The Commonwealth Medical College

Scranton, PA

Final Report 2013

Xiao Ye Zheng

Structural Option

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04/03/2013

TCMC®
THE COMMONWEALTH MEDICAL COLLEGE

MEDICAL SCIENCE BUILDING
Scranton, PA

PROJECT TEAM

Owner	The Commonwealth Medical College
Architects	Highland Associates & HOK
Structural/M.E.P. Engineers	Highland Associates
Construction Management	**Quandel Construction Group**
Landscape Architecture	McLane Associates
Interior Architecture	Highland Associates & HOK

GENERAL INFO

Function	Medical College
Size	185,000 square feet
Height	93 feet, 5 Stories
Cost	$120 Million
Construction Dates	May 2009 to Oct 2011
Delivery Method	Design-Bid-Build

ARCHITECTURE

The Commonwealth Medical College (TCMC), also known as The Medical Sciences Building (MSB), opened its doors on April 2011 to serve over 675 occupants. It has over 3 acres of prime space in the heart of Scranton, PA. TCMC has two wings, east and west, connected by a grand lobby where everyone gets together and socialize. The lobby is surrounded by a huge courtyard, both north and south side, which provides a very nice green view that the city cannot provide. The exterior facade of TCMC are glass, granite and limestone veneers. High performance glazing and honeycombed transom glazing, for integrated daylight control, are used all over the building. TCMC is a brand new state of the art building that provides the most luxurious feeling a building can have.

STRUCTURAL

The west wing rests on mat slab foundation while the east wing, and the link between them, rest on drilled caissons. The framing system of TCMC is primarily W-shape, ASTM A992 steel. Lateral forces are resisted by moment connections. All floors are concrete slab on a composite steel deck. The roof however, is a roof deck with no concrete.

MEP SYSTEMS

Air Handling Units: Different sizes, totaling over 200,000 cfm, with heat recovery system
Lab Exhaust System: Five 20,000 cfm, Strobic fans with pre-filter, heat recovery coils
Refrigeration: Four 300 ton, McQuay magnetic bearing chillers for central cooling
Three 800 ton, cooling towers
Heating Plant: Three 80 psig, 100 bohp, Fulton high pressure steam boilers
Lighting: All motion-sensor, using 277V. Fluorescent Lighting System
Electric: Both 208Y/120 V and 480Y/277V 3 phase

XIAO YE ZHENG | STRUCTURAL

http://www.engr.psu.edu/ae/thesis/portfolios/2013/xyz5035/

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Executive Summary

The Commonwealth Medical College is a brand new state of the art medical science building, completed in April, 2011 with over 185,000 square foot of building space. It is located on an urban setting, in Scranton, PA. The cost of the entire project was around $120 million, at over $600 per square foot. TCMC is clad in brick, stone, and glass, with a modern architectural look compared to the surrounding buildings. The main gravity system is composite steel deck with concrete topping and steel beams resting on steel columns. The lateral system consists of 15 moment frames scattered throughout the building.

This report emphasized on two redesigns of the original lateral structure, from a given problem statement that the author was interested in. Because the existing structure is so well designed to meet all code requirements, nothing can be done to improve the building under the current scenario. Therefore, a new scenario was created in which The Commonwealth Medical College was proposed to be built on a typical urban site in Miami, FL. The new structures were designed to be adequate for both strength and serviceability at this new site.

The two new redesigns were steel moment frame and chevron braced frame. Having steel moment frames will increase the current building weight by approximately 5%, compared to a 1% increase by braced frames. Also, moment frames are around three times the cost of braced frames. It was determined that braced frames are a much better choice than moment frames in terms of strength, serviceability, cost, and constructability. However, moment frames have more architectural freedom.

In addition to the lateral system redesigns, three breadths were also undertaken. The first breadth was on façade design. A rainscreen cladding system, TerraClad Rain Screen, made by Boston Valley Terra Cotta, was chosen for the new outer façade of TCMC because of its advantages in the new site. As for glazing, laminated glass units designed as a sacrificial ply were used to handle debris loading.

The second breath was on solar panel design. It was easy to see the great opportunities for solar energy in Florida, so a solar panel system was designed. The model of the panels chosen was the HIT Power 220A, made by Panasonic. This model has the highest output of energy on cloudy days. The inverter was chosen to be SMA Sunny Boy 3800 because this was recommended by Panasonic for this 220A model and this inverter is built to cool itself, which increases its lifespan. The solar panels would save the owner approximately $10,000 per year and the whole system will have a payback period of approximately 27 years.

The last breath was on small mechanical and electrical modifications. The number of steam boilers was cut down because it wasn't needed anymore. Most importantly, a more powerful dehumidifier was added because Miami is very humid compared to Scranton. The model chosen for the dehumidifier was the RLNL-G dehumidifier, made by Rheem. The only main electrical change was from a simply electrical gird connection to a gird-tied connection. This allows TCMC to use the energy from the solar panels and energy from the electrical supplier at the same time.

Acknowledgements

I'd like to extend my gratitude to the following companies and people for their help during the completion of my senior thesis course:

Penn State Architectural Engineering Faculty

> The entire AE faculty, for the help and support throughout my time at Penn State, including Heather Sustersic, for being my advisor and helping me under take challenges

Highland Associates

> Highland Associates, for providing me with the project and the owner permission form. A special thanks to Eric McAndrew for your quick responses to my questions and your willingness to assist me along the way.

The Commonwealth Medical College

> TCMC, for giving me the opportunity to use their building as my thesis project.

Family and Friends

> Family and friends, for your unwavering support throughout the years. Thank you very much for being there when you are needed most.

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.

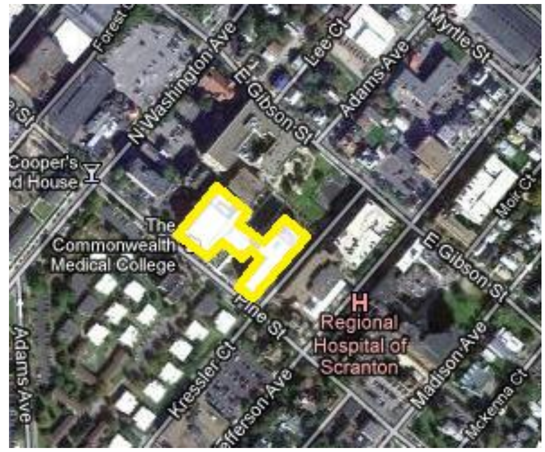


Figure 1 Aerial map from Google.com showing the location of the building site

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

Table 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.

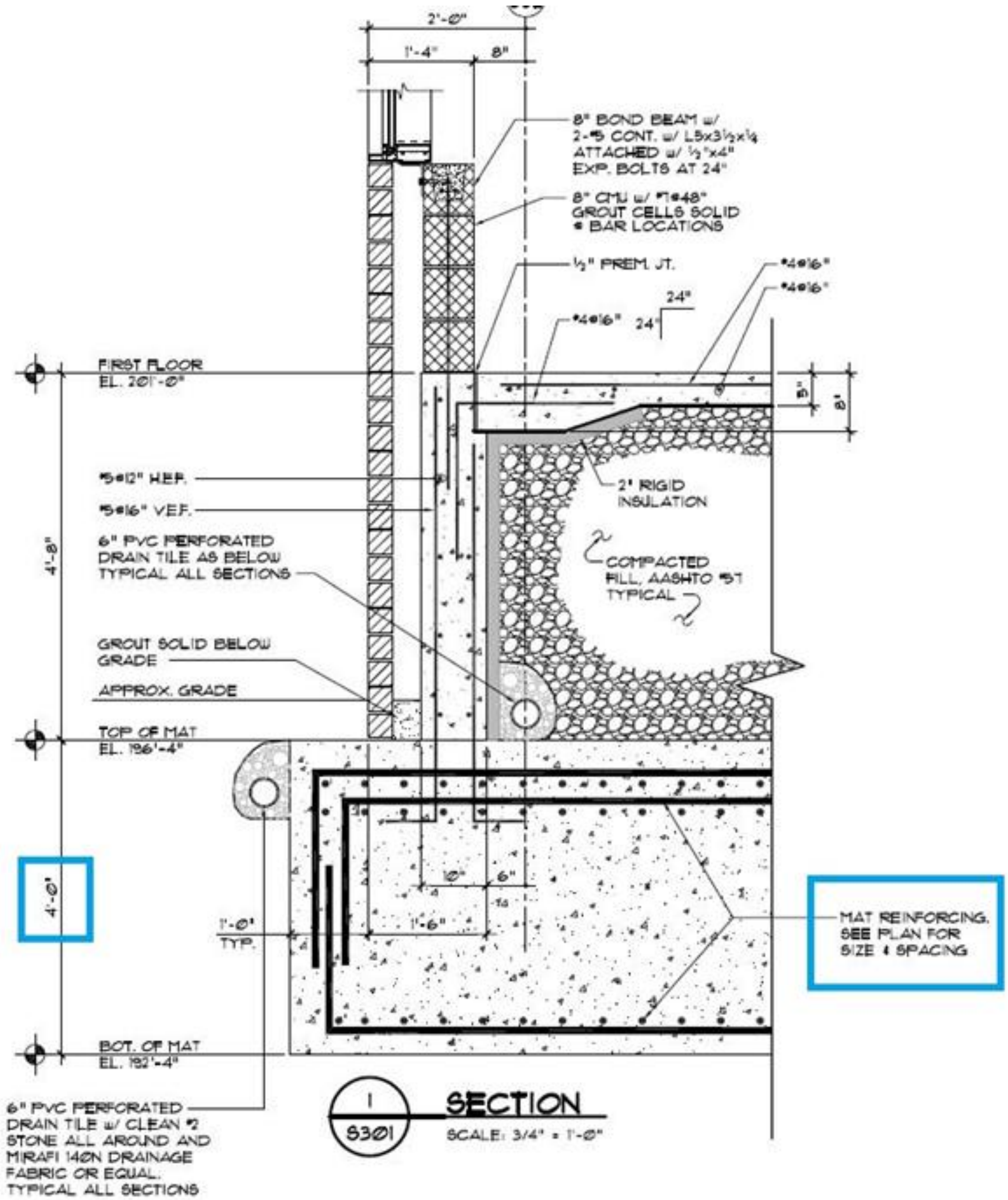


Figure 5 A typical Section cut showing the mat slab foundation. Courtesy of Highland Associates

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.

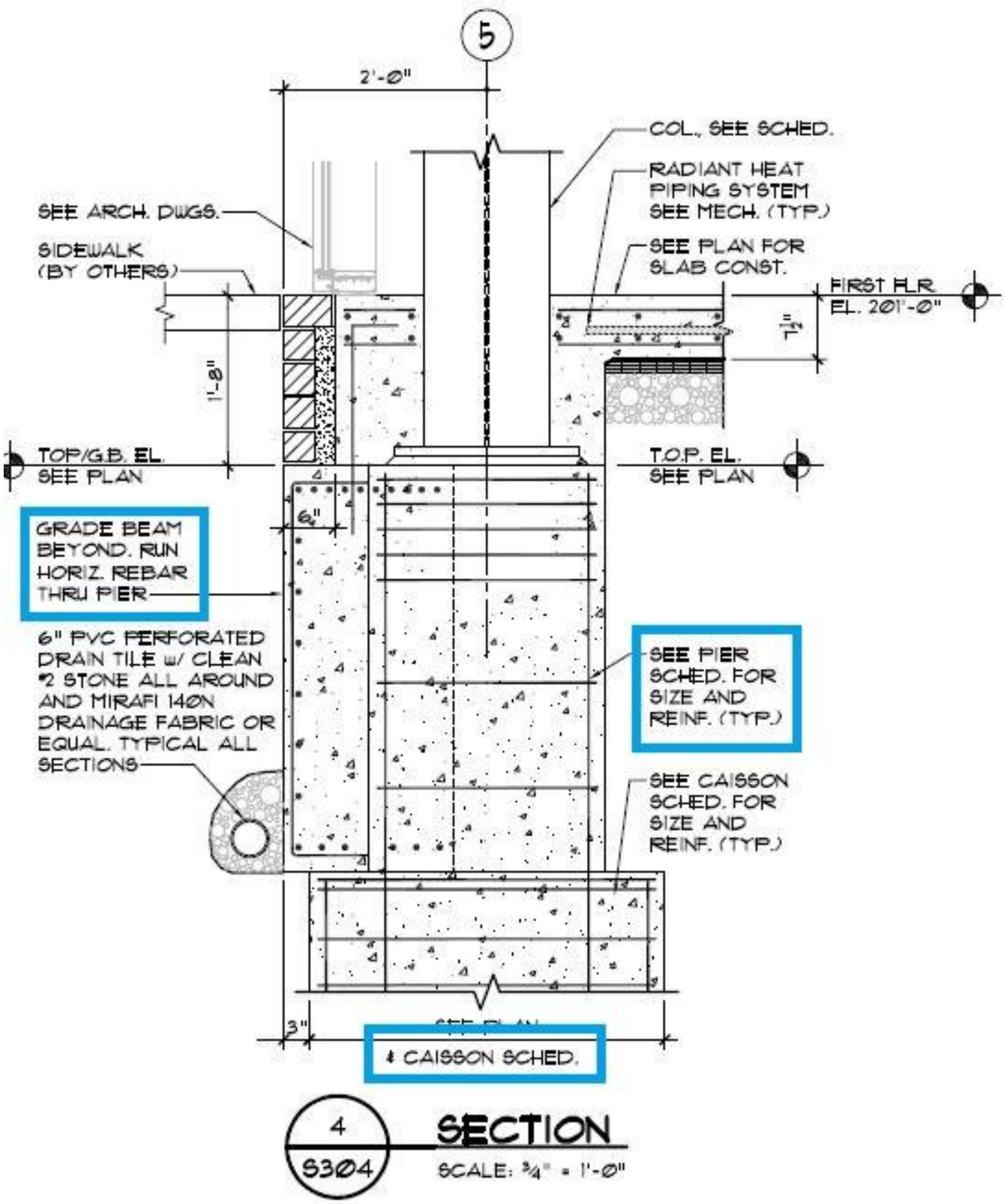


Figure 6 A section cut of a drilled caisson foundation. Courtesy of Highland Associates

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.

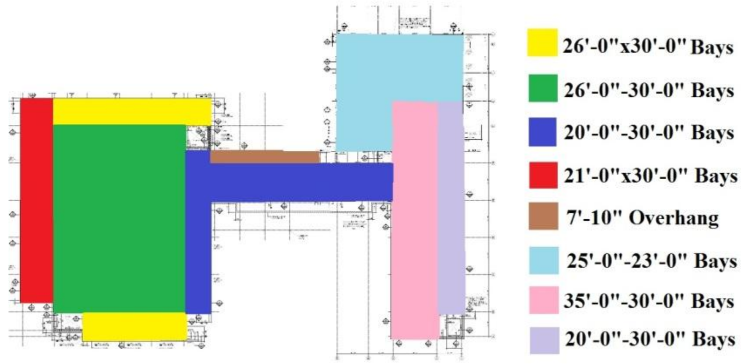


Figure 7 Different Bay sizes respective to their color

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".

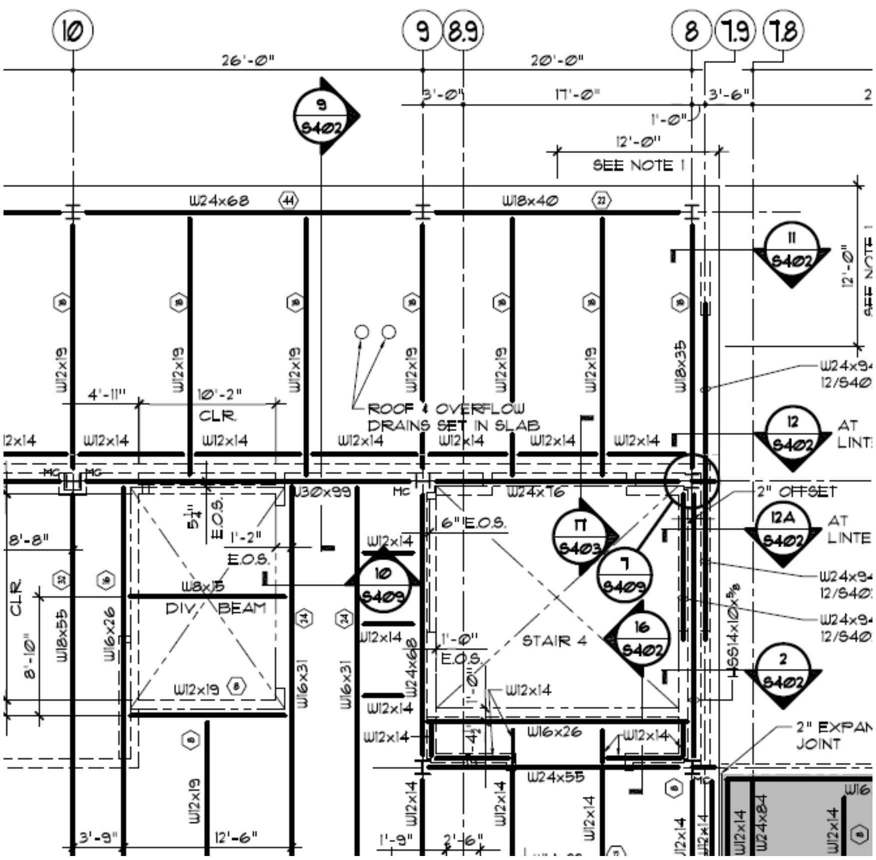


Figure 8 Partial plan showing the second floor, northeast corner of the west wing

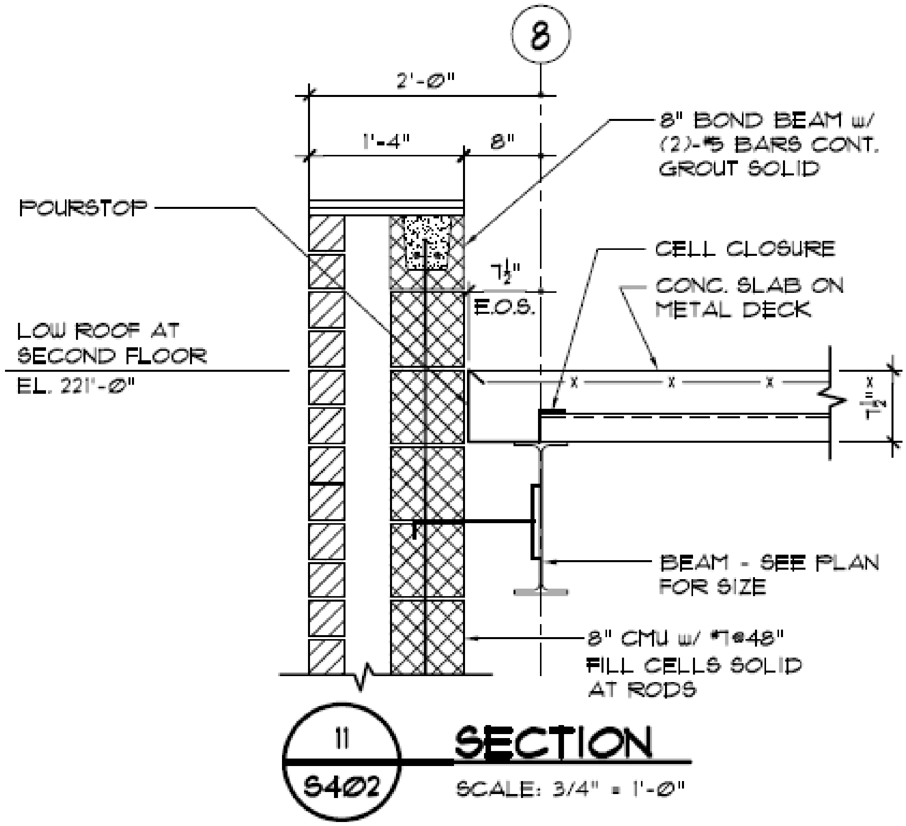


Figure 9 Section cut 11 from Figure 8

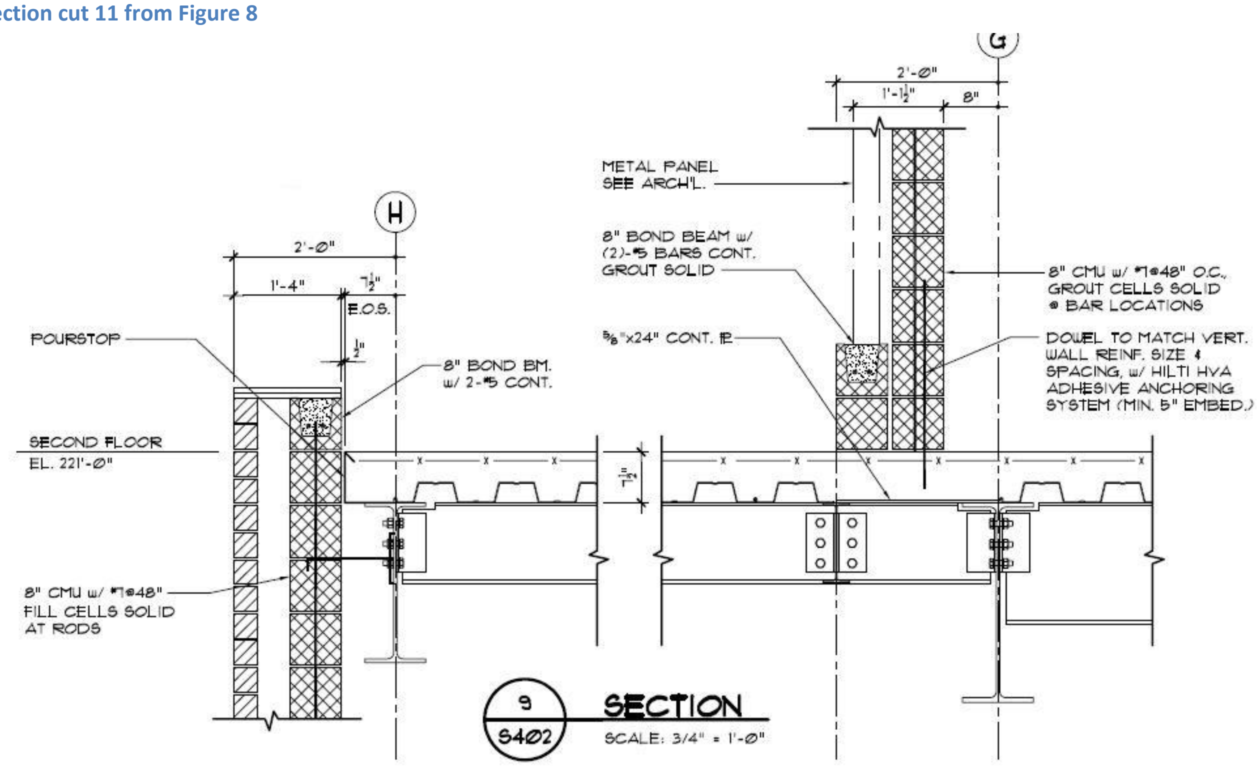


Figure 10 Section cut 9 from Figure 8

Roof Systems

TCMC has over 9 different roof heights, as shown in Figure 11.1 and Figure 11.2, 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’.

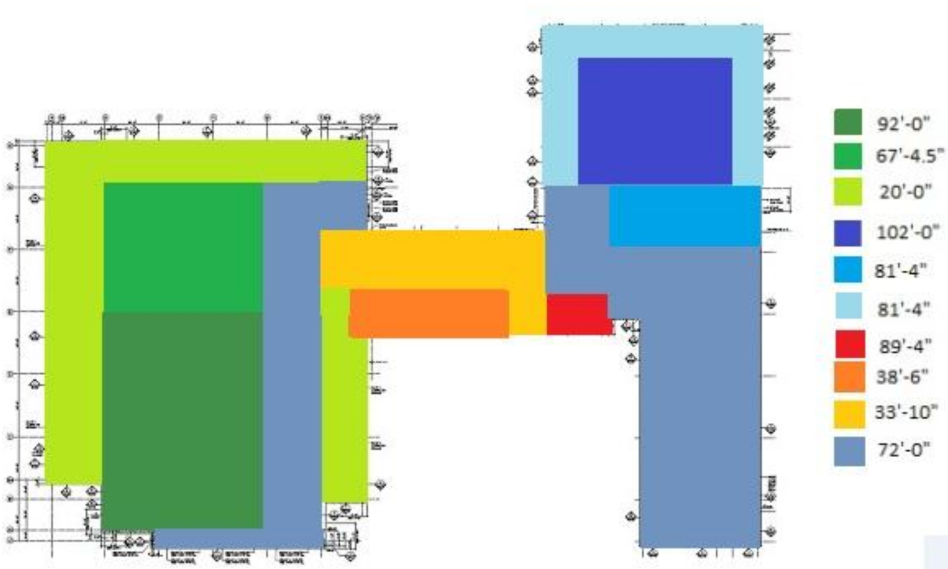


Figure 11.1 Plan showing the different roof heights; the darker, the higher.

Figure 11.2 Google Map Image showing the different roof heights of TCMC

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.

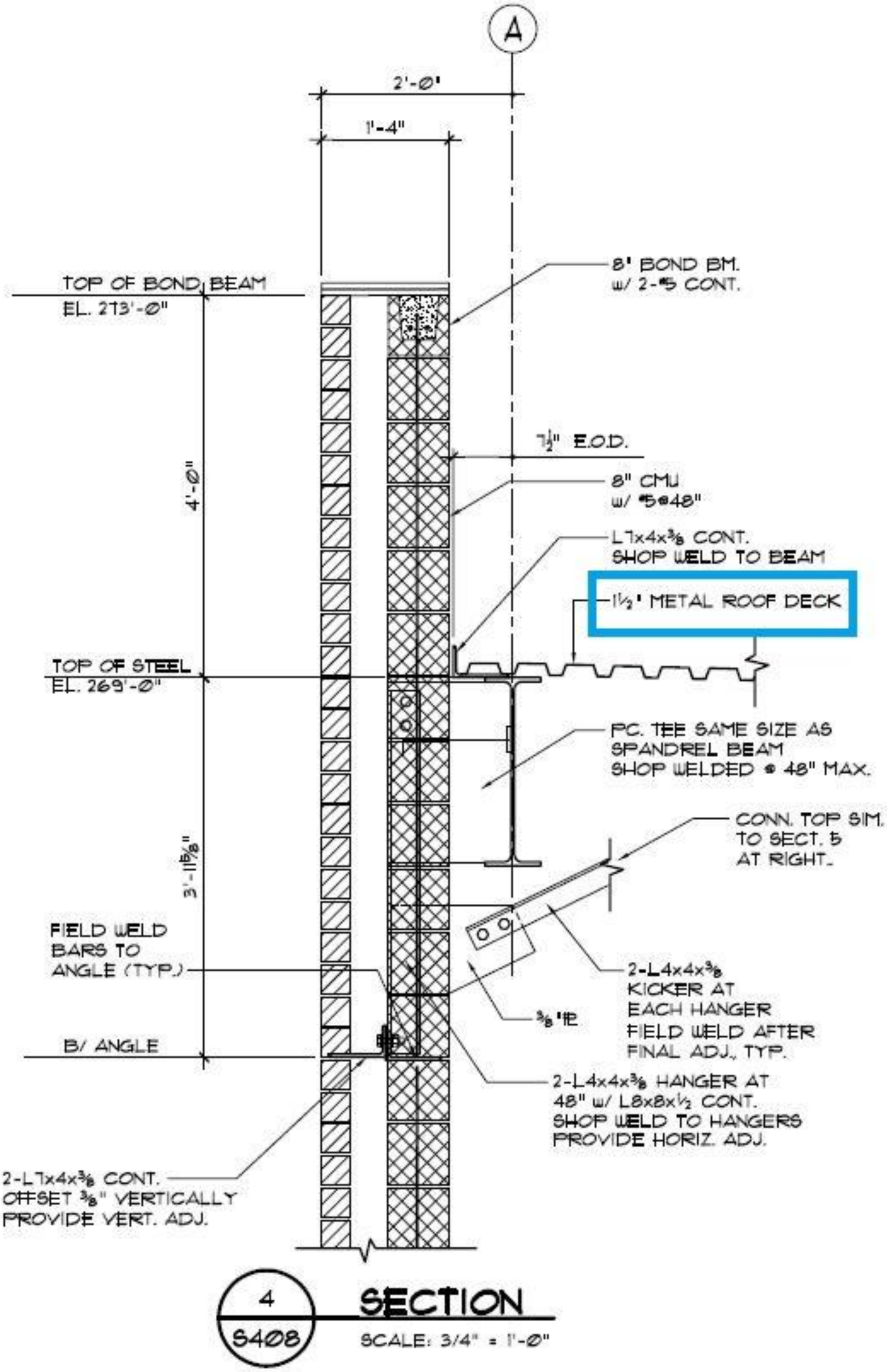


Figure 12 Typical roof section cut showing the roof deck. Courtesy of Highland Associates

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.

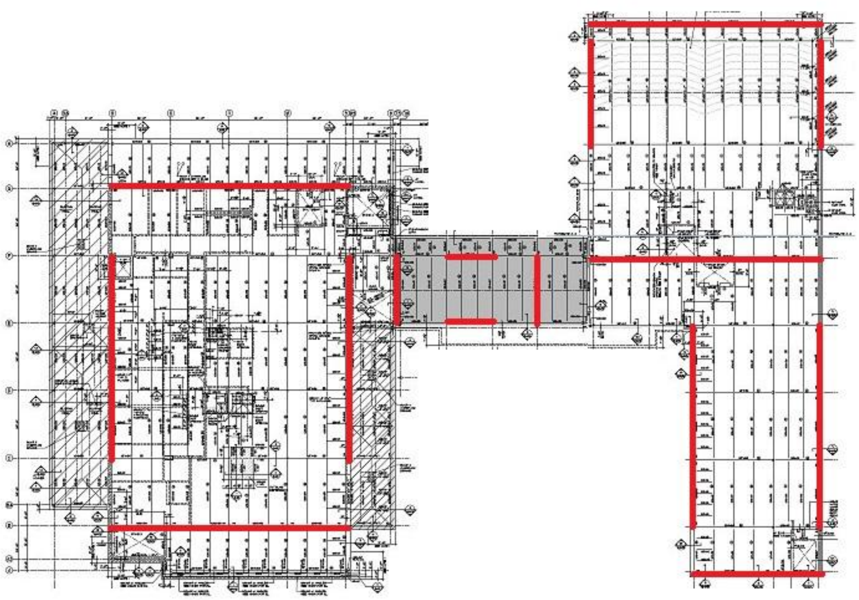


Figure 13.1 Locations of Moment Frames at TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

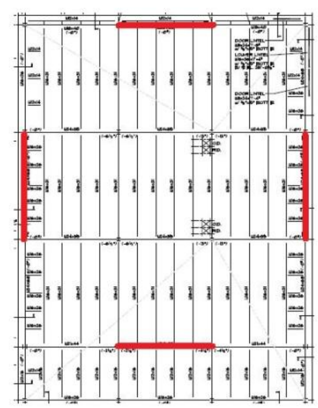


Figure 13.2 Locations of Moment Frames at the Penthouse of TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

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 Table 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 Table 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

Table 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

Table 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 Table 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	

Table 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. Table 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

Table 17 Variable for snow load obtained from S201B

Lateral Loads

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.

