 1 of 2

Frame D will be used for Spot Check.

Spot capacity on column G12 – axial
– bending

G12 = W14x257

Frame D



G12

Tributary area = 26' x 25' = 650 ft²

Load = 3 Floors + 1 roof

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4\cdot 3(650)}} = .42$$

$P_L = 100(.42)(650) = 27,300$ 27,300(3) = 81,900

$P_D = 93(650) = 60450$ 60450(3) = 181350

$P_{LR} = 20(650) = 13,000$

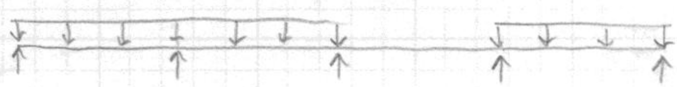
$P_{DR} = 20(650) = 13,000$

$P_S = 30(650) = 19,500$

$P_u = 1.2(181350 + 13,000) + 1.6(81,900 + 13,000) + .5(19,500)$

= 394.9 kips

The following loadings give the largest moments on beams.

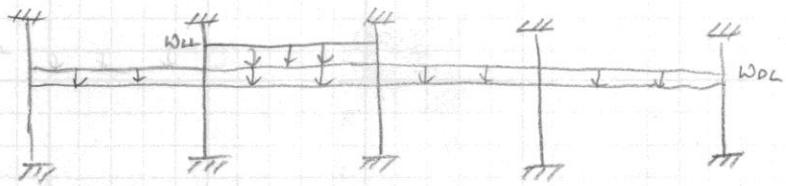


$-0.121 w l^2$ = largest negative moment

+.0996 = largest positive moment

Spot Checks | Tech 3 | Page 2 of 2

Columns continue



$w_u = 1.2(93)(25) + 1.6(100)(25)(.67) = 6.79$ klf

$LL_{red} = 0.25 + \frac{15}{\sqrt{2(25)(26)}} = .67$

Dead load unbalanced moment ≈ 0

Live load unbalanced moment

$w_{LL} = 1.6(100)(25)(.67) = 2.68$ klf

$FEM_u = \frac{2.68(26)^2}{12} = 151$ k·ft

$\frac{1}{2}FEM = 75.5$ k·ft

$M=0$ $FEM = 151$ k·ft

75.5 k·ft

$P_u = 394.9$ kips

$M_u = 75.5$ k·ft

$P_{eq} = 394.9 + \frac{25(75.5)}{14} = 530^k$

Use Column W14 x 90 $\phi P_n = 877^k$ @ 20ft

Beam sizes

$w_L = \frac{.65(100)(25)}{1000} = 1.625$ klf

$w_D = \frac{93(25)}{1000} = 2.325$ klf

$w_u = 1.2(2.325) + 1.6(1.625) = 5.39$ klf

$\frac{w_u l^2}{12} = \frac{5.39(30)^2}{12} = 404.25$ k·ft

W18x97 with $\phi M_p = 495$ k·ft @ 30' unbraced

compared with TCMC W30 x 99.