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. The structural drawing, sheet S201B, provided the basic wind load variables needed; see Table 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. Table 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

Table 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

Table 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)	
	Ground	0	9.41	3.62	-3.62	13.02	5.79
	2nd	21	9.41	3.62	-3.62	13.02	5.79
Windward	3th	37	9.94	3.62	-3.62	13.55	6.32
Walls	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

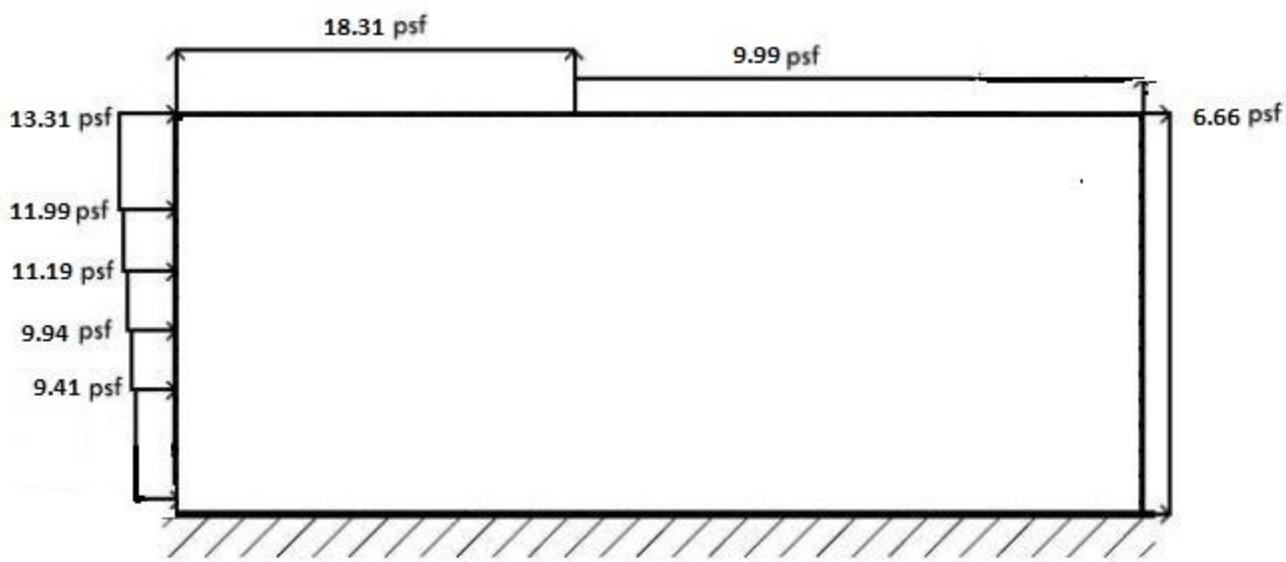


Figure 20 Wind Pressures acting on the West Wing, North and South facades

West Wing Wind Forces N-S Direction								
Floor	Height (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	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

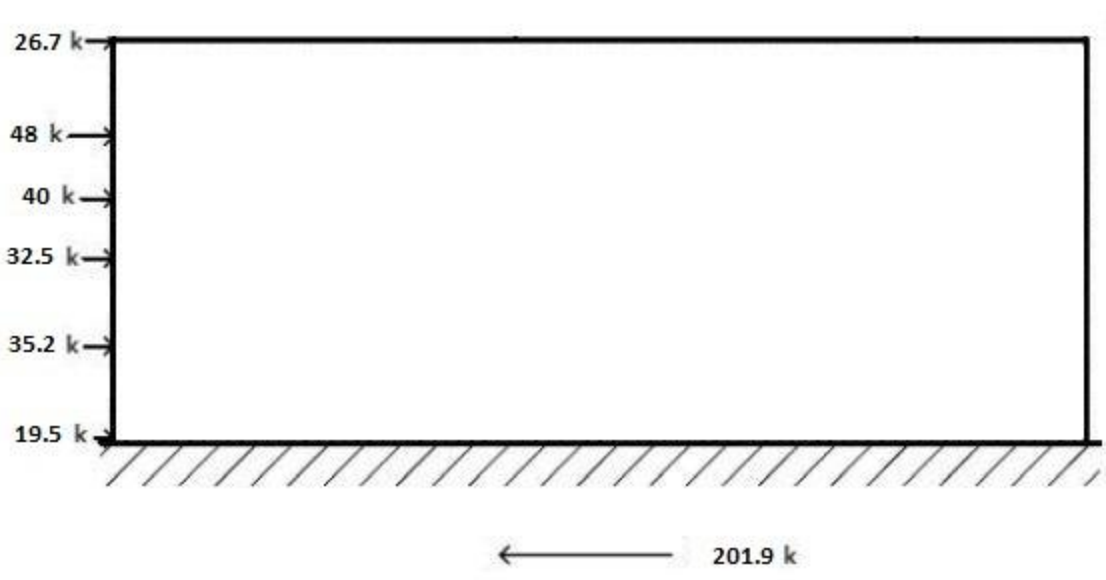


Figure 21 Wind Forces acting at each floor level on the West Wing, North and South facades

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

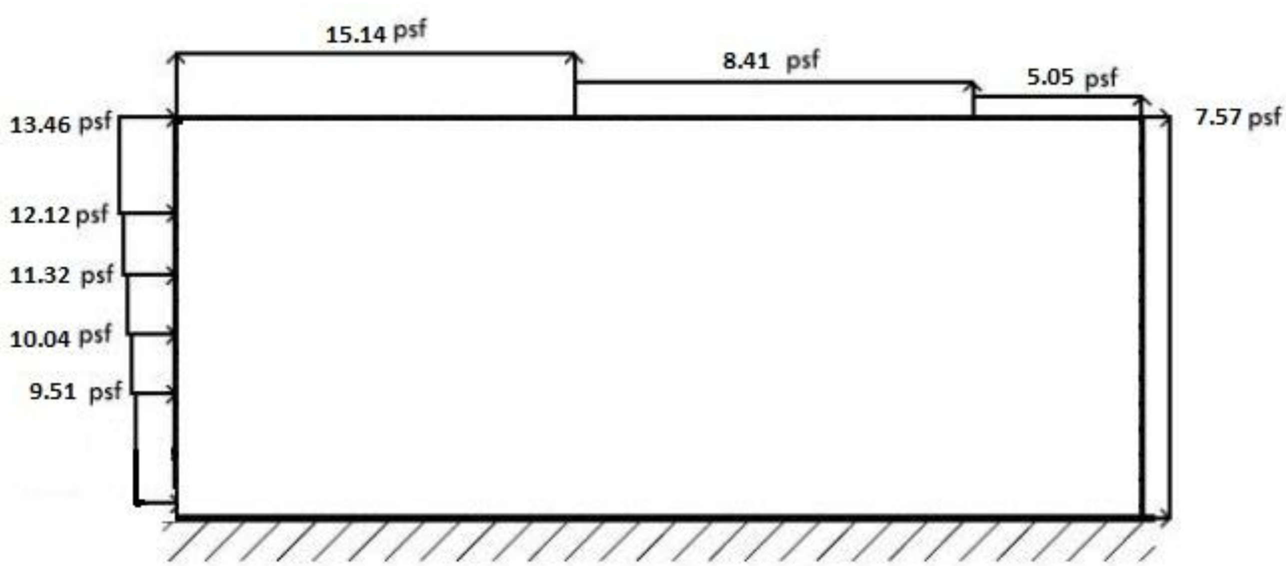


Figure 22 Wind Pressures acting on the West Wing, East and West facades

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

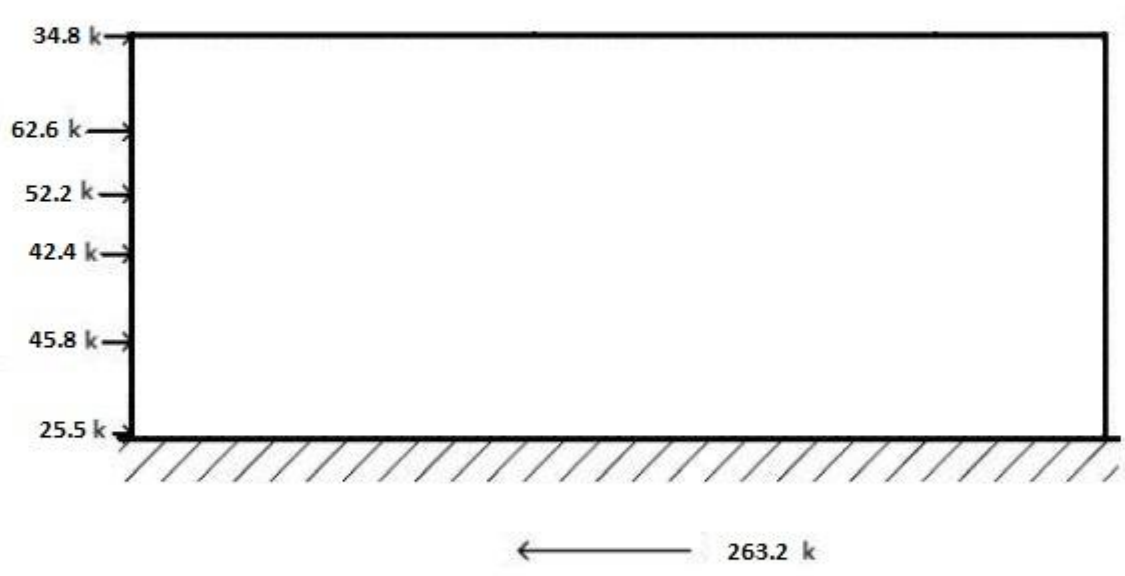


Figure 23 Wind Forces acting at each floor level on the West Wing, East and West facades

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

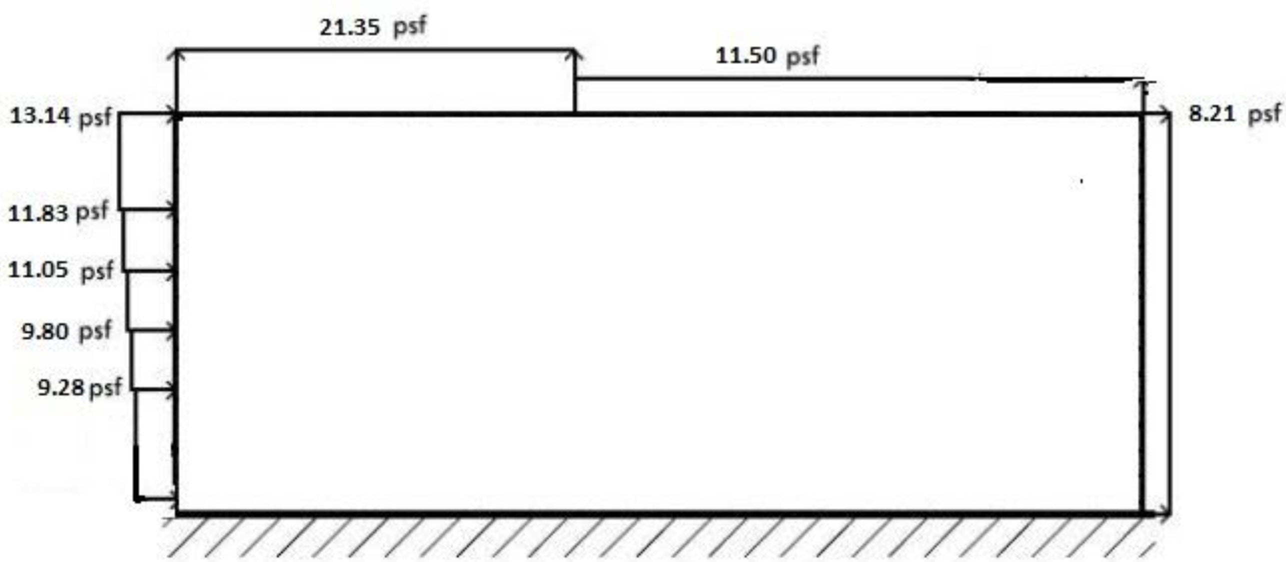


Figure 24 Wind Pressures acting on the East Wing, North and South facades

East Wing Wind Forces N-S Direction								
Floor	Height (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	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

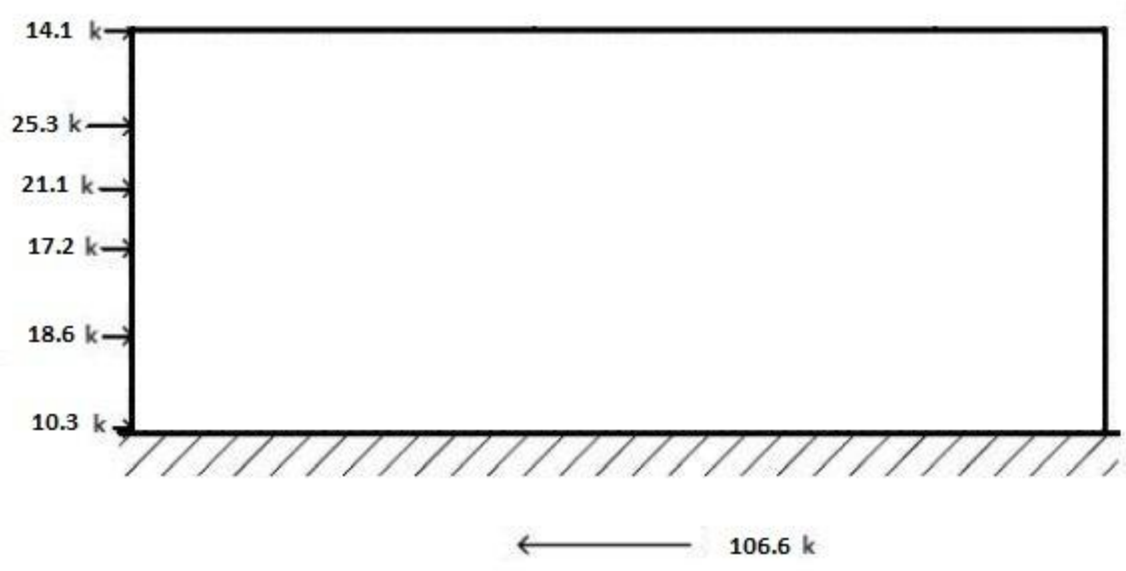


Figure 25 Wind Forces acting at each floor level on the East Wing, North and South facades

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

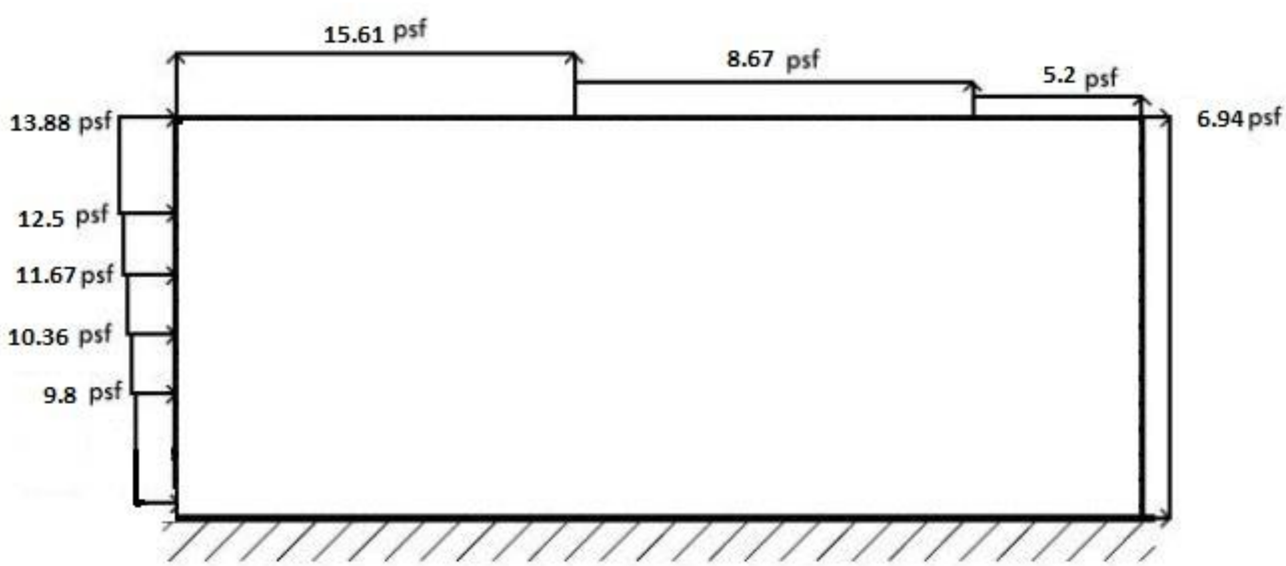


Figure 26 Wind Pressures acting on the East Wing, East and West facades

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

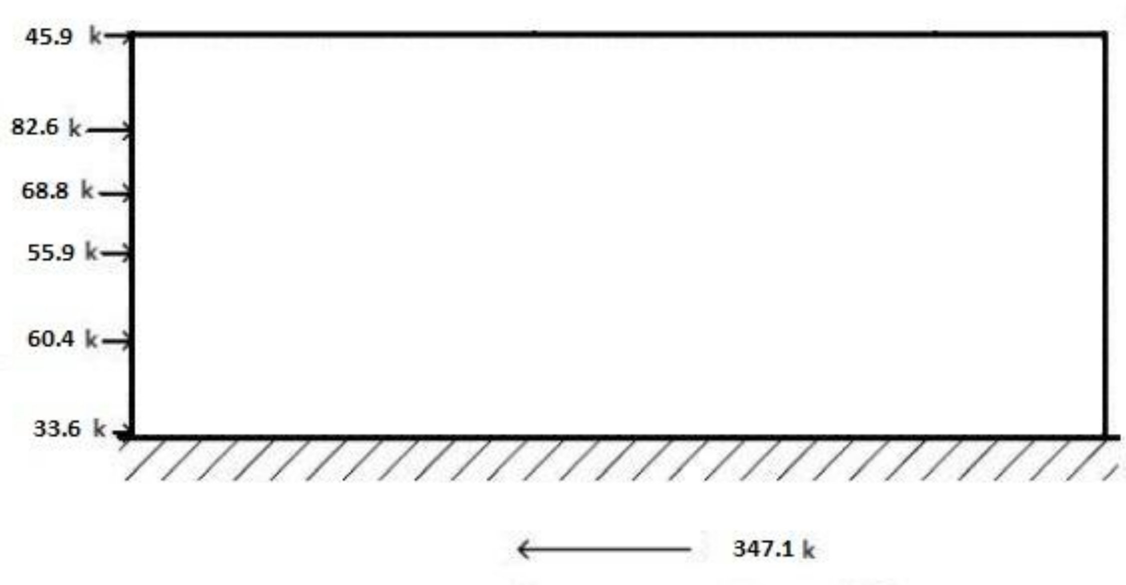


Figure 27 Wind Forces acting at each floor level on the East Wing, East and West facades

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. Table 28 is from S201B, showing the variables used. Table 29 shows the excel chart of the calculated variables.

Through this analysis, the base shear was found to be 130 kips in both the North-South and East-West direction of the West Wing, and 120 kips in both the North-South and East-West direction of the East Wing. 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 and 31, on the following pages, show 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

Table 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
T	1.05
T_L	6
Cs	0.001

Table 29 Calculated Variables for Seismic

Vertical Distribution of Seismic Forces West Wing							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	476	153804	0.110	14.33	14.33	1332.9
Penthouse	72	2163	504725	0.362	47.03	61.37	3386.4
4th	52	2497	384933	0.276	35.87	97.24	1865.3
3th	36	2497	240861	0.173	22.45	119.68	808.0
2nd	20	2429	110725	0.079	10.32	130.00	206.4
1st	0	2835	0	0.000	0.00	130.00	0.0
Total			1395049.35	1.000	130.00	N/A	7599.0

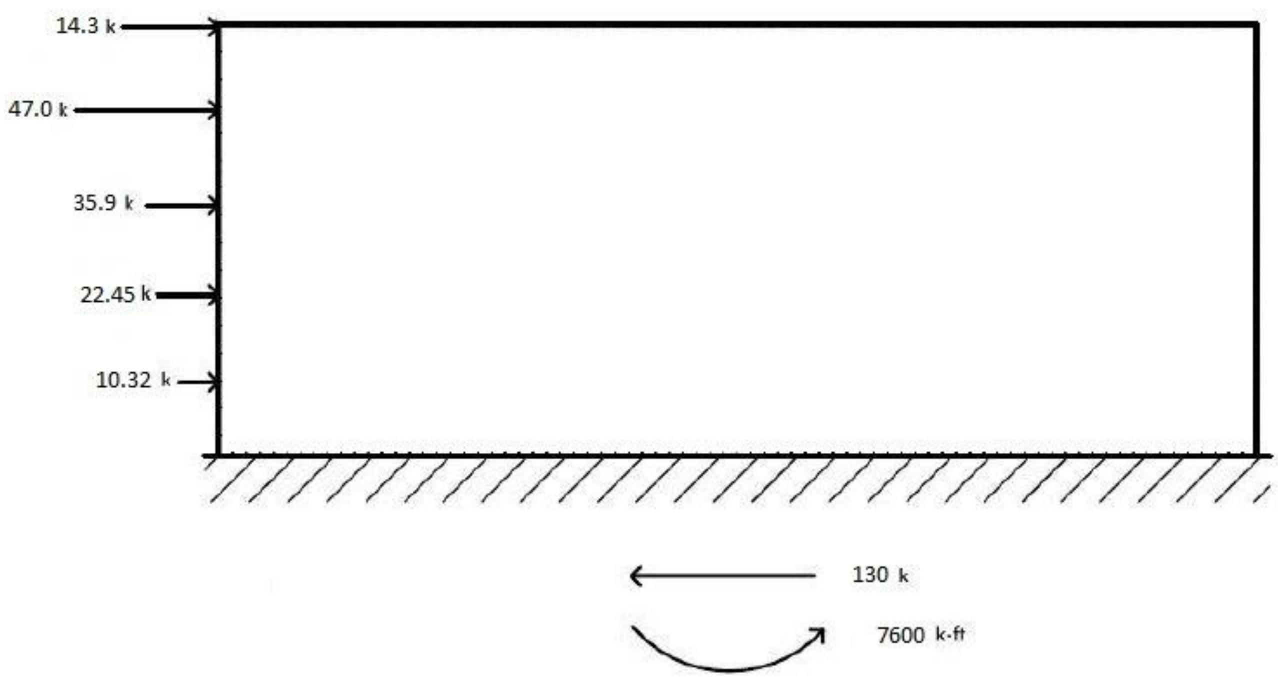


Figure 30 Table showing the vertical distribution of seismic forces with an elevation view. The same forces apply to both N-S and E-W direction for the West Wing

Vertical Distribution of Seismic Forces East Wing							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	454	146707	0.109	13.13	13.13	1221.0
Penthouse	72	2063	481435	0.359	43.08	56.21	3102.1
4th	52	2417	372516	0.278	33.34	89.55	1733.5
3th	36	2417	233091	0.174	20.86	110.41	751.0
2nd	20	2351	107154	0.080	9.59	120.00	191.8
1st	0	2264	0	0.000	0.00	120.00	0.0
Total			1340902.86	1.000	120.00	N/A	6999.4

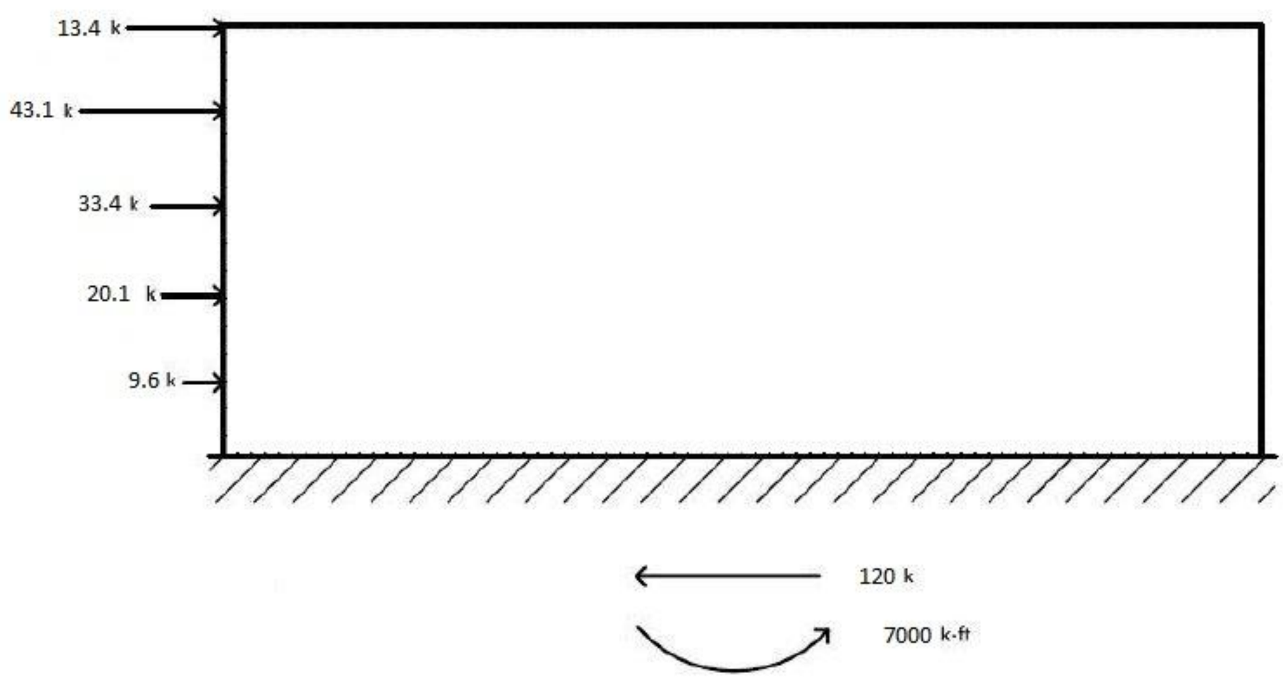


Figure 31 Table showing the vertical distribution of seismic forces with an elevation view. The same forces apply to both N-S and E-W direction for the East Wing.

Problem Statement

Under the current design, there is very little that can be done to improve the design of The Commonwealth Medical College. All structural elements met well above minimum code requirement. For the given architecture that the owner and the architect wanted; the current design is the best system in comparison to other alternatives. The author of this report was interested in steel design so TCMC was a great project to be worked on from the beginning.

The author of this report is also interested in building design for large lateral loads, especially from wind load. Therefore, a scenario was created in which TCMC was proposed to be built on a typical urban site in Miami, FL, instead of the original site. Wind velocity can reach up to 150 mph, as defined by code. Miami is considered to be one of the place with the highest wind velocity so creating this scenario will help the author better understand buildings under heavy wind load conditions.

A new viable lateral system must be designed to provide adequate strength and serviceability requirements to achieve minimum code standards. Loads that will be considered are dead load, live load, seismic load, and wind load. Lastly, the aim of the designs was to have the least amount of impact or change with the current architecture, schedule, and cost, as possible.

Not only the weather is different, but the site is also different in Miami and Scranton. The site in Scranton has a much higher bearing capacity, while the site in Miami has a lot less bearing capacity due to its sandy nature. Therefore, a new foundation will need to be designed.

Proposed Solution

To meet the new requirements of design for TCMC in Miami, FL, the lateral system were redesigned along with a new foundation design. The codes that were used to redesign TCMC in Miami are the Florida Building Code 2010, and ASCE 7-05. Two lateral system solutions, both in steel, have been proposed and analyzed for comparison. The two lateral systems were the following,

- Steel Moment Frames
- Steel Chevron Braced Frames

Moment frames are the original system for TCMC. It is now redesigned to withstand a larger wind load since it is now in Miami. This frame was then compared to a chevron braced frame system, which was the second solution proposed. Both systems kept their original gravity system.

The foundation was redesigned to account for the different soil condition in Miami Florida. To accomplish this, geotechnical research was conducted on a nearby location. In this case, two new foundation designs were done because the foundation for the two new proposed lateral systems are different due to the different load and length of the lateral frames. Because of the high load the building faces, and with a low bearing capacity on the site, a mat-slab foundation was chosen for the entire building because mat foundations are preferred when soil have low bearing capacity.

MAE Material Incorporation

The information learned in AE 534, Steel Connections, was utilized to design a typical chevron braced frame connection and a typical moment frame connection for TCMC. In addition, information learned in AE 542, Building Enclosures, was used to design and detail the new façade for impact and pressure resistance, waterproofing, and heat transfer. Lastly, information learned in AE 530, Computer Modeling was used to model the appropriate moment frame system and braced frame system on ETABS and confirmed with STAAD and hand calculations.

Breadth Studies

By relocating TCMC to Miami, Florida, the climate will be very different from which TCMC was originally designed for. In addition to impact loadings, the proposed new façade redesign incorporate heat transfer and waterproofing considerations. Breadth one focused on the redesign of TCMC façade to perform better in heat transfer, waterproofing, and impact against debris.

Being a LEED silver certified building, adding solar panels on the roof of TCMC increased its efficiency of energy usage. Research was done to confirm this along with finding the most efficient placement of the solar panels. The climate in Florida will make the solar panel system very beneficial because more sunlight can be converted into electricity. Solar panel design was investigated along with an inverter that will make the system work. This ‘free’ electric will be used for lighting, emergency system, and other usages.

And the last breath design was on small mechanical and electrical changes. The weather conditions in Miami are very different compared to Scranton. Because of this, new mechanical systems need to be analyzed. Heating units were replaced by cooling units under the warmer climate. The addition of solar panels impacted the electrical wirings. TCMC’s electrical connection will be change from simply electrical grid connection to a grid-tied connection. Lastly, a more powerful emergency backup power was installed in case of a large hurricane, which is unlikely to happen in Scranton; therefore it is not currently designed for it.

Structural Depth: Steel Redesigns

Moving TCMC to Miami means the structure will see a new site and new loads that it was not previously designed for. Therefore, this section will show the redesigns of TCMC that met both strength and serviceability requirements. Two lateral systems were designed, moment frames and chevron braced frames, and later compared to see which system is preferred. It is clear that chevron braced will be more effective but the author is interested in learning how much more efficient braced frames are over moment frames. The depth of the mat-slab foundation was also found for the two systems. As for the MAE requirement, a typical welded braced connection was designed for a brace on the second floor.

Miami Site Overview

A geotechnical report was never found for a typical urban site in Miami, FL. The closest site that a report was found for was an urban site in Orlando, FL. This gives a sense on how the site in Miami could be like but is not accurate. Therefore, some assumptions were made for this site, shown later in this report.

Figure 32, shows the location of the Miami site, shaded in orange, that TCMC was designed to be built on. Shaded in blue (not drawn to scale) is a footprint of TCMC. This site is slightly larger than the current site of TCMC which confirms that the building will fit. The footprint shows the orientation of the building on the site.

This land is currently used as a parking lot. It was chosen because many buildings in this area are related to the medical field, such as hospitals and other medical schools.

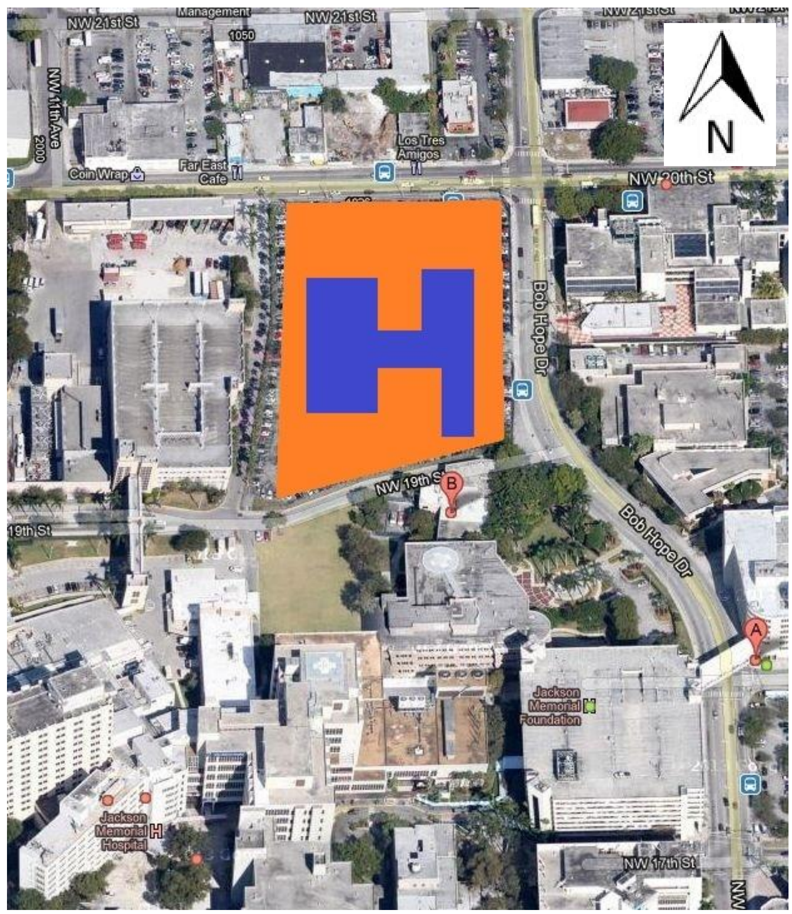


Figure 32 Shaded in orange is the area where TCMC will be build. Shaded in blue is a footprint, but not to scale, of TCMC.

Miami, FL, Wind Load

The main focus in Miami was calculating wind loads. A wind study was performed on TCMC for Miami, FL, using ASCE 7-05, MWFRS Analytical Procedure. This was done the same way as TCMC in Scranton, PA. 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. Table 33, provided the basic wind load variables needed. A factored base shear of 560.5k was found for the West wing in the North-South direction. A factored base shear of 295.9k was found for the East wing in the North-South direction. As for the East-West direction, a factored base shear of 730.8k was found for the West and a factored base shear of 963.7k 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. Again, this 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. Table 34 gives the summary of the wind loads. Figures 35 to 42 on the next couple pages show the wind pressures and wind forces acting on the West and East wing of TCMC, along with an elevation view.

Wind Load Variables	
Basic Wind Speed	150 mph
Importance Factor	1.15
Exposure Category	B

Table 33 Show the wind load variables used in design

Summary: Wind Loads on TCMC				
	West Wing		East Wing	
NS Base Shear	560.0	k	296	k
NS Overturning Moment	27500.0	k-ft	14500	k-ft
ES Base Shear	731.0	k	960	k
EW Overturning Moment	35800.0	k-ft	47220	k-ft

Table 34 Summary of the new Wind Loads on TCMC

West Wing Wind Pressures N-S Direction							
Type	Floor	Distance	Wind Pressure	Internal Pressure		Net Pressure	
		(ft)	(psf)	(psf)		(psf)	
	Ground	0	26.2	10.0	-10.0	36.2	16.1
	2nd	21	26.2	10.0	-10.0	36.2	16.1
Windward	3th	37	27.6	10.0	-10.0	37.7	17.6
Walls	4th	53	31.0	10.0	-10.0	41.0	21.0
	Penthouse	69.5	33.3	10.0	-10.0	43.3	23.2
	Roof	93	37.0	10.0	-10.0	47.0	26.9
Leeward Walls	All	All	-18.5	10.0	-10.0	-8.4	-28.5
Side Walls	All	All	-32.3	10.0	-10.0	-22.3	-42.4
Roof	N/A	0-46.5	-50.8	10.0	-10.0	-40.8	-60.9
	N/A	46.5-186	-27.7	10.0	-10.0	-17.7	-37.8

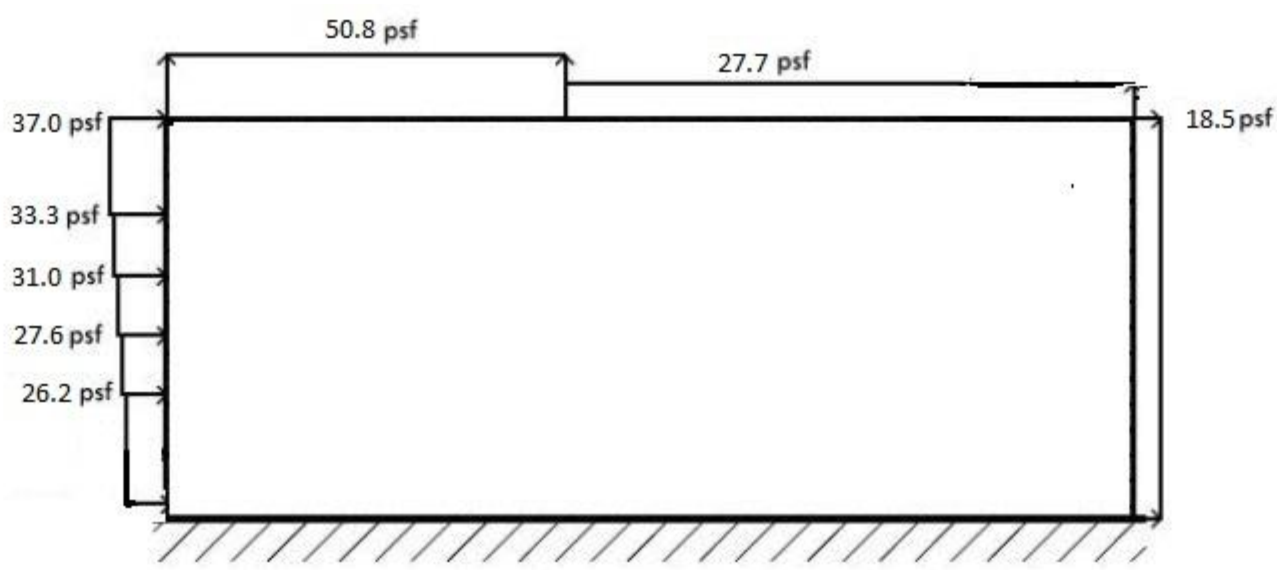


Figure 35 Wind Pressures acting on the West Wing, North and South facades

West Wing Wind Forces N-S 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	1500	54.3	560.5	0.0
2nd	20	10	1500	8	1200	97.8	506.2	1955.3
3th	36	8	1200	8	1200	90.4	408.4	3254.3
4th	52	8	1200	10	1500	110.8	318.0	5762.6
Penthouse	72	10	1500	10.5	1575	133.1	207.2	9585.8
Roof	93	10.5	1575	0	0	74.0	74.0	6885.2
Total						560.5	N/A	27443.3

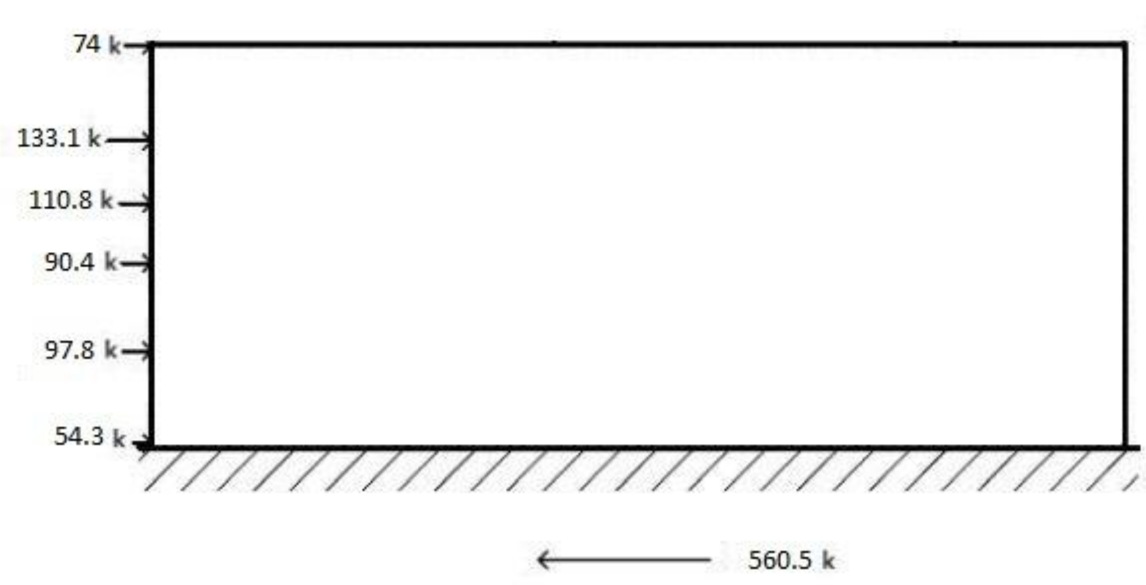


Figure 36 Wind Forces acting at each floor level on the West Wing, North and South facades

West Wing Wind Pressures E-W Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
	Ground	0	26.4	10.0	-10.0	36.5	16.4
	2nd	21	26.4	10.0	-10.0	36.5	16.4
Windward Walls	3th	37	27.9	10.0	-10.0	38.0	17.9
	4th	53	31.3	10.0	-10.0	41.4	21.3
	Penthouse	69.5	33.6	10.0	-10.0	43.7	23.6
	Roof	93	37.4	10.0	-10.0	47.4	27.3
Leeward Walls	All	All	-21.0	10.0	-10.0	-11.0	-31.1
Side Walls	All	All	-32.7	10.0	-10.0	-22.6	-42.7
Roof	N/A	0-93	-42.0	10.0	-10.0	-32.0	-52.1
	N/A	93-186	-23.4	10.0	-10.0	-13.3	-33.4
	N/A	>186	-14.0	10.0	-10.0	-4.0	-24.1

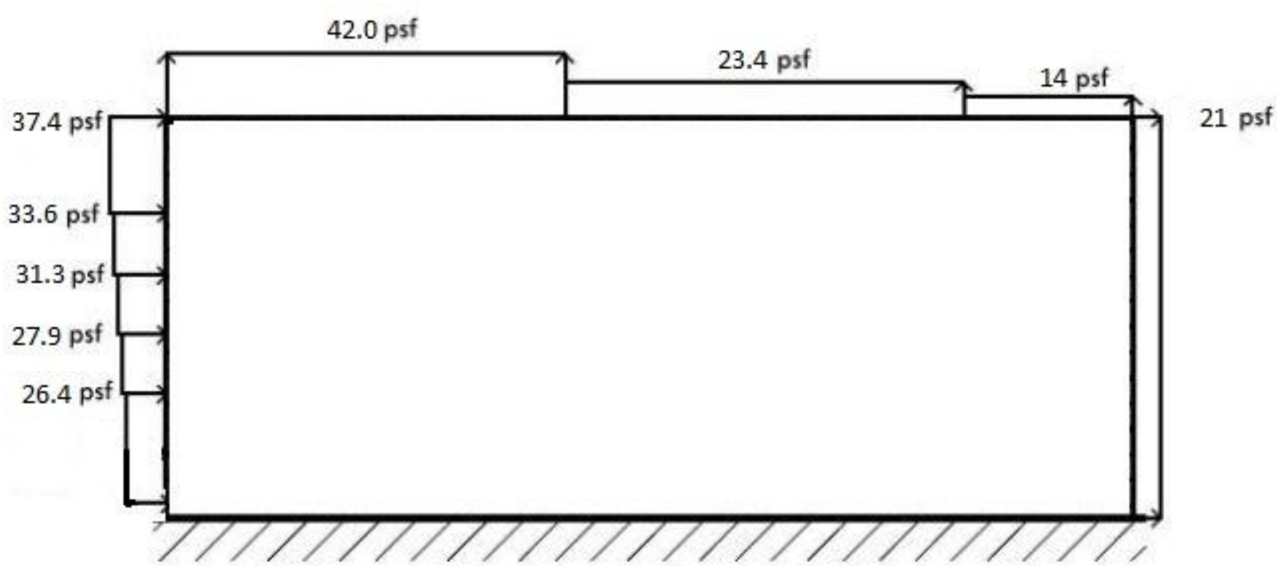


Figure 37 Wind Pressures acting on the West Wing, East and West facades

West 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	1940	70.8	730.8	0.0
2nd	20	10	1940	8	1552	127.4	660.0	2548.7
3th	36	8	1552	8	1552	117.8	532.5	4242.5
4th	52	8	1552	10	1940	144.5	414.7	7514.2
Penthouse	72	10	1940	10.5	2037	173.6	270.2	12501.2
Roof	93	10.5	2037	0	0	96.6	96.6	8981.0
Total						730.8	N/A	35787.5

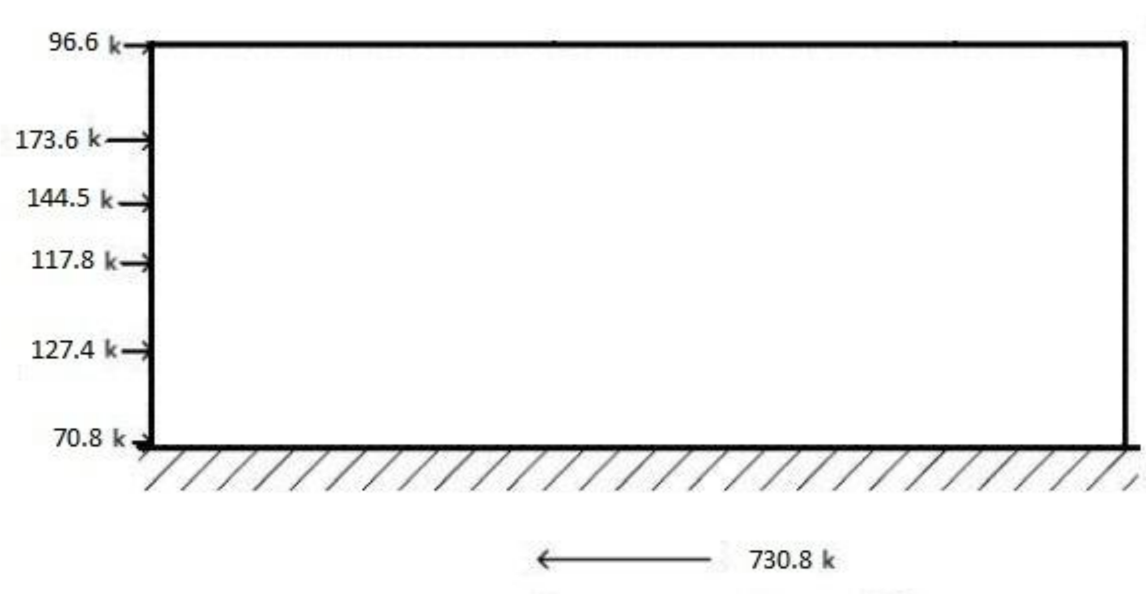


Figure 38 Wind Forces acting at each floor level on the West Wing, East and West facades

East Wing Wind Pressures N-S Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
	Ground	0	25.8	10.0	-10.0	35.9	15.8
	2nd	21	25.8	10.0	-10.0	35.9	15.8
Windward Walls	3th	37	27.3	10.0	-10.0	37.3	17.2
	4th	53	30.6	10.0	-10.0	40.6	20.5
	Penthouse	69.5	32.8	10.0	-10.0	42.9	22.8
	Roof	93	36.5	10.0	-10.0	46.5	26.4
Leeward Walls	All	All	-22.8	10.0	-10.0	-12.8	-32.8
Side Walls	All	All	-31.9	10.0	-10.0	-21.9	-42.0
Roof	N/A	0-46.5	-59.3	10.0	-10.0	-49.2	-69.3
	N/A	46.5-186	-31.9	10.0	-10.0	-21.9	-42.0

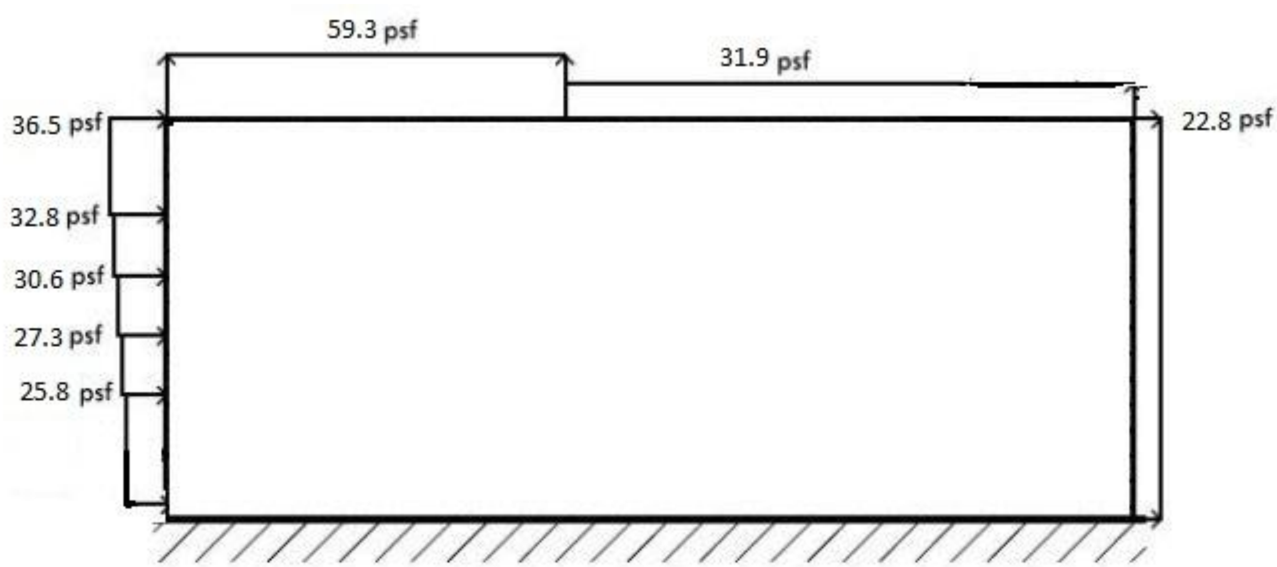


Figure 39 Wind Pressures acting on the East Wing, North and South facades

East Wing Wind Forces N-S Direction								
Floor	Height (ft)	Trib Below		Trib Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
		height (ft)	area (sf)	height (ft)	area (sf)			
Ground	0	0	0	10	800	28.7	295.9	0.0
2nd	20	10	800	8	640	51.6	267.2	1032.8
3th	36	8	640	8	640	47.7	215.6	1718.7
4th	52	8	640	10	800	58.5	167.9	3042.6
Penthouse	72	10	800	10.5	840	70.3	109.4	5060.3
Roof	93	10.5	840	0	0	39.1	39.1	3633.7
Total						295.9	N/A	14488.1

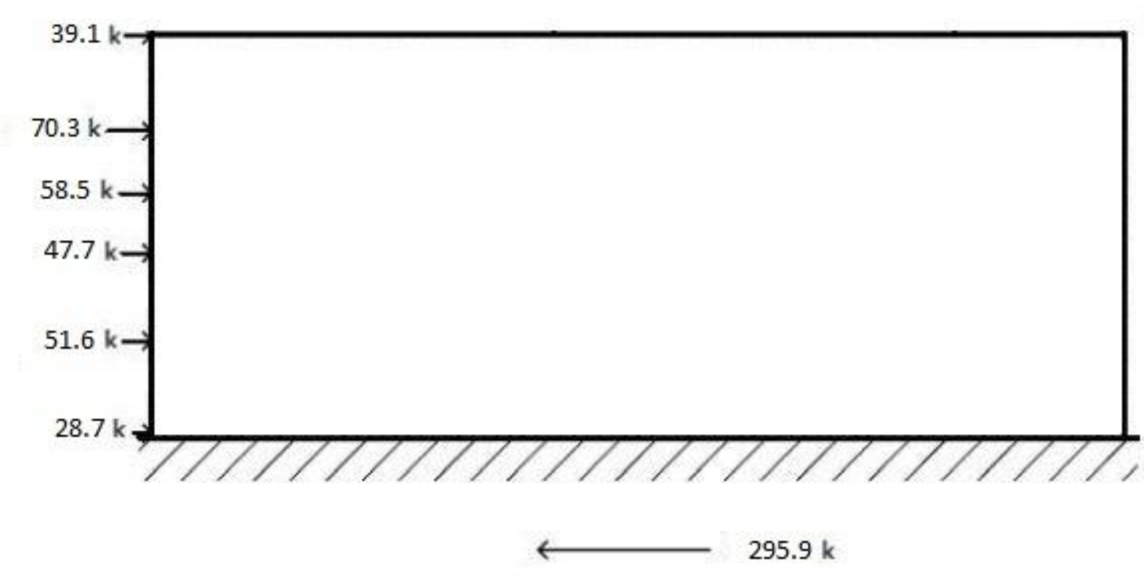


Figure 40 Wind Forces acting at each floor level on the East Wing, North and South facades

East Wing Wind Pressures E-W Direction							
Type	Floor	Distance (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
Windward Walls	Ground	0	27.3	10.0	-10.0	37.3	17.2
	2nd	21	27.3	10.0	-10.0	37.3	17.2
	3th	37	28.8	10.0	-10.0	38.8	18.7
	4th	53	32.3	10.0	-10.0	42.4	22.3
	Penthouse	69.5	34.7	10.0	-10.0	44.7	24.6
	Roof	93	38.5	10.0	-10.0	48.6	28.5
Leeward Walls	All	All	-19.3	10.0	-10.0	-9.2	-29.3
Side Walls	All	All	-33.7	10.0	-10.0	-23.7	-43.8
Roof	N/A	0-93	-43.3	10.0	-10.0	-33.3	-53.4
	N/A	93-186	-24.1	10.0	-10.0	-14.0	-34.1
	N/A	>186	-14.4	10.0	-10.0	-4.4	-24.5

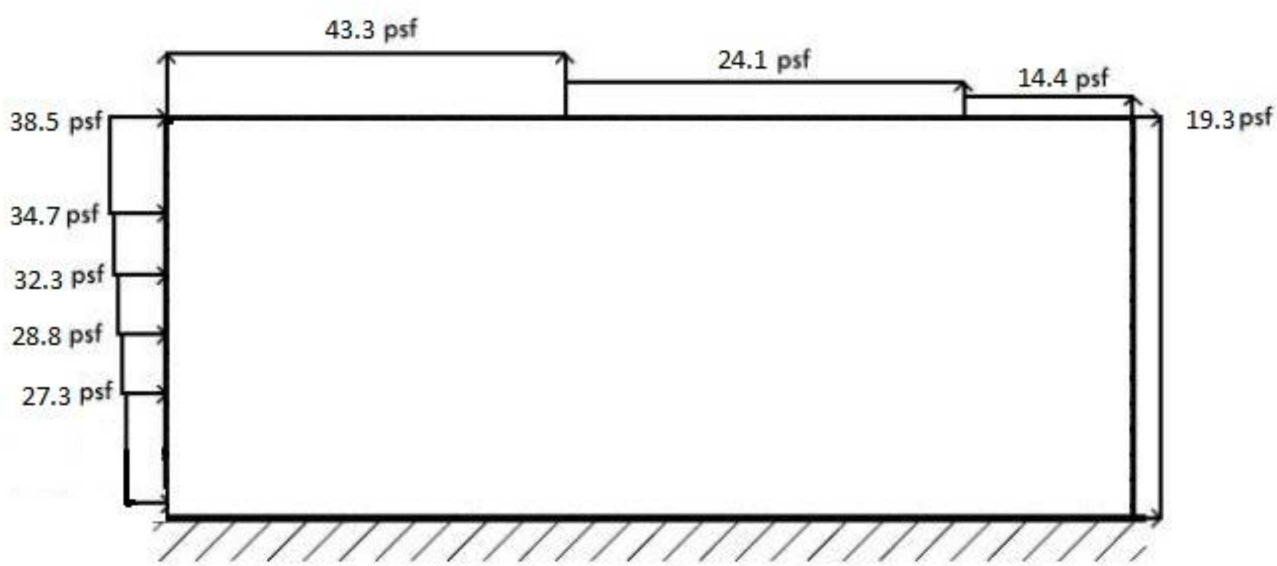


Figure 41 Wind Pressures acting on the East Wing, East and West facades

East 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	2500	93.3	963.7	0.0
2nd	20	10	2500	8	2000	167.9	870.4	3358.3
3th	36	8	2000	8	2000	155.3	702.5	5592.0
4th	52	8	2000	10	2500	190.6	547.2	9911.0
Penthouse	72	10	2500	10.5	2625	229.1	356.6	16495.1
Roof	93	10.5	2625	0	0	127.5	127.5	11856.7
Total						963.7	N/A	47213.2

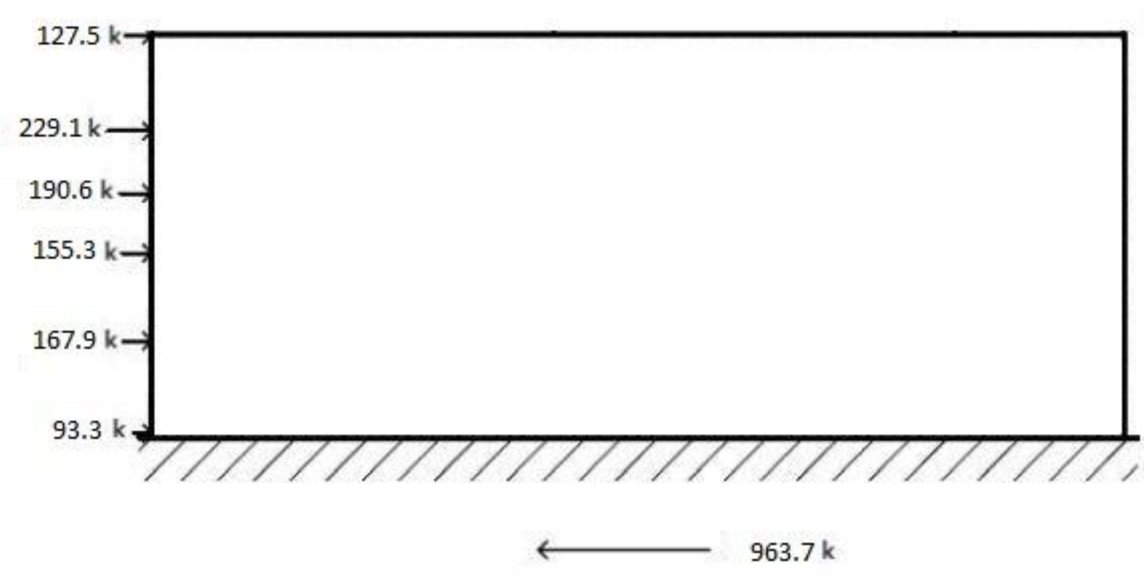


Figure 42 Wind Forces acting at each floor level on the East Wing, East and West facades

Miami, FL, Seismic Load

Seismic loads were calculated using ASCE 7-05, chapters 11 and 12, same procedure when it is located in Scranton, PA. The only difference in the calculation was the building's weight. Because larger steel members were needed, it was assumed that the building overall weight was increased by 5%. This did not have a huge impact on the seismic load on the building. Table 43 shows the new seismic design data and calculated variables.

Through this analysis, the base shear was found to be 136 kips in both the North-South and East-West direction of the West Wing, and 126 kips in both the North-South and East-West direction of the East Wing. This is only a small increase and it is due to the change of the building's weight. 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. Figures 44 and 45, on the next following pages, show the table with the distribution of forces, along with an elevation view.

Calculated Variables	
Fa	1
Fv	1
Sms	0.08
Sm_1	0.047
S_{DS}	0.053
S_{D1}	0.031
R	3
T	1.05
T_L	6
Cs	0.001

Table 43 Calculated Variables for Seismic

Vertical Distribution of Seismic Forces West Wing							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	499	161494	0.110	14.99	14.99	1394.4
Penthouse	72	2271	529962	0.362	49.20	64.20	3542.7
4th	52	2622	404180	0.276	37.53	101.72	1951.4
3th	36	2622	252904	0.173	23.48	125.21	845.3
2nd	20	2550	116262	0.079	10.79	136.00	215.9
1st	0	2977	0	0.000	0.00	136.00	0.0
Total			1464801.82	1.000	136.00	N/A	7949.7

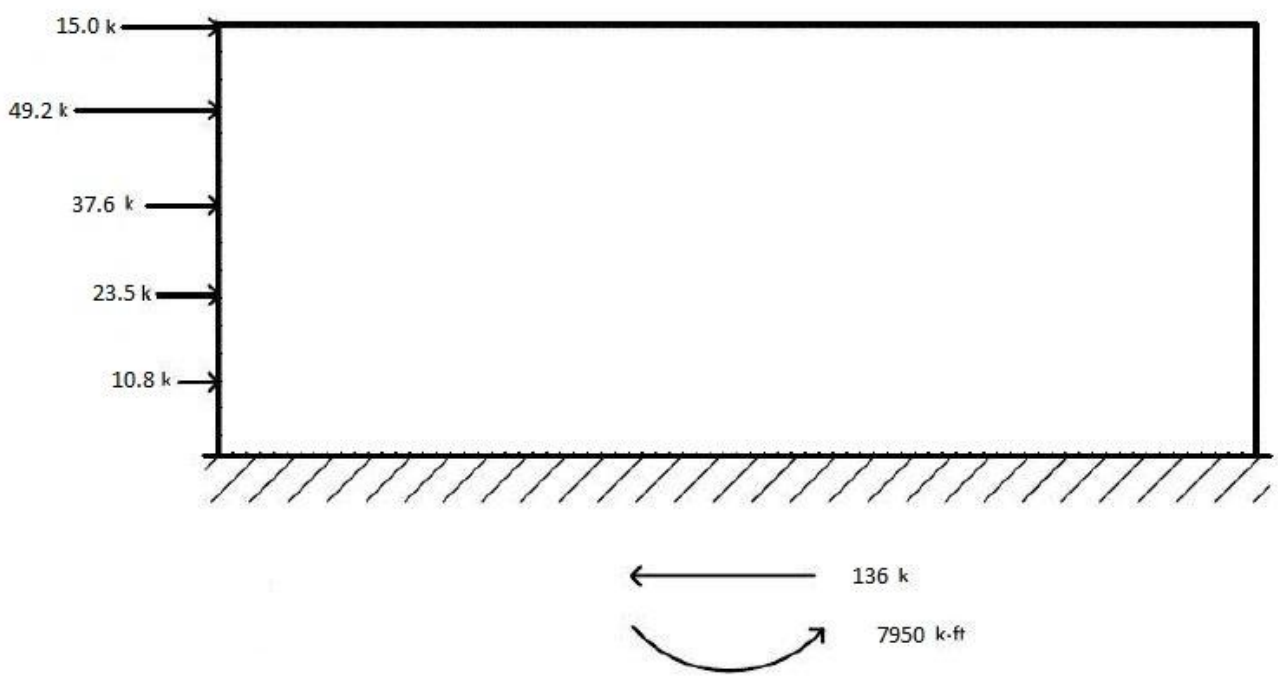


Figure 44 Table showing the vertical distribution of seismic forces on the West Wing with an elevation view. The same forces apply to both N-S and E-W direction.

Vertical Distribution of Seismic Forces East Wing							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	476	154042	0.109	13.79	13.79	1282.1
Penthouse	72	2166	505507	0.359	45.24	59.02	3257.2
4th	52	2538	391142	0.278	35.00	94.03	1820.2
3th	36	2538	244746	0.174	21.90	115.93	788.5
2nd	20	2468	112511	0.080	10.07	126.00	201.4
1st	0	2377	0	0.000	0.00	126.00	0.0
Total			1407948	1.000	126.00	N/A	7349.3

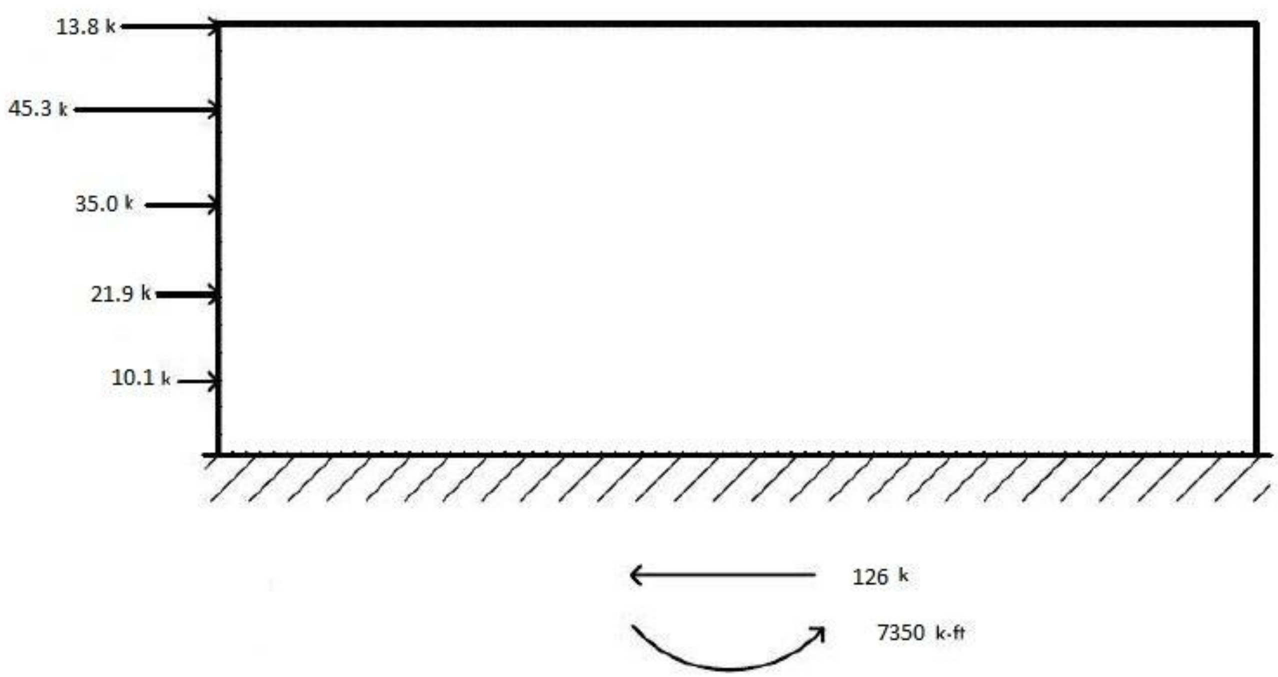


Figure 45 Table showing the vertical distribution of seismic forces on the East Wing with an elevation view. The same forces apply to both N-S and E-W direction.

Comparison of Wind and Seismic Forces

By comparing the lateral loads produced by wind and seismic forces, it shows that wind forces greatly controlled over seismic forces in both North-South and East-West direction, as shown in Table 46. The shear values have been factored by 1.6 for wind loads to allow for LRFD comparison between the two loads.

Comparison of Seismic and Wind Forces						
	West Wing			East Wing		
Miami, FL	Wind, N-S	Wind, E-W	Seismic	Wind, N-S	Wind, E-W	Seismic
Base Shear (k)	560	730	136	300	970	126
Overturning Moment (k-ft)	27500	35800	7950	14500	47300	7350
Scranton, PA	Wind, N-S	Wind, E-W		Wind, N-S	Wind, E-W	Seismic
Base Shear (k)	200	270	130	110	350	120
Overturning Moment (k-ft)	10000	12900	7600	5230	17100	7000

Table 46 Comparison of Seismic and Wind Forces

Moment Frame Design

This design was created for the purpose of keeping the new structure as similar to the existing structure as possible. Because moment frames are utilized for the new building, minimum changes were made to the overall project. This will allow TCMC to keep its original architectural look. The layout of the moment frames were kept the same. This is because the original layout was already designed for maximum efficiency. Additional bays cannot be added to frames without having to increase the building's length. Figure 47 shows the layout of the moment frames, in red. The frames were lettered for ease of reference. Figure 48 shows the moment frames of the penthouse, which is the last story in the West Wing. Only one bay from Frames B and C extends up to the penthouse, as shown in that figure.

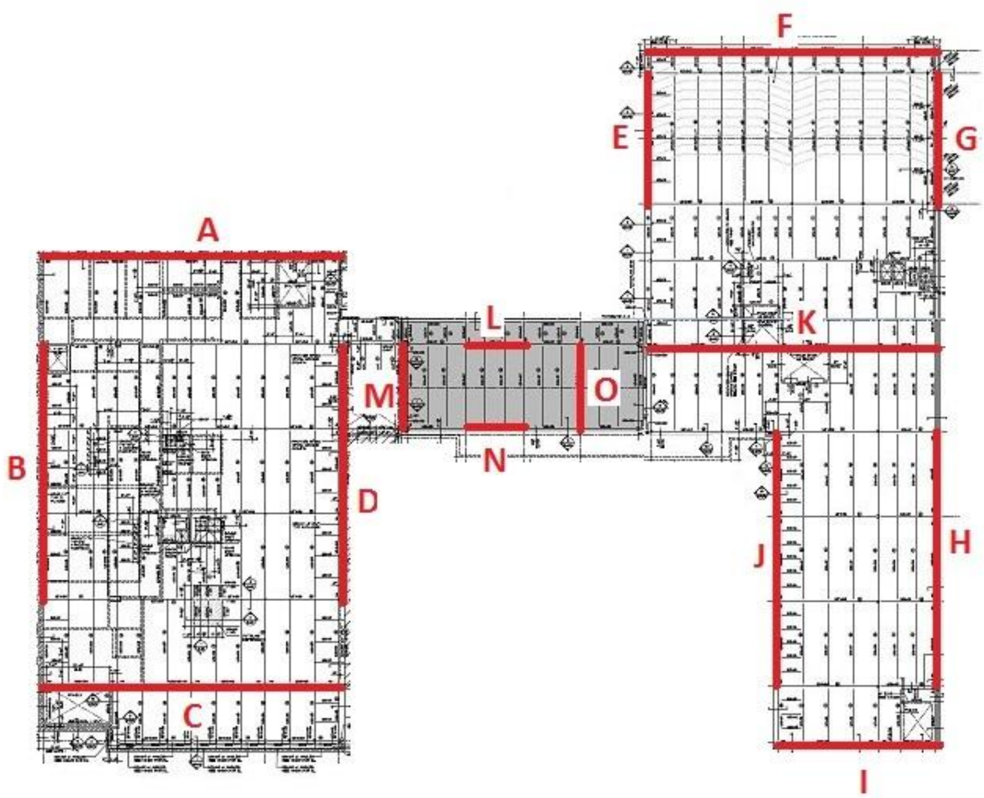


Figure 47 New Moment Frames Layout

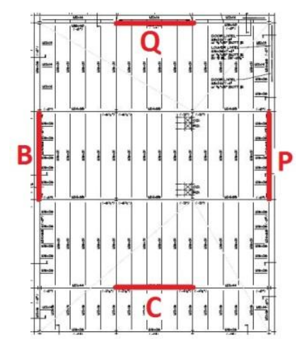


Figure 48 Moment Frame Layout on the Penthouse in the West Wing

This layout was used to produce ETABS models. A total of three separate models were designed, the West Wing, East Wing, and the Link, as shown in Figures 49.1 to 49.3. Through calculations of strength design, shown in Appendix D, preliminary sizes where chosen for the members to be inputted into ETABS. Inputting the wind loads, dead loads, live loads, load combinations, and setting a drift limit of 0.02h into ETABS, the members were redesigned by ETABS. All diaphragms were modeled as rigid because it has a composite steel deck. The finalized size of the members met both strength and drift requirements. All modeling designs were confirmed with STAAD and hand calculations to make sure the model does not have error.

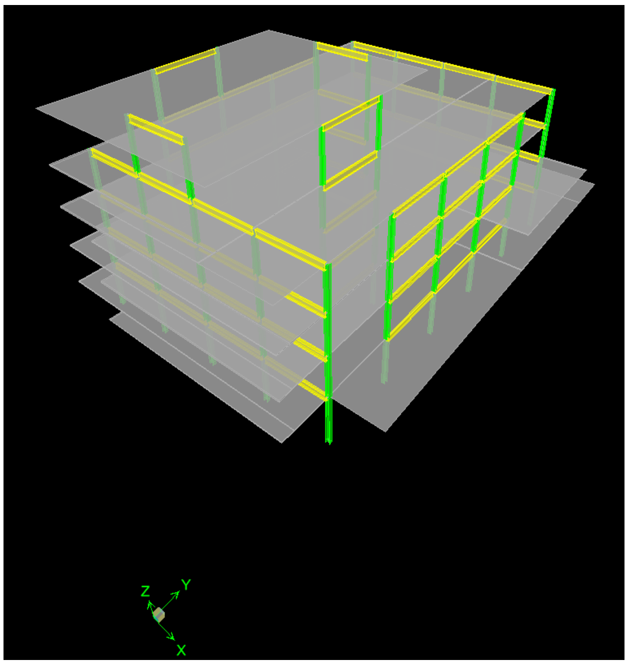


Figure 49.1 West Wing Moment Frame ETABS model

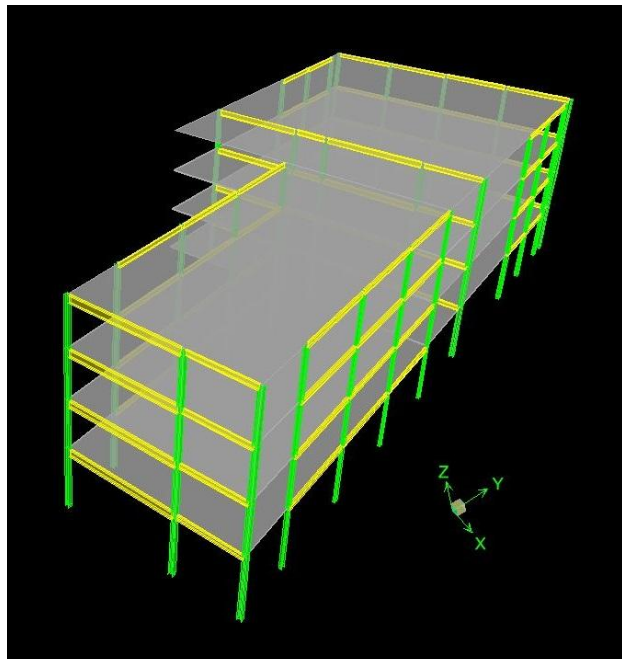


Figure 49.2 East Wing Moment Frame ETABS model

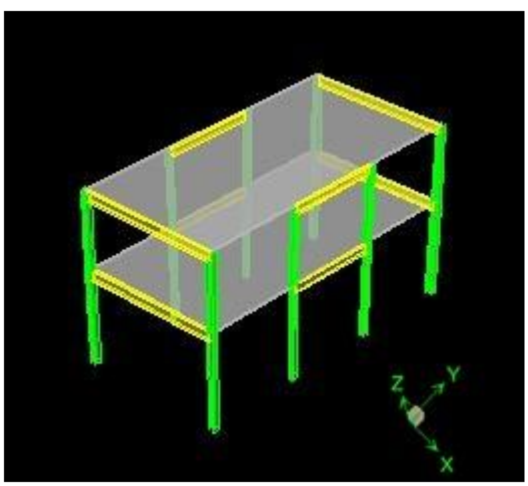
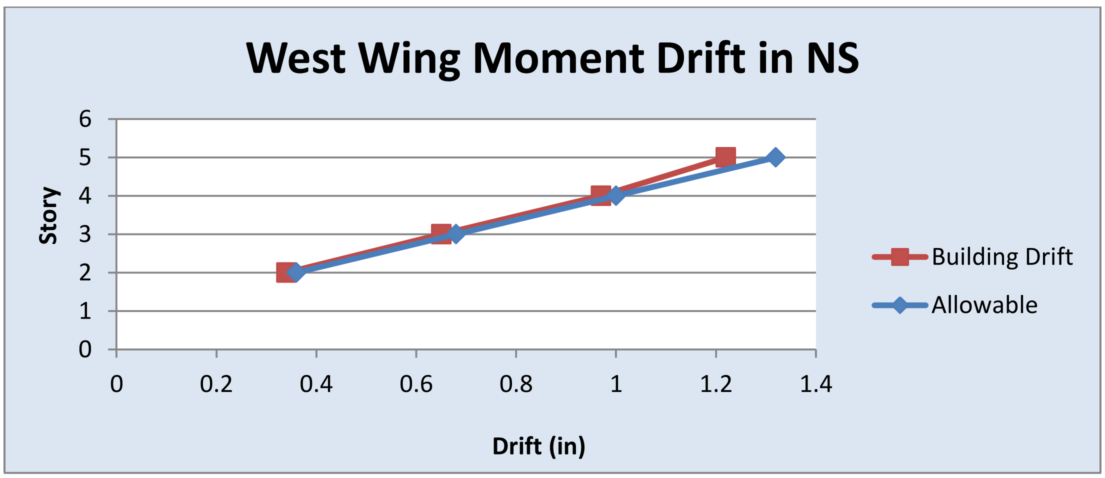
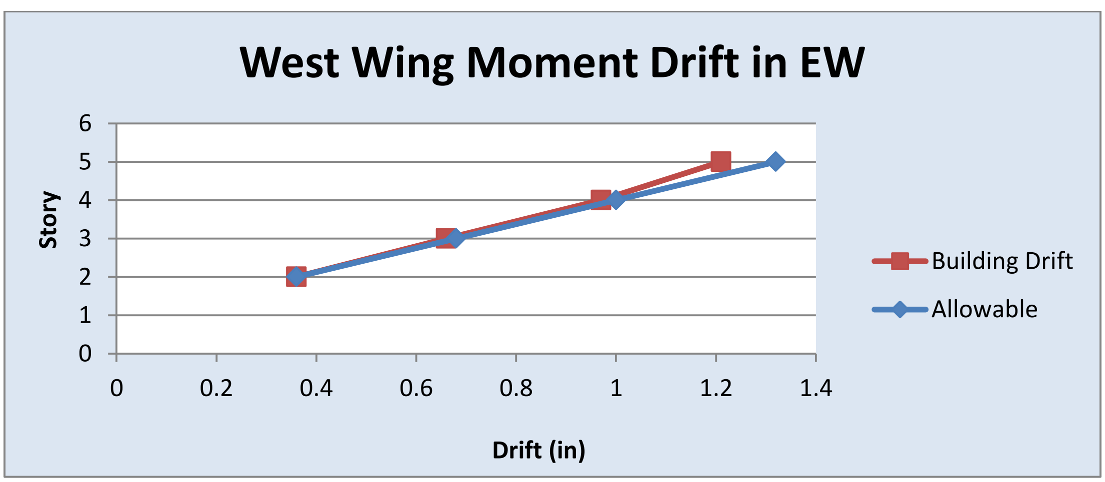


Figure 49.3 Link Moment Frame ETABS model

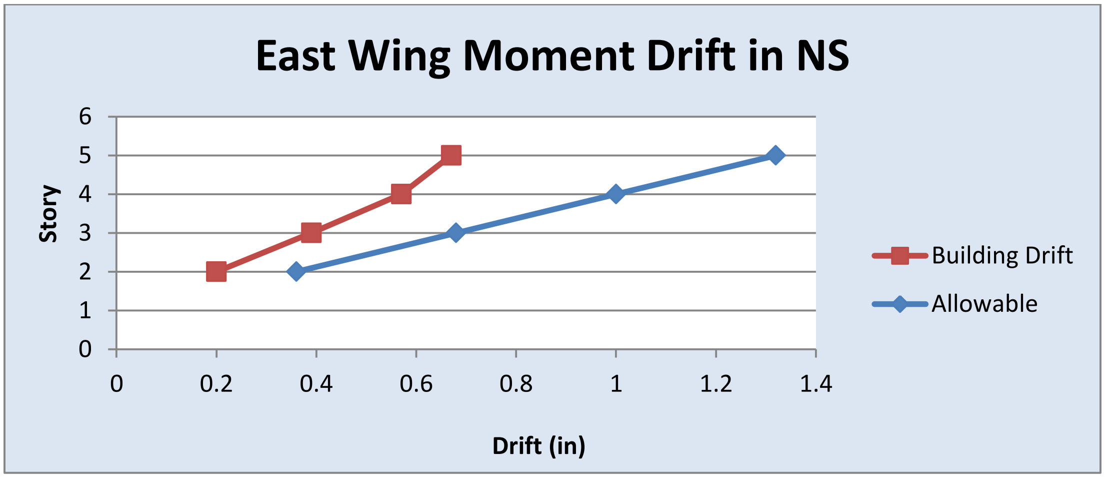
From hand calculations, it is reasonably effective in finding the strength design of the members. However, calculating drift is inaccurate. If drift controls, then the members will have to be re-sized until they are within the drift limit. For moment frames, drift limit greatly controls the design. Graph 50.1 and 50.2, shows how close the drift experienced from the building, in red, is to the allowable drift limit, in blue. If strength is the only factor, then W27x146, for a typical beam in Frame A, would be enough, but to be within the drift limit, the beam size increased greatly, to W40x372. Because TCMC has an occupancy category of III, drift limit is set to be within 0.02h, rather than the typical 0.025h. This difference has a huge impact when determining member sizes. The final sizes for Frame A are shown in Figure 51.1. This can be compared to Frame A on the original design sizes, Figure 51.2. The members are clearly a lot larger in the new moment frame. With the unreasonably large size of the members, construction will be very difficult. However, if construction is possible, using moment frames will give the architect more architectural freedom.

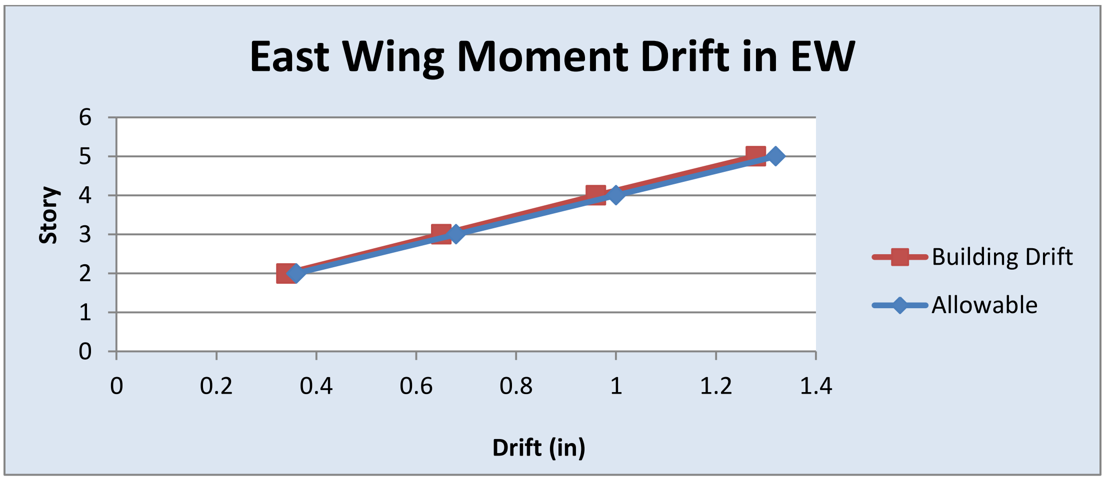
Once the ETABS model was deemed to be adequate for both strength and serviceability, the total weight and total cost of the building were calculated. This will be used to compare with the braced frame. The total weight of just the lateral frames was found to be approximately 1,220 kips. This makes the overall building weight to be approximately 19,290 kips. This is a 5% increase to the overall building weight.





Graph 50.1 Building and Allowable Drift for West Wing Moment Frame





Graph 50.2 Building and Allowable Drift for Eest Wing Moment Frame

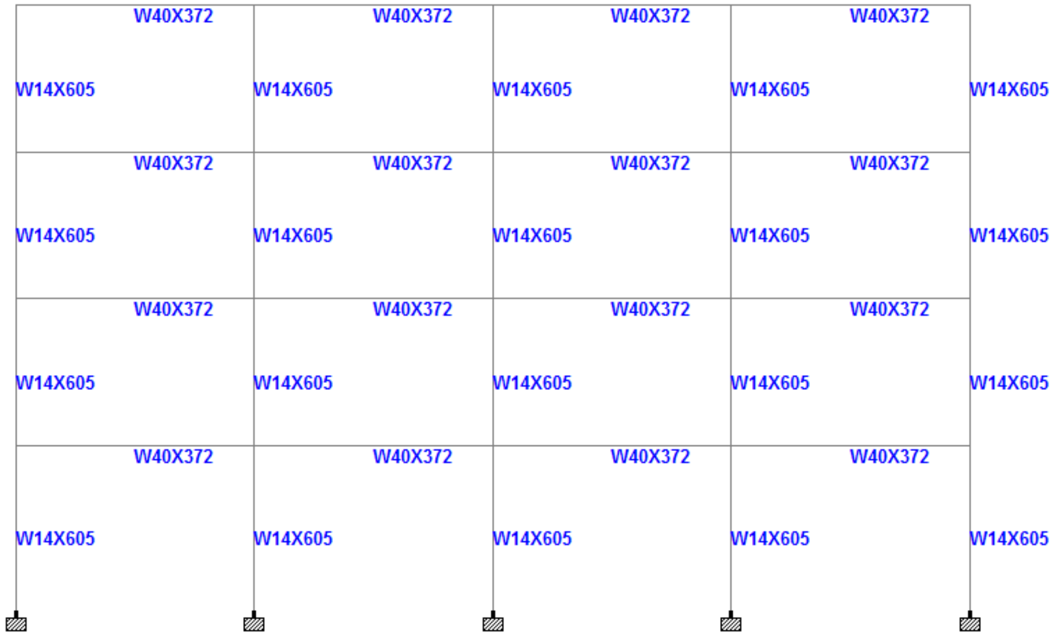


Figure 51.1 Final Member Sizes for Moment Frame A, in Miami FL

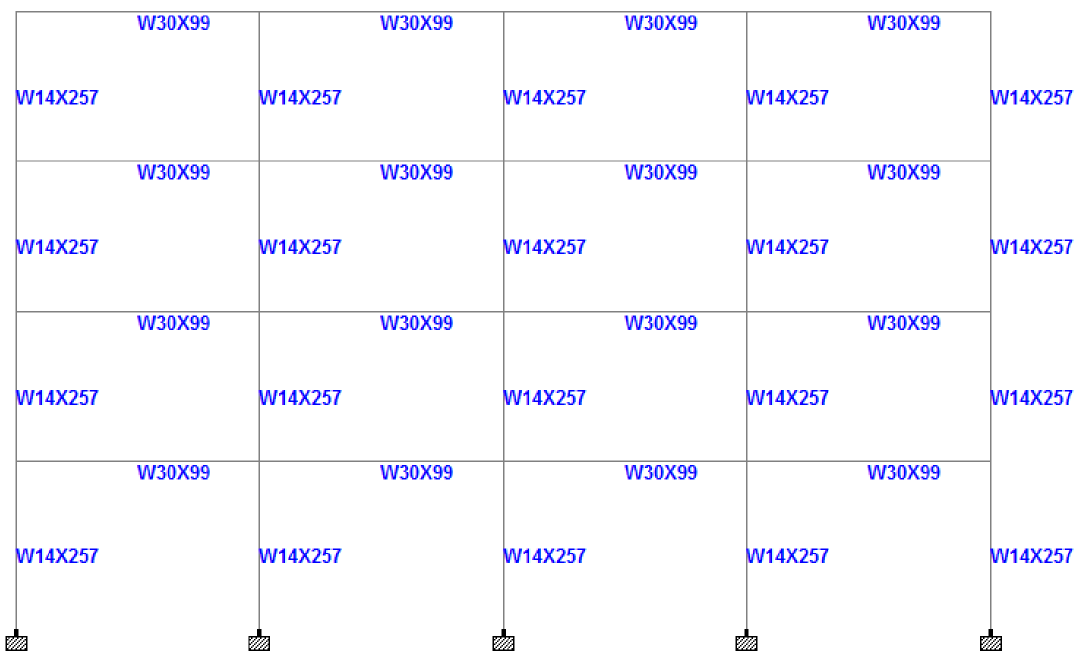


Figure 51.2 Sizes used in the Existing TCMC, in Scranton PA

Chevron Braced Frame Design

It is the author's interest to determine how much more effective is a braced frame system over a moment frame system. Therefore, for comparison, a second lateral system was designed as a chevron braced frame. This frame was chosen over other braced frame structures because the stiffness of the frame is the same in both directions that it is resisting and the frame is symmetrical in each bay providing a better architectural look.

Because braced frames are more effective compared to moment frames, the number of bays were cut down for majority of the frames. The layout of the braced frame is shown on Figures 52.1 and 52.2. Frame E and G are placed closer to the center of the building because the mechanical room on the third story will not allow bracing over the vent. Therefore, the frames were shifted inward. To keep the center of rigidity closer to the center of mass, Frame K and L were also shifted inward. Lastly, the frames in the Link were kept as moment frames (same as previous design), because the Link is mostly enclosed in glass. Also, because the beams and columns were still relatively small, bracing this structure may not be worth the cost when compared to architectural aesthetics.

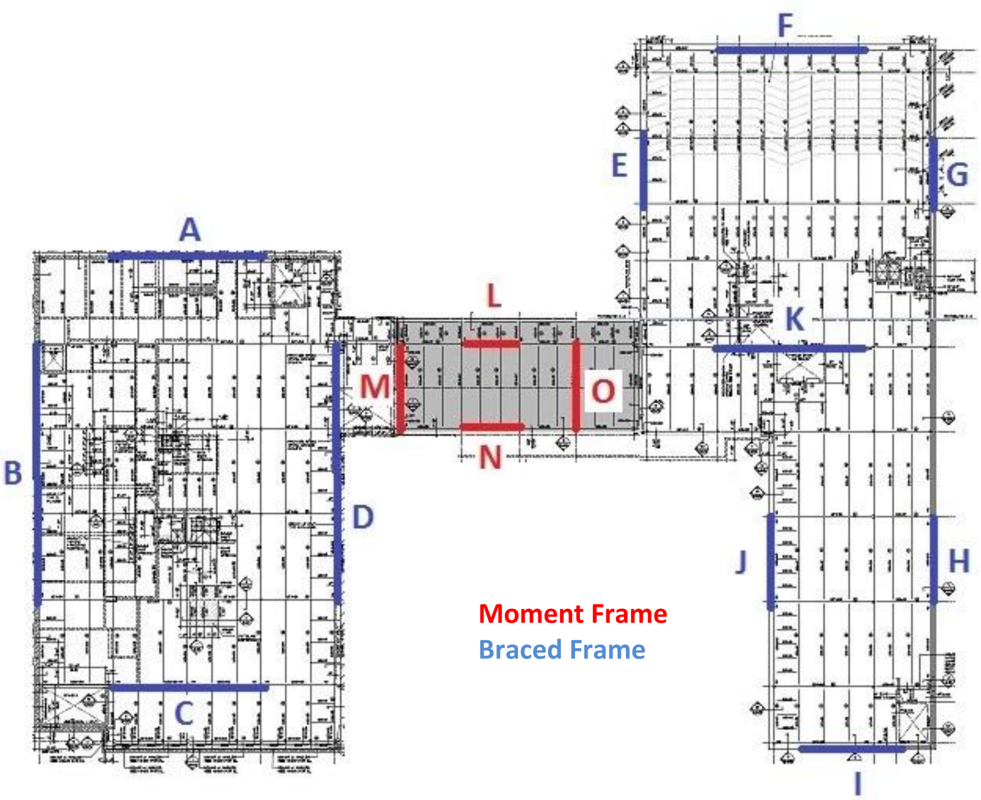


Figure 52.1 Braced Frame Layout

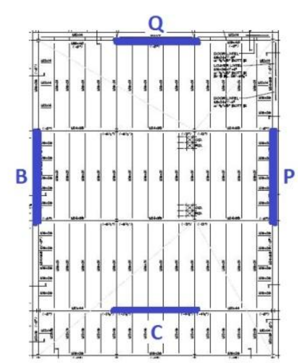


Figure 52.2 Braced Frame Layout in the Penthouse in the West Wing

Similar to moment frame design, the calculations on strength design, shown in Appendix E, resulted in preliminary sizes that were chosen for the members to be input into ETABS. Two models were designed, the West Wing and the East Wing, shown in Figures 53.1 and 53.2. As stated before, the Link was kept as moment frames, so it was not modeled again. The diaphragm was kept as rigid since it didn't change. Again, inputting the wind loads, dead loads, live loads, load combinations, and setting a drift limit of 0.02h into ETABS, the members were redesigned.

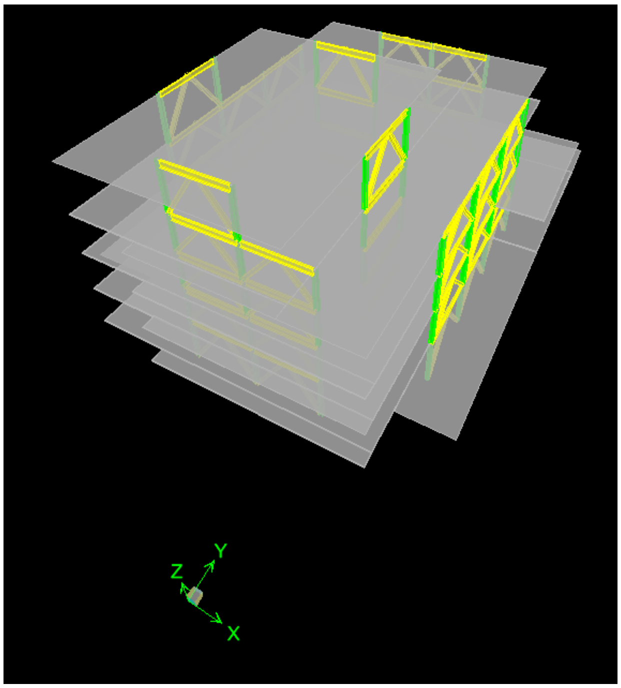


Figure 53.1 West Wing Braced Frame Model on ETABS

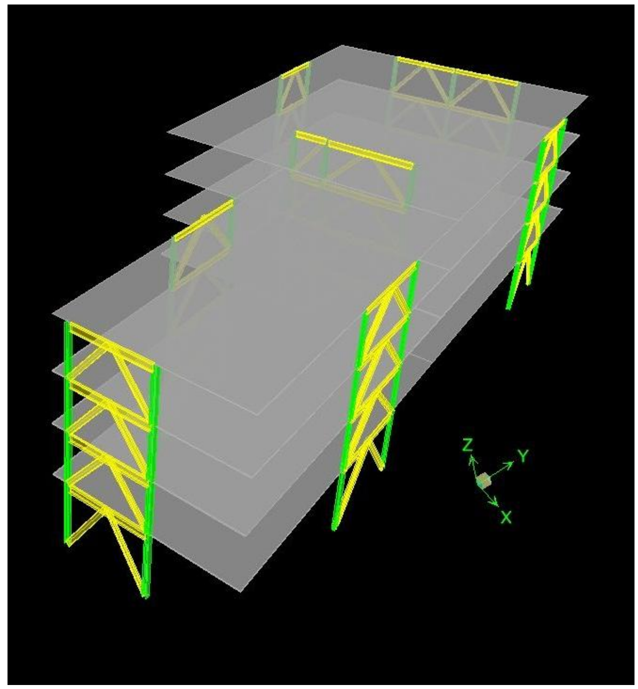
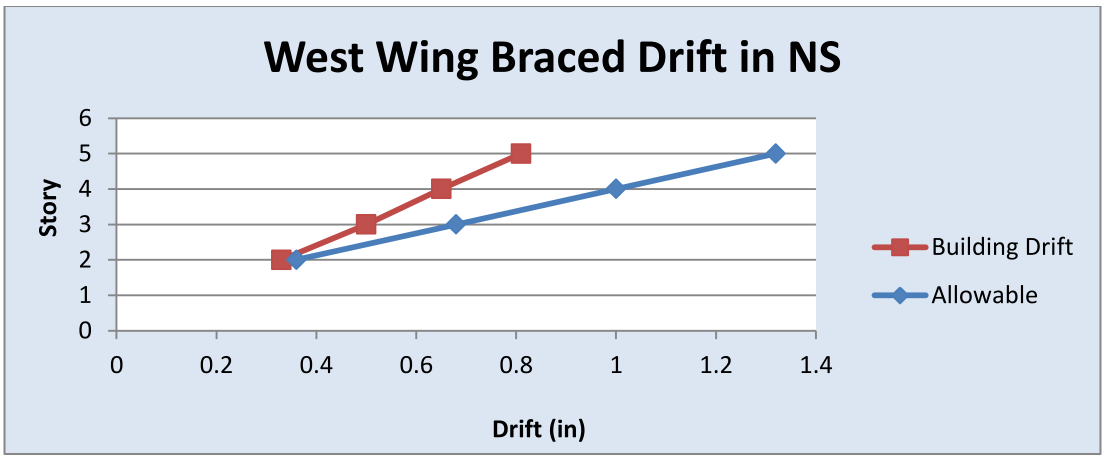
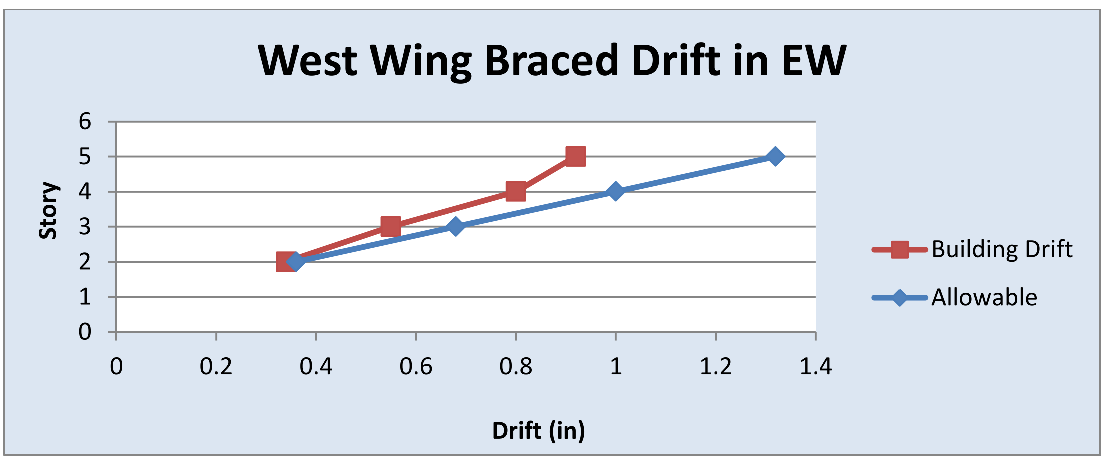


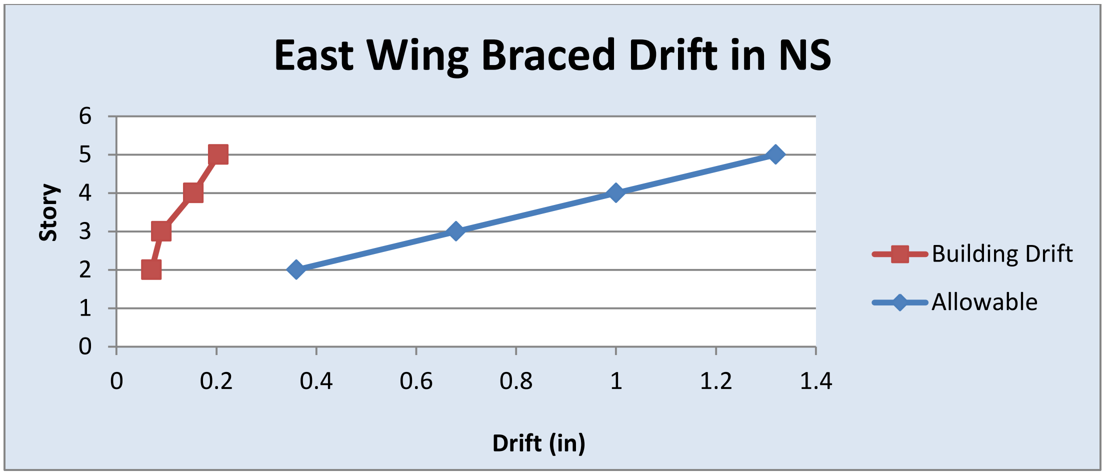
Figure 53.2 East Wing Braced Frame Model on ETABS

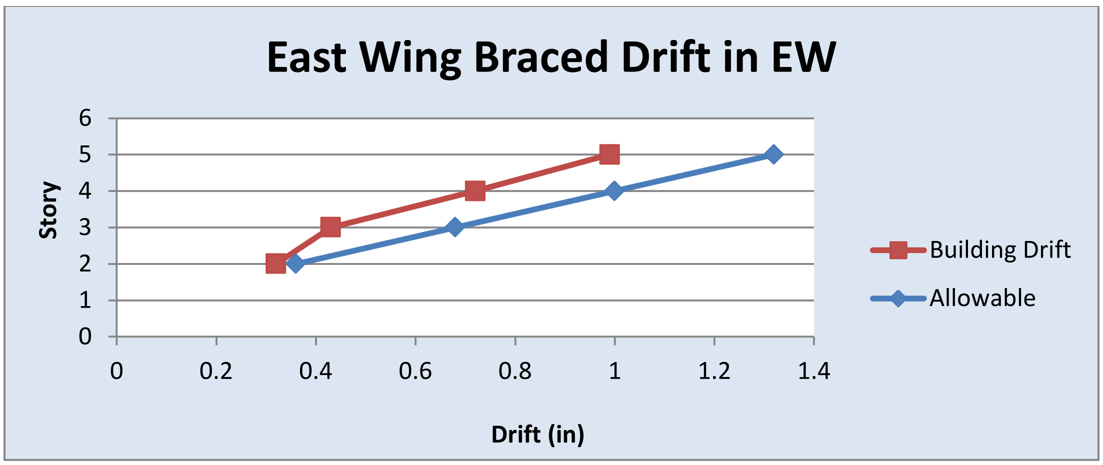
Majority of the braced frames were controlled by strength rather than by drift. Figures 54.1 and 54.2 show graphs of the building drift with the allowable drift. Notice, from the graphs, that the drift experienced by the building, in red, is not as close to the allowable drift, in blue, compared to the moment frame design. This shows that braced frames work a lot better when it comes to drift. The drift values were obtained from the ETABS model and the allowable drift limit is 0.02h, for a building with an occupancy category of III.





Graph 54.1 Building and Allowable Drift for West Wing Braced Frame





Graph 54.2 Building and Allowable Drift for East Wing Braced Frame

Figure 55.1 on the following page shows the axial force experienced by Frame A under the load combination of 0.9D + 1.6W. The red region is the member under compression and the blue region is the member in tension. This figure shows that it is possible for TCMC to experience tension on columns when there is minimum dead and live load in the building. It was found that TCMC may experience up to 239 kips of tension force in the outer column under the given load combination. This is important later on when we consider foundation design.

Figure 55.2 shows the axial force experienced by Frame A under the load combination of 1.2D + 1.6W +0.5L. This load controls the strength design for side columns. Figure 55.3 shows the axial force experienced by 1.2D + 1.6L. This controls the design for the center column in Frame A. Lastly, Figure 56 shows the final sizes for the members in Frame A.

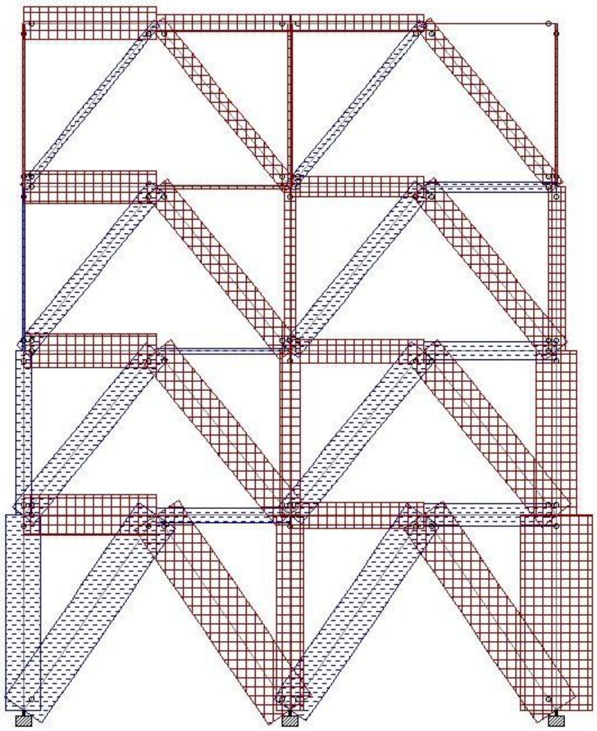


Figure 55.1 Frame A experiencing axial load from the load combination, 0.9D+1.6W. This load combo controls uplift design.

Compression
Tension

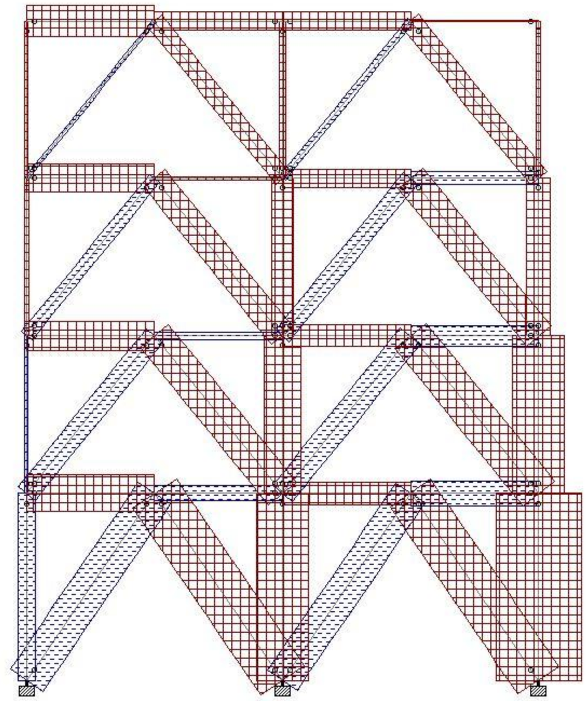


Figure 55.2 Frame A experiencing axial load from the load combination, 1.2D+1.6W+.5L. This load controls the strength design for the side columns

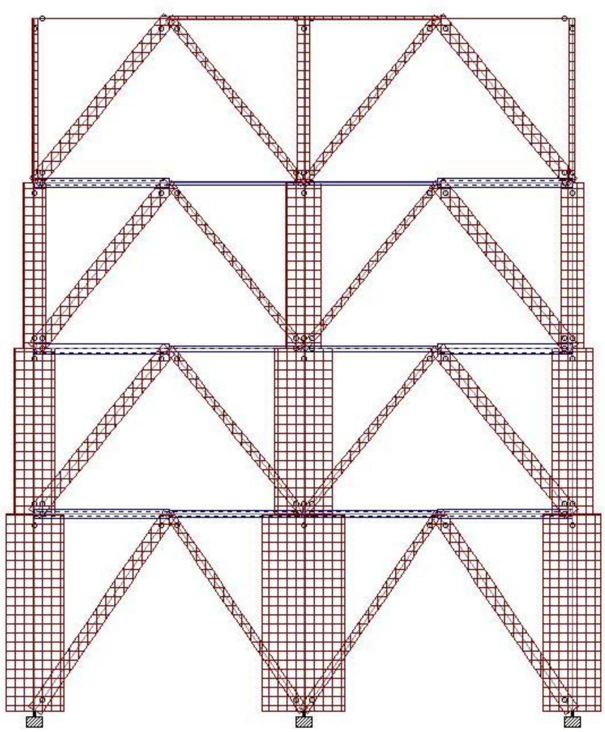


Figure 55.3 Frame A experiencing axial load from the load combination, 1.2D+1.6L. This load controls the strength design for the center column.

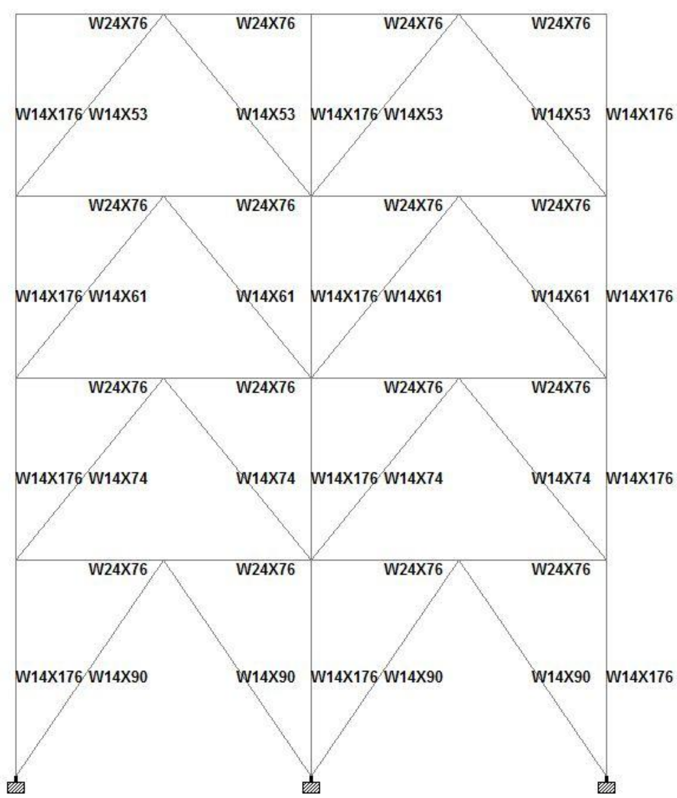


Figure 56 The final sizes for the members in Frame A

Once the ETABS model was deemed to be adequate for both strength and serviceability, the total weight and total cost for the braced frame structure were calculated. The total weight of just the lateral frames was found to be approximately 256 kips, a lot less weight compared to moment frames. However, the overall building weight still is approximately 1% larger than the original design, at 18,600 kips. This is because when the lateral frames were cut down in bay numbers, gravity beams and columns replaces them. This adds to the overall building weight, which caused it to be heavier than the original design even when all the members for this lateral system are smaller.

The draw back in braced frames for TCMC is the outer architecture. Changing the building from moment frames to brace frames has minimum effect on the floor plan, but it does have a little impact on the look of the exterior glazing. Also, windows will have to be carefully positioned to avoid bracing locations. TCMC has three façades with glazing that will be affected. The bracing however, will look symmetrical through the glazing so it won't hinder the architecture as much. With the overwhelming benefits of having braced frames over moment frames when it comes to strength, serviceability, and cost, the author recommends a braced frame design. The decision ultimately will to be decided by the owner or the architect.

Comparison

For ease of comparison, three tables were created to compare the original moment frame system, the new moment frame system, and the chevron braced system. Table 57 shows the typical member sizes between first and second floor on frame A. Notice how the members in the new moment frame design is a lot larger than the ones in the original member. The beams and columns in the braced frame are a lot smaller, even when the frame has fewer bays. This is all due to the effect of the bracing.

Typical Member Size between 1st and 2nd Floor on Frame A			
	Original	Moment	Braced
Beam in NS	W24x68	W36x256	W21x68
Beam in EW	W30x99	W40x372	W24x76
Column	W14x257	W14x605	W14x176
Bracing	N/A	N/A	W14x90

Table 57 Typical Member Size chosen for Frame A, 2nd floor

Table 58.1 gives the weight comparison between the three systems. As stated before, the weight of the new moment frame system is approximately 5% more than the original while the braced frame system is approximately 1% more. Table 58.2 gives the preliminary cost comparison between the three systems using data from RSMeans 2012. Moment frames turns out to cost over three times the amount of the original structure. Braced frame on the other hand, had only approximately 9% increase in cost (the gravity beams and columns that replaced the missing bays were included in this cost).

Weight Comparison			
	Original	Moment	Braced
Lateral Resisting Members	330 k	1220 k	256 k
Total Building Weight	18400 k	19290 k	18600 k
Percentage	100%	105%	101%

Table 58.1 Weight Comparison between the three structures

Cost Analysis For Frame A				
Original Design				
Member	Length (ft)	# of Members	Total Cost Per Linear Foot	Total
W14x257*	18	5	$ 411.20	$ 37,008.00
W14x257*	16	15	$ 411.20	$ 98,688.00
W24x76	26	16	$ 121.60	$ 50,585.60
			Total	$ 186,281.60
Moment Frame Design				
W14x605*	18	5	$ 968.00	$ 87,120.00
W14x605*	16	15	$ 968.00	$ 232,320.00
W40x372*	26	16	$ 595.20	$ 247,603.20
			Total	$ 567,043.20
Braced Frame Design				
W14x176*	18	3	$ 281.60	$ 15,206.40
W14x176*	16	9	$ 281.60	$ 40,550.40
W24x76	26	16	$ 121.60	$ 50,585.60
W14x61	30	4	$ 97.60	$ 11,712.00
W14x74	30	4	$ 118.40	$ 14,208.00
W14x90	30	4	$ 144.00	$ 17,280.00
W14*x109	30	4	$ 174.40	$ 20,928.00
W12x152	18	2	243.2	8755.2
W12x152	16	6	243.2	23347.2
			Total	$ 202,572.80
		Original	Moment	Braced
% Increase for Redesigned Structure		100%	304%	109%

Table 58.2 Cost Analysis comparing the systems. Source: RSMeans 2012. *The Cost of these members were interpolated from data

Foundation Design

As explained in the earlier section, a geotechnical report was never found for a typical urban site in Miami, FL. The closest site that a report was found for was an urban site in Orlando, FL. This gives a sense on how the site in Miami could be like but is not accurate. Therefore, assumptions were made for this site. It is assume that the site has at least a bearing capacity of 2500psf (the original site has a bearing capacity of 3000psf).

Because of the high load the building faces, and with a low bearing capacity on the site, a mat-slab foundation was chosen for the entire building because mat foundations are preferred when soil have low bearing capacity. This foundation distributes heavy column and wall loads across the entire building area to lower the contact pressure, which is what we wanted for TCMC. Through a preliminary calculation with a soil bearing capacity at 2500psf, the depth of the mat-slab needed was found, along with the factor of safety, F.S. Table 59.1 shows these variables. Both designs required a thicker mat-slab, which is correct because of the higher load. The mat-slab for the braced frame system requires the largest thickness because its critical section, the outer columns of the frames, experienced the largest load. The table also shows that strength design is what controlled this design because it has the lowest factor of safety. Lastly, due to privacy issues, a geotechnical report was never obtained for the original site of TCMC, so most values are unknown, as noted in the table.

The author's ability is limited in designing an accurate mat-slab foundation. This is because a full analysis is very complex. Therefore, only a preliminary calculation was done, and is shown in Appendix F. Calculation for reinforcing was never done because of its complexity.

Foundation Summary			
	Original	Moment	Braced
F.S. Bearing	N/A	2.8	2.8
F.S. Uplift	N/A	Not an issue	4.4
F.S. Strength	N/A	2.5	2.5
Depth into Earth	8'-8"	10'	11'-6"
Thickness of MAT	4'	6'	7'-6"

Table 59.1 Summary of the Foundations

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 59.2 below shows the overturning moment that the lateral forces had cause. For the West wing, the controlling moment, from wind in the East-West direction, is 35786 k-ft. However, the West wing's resisting moment for the East-West direction was found to be 839,542k-ft, a lot larger. For the East wing, the controlling moment, also from wind in the East-West direction, was found to be 47,213-ft. The resisting moment here is 439,705 k-ft, over ten times greater. Foundations are designed with a high safety factor because the whole building depends on it to work properly.

West Wing 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	15	1395	74	6882	96.6	8983.8
Mainroof	72	49.2	3542.4	133	9576	173.6	12499.2
4th	52	37.5	1950	111	5772	144.5	7514
3th	36	23.5	846	90.5	3258	117.8	4240.8
2nd	20	10.8	216	98	1960	127.4	2548
Overturning Moment		Sum=	7949	Sum=	27448	Sum=	35786
Resisting Moment =			839542		1286395		839542

East Wing 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	13.8	1283.4	39.1	3636.3	127.5	11857.5
Mainroof	72	45.3	3261.6	70.3	5061.6	229.1	16495.2
4th	52	35.0	1820	58.5	3042	190.6	9911.2
3th	36	21.9	788.4	47.7	1717.2	155.3	5590.8
2nd	20	10.1	202	51.6	1032	167.9	3358
Overturning Moment		Sum=	7355	Sum=	14489	Sum=	47213
Resisting Moment =			439705		1532686		439705

Table 59.2 Show the overturning moment caused by seismic lateral force and the resisting moment of TCMC.

MAE Material Incorporation

Information gained from three classes helped the author fulfill the MAE requirement part of this thesis. They are, AE 530-Computer Modeling, AE 534-Steel Connection, and AE 542-Building Enclosures. As shown in the previous sections, ETABS and STAAD models were designed using the knowledge from AE 530. The knowledge from AE 534 allows the author to design a typical welded braced connection, which will be shown more below. And lastly, the knowledge from AE 542 helped the author in façade design, which is part of breadth 3.

A typical braced connection on the 1st floor was designed. It was designed as a pinned, welded connection; the brace is connected to the beam and column by two WT 7x24, shown in Figure 60. This leaves 2” of clear space left for welding. The effective length, ℓ_d, (shown in the figure) is 10” for enough strength in the weld. The welding required is at least a $^{9}/_{16}$” fillet weld, on all three sides of contact. Full calculation for this connection is shown in Appendix G. Overall, the connection is controlled by tension yielding of the WT, which has a maximum allowable axial load of 636 kip on the brace. This brace experiences a 519 kip axial load so this connection works.

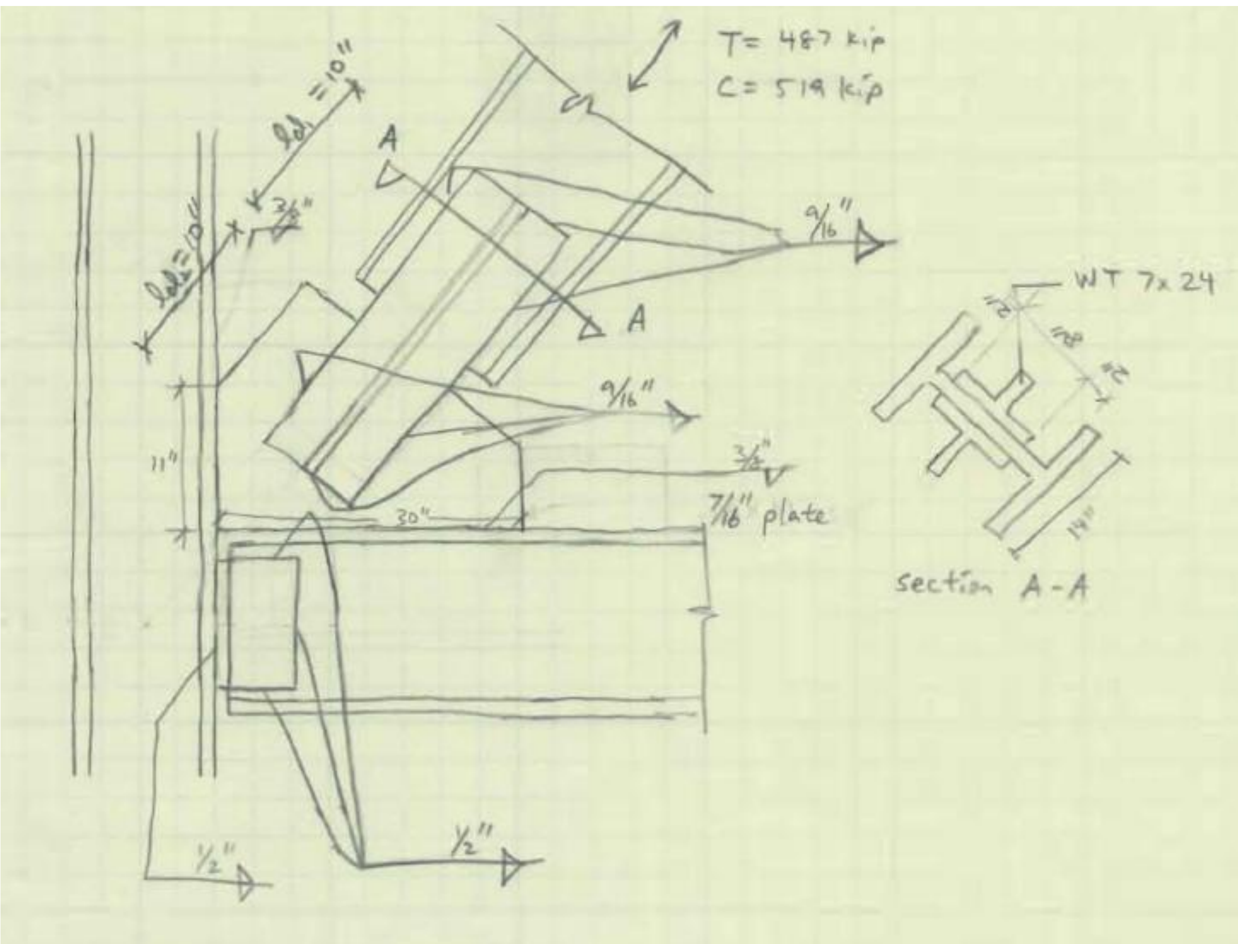


Figure 60 Pin Welded Connection Designed for the Brace on the Second Floor, Frame A. This connection takes up to 636 kip of axial load.

Breadth One: Façade Design

The purpose of this breadth was to investigate how the new setting of the building will affect the façade of TCMC. As mentioned before, the climate is very different in Miami, Florida, compared to Scranton, Pennsylvania. During a hurricane, not only that there is large wind pressures, but there will be impact from debris also. The new façade is designed to resist these impacts. Heat transfer and waterproofing were also kept in mind when designing this new façade. Heat loss or heat gain through a façade is very important to a building. The more heat that can be transfer through a wall, the more energy is required to bring the building to optimum condition. This will leave to huge energy loss and cost for the building owner. When designing the new façade considering impacts, heat transfer, and waterproofing, a new glazing type and a new exterior wall material were used.

Façade Type

When determining a probable façade for this situation, many systems were first researched. It was determined that a rainscreen cladding system that uses individual wall-cladding panels will be a very good choice. A rainscreen cladding acts as a ventilated outer skin that is attached to the exterior wall. This system has two features that made it desirable to be used in Miami. The primary feature is that water from rain can escape through the rainscreen cladding easily so the wall will not be damaged from excess rainfall. The second feature is that the rainscreen cladding can dissipate heat from the sun so the building would remain cool. This acts as an extra insulation.

The next challenge is to find a manufacturer that produces a reliable rainscreen cladding suitable for the environment of Miami. After researching several companies, Boston Valley Terra Cotta seems to be one of the best companies that produce a rainscreen cladding system. Boston Valley Terra Cotta's manufactured a rainscreen system known as TerraClad Rain Screen, shown in Figure 61. The following are some benefits in having this system installed on TCMC;

- It is one of the few rainscreen producers to be manufactured in North America.
- very simple to install, leading to less time during construction
- shields the building from wind driven rain
- acts a sunshade to keep the building cool during summer
- have LEED credit opportunities
- many different colors and sizes to choose from

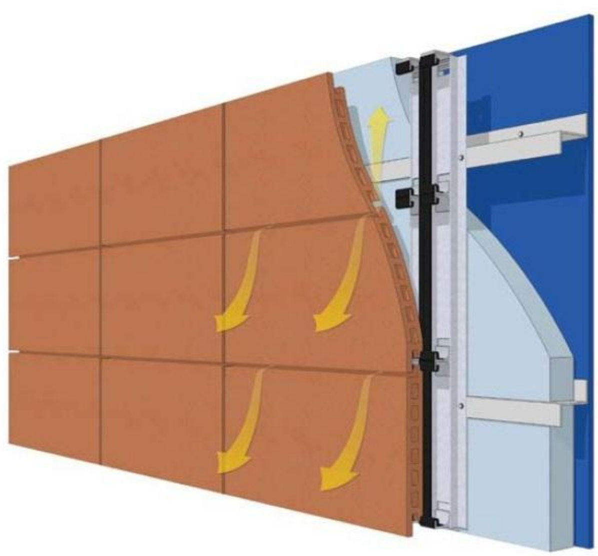
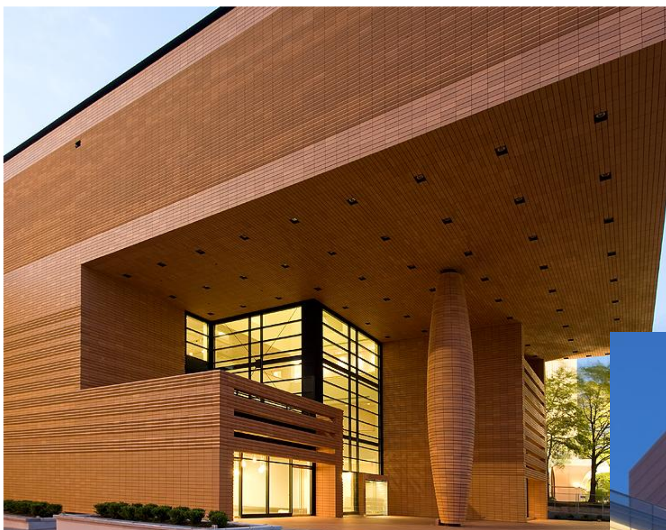


Figure 61, Taken directly from Boston Valley Terra Cotta's website. This shows a typical TerraClad Rain Screen.

Figure 62 and 63 gives a sense of how TCMC's facade would look like when this system is installed. From a distance, this system will look very similar to the existing façade. Because the rainscreen cladding is going to be installed to a building in Miami, FL, making sure that the TerraClad Rain Screen can be used there is the most important factor. Boston Valley Terra Cotta already confirmed that their TerraClad Rain Screen met Florida Building Code. Figure 64 shows the Florida Building Code that had been met by this product. It is also tested for high velocity hurricane zone. Lastly, large missile impacts were also tested and have met code. This makes the TerraClad Rain Screen a very favorable system to be used on TCMC in Miami, FL.



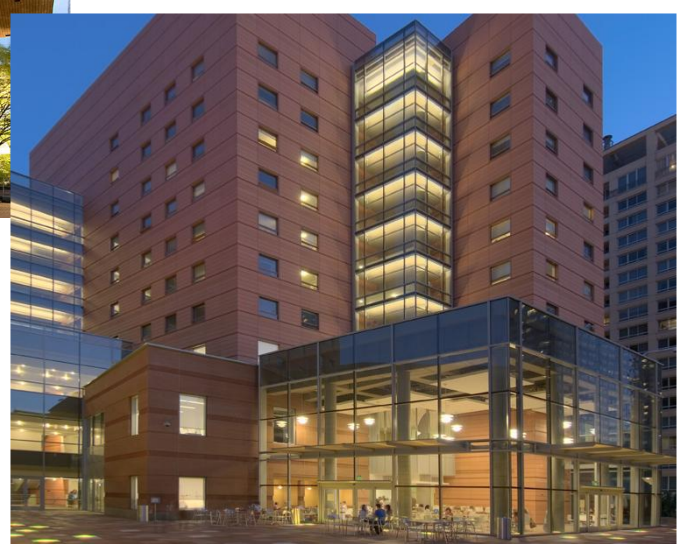


Figure 62, Top, Taken directly from Boston Valley Terra Cotta's website, is The Bechtler Museum of Modern Arts. The rainscreen cladding here are the same size of a normal brick. This shows that many sizes are possible when using TerraClad Rain Screen. This building also shows the color of stones similar to ones that are used in TCMC. The size and color can keep the architectural look similar to the original.

Figure 63, Right, is The Colburn School of Performing Arts. This building gives a sense of how TCMC would look like. It shows how the glazing will look with the TerraClad Rain Screen.

Florida Building Code - High Velocity Hurricane Zone Testing, Miami-Dade County NOA08-1014.03

TAS 201-94

Impact Test Procedures - Large Missile Impact

TAS 202-94

Criteria for Testing Products Subject to Cyclic Wind Pressure Loading

TAS 203-94

Criteria for Testing Impact & Non Impact Resistant Building Envelope Components Using Uniform Static Air Pressure

Figure 64 Taken directly from Boston Valley Terra Cotta's website on the testing and code requirements that are met for their product, TerraClad Rain Screen. This shows that their rainscreen are workable in Miami , Florida. It can resist damage from hurricanes and also large missile impacts.

The performance on the wall is later checked for its efficiency in heat transfer and to make sure condensation does not occur within the wall. Using H.A.M., it was determined that the R-value of the typical wall of TCMC is 23.44. This is very efficient for a wall when it comes to insulation. Again, using H.A.M., this wall also shows no sign of condensation issues in winter and summer when a vapor barrier is placed within the wall. This full analysis is shown in Appendix H.

Window Design

Glazing is the only part of the exterior wall that still needs to be looked at. Because of larger wind loads, all glazing in TCMC was redesigned to meet the new load requirements from wind pressure, and debris impact. There are mainly two different window sizes. Smaller windows are typically 2'x4' and larger windows are typically 6'x10'. Using a simplified window design calculation, from Minor and Norville, it was determined that the smaller windows need to be $^{3}/_{16}$" thick and the larger windows need to be $^{5}/_{8}$" thick in order to resist up to 60 psf.

As for impact on windows, a sacrificial ply design will be implemented. This requires the windows to be designed as laminated glass units, LGUs. LGUs are two lites of glass having a protective vinyl later of material between them. LGUs are recommended when it comes to safety (from shattered glass), sound reduction, and impact resistance. When including the design of a sacrificial ply into the LGU, it makes the window performed even better in a hurricane prone region. The concept of a sacrificial ply is to allow the outer ply to fracture on a debris impact. The inner ply is prevented from breaking. The glass fragments on the outer ply remain bonded to the protective vinyl layer; therefore, safety is not an issue. This ply can be any size, but for this case, TCMC will use an outer ply of $^{1}/_{8}$". The inner ply will be the only one designed to resist wind pressure. Table 65 below outlines the window design summary. Figure 66 shows the concept of a sacrificial ply.

Typical Window Design				
	Width	Height	Outer Ply Thickness	Inner Ply Thickness
2'x4'	2'	4'	1/8"	3/16"
6'x10'	6'	10'	1/8"	5/8"

Table 65 Final Window Design Values

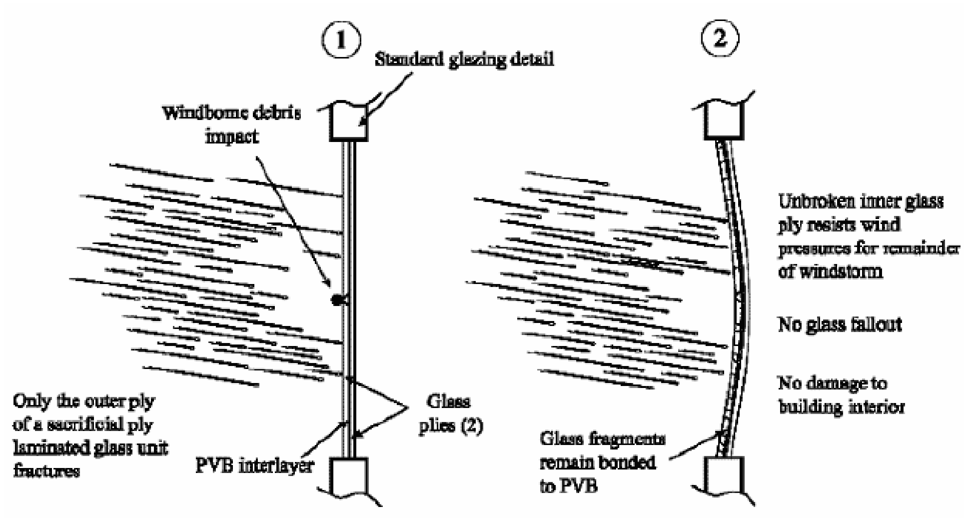


Figure 66, "Sacrificial Ply Concept" Founded by Nathan Kaiser, Richard Behr, Joseph Minor, Lokeswarappa Dharani, Fangsheng Ji, and Paul Kremer. Image from AE542 Class Notes.

Heat Transfer

Heat loss and heat gain through the façade system are very important when it comes to building design. The more heat that is allowed to go through the wall, the costlier it is because of the extra energy required to recondition the interior environment back to comfortable level. After the façade and glazing has been designed, a heat transfer analysis was done on the first floor of West Wing to calculate how much heat can transfer through the walls and windows. It was found that the average temperature in Miami is 91^{o}F during summer and is 46^{o}F during winter. Having the interior of the building maintained at 70^{o}F at all times, will made the temperature difference to be around 24^{o}F. Along with knowing the R value of the walls and windows, the amount of heat transferred can be calculated. From H.A.M, the R value of the wall was found to be 23.44, and a typical LGU will have an R value of 3.0. Table 67 shows the calculation of heat loss during winter or heat gained during summer, for the first floor, West Wing. Since the temperature difference is relatively close, the answer will be close; therefore only one table was produced. The table only shows heat gained or heat loss through the walls and windows, but in reality, many factors need to be considered also, such as heat gained from equipment or latent heat gained from people. Based on the calculations, the HVAC will have to accommodate an extra 24,000 Btu/hr of cooling during summer or of heating during winter on the first floor of the West Wing due to the transfer of heat through the walls.

Heat Loss or Gained on First Floor West Wing		
Area of Wall =	7900	ft^2
Area of Glass =	1980	ft^2
R of Wall =	23.44	
R of Glazing =	3	
Temperture Difference =	24	^{o}F
	Wall	Glazing
Sensible Heat Loss	8089	15840
Latent Heat Loss	Neglected due to Vapor Barrier	
Total	23929	Btu/hr

Table 67 Approximate Heat Loss or Gain in 1st Floor, West Wing

Breadth 2: Solar Panel Design

When designing solar panels for TCMC, the intent was never to remove the building off the electric grid because it is impractical due to the huge consumption of electricity. Therefore, TCMC's electrical connection will be change from simply electrical grid connection to a grid-tied connection. This will be explained more in depth in Breath 3.

The location and placement of the solar panels is very important when it comes to solar design. Since the building is in Florida, there will be plenty of sunlight so that a solar panel system will be a feasible investment over time. After a conducted solar shading study, the best placement for the panels is the area on the flat roof. Figure 68 shows the different solar angle in Miami, FL, during summer and winter.

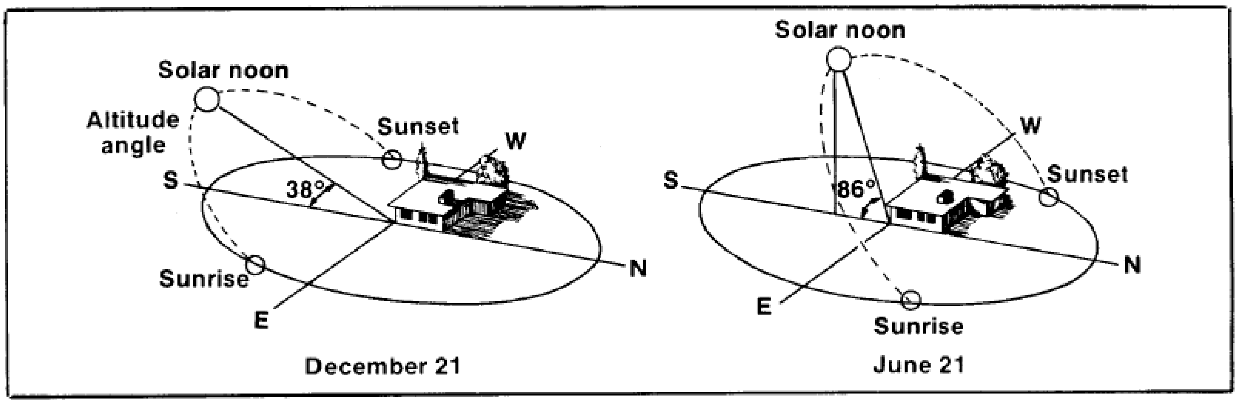


Figure 68, Solar Angle in Miami, FL. Image from Florida Solar Energy Center

The solar panels chosen for this design were HIT Power 220A Photovoltaic Module, made by Panasonic, Figure 69.1. This was chosen because of its great quality, ease of placement (apply to the mounting member using nuts, bolts, and metal clamp, see Figure 69.3), and one of the top energy producers. The panels have an efficiency of 19.8%, and its "hybrid cell produces the highest output on cloudy days," as mentioned on its data sheet. In Miami, around 20% of the year is cloudy due to the rain season. Therefore, being able to produce the most energy during cloudy days separates this panel model from others. More importantly, it can withstand wind pressure of 60psf. Under the new wind load, TCMC, experience a maximum wind uplift pressure of 59.3psf. The HIT Power 220A is one of the few models that can withstand pressures up to 60psf.

The inverter chosen was the SMA Sunny Boy 3800, Figure 69.2. An inverter is an electrical device that converts direct current (DC), produced from the solar panels, to alternating current (AC), used in a building. The manufacturer, SMA, was chosen because the company is known as the current market leader for innovative solar inverters, for their product quality and efficiency. This product has a product warranty of 5 years for any defects. This inverter also has a build in OptiCool temperature management system that ensures it stays cool. This is one reason why Sunny Boy 3800 is very efficient and keeping it cool also increases the life of the inverter. The inverter and the solar panels data sheet can be found in Appendix I.

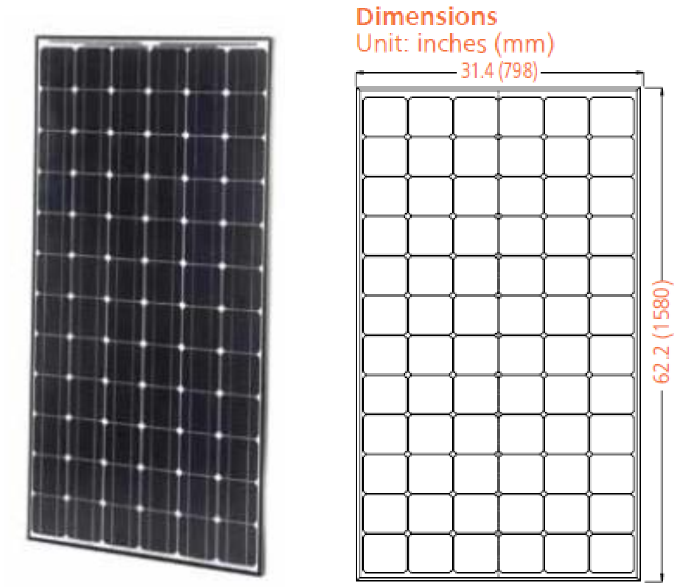


Figure 69.1, HIT Power 220A Photovoltaic Module. Image from Panasonic Sanyo HIT Technology

Figure 69.2, SMA Sunny Boy 3800 inverter. Image from The Solar Electricity Company.

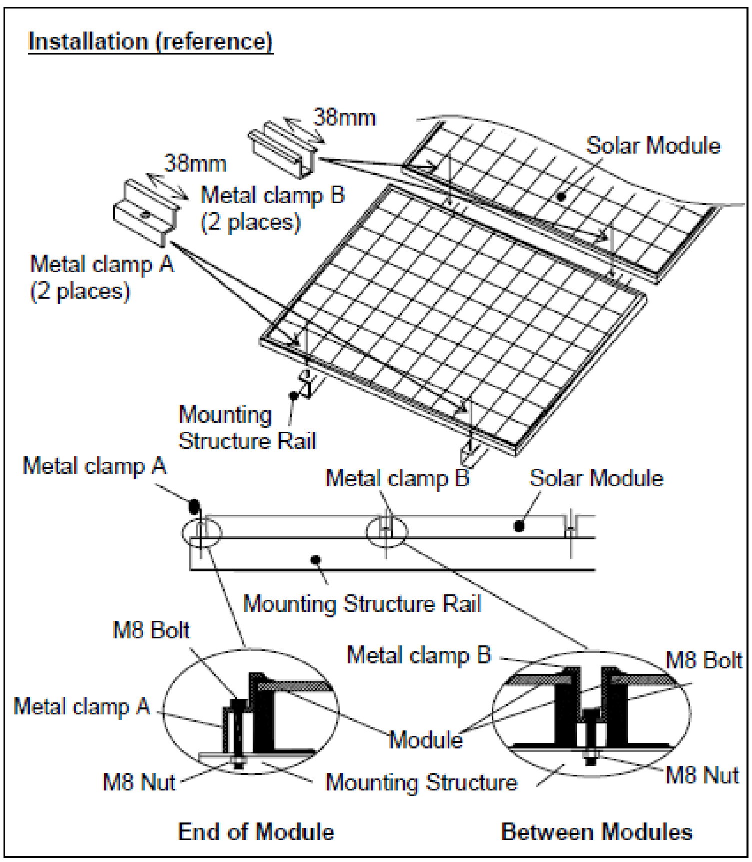


Figure 69.3, HIT Power 220A installation reference. Image from Panasonic Sanyo HIT Technology

Due to problems with shading, only the main part of the roof is the best place for placing the solar panels. Figure 70.1 shows the area, in blue, where the panels were placed. This has around 6500 ft^2 of available space. Figure 70.2 shows how TCMC roof will look like with the installed solar panels.

Figure 70.1, Area in blue is where the solar panels were placed. Image from google maps, edited by the author

Figure 70.2, Image of installed HIT 220A solar panels. This is how TCMC roof, where solar panels were placed, will look. Image from Panasonic Sanyo HIT Technology

Final calculations were performed to see how much will the system cost and how many kilowatt-hour the system will produce. Initial cost of the system was estimated to be around $336,500, which includes 430 solar panels and installation. This information is used to determine the life cycle cost of the entire system, for 20 years, as shown in Table 71. Since an exact cost cannot be obtained, all costs were estimated using data from the author's research from cost of similar products. As for the tax incentive, the government currently pays for 30% of the cost of the installed system. This incentive will end in 2016. The resulting life cycle cost for this system over 20 years will be around $279,000.

Estimated Life-Cycle Cost - Solar Panel System for 20 Years				
Cost Description	Cost	# of Years	Present Value Factor	Present Value
Initial Cost	$ 336,500	1	1	$ 336,500
Inspections	$ 1,000	20	0.91	$ 18,200
Repair & Replacements	$ 10,000	5	0.67	$ 33,500
Salvage	$ (25,000)	1	0.48	$ (12,000)
Tax Incentives	$ (100,950)	1	0.962	$ (97,114)
			Total =	$ 279,086

Table 71 Estimated Life-Cycle Cost of the Solar Panel System

The estimated payback period was also determined. Using the current average electric cost of $0.10, for commercial consumers, and with an inflation of 3% per year, the payback period was determined to be 27 years. The owner will save over $10,000 per year in electric cost. This calculation can be seen in Table 72 below.

Estimated Payback Period - Solar Panel System	
Total Power of System (kW)	86
Total Power (kWh) per year	100448
Cost of Current Power	$ 0.10
Total Savings per Year	$ 10,045
Payback Period (years)*	26.97
*calculated with an electric inflation cost of 3% per year	

Table 72 Estimated Payback Period

Breadth 3: Mechanical and Electrical Changes

Mechanical Changes

The mechanical system of TCMC needs to be redone due to the new location. This breadth concentrates on what new mechanical system should be used. Because this is a breadth topic, a full mechanical analysis on the building was not carried out. Before moving any further, a basic understanding of the climate in Miami was needed. Miami has a very high humidity, with an average temperature of 70^{o}F to 77^{o}F. Knowing this, the systems needed to make the environment comfortable for the occupants were looked up. The original TCMC in Scranton, PA, has four McQuay chillers for cooling and three steam boilers for heating. Now since it is in Miami, FL, this changed. Only one steam boiler per wing was needed due to the warmer climate. The number of chillers remained the same because Miami is not really high in temperature.

The main problem was the humidity. Because of this, a more powerful dehumidifier was installed. The system that was chosen to handle this is the RLNL-G dehumidifier produced by Rheem. This system can deliver dry neutral air when humidity is high. The following is a list of its benefits,

- Money-Saving Efficiency
- Quiet Operation
- ClearControl- remote monitoring and control
- Quality- Rheem claim that it will last longer than its competitors

Having this system installed, it will take care of the humidity and latent heat which are the main problems in Miami.

Electrical Changes

As explained in Breath 2, when designing solar panels, the intent was never to remove TCMC off the electric grid because it is impractical due to the huge consumption of electricity. Therefore, TCMC's electrical connection will be changed from simply electrical grid connection to a grid-tied connection. This type of connection is where the energy created from the solar panels will be transferred to the building, while the building is still connected to the electric grid. The advantage of grid-tied systems is the net metering, where the electric meter, from the electric company, runs forward when the power is purchased, and runs backward when the power is returned. The customer, in this case, TCMC, only needs to pay for the "net" use of electric. Figure 73 shows a typical grid-tied system, showing how the solar panels, inverter, and the electric meter are connected.

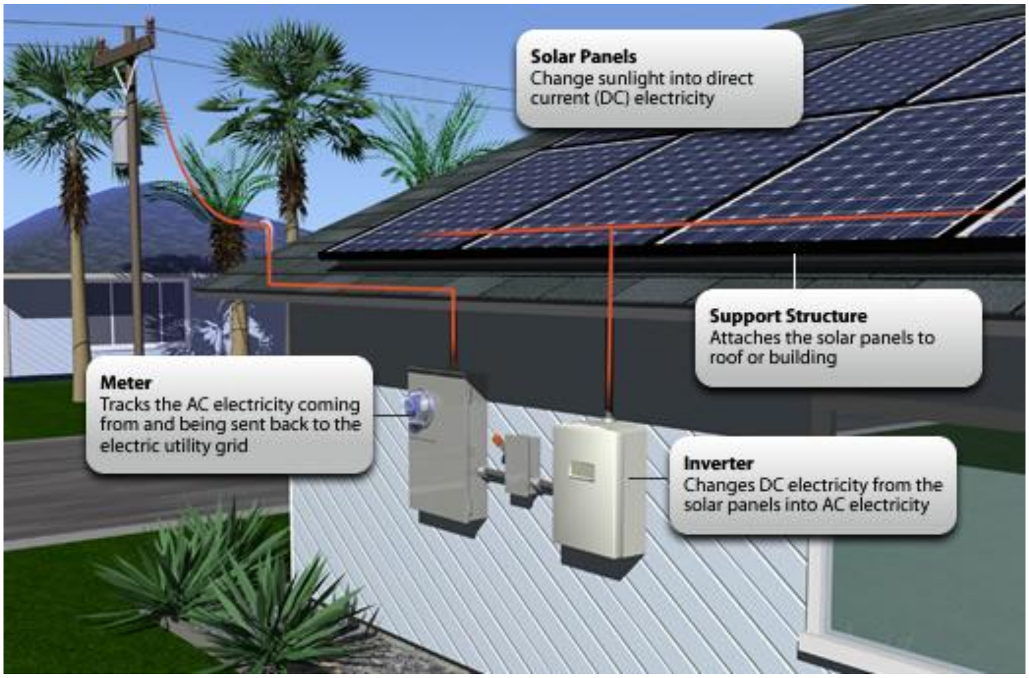


Figure 73, Image showing a typical grid-tied solar panel electrical connection

Lastly, we will look at the back up emergency system for the new TCMC. Because it is more likely for TCMC to lose power in Miami due to hurricane storms, a new backup system is preferred. The system that was chosen for TCMC was the Diesel Engine Generator 2800KW, from Kentech. Kentech was chosen because they provide quality commercial/industrial generators. With over 25 years of experience, they supply emergency backup systems to many fields, including schools and hospitals. This gives Kentech the credibility to work on TCMC. With this system, TCMC will be prepared for any power outages.

Conclusion

Because the existing TCMC was so well designed to meet all code requirements, nothing can be done to improve the building under the current scenario. Therefore, the new scenario was created in which The Commonwealth Medical College was proposed to be built on a typical urban site in Miami, FL. Two new structures were designed to be adequate for both strength and serviceability at this new site.

The two new redesigns were steel moment frame and chevron braced frame. Having steel moment frames increase the current building weight by approximately 5%, compared to 1% by braces frames. It was determined that braced frames is a lot more efficient than moment frames in terms of strength, serviceability, drift, and cost. More importantly, the sizes of the moment frames came out to be unreasonably large, which is extremely difficult for construction. However, using moment frames will give the architect more architectural freedom. The author recommends using the chevron braced frame system because it is very efficient and easy to construct compared to the moment frames. Braced frame members are very small relatively, which is the main reason why it is approximately four times cheaper over moment frames. But ultimately, the decision between either moment frames or braced frames will be decided by the owner or the architect.

In addition to the lateral system redesigns, three breadths were considered. The first breadth was on façade design. A rainscreen cladding system, TerraClad Rain Screen, made by Boston Valley Terra Cotta, was chosen for the new outer façade of TCMC because of its advantages in the new site. As for glazing, laminated glass units designed as a sacrificial ply will be used to handle debris loading.

The second breath was on solar panel design. It was easy to see the great opportunities for solar energy in Florida, so a new photovoltaic system was designed. The model of the panels chosen was the HIT Power 220A, made by Panasonic. This model has the highest output of energy on cloudy days. The inverter was chosen to be SMA Sunny Boy 3800 because this was recommended by Panasonic for this model and it is built to cool itself, which increases its lifespan. The solar panels would save the owner approximately $10,000 per year and the whole system will have a payback period of approximately 27 years.

The last breath was on small mechanical and electrical modifications. The number of steam boilers was cut down because it wasn't needed anymore. Most importantly, a more powerful dehumidifier was added because Miami is very humid compared to Scranton. The model chosen for the dehumidifier was the RLNL-G dehumidifier, made by Rheem. The only main electrical change was from a simply electrical gird connection to a gird-tied connection. This allows TCMC to use the energy from the solar panels first and when needed, energy from the electrical supplier.

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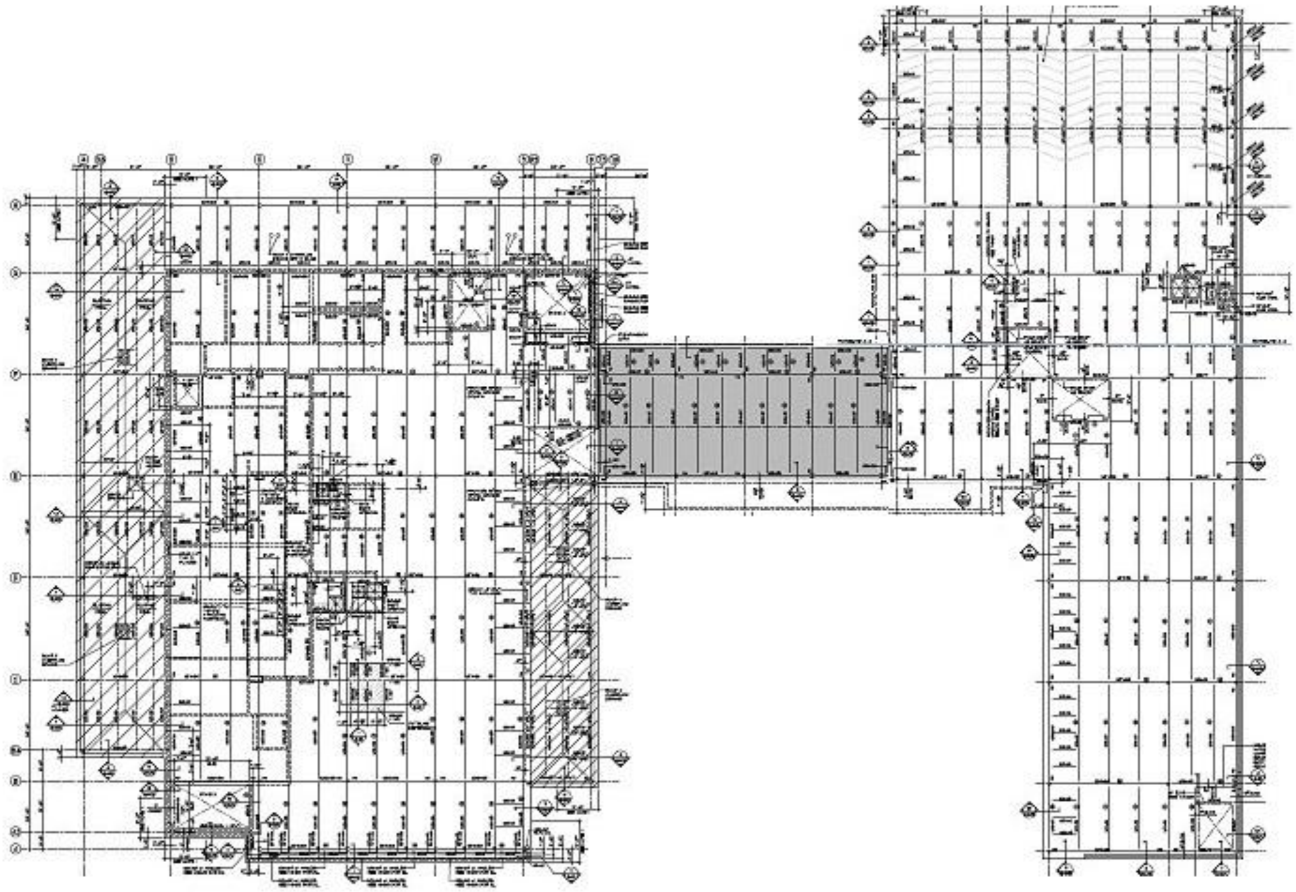
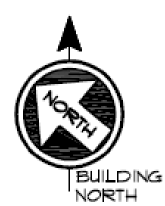
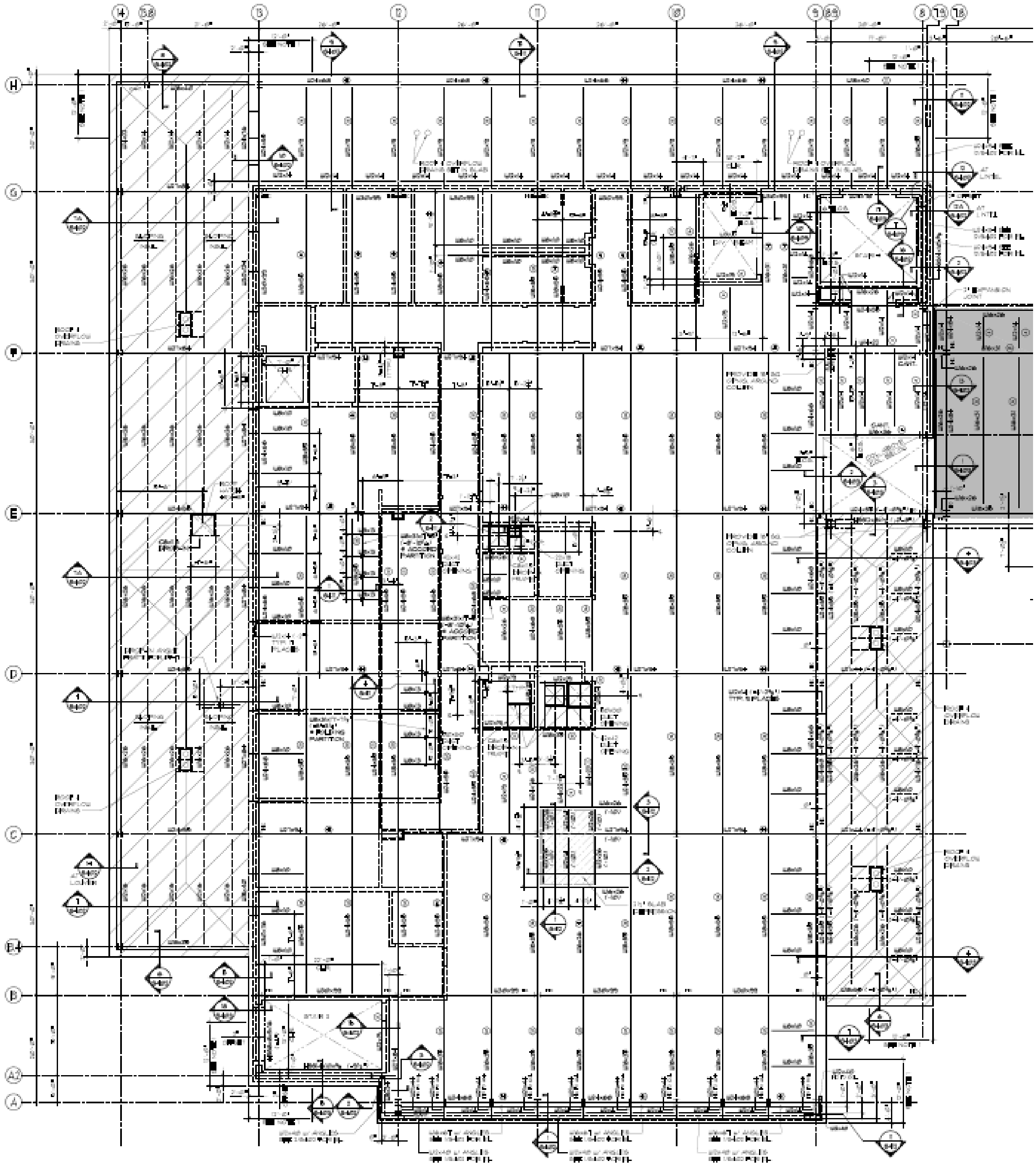
Appendix A: Typical Floor Plans

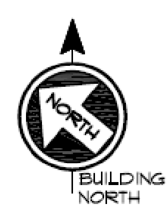


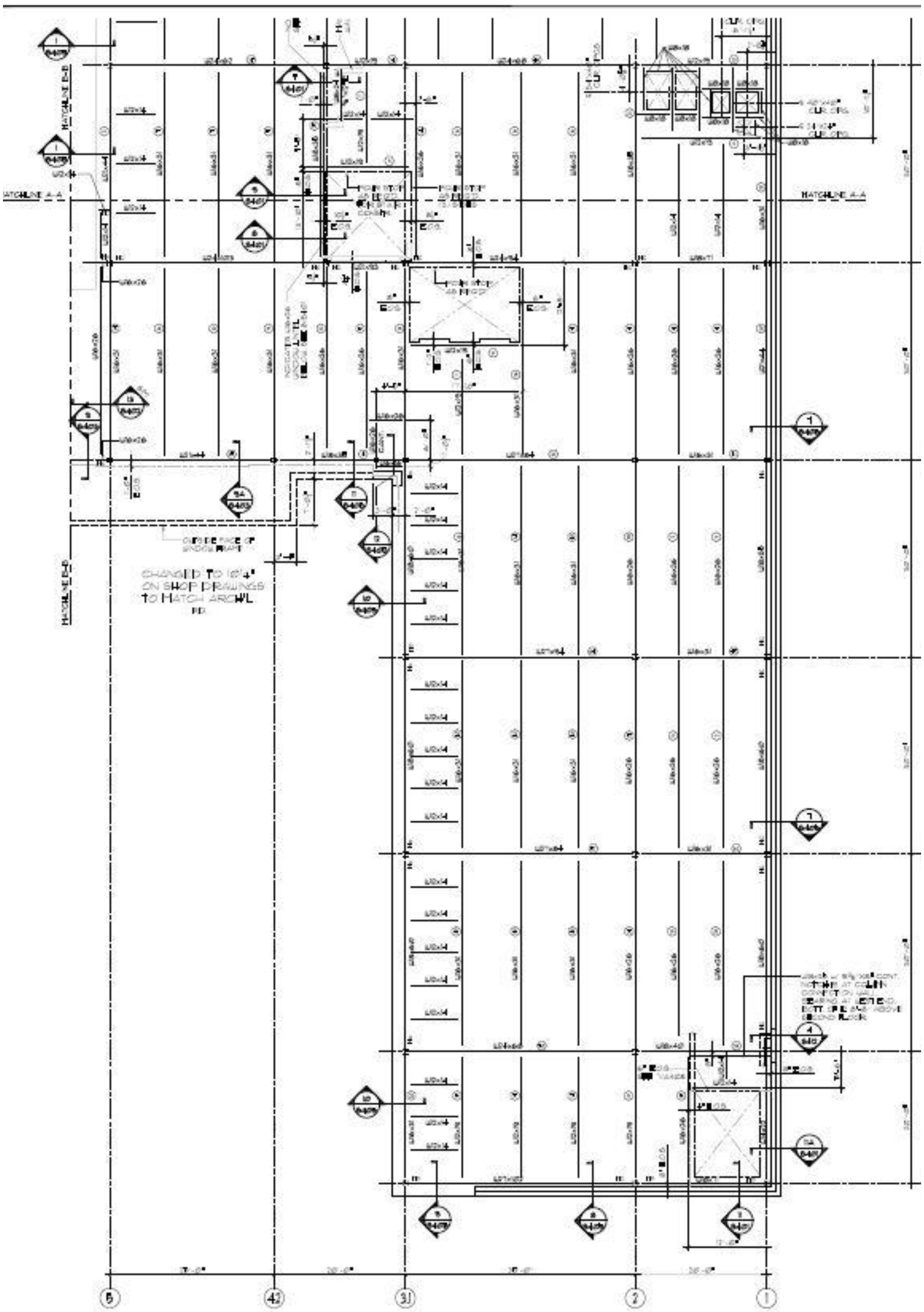
Framing Plan of the 2nd Floor, Courtesy of Highland Associates



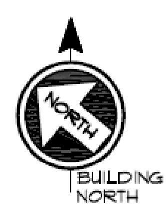


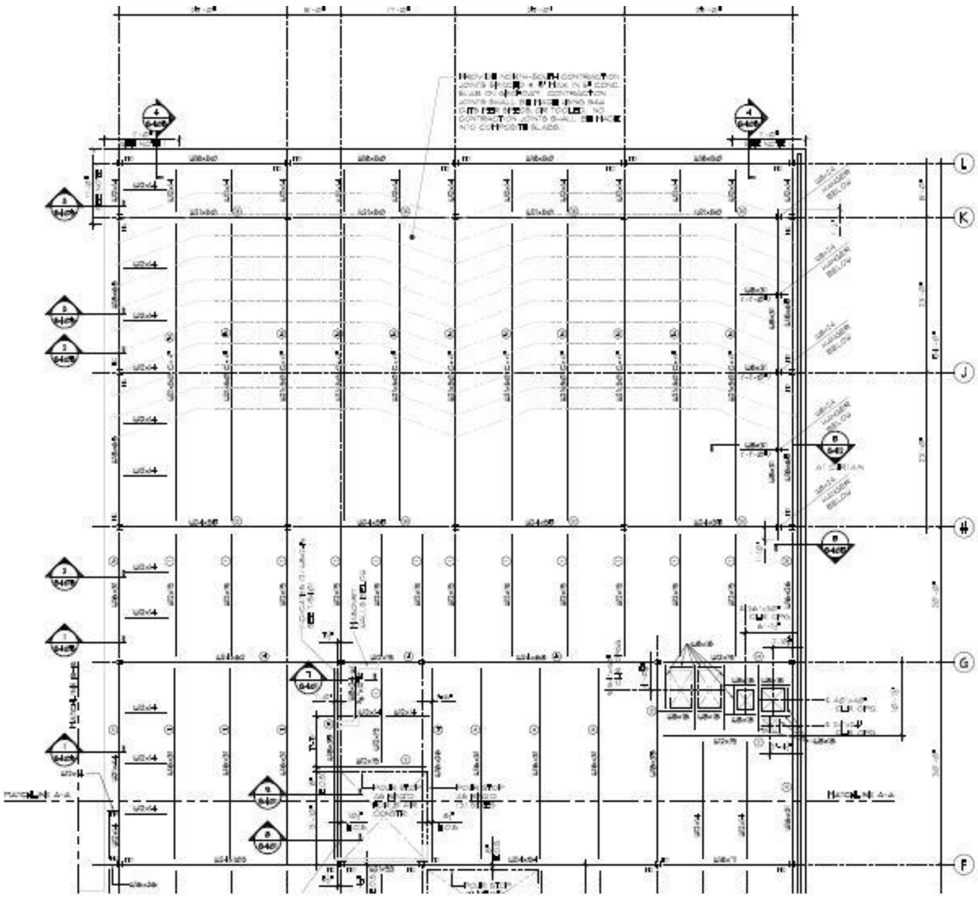
2nd Story frame, west wing, Courtesy of Highland Associates



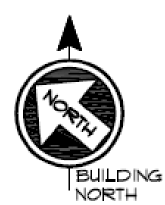


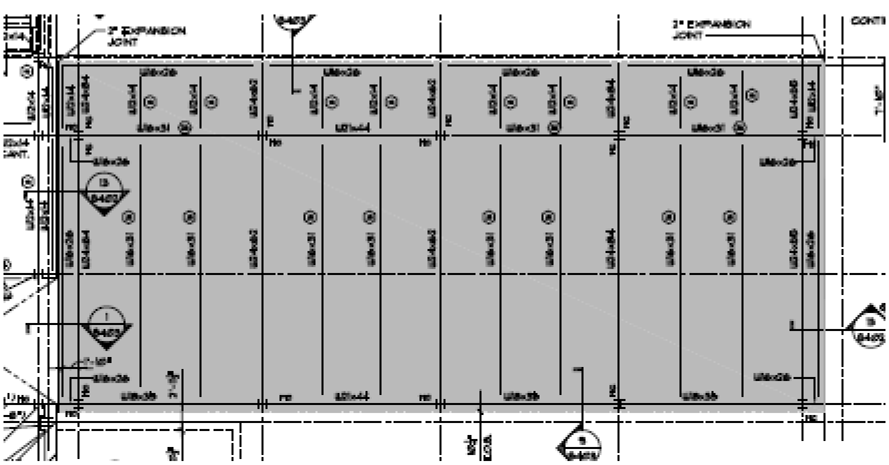
2nd Story frame, east wing (south), Courtesy of Highland Associates



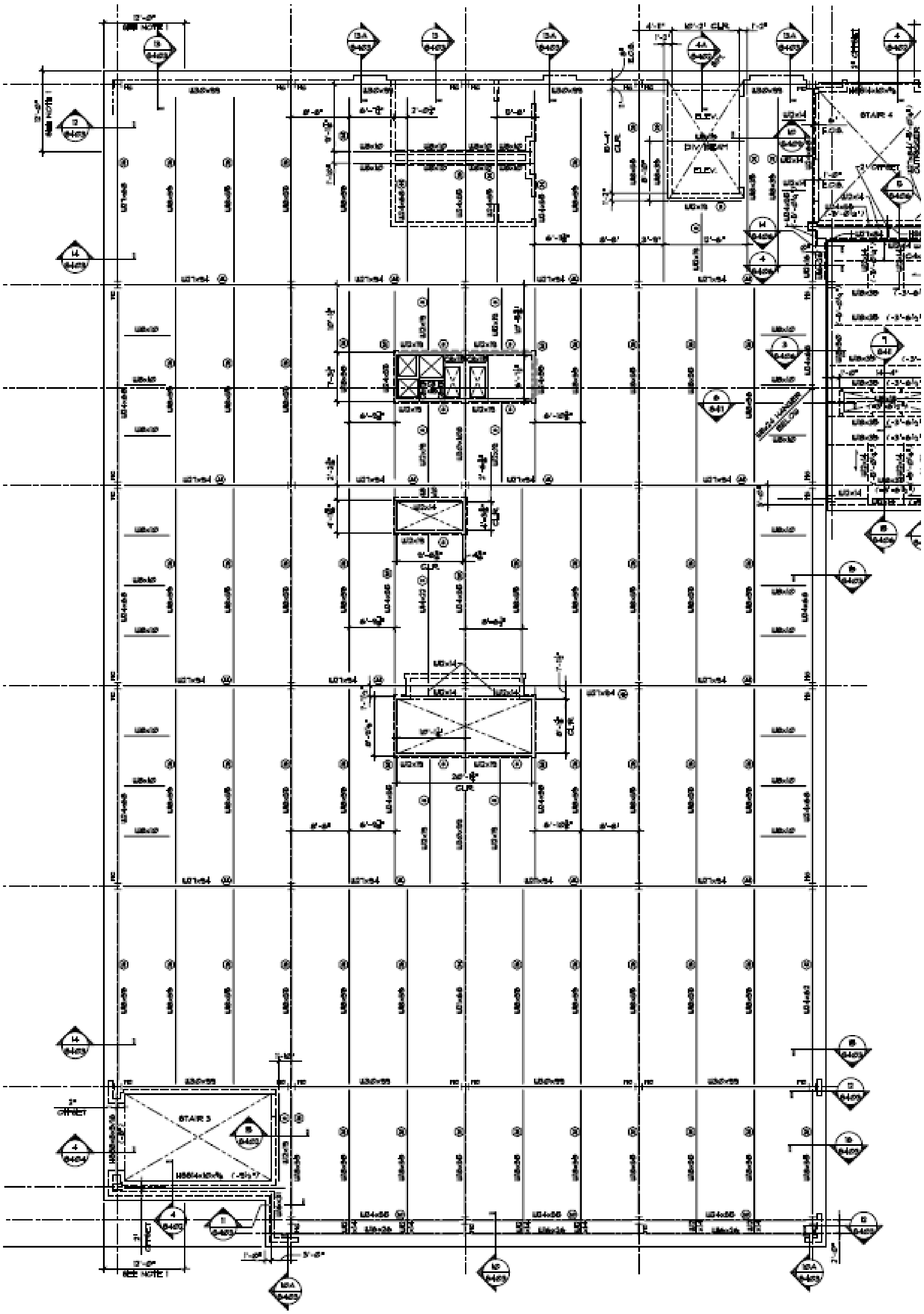


2nd Story frame, east wing (north), Courtesy of Highland Associates

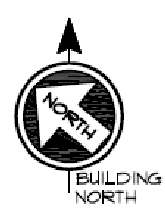


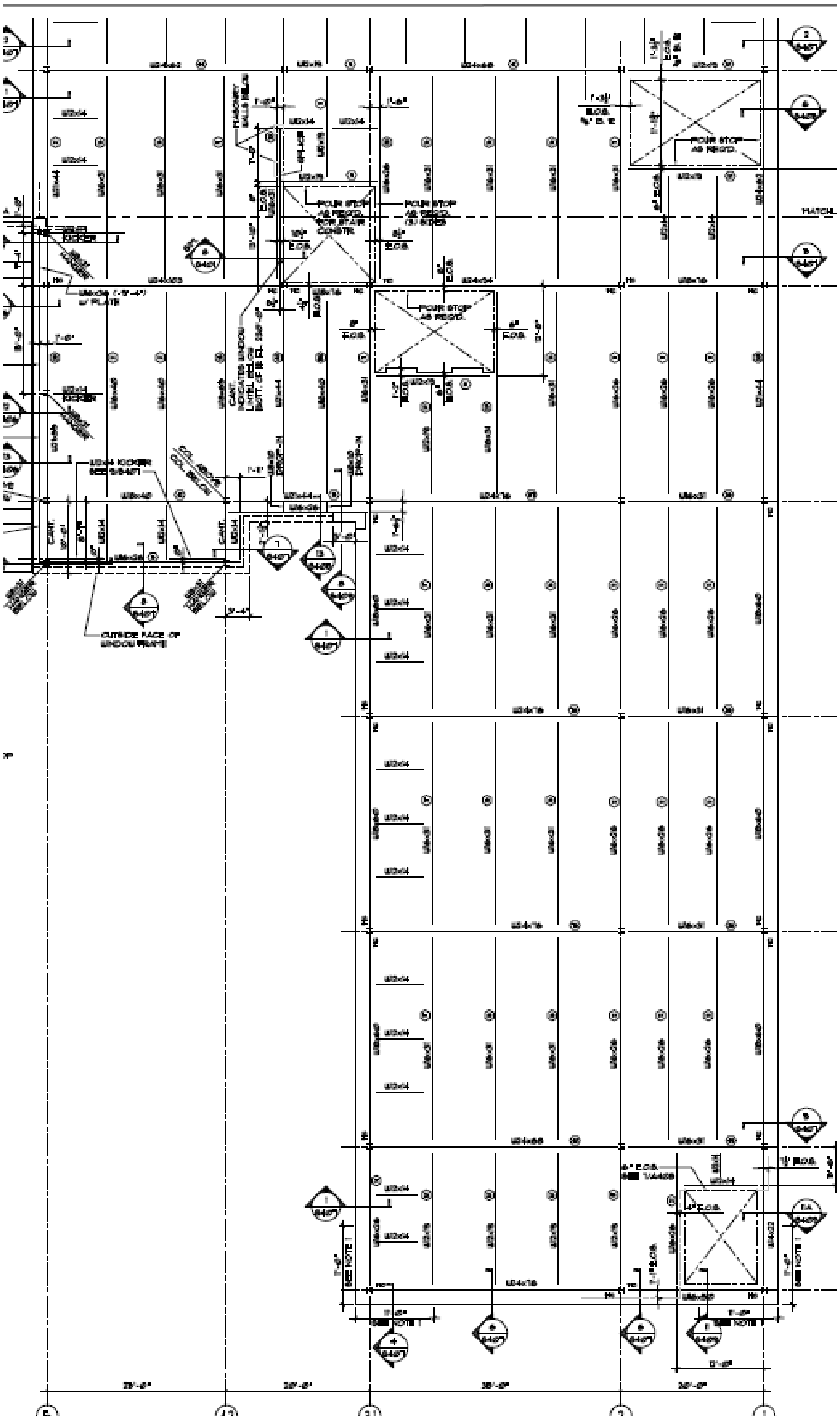


2nd Story frame, Link, Courtesy of Highland Associates

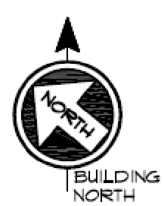


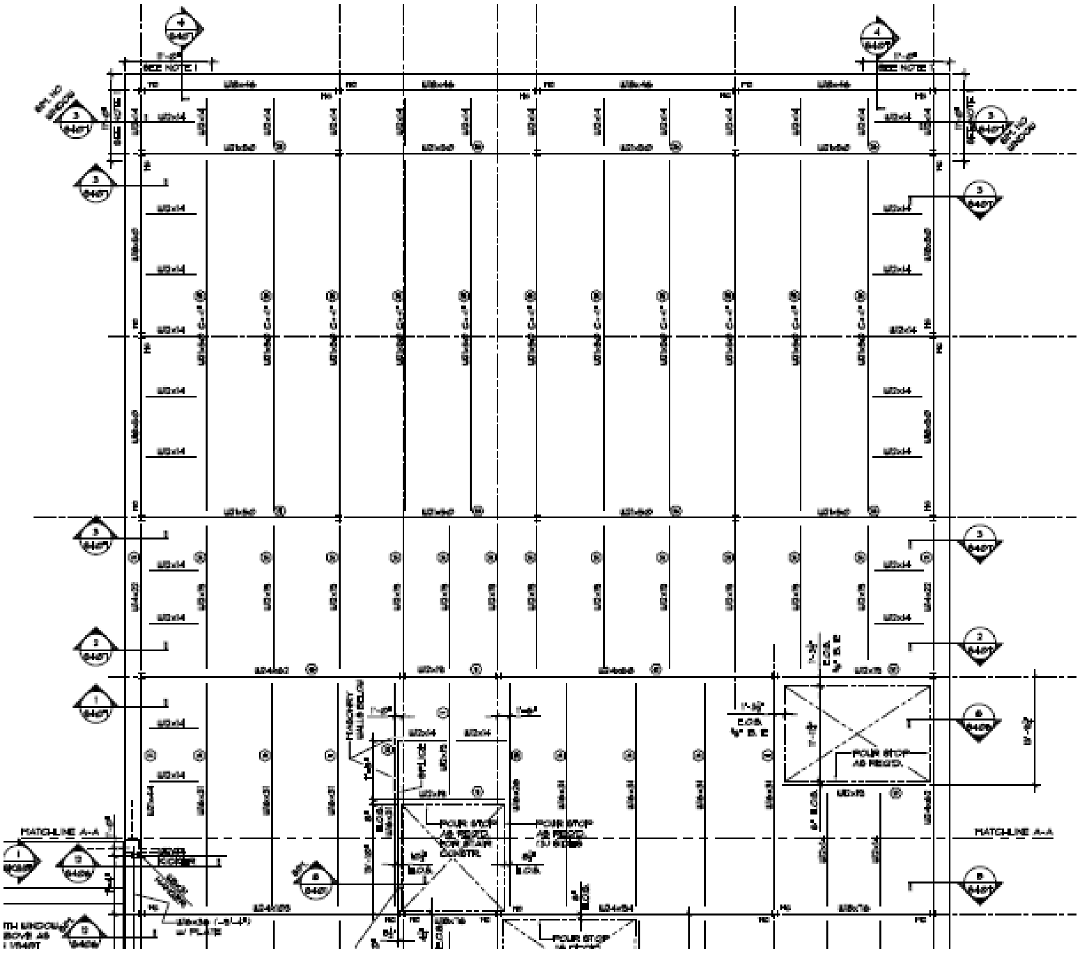
3rd Story frame, west wing, Courtesy of Highland Associates



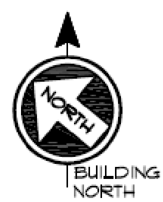


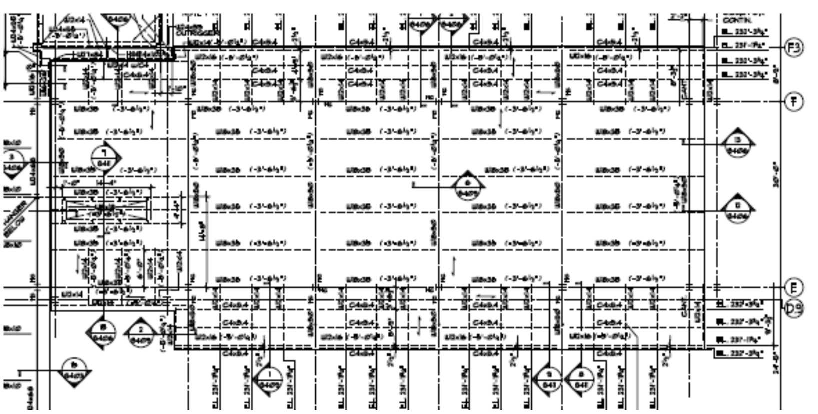
3rd Story frame, east wing (south), Courtesy of Highland Associates



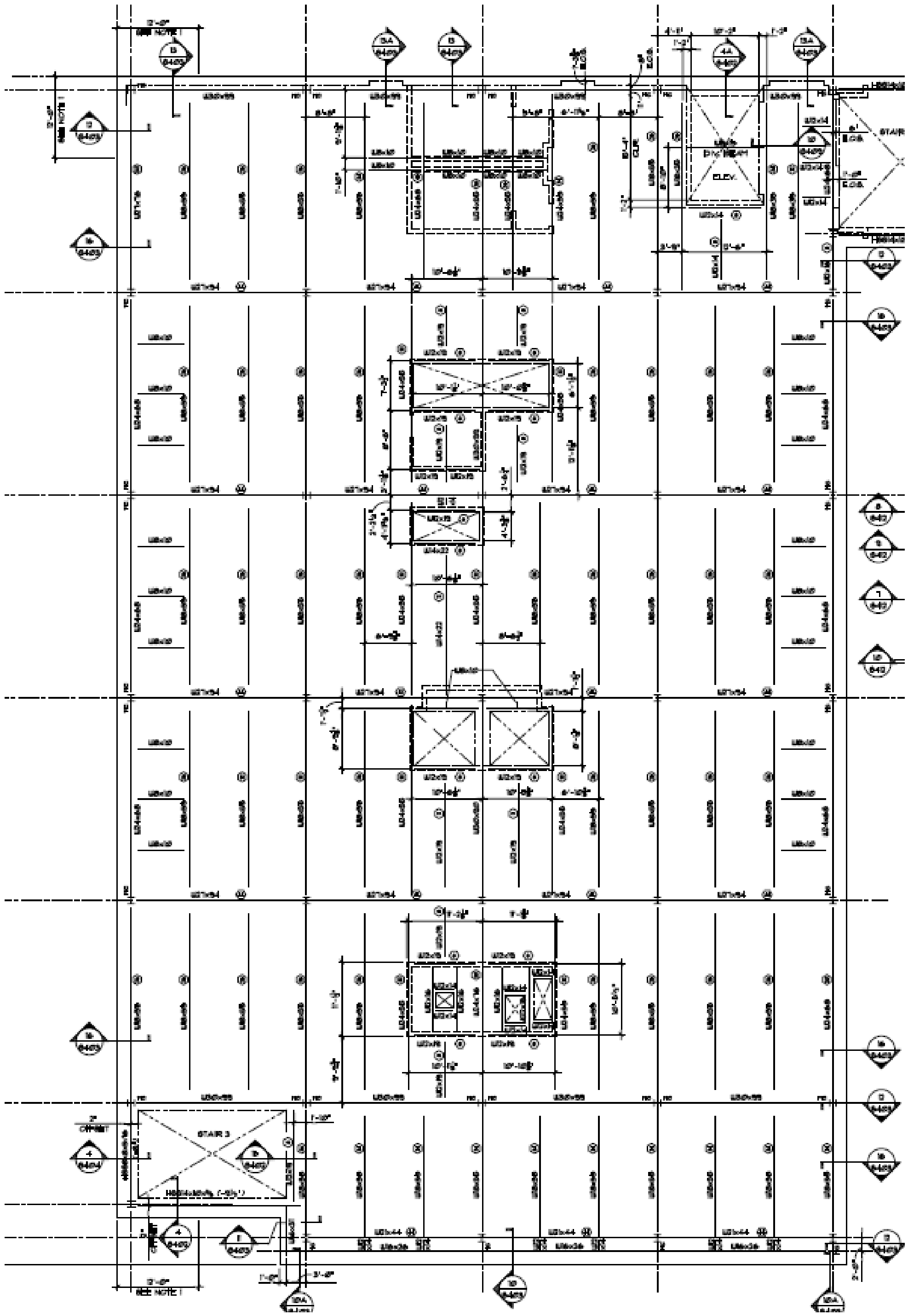


3rd Story frame, east wing (north), Courtesy of Highland Associates

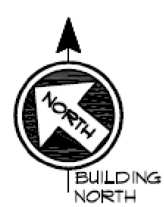


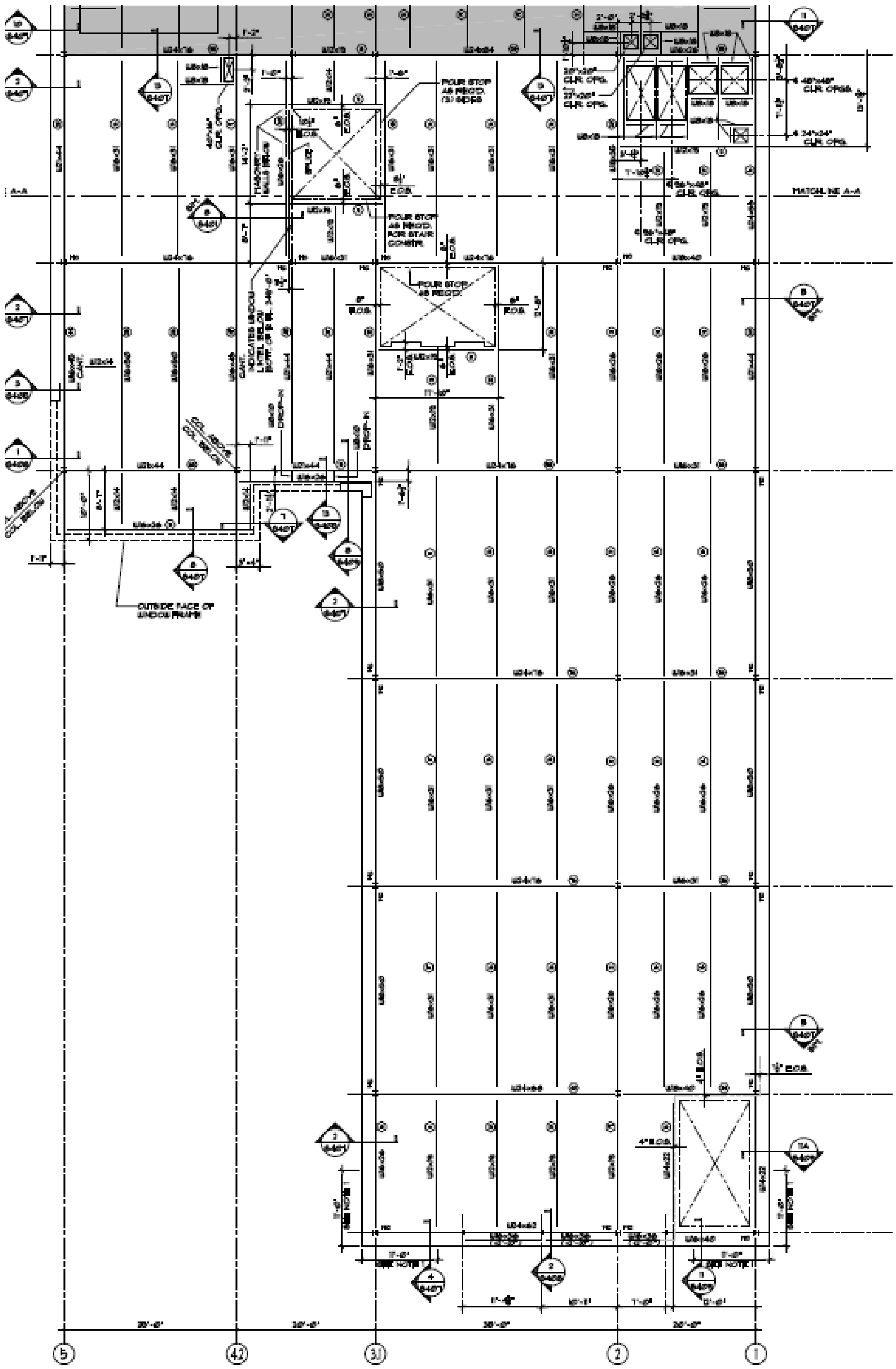


3rd Story frame, Link, Courtesy of Highland Associates

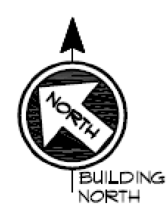


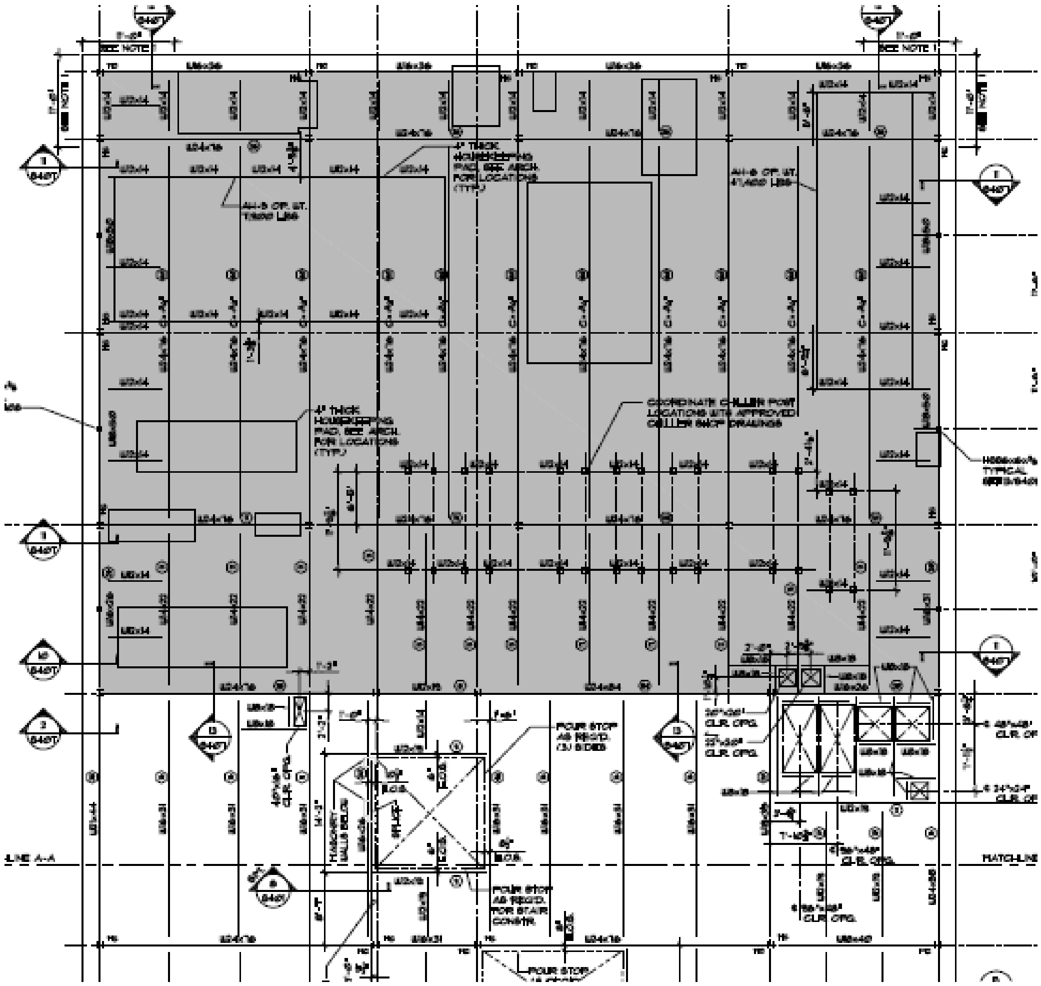
4th Story frame, west wing, Courtesy of Highland Associates



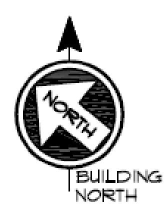


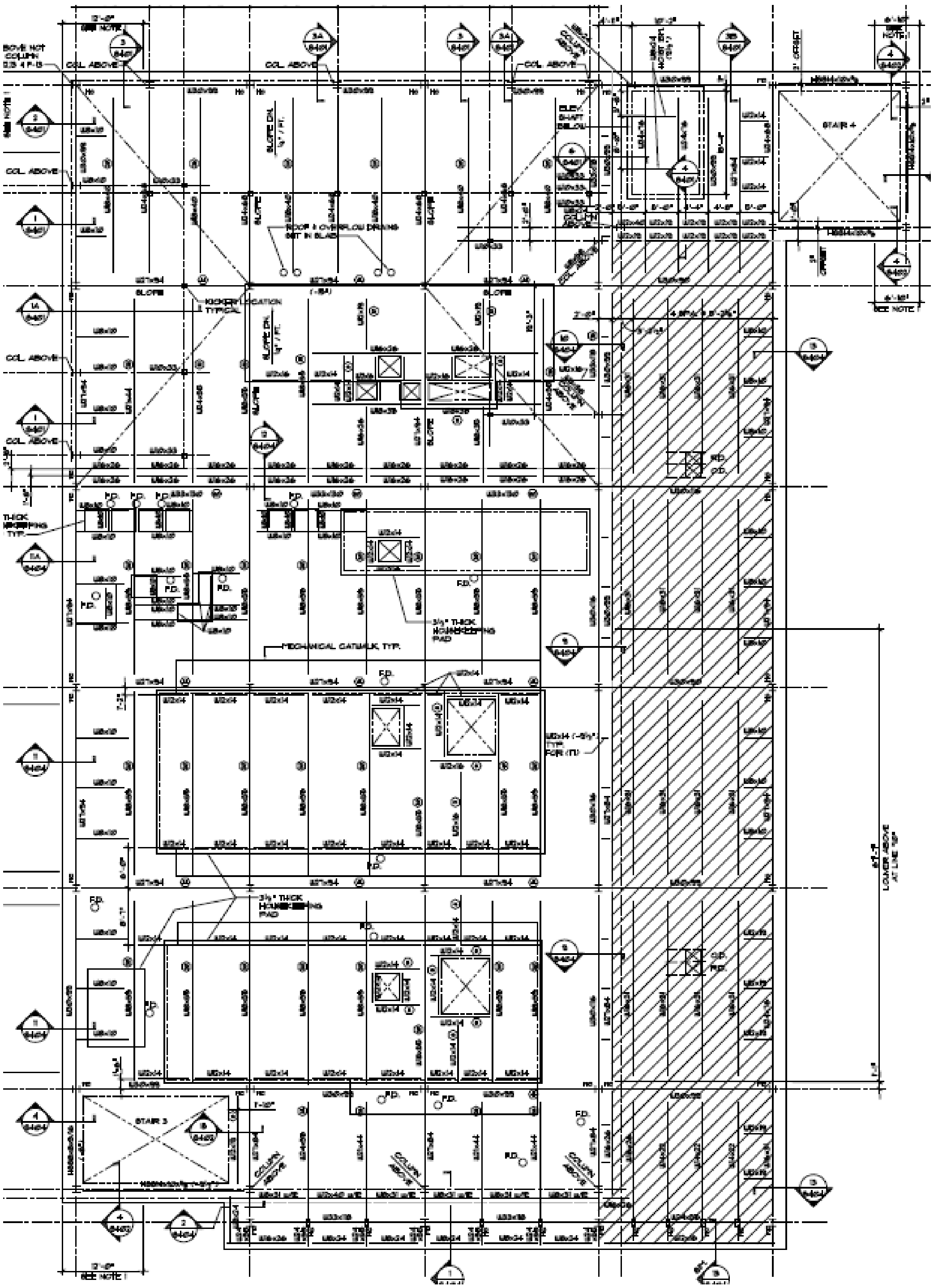
4th Story frame, east wing (south), Courtesy of Highland Associates



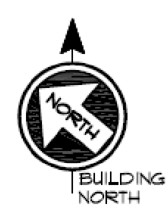


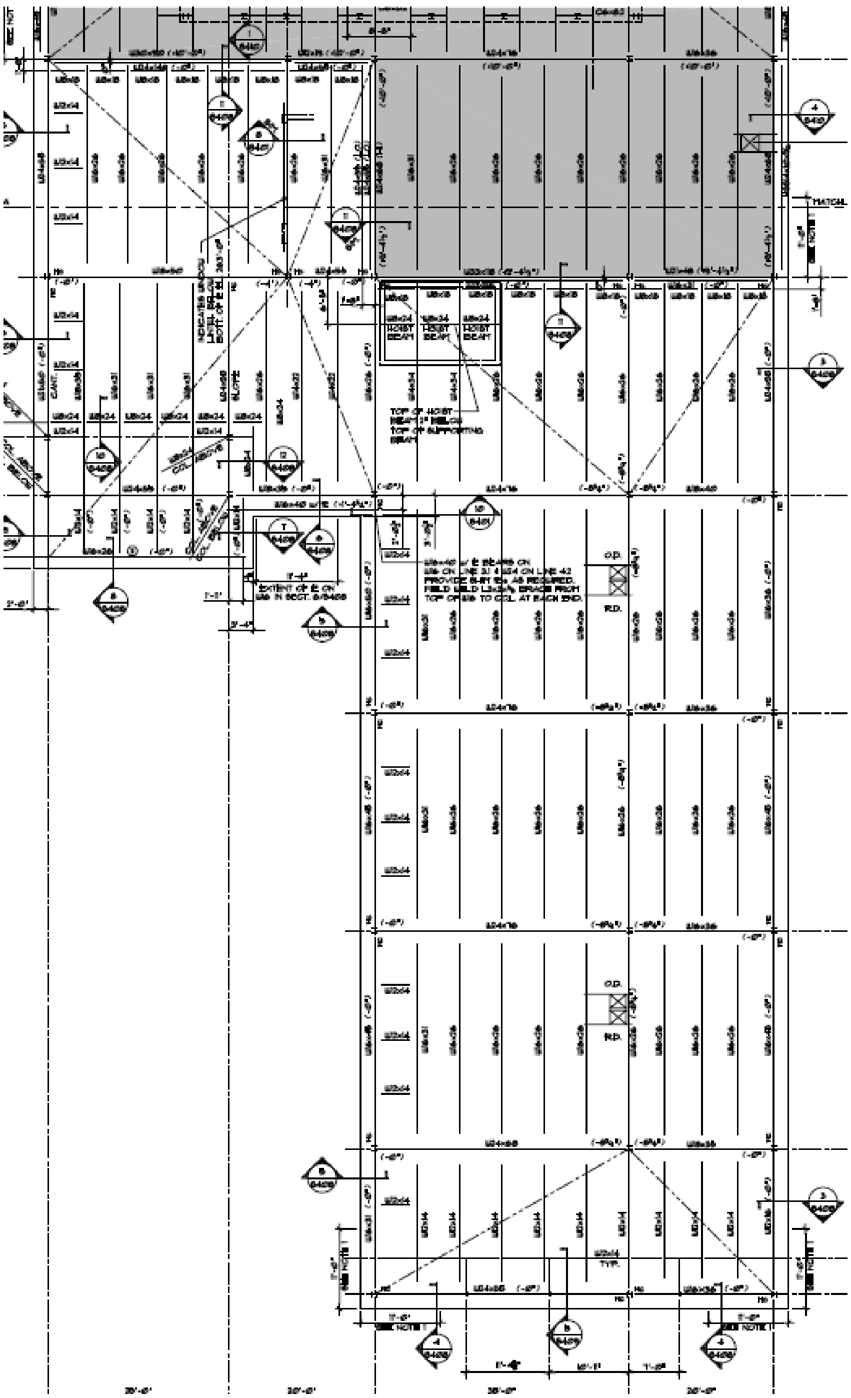
4th Story frame, east wing (north), Courtesy of Highland Associates



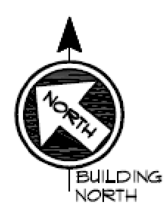


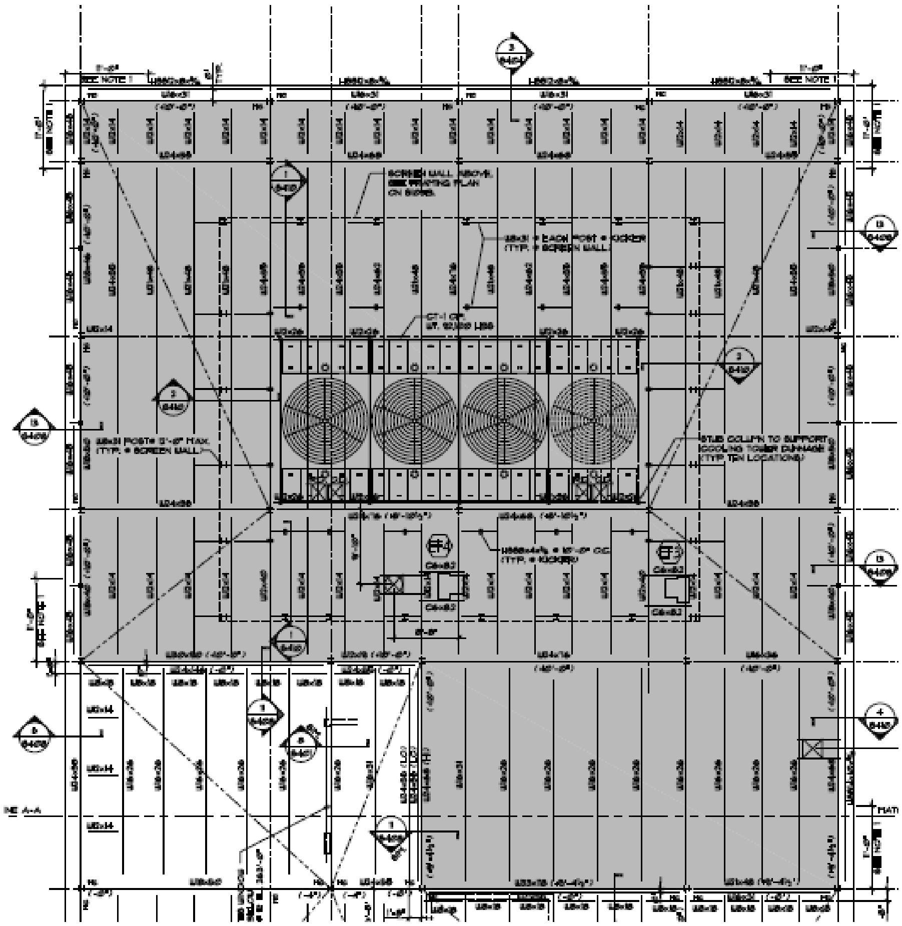
Main Roof Story frame, west wing, Courtesy of Highland Associates



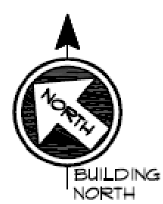


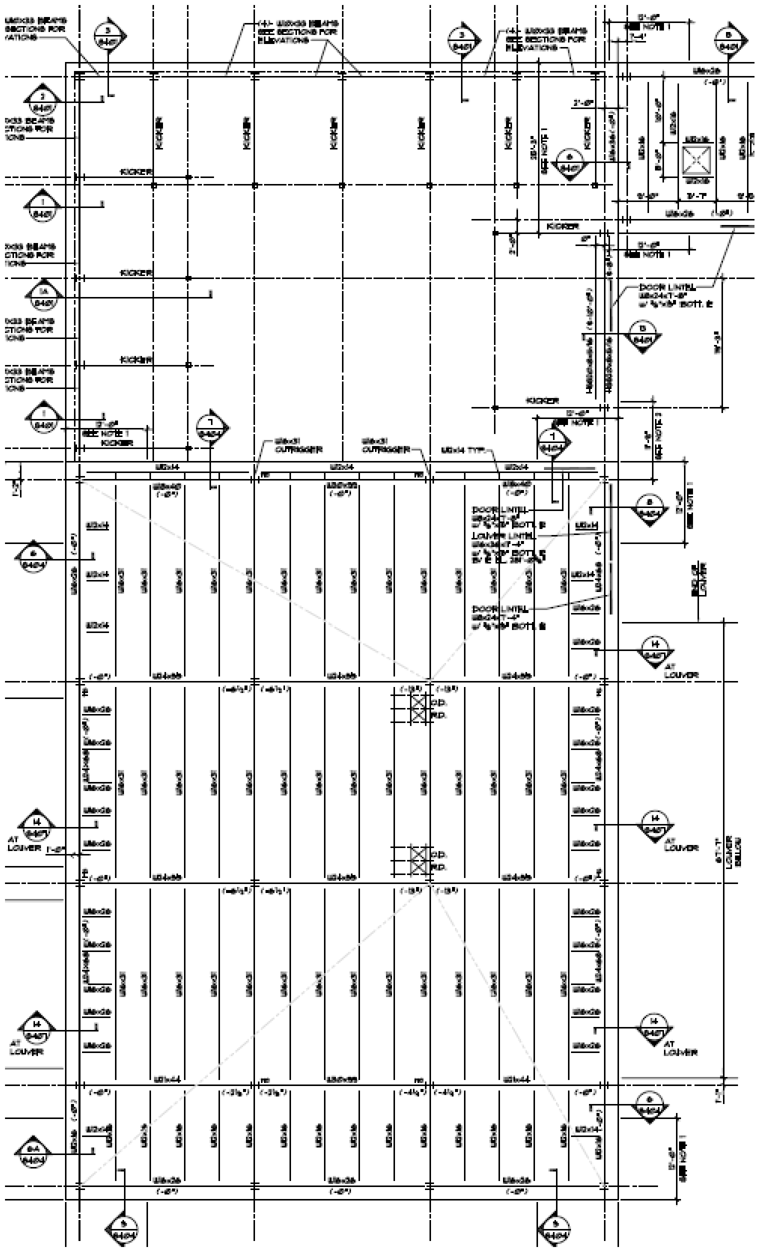
Main Roof Story frame, east wing (south), Courtesy of Highland Associates



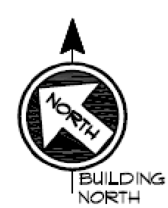


Main Roof Story frame, east wing (north), Courtesy of Highland Associates



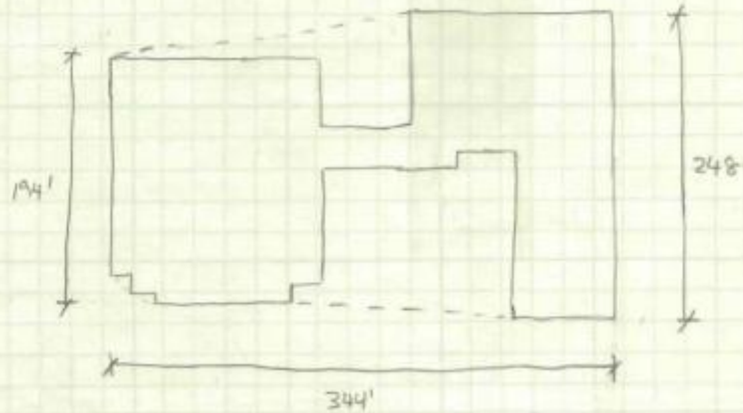


Penthouse Roof Story frame, west wing, Courtesy of Highland Associates

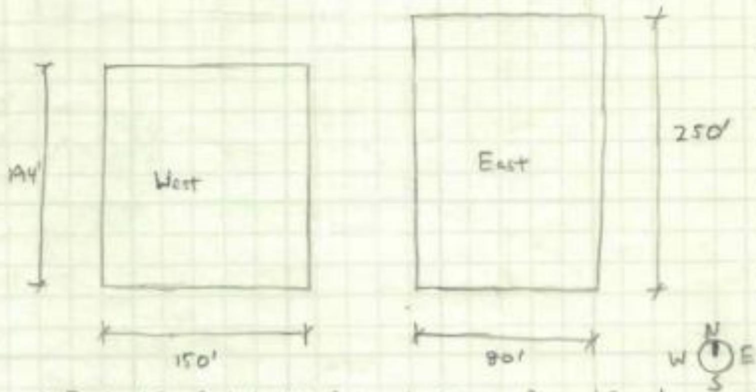


Appendix B: Miami, FL, Wind Load Calculations

Miami
Wind Design



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, $V_{3s} = 150$ MPH, Miami Florida

$I_w = 1.15$ = 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

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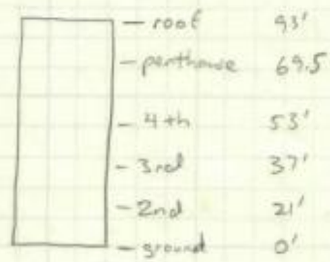

$q_z = 0.00256 K_z K_{zt} K_d V^2 I_w$

$= 0.00256(.99)(1.0)(.85)(150)^2(1.15)$

$q_{z_r} = 55.8$ psf for roof

$= 0.00256(.89)(1.0)(.85)(150)^2(1.15)$

$q_{z_p} = 50.2$ psf for penthouse floor

$= 0.00256(.83)(1.0)(.85)(150)^2(1.15)$

$q_{z_4} = 46.8$ psf for 4th story floor

$= 0.00256(.74)(1.0)(.85)(150)^2(1.15)$

$q_{z_3} = 41.7$ psf for 3th story floor

$= 0.00256(.70)(1.0)(.85)(150)^2(1.15)$

$q_{z_2} = 39.5$ psf for 2nd story floor

$= 0.00256(.70)(1.0)(.85)(150)^2(1.15)$

$q_{z_g} = 39.5$ 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

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

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

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

Wind Design | | 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

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

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

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

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

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

$$= 381.2$$

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

$$= 64.0$$

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

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

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West Wing

Is not required, does not control design

N-S Direction

$h = 93$ ft

$L = 150$ ft

$B = 194$ ft

E-W Direction

$h = 93$ ft

$L = 194$ ft

$B = 150$ ft

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

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

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

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

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

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

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

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

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

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

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

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

Wind Design

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 $> 2h$

$p = 55.8(.828)(0.8) - 55.8(\pm .18) =$

$= 37.0 \pm 10.1$

$= 47.1$ or 26.9 psf

at roof height, windward wall

* see excel for rest of calculations

Wind Design | Page 6 of 7

East Wing

N-S Direction

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

E-W Direction

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

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

N-S Direction:

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

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

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

$R_h = .130$

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

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

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

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

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

$G_f = .817$

E-W Direction:

$\eta_h = 7.15$

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

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

$R_h = .130$

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

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

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

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

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

$G_f = .863$

Wind Design

East Wing Pressures

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

Wall

N-S.

Windward $C_p = 0.8$

Side Wall $C_p = -0.7$

Leeward $C_p \Rightarrow L/B = 80/250 = .32$

$C_p = -0.5$

E-W

$C_p = 0.8$

$C_p = -0.7$

$C_p = 250/80 = 3.125$

$C_p = -0.25$ by interpolation

Roof

$\theta = 0°$ $h/L = 93/80 = 1.162$

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

$2h = 186$

Roof area >> 1000 sf R.F = .8

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

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

$h/L = 93/250 = .372$

$C_p = -0.9$ for 0 to h

$C_p = -0.5$ for h to 2h

$C_p = -0.3$ for >2h

$p = 55.8(.817)(0.8) - 55.8(\pm .18) = 9.51$

$= 36.5 \pm 10.1$

$= 46.6$ or 26.4 psf

at roof height, windward wall

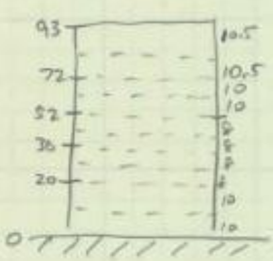
$p = 55.8(.863)(0.8) - 55.8(\pm .18)$

$= 38.6 \pm 10.1$

$= 48.7$ or 28.5 psf

* See Excel for rest of calculations

Wind Forces Breakdown



Appendix C: Miami, FL, Seismic Load Calculations

Seismic Analysis

For Miami Florida, Site Class D

$S_s = 0.050g$ $S_{MS} = 0.080g$ $S_{DS} = 0.053g$

$S_1 = 0.019g$ $S_{M1} = 0.047g$ $S_{D1} = 0.031g$

From Table 12.2-1

R = 3.5 for ordinary steel Moment Frames

$\Omega = 3$

$C_d = 3$

R = 3.25 for ordinary steel concentrically braced frames

$\Omega = 2$

$C_d = 3.25$

From Table 11.6-1 ASCE 7-05

$S_{DS} < 0.167$ Occupancy Category III $\Rightarrow$

Seismic Category A.

Section 11.7 of code

$V = .01(W)$ $W \approx 23{,}000$ kips

$V = .01(13{,}600) = 136$ kips

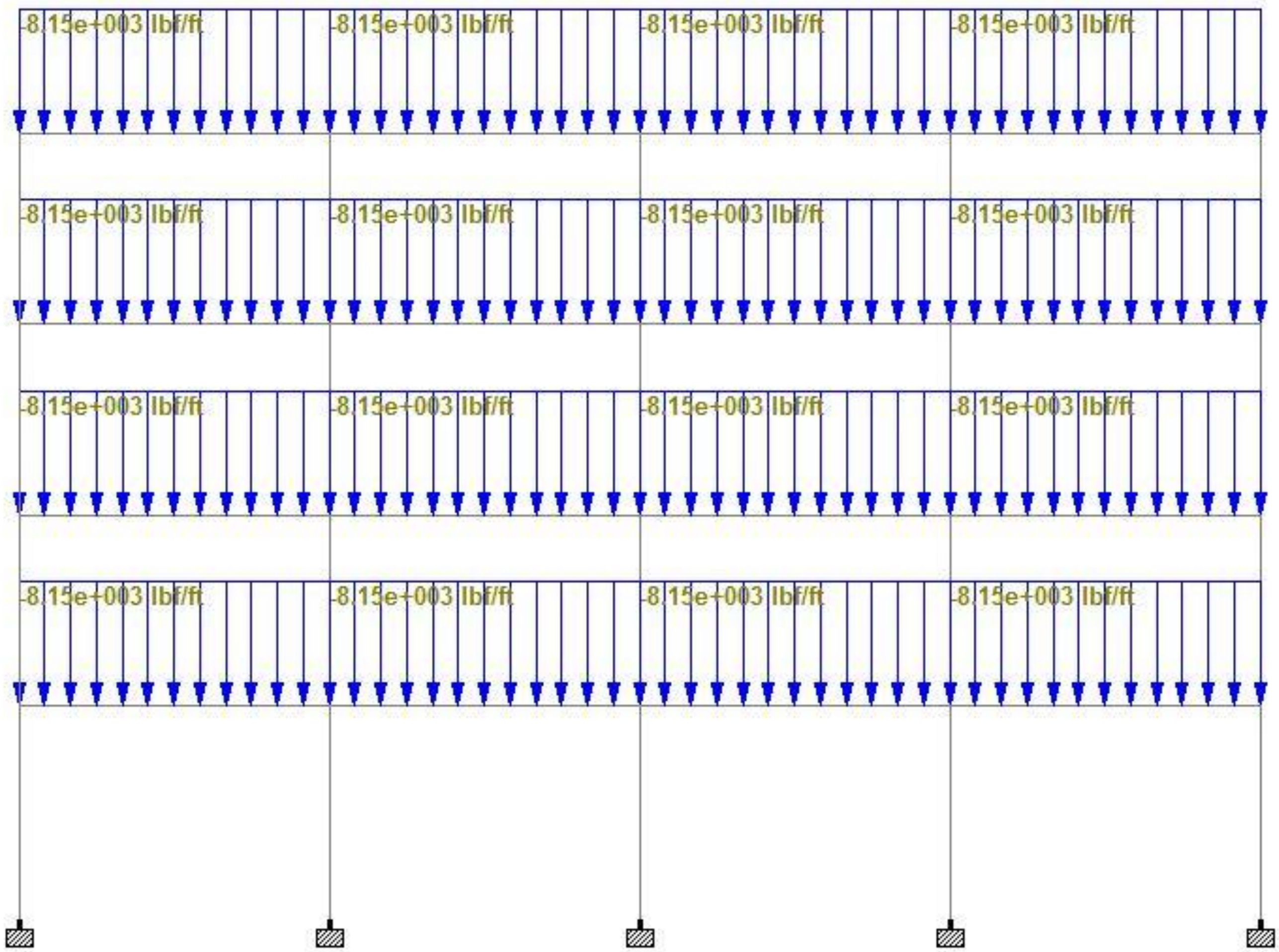
$F_x = 0.01 w_x = 130$ kips at 1st for West Wing

See rest on excel

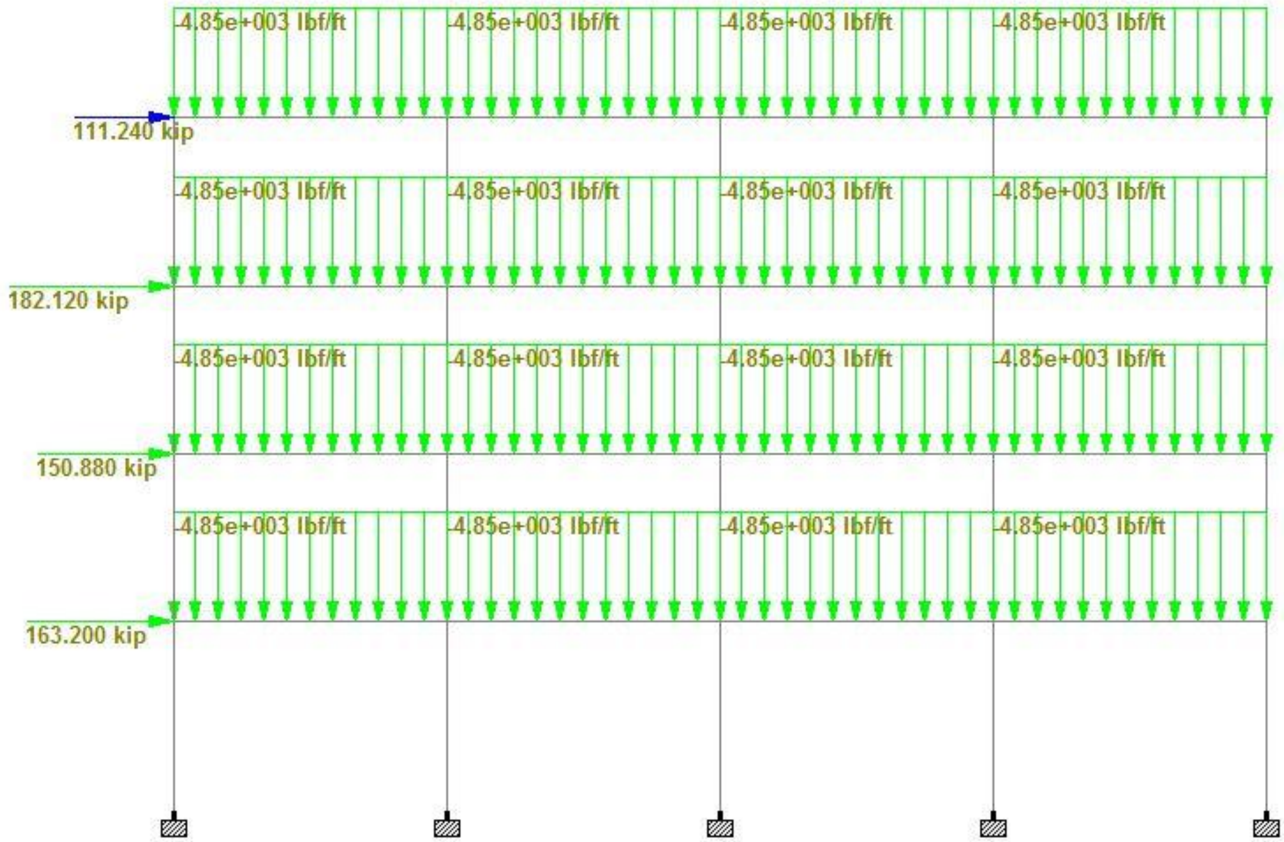
$F_x = 0.01(12{,}600) = 126$ kips at 1st for East Wing

See rest on excel

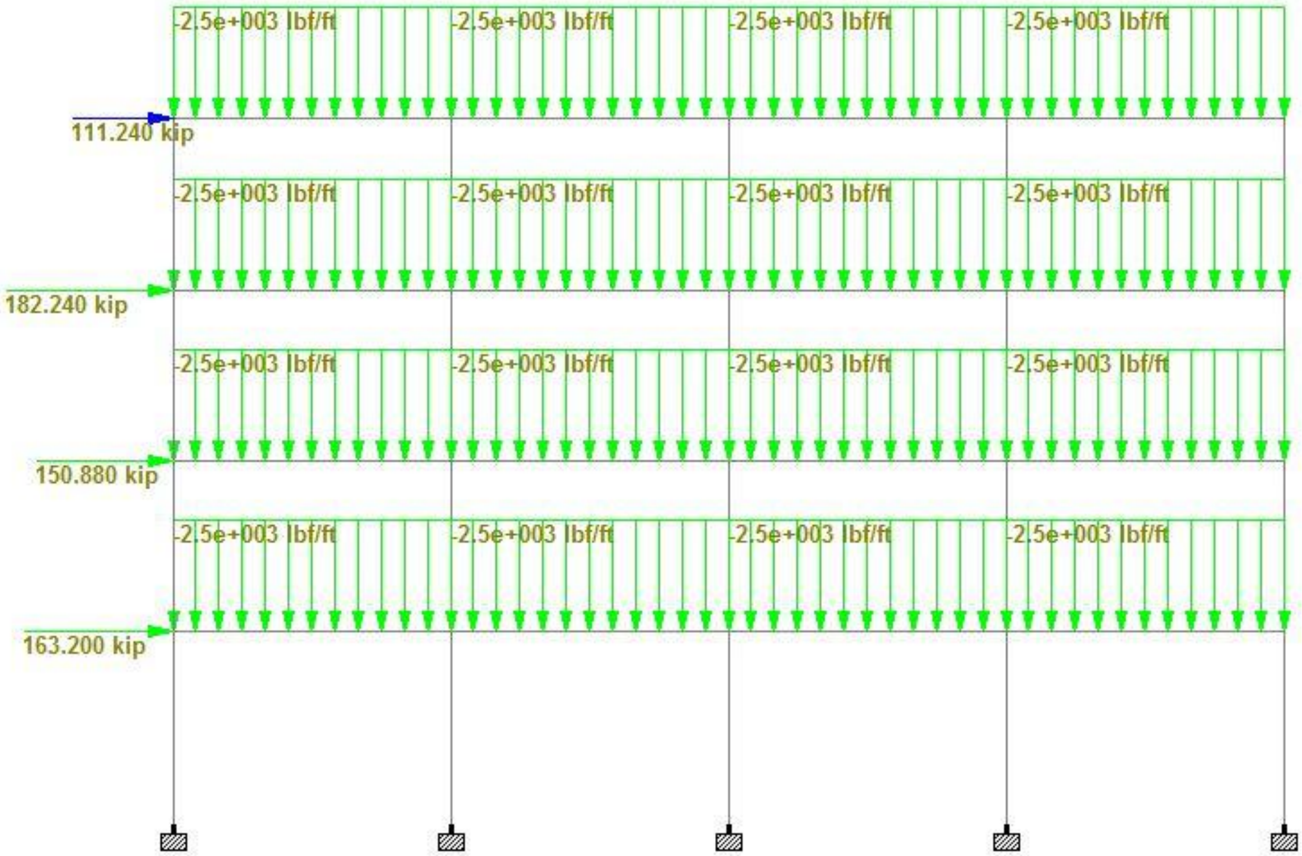
Appendix D: Moment Frame Design



1.2D+1.6L on Frame A



1.2D+1.6W+0.5L on Frame A



0.9D+1.6W on Frame A

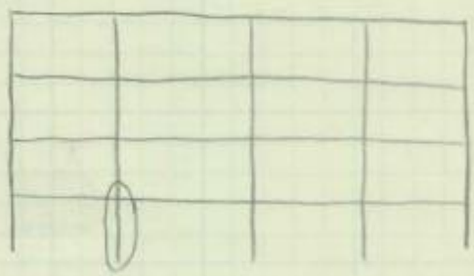
Moment Frame Design | Thesis | Page 1 of 3

Frame A will be used for this preliminary design

Check for dead + live load.

G12 = W14x257

Frame D



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

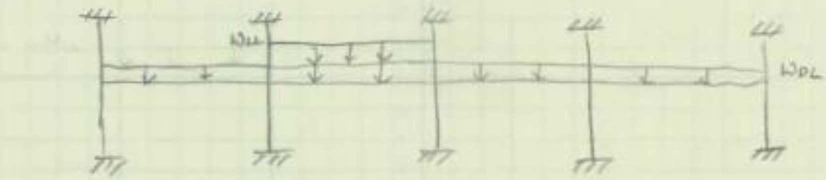
The following loadings give the largest moments on beams.

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

+.0996 = largest positive moment

Moment Frame Design | Thesis | Page 2 of 3

Columns continue



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

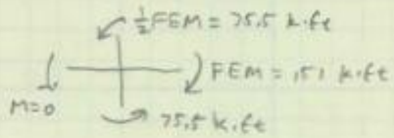
$LL_{red} = 0.25 + \frac{15}{\sqrt{2(25)(26)}} = .67$

Dead load unbalanced moment ≈ 0

Live load unbalanced moment

$w_L = 1.6(100)(25)(.67) = 2.68$ klf

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

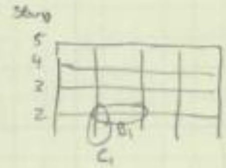
$w_u = 1.2(2.325) + 1.6(1.625) = 5.39$ klf

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

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

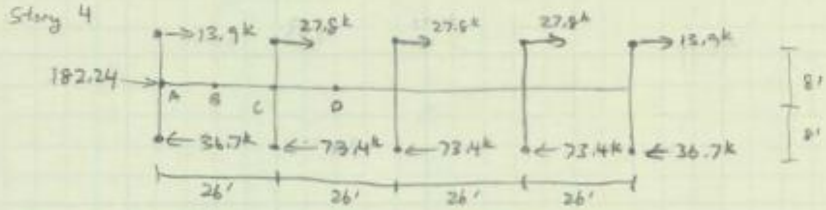
Moment Frame Design — pg 3 of 3

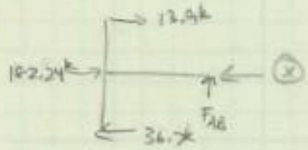
Portal Method Analysis



Frame A

1.6W load





$\Sigma M_A = 13.9(8) + 36.7(8) - F_{AB}(13) = 0$

$F_{AB} = 31.1^k$

$M_{AC} = 31.1(13) = 404.3$ k-ft $\approx$ 380 STAAD model ✓ Good.

Use load from STAAD model

Determining sizes

For C_1 $\Rightarrow$ $P_r = 864$ kip

$M_r = 923$ k-ft

Try W 14x342 $\Rightarrow$ $P_c = 3760$

$M_c = 1270$

$\frac{P_r}{P_c} = \frac{864}{3760} = .23 \geq 0.20$ $\quad \frac{M_r}{M_c} = \frac{923}{1270} = .73$

$.23 + \frac{8}{9}(.73) = .88$ works

Try W 14x311 $\Rightarrow$ $P_c = 3390$

$M_c = 1140$

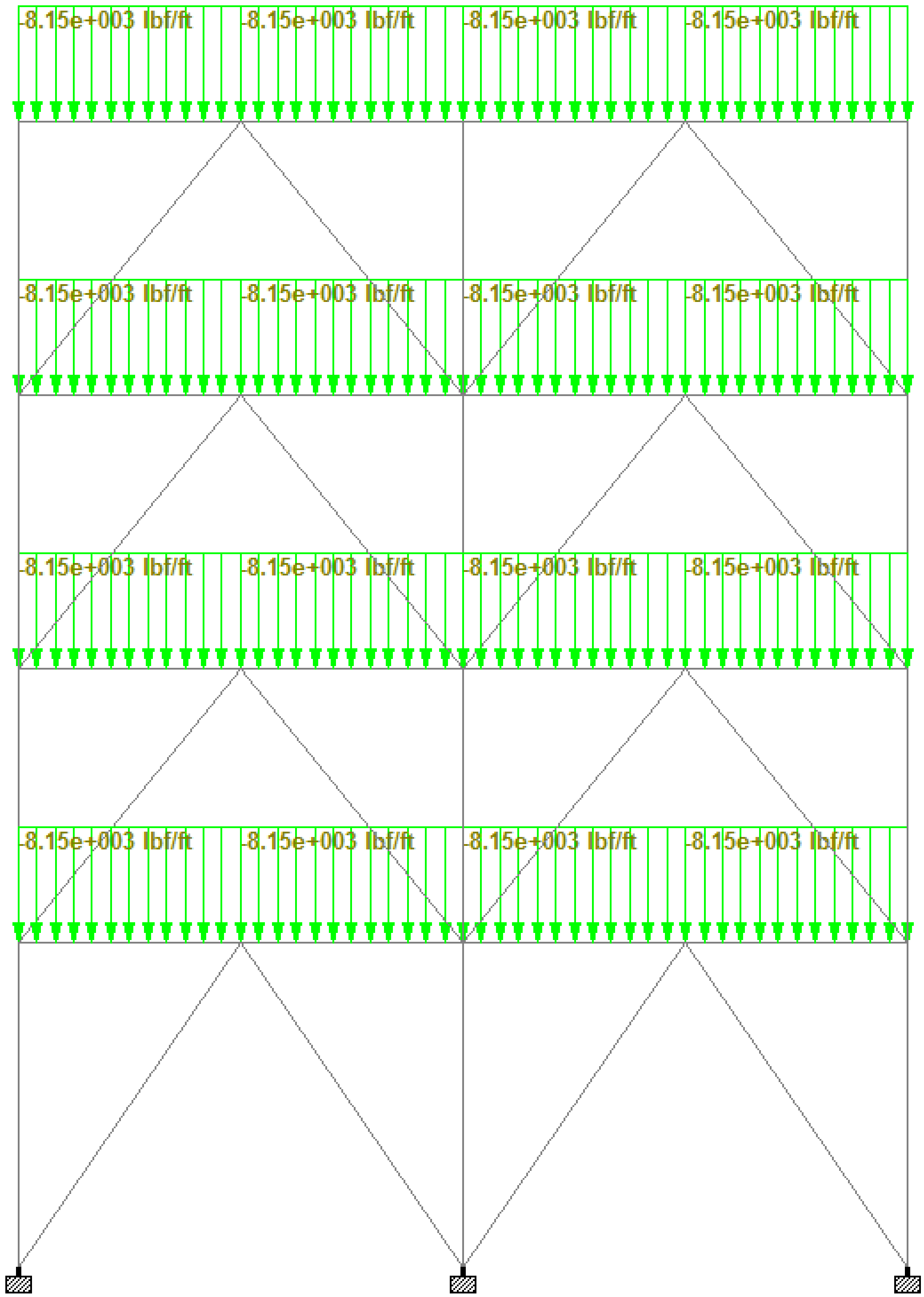
$\frac{P_r}{P_c} = \frac{864}{3390} = .254 \geq 0.20$ $\quad \frac{M_r}{M_c} = \frac{923}{1140} = .81$

$.254 + \frac{8}{9}(.81) = .974$ works ✓

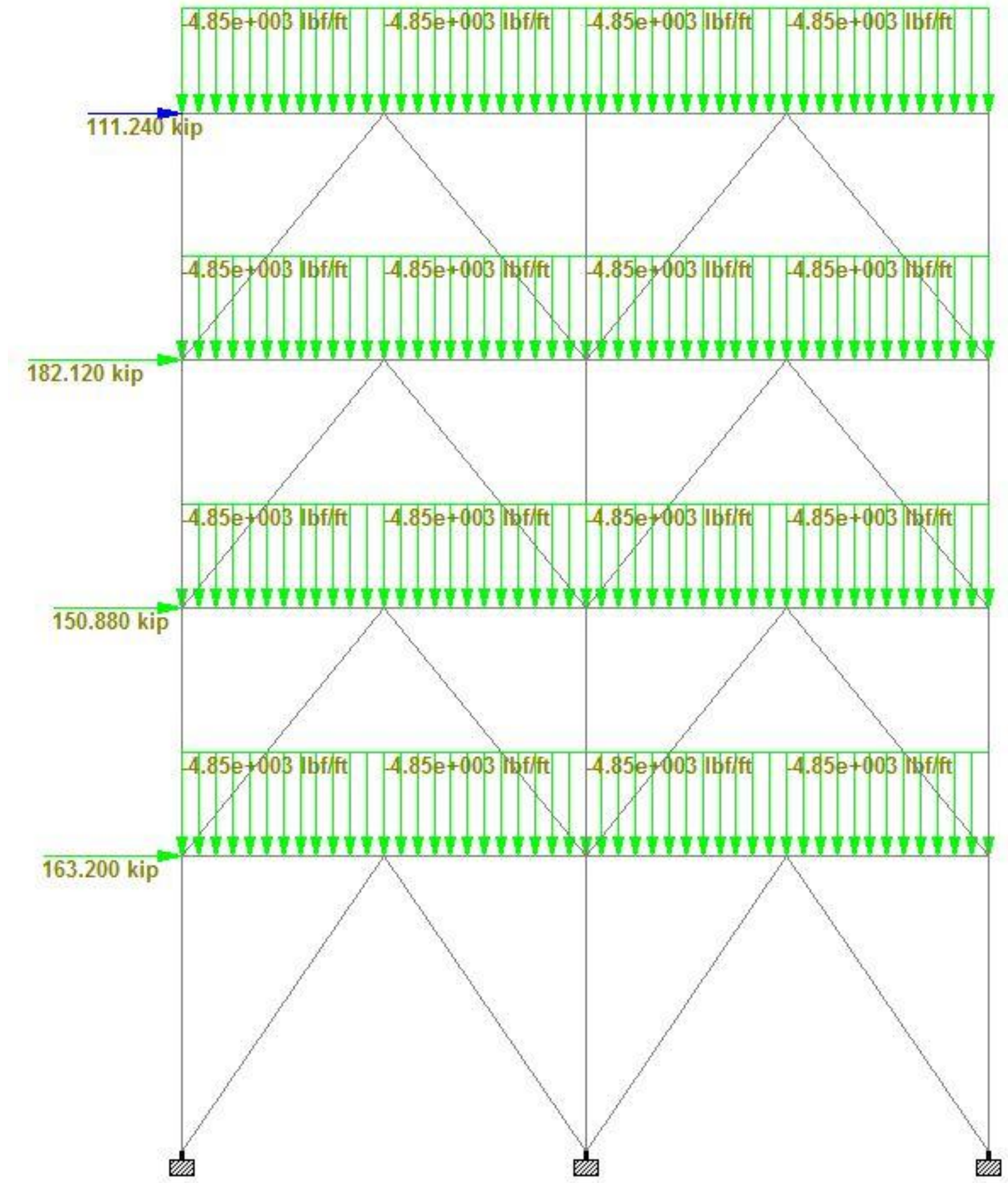
For B_1 $\Rightarrow$ $M_u = 1189$ k-ft

Most economical = W 27 x 146 Table 3-10 Steel Manual.

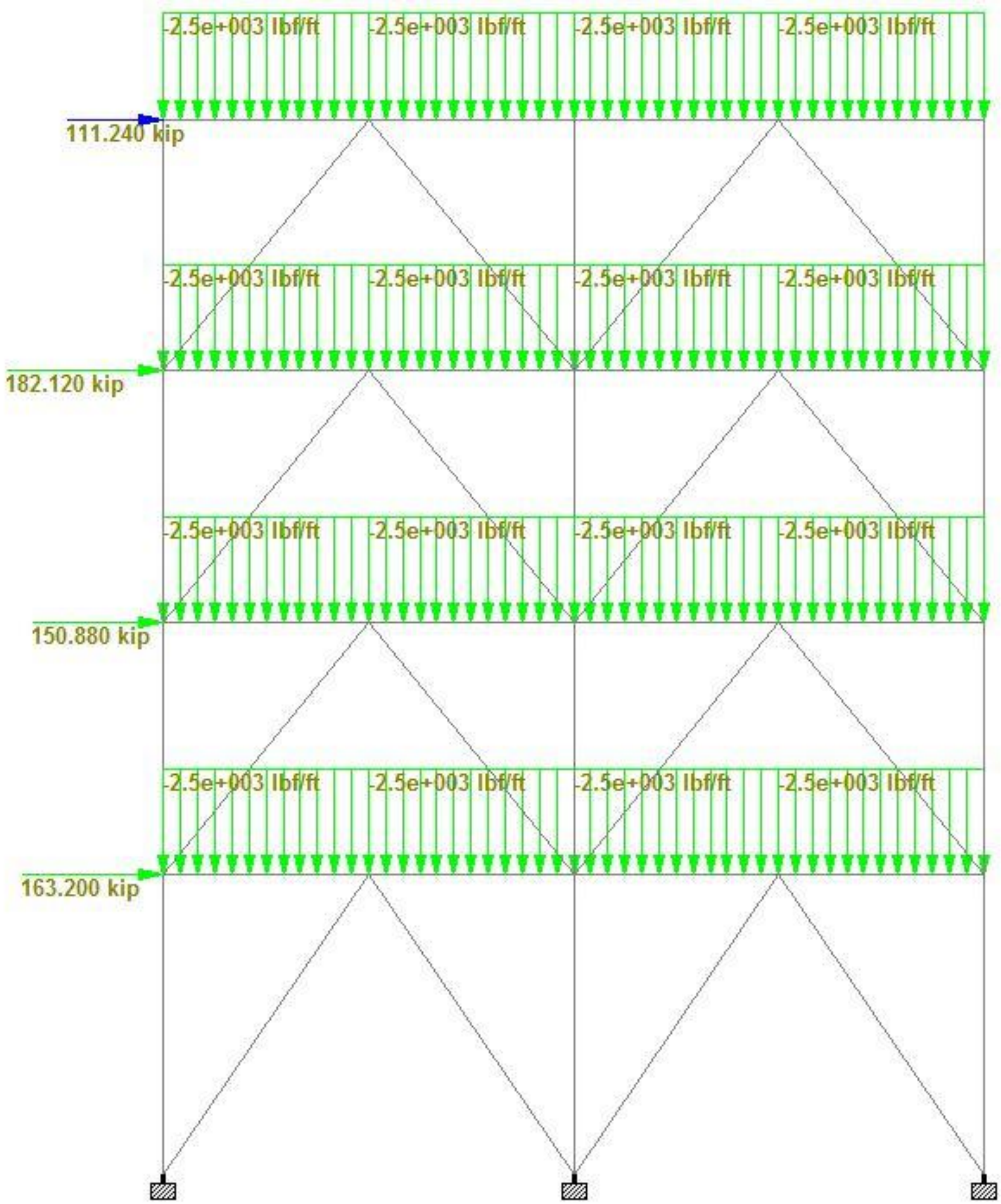
Appendix E: Chevron Braced Frame Design



1.2D+1.6L on Frame A



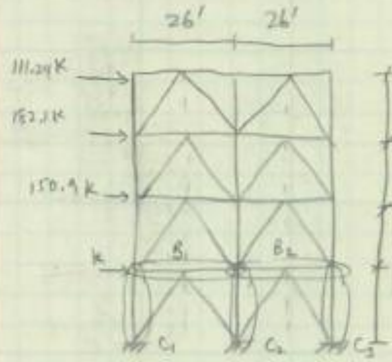
1.2D+1.6W+0.5L on Frame A



0.9D+1.6W on Frame A

Calculation Braced Frame | Thesis | Page 1 of 3

Preliminary Calculations for Braced Frame G in the West Wing



Frame A
West Wing

Select Trial Sizes
by strength design

- Columns
  We loads obtained from STAAD.

  Set of loads on columns. Columns will be the same size.
  So column size $C_1 = C_2 = C_3$

  Set of loads $C_1 \rightarrow$ $P_u = 417$ k $M_u = 12$ k·ft X won't control
  $P_u = 14$ k $M_u = 73$ k·ft $\rightarrow$ try

  $C_2 \rightarrow$ $P_u = 592$ k $M_u = 0$ $\rightarrow$ try
  $P_u = 354$ k $M_u = 55$ k·ft X won't control

  $C_3 \rightarrow$ $P_u = 508$ k $M_u = 57$ k·ft $\rightarrow$ try

  $\boxed{W14 \times 109}$ is more than sufficient, based on inspection.
  $\phi P_n = 1130$k $\phi M_n = 346$ k.ft
  Can try smaller, depending on drift.

- Beams

  $B_1 \rightarrow$ $P_u = 250$k $M_u = 47$ k·ft
  $P_u = 68$ k $M_u = 190$ k·ft
  $P_u = 230$k $M_u = 102$ k·ft
  $B_2 \rightarrow$ $P_u = 137$k $M_u = 125$ k·ft

  $\boxed{W24 \times 62}$ is more than sufficient

Calculation Braced Frame Page 2 of 3

• Bracings

$P_u = 324$ k on 1st Floor
$P_u = 240$ k on 2nd Floor
$P_u = 181$ k on 3rd Floor
$P_u = 113$ k on 4th Floor

} Controlling loads

Check for buckling.

$$F = \frac{\pi^2 E I}{(Kl)^2}$$

$$324 = \frac{\pi^2 (29{,}000)(I_{req})}{(23 \cdot 12)^2} \Rightarrow I_{req} = 86.3 \text{ in}^4$$

$$240 = \frac{\pi^2 (29{,}000)(I_{req})}{(21 \cdot 12)^2} \Rightarrow I_{req} = 53.3 \text{ in}^4$$

$$181 = \frac{\pi^2 (29{,}000)(I_{req})}{(21 \cdot 12)^2} \Rightarrow I_{req} = 40.2 \text{ in}^4$$

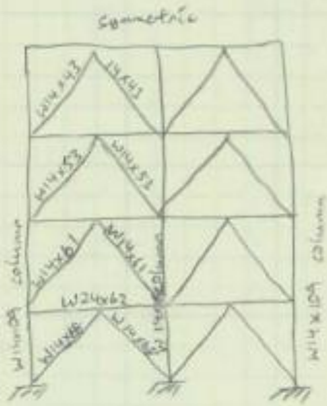
$$113 = \frac{\pi^2 (29{,}000)(I_{req})}{(21 \cdot 12)^2} \Rightarrow I_{req} = 25.1 \text{ in}^4$$

Buckling isn't much a problem.

1st Floor ⇒ W14x68 = $\phi P_n = 330$ K @ 24' unbraced length $I = 722$ in^4
2nd Floor ⇒ W14x61 = $\phi P_n = 345$ k @ 22' unbraced length $I = 640$ in^4
3rd Floor ⇒ W14x53 = $\phi P_n = 156$ k @ 22' unbraced length $I = 541$ in^4
4th Floor ⇒ W14x43 = $\phi P_n = 146$ k @ 22' unbraced length $I = 428$ in^4

Calculation Braced Frame | Thesis | Page 3 of 3

Preliminary sizes chosen for braced frame.



Final sizes will be chosen on ETAB design

Must be at least this size

Appendix F: Foundation Design

Mat Foundation | Thesis | Page 1 of 3

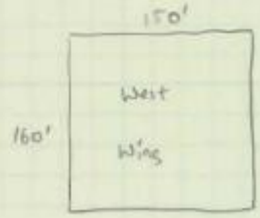
Designing a typical Mat Foundation

Simplified with assumptions due to lack of geotech report,

For west wing

Load Combos
D
D+L
D+W
D+.75W+.75L
0.6D+W

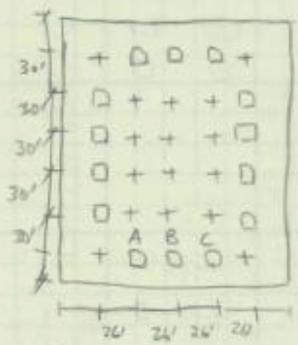
- Try $D_f = 10$ ft
- Assume soil has improvements to make $c_u = 2500$ lb/ft²
- Total dead + live load = 18000 k

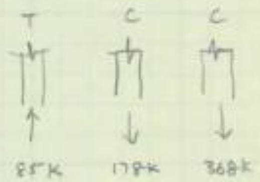


- We need to check for bearing and uplift in foundation design.
- 0.6D+W load combo controls foundation design because it causes an uplift on one side of the foundation.
- The other load combo may control for bearing.

Braced West Wing

□ = lateral load columns
+ = gravity columns





Column A needs to be designed for uplift
Column C needs to be designed for bearing capacity

Since it's a mat foundation, even its huge which lowers the stress on the soil.

Frame in the N-S direction does not control design.

$A = 150(160) = 24{,}000 \text{ ft}^2$

$I_x = \frac{1}{12}(150)(160)^3 = 51.2E^6 \text{ ft}^4$

Q = Axial load on Columns = 18,000 K

$I_y = \frac{1}{12}(160)(150)^3 = 45E^6 \text{ ft}^4$

$e_x = x' - \frac{B}{2} = .3$

$e_y = y' - \frac{L}{2} = .2$

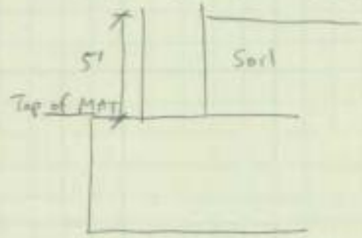
$$q = \frac{18{,}000}{24{,}000} \pm \frac{.3(18000)y}{45E^6} \pm \frac{.2(18000)x}{51.2E^6}$$

For $A = .75 - .09 - .06 = .6 = 600 \text{ lbs/ft}^2$

For $C = .75 + .09 + .06 = .9 = 900 \text{ lbs/ft}^2 < 2500 \text{ lbs/ft}^2$ ✓ $\frac{2500}{900} = F.S. = 2.8$

Mat Foundation | Thesis | Page 2 of 3

Amount of soil over column A to stop uplift.



Assume compact fill is $\gamma = 120\ \text{lb/ft}^2$

$4'(120)(30)(26) = 374\text{k}$ of soil keeping column A down, from uplift.

$$F.S. = \frac{85}{374} = 4.4$$

So bearing and uplift has been satisfied

Design foundation size and reinforcing

$$q_{au} = \frac{900 + 600}{2} = 750\ \text{lb/ft}^2$$

$$q_{au}BL = 750(26)(160) = 312\ \text{Kips}$$

Thickness of Mat Slab

* Assume load factor of 2.5

Critical Section is for diagonal tension shear at column A.

$$b_o = \left(0.5 + \frac{d}{2}\right) + \left(0.5 + \frac{d}{2}\right) + \left(0.5 + d\right) = 1.5 + 2d$$

$$U = 2.5\left(\frac{365}{4}\right) = 230$$

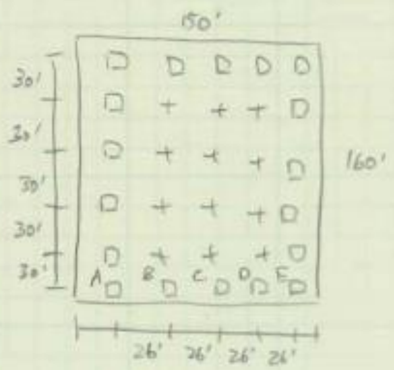
$$230{,}000 = (1.5 + 2d)(d)\left[0.85(.34)\sqrt{4000}\right]$$

$$15724 = 1.5d + 2d^2$$

$$d = 87'' \approx 7.3\ \text{ft} \approx \text{use } \boxed{7'\text{-}6''\ \text{ft}} \Rightarrow D_f = 11'\text{-}6''$$

Mat Foundation | Thesis | Page 3 of 3

For Moment West Wing



Bearing is less critical here since it is more spread out, So using a $D_f = 10'$ or 4' soil above MAT is sufficient

Thickness of MAT

Critical section at Column E with 284 k, Column D = 258 k

$U = (2.5)\left(\frac{284}{4}\right) = 162$

$162{,}000 = (1.5 + 2d)(d)\left[0.85(.34)\sqrt{4000}\right)\right]$

$8100 = 1.5d + 2d^2$

$d = 67.2'' \approx 5.6 \text{ ft} \approx \text{use } 6 \text{ ft} \Rightarrow D_f = 10'$

Reinforcing

Same as braced frame.

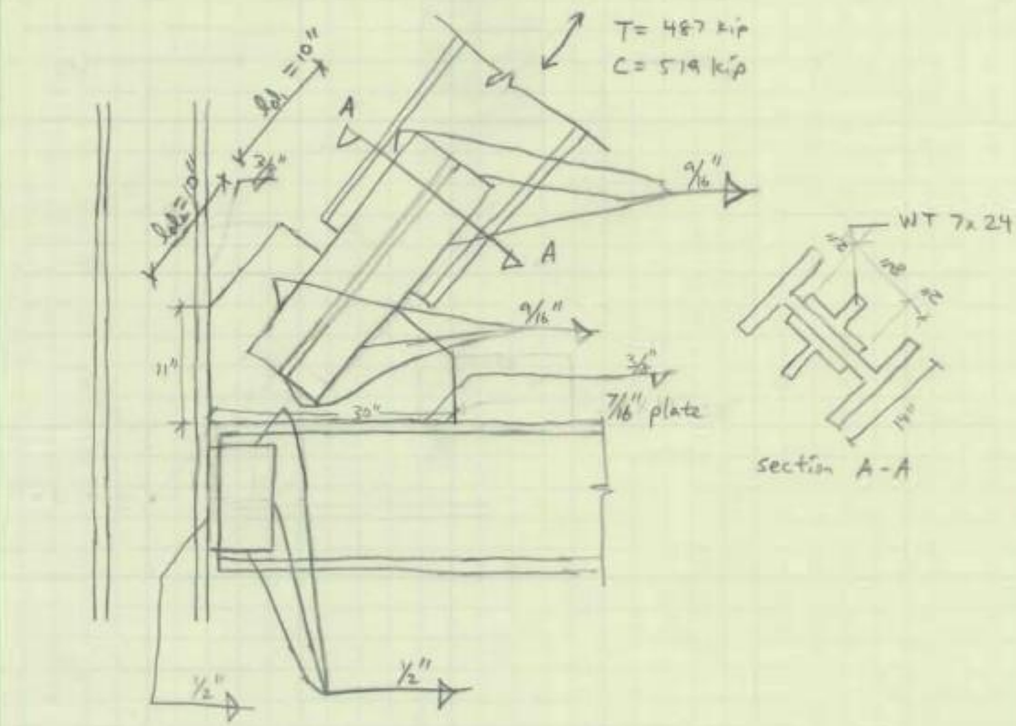
Summary

Typical Mat Foundation

	Braced	Moment	Current
F.S. for strength =	2.5	2.5	✓
F.S. for uplift =	4.4	not an issue	✓
F.S. for bearing =	2.8	2.8	✓
Depth into Earth =	11'-6"	10'	8'-8"
Thickness of Mat =	7'-6"	6'	4'

Appendix G: Welded Braced Connection Design

Connections | Thesis | Page 1 of 4



W14x74
A=21.8
t_w=.45
t_f=.785

WT 7x24
A= 7.07
t_w=.34
t_f=.595
b_f = 8.03
d=6.9

A: Limit States for Brace, W14x74

WT 7x37 $\bar{y}$=1.32

$U = 1 - \frac{1.32}{10} = .868$

Tension Yielding: $\phi R_n = 0.9(50)(21.8) = 981^k$

Tension Rupture: $\phi R_n = 0.75(65)(.868)(21.8) = 922^k$

Weld Rupture: $\phi R_n = 0.75(0.6)(70)(.707)(7/16)[9(20) + 9(8)](2)$

$= 1.392[9(20)+9(8)](2) = 701^k$

'or' take max

$= 1.392[0.85(9)(20) + 9(8)(1.5)](2) = 726^k$

both ok!

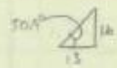
Base metal: $\phi R_n = 0.75(0.6)(65)(.45'')(28)(2) = 789^k$

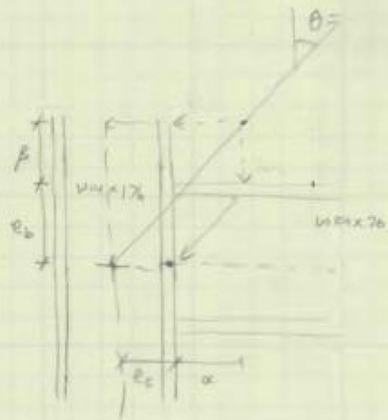
Block Shear: $\phi R_n = 0.75[(0.60)(65)(20) + 1.0(65)(8)](2) \le 0.75[(0.60)(50)(20) + (65)(8)](2)$

$= 1740^k \le 1344^k = 1344^k$ controls good!

Controlling $= 726^k > 519\ k$ ok

Page 2 of 4





$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$

$\alpha = \frac{30}{2} = 15''$

$\beta = \frac{11}{2} = 5.5''$

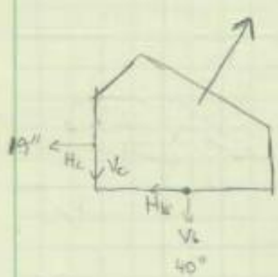
$e_b = 11.9''$

$e_c = 6.1$ in

$\frac{e_c + \alpha}{e_b + \beta}$

$\frac{6.1+15}{11.9+5.5} = \tan\theta$

$\theta = 50.5 \approx 50.9$ ok



$r = \sqrt{(6.1+15)^2 + (11.9+5.5)^2}$

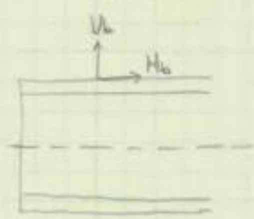
$r = 27.3$

$V_c = \frac{5.5}{27.3}(519) = 104.5^k$

$H_c = \frac{6.1}{27.3}(519) = 116^k$

$H_b = \frac{15}{27.3}(519) = 285^k$

$V_b = \frac{11.9}{27.3}(519) = 226^k$



B: Brace – to – Gusset WT_s

Tension Yielding = $\phi R_n = 0.9(50)(7.07)(2) = 636^k$ ok

Tension Rupture = $\phi R_n = 0.75(65)(7.07)(2)(1) = 780^k$

Weld Rupture = "same" = $\phi R_n = 726^k$

Base Metal = $0.75(0.6)(65)(.65)(26)(2) = 1064^k$

Block Shear = Does not control by inspection.

Compression = $\phi R_n = 0.90(50)(7.07)(2) = 636^k$ $\quad \frac{KL}{r} = \frac{20.5}{1.88} = 10.9 < 25$

Controlling = $636^k > 519^k$ ok

C: Gusset Plate

Gusset

$A \approx 19.9 in^2$

Tension Yielding = $\phi R_n = 0.9(36)(19.9) = 645^k$

Tension Rupture = $\phi R_n = 0.75(58)(19.9) = 865^k$

Weld Rupture = "same" when WT is welded 10" on gusset plate.

$\phi R_n = 726^k$

Base Metal = $\phi R_n = 658^k$ "same thickness"

Block Shear = $\phi R_n = 1344^k$

Compression = $\phi R_n = 0.90(36)(19.9) = 645^k$

Controlling = $645^k > 519^k$ ok

D: Gusset to Beam Connection (look at pg 2 for forces)

W 24x76

$A = 22.4$

$d = 23.9$

$t_w = .44$

$t_f = .68$

Weld Rupture: $\phi R_n = 1.392\left((2)(6)(30)(1.5)\right) = 751^k > 226^k = V_b$ ✓

: $\phi R_n = 1.392(2)(6)(30)(1.0) = 501^k > 285^k = H_b$ ✓

Beam web yielding

$\phi R_n = 1.0(5(1.18) + (30))(50)(.44) = 789.8^k$ ok

Beam web crippling

$$\phi R_n = 0.75(0.80)(.44)^2\left[1 + 3\left(\frac{30}{23.9}\right)\left(\frac{.44}{.68}\right)^{1.5}\right]\sqrt{\frac{29{,}000(50)(.68)}{.44}}$$

$= 521^k$ ok

E: Gusset to Column Connection

W14x176

$A = 51.8$

$k_{des} = 1.91$

$t_w = .83$

$t_f = 1.31$

$d = 15.2$

Weld Rupture: $\phi R_n = 1.392\left((2)(6)(11)(1.5)\right) = 275^k > 116^k$ ✓

: $\phi R_n = 1.392(2)(6)(11) = 183^k > 105^k$ ✓

F: Beam to Column Connection (Double Angle 2L 4x4x½x16)

weld Rupture due to eccentricity shear

$C = 2.65, \ C_1 = 1.0, \ D = 7/16"$

$\phi R_n = .75(1.0)(2.65)(7)(16)(2) = 445.2^k > 251^k$

ok.

$l = 16"$

$k = \frac{3.5}{16} = .22$

$x = .033$

$xl = .53"$

$a = \frac{4 - .53}{16} = .216$

Page 4 of 4

F: continue

Beam Web Strength at Weld

$$\phi V_n = \frac{0.75(.6)(65)(.44)(1.0)}{1.392(7)(1.0)}(445.2) = 588^k > 68^k$$

Angle: Tension Yield $\phi R_n = 0.9(36)(16)(1/2)(2) = 518.4^k$

Tension Rupture $\phi R_n = .75(58)(16)(1/2)(2) = 696^k$

Column Weld Rupture Strength.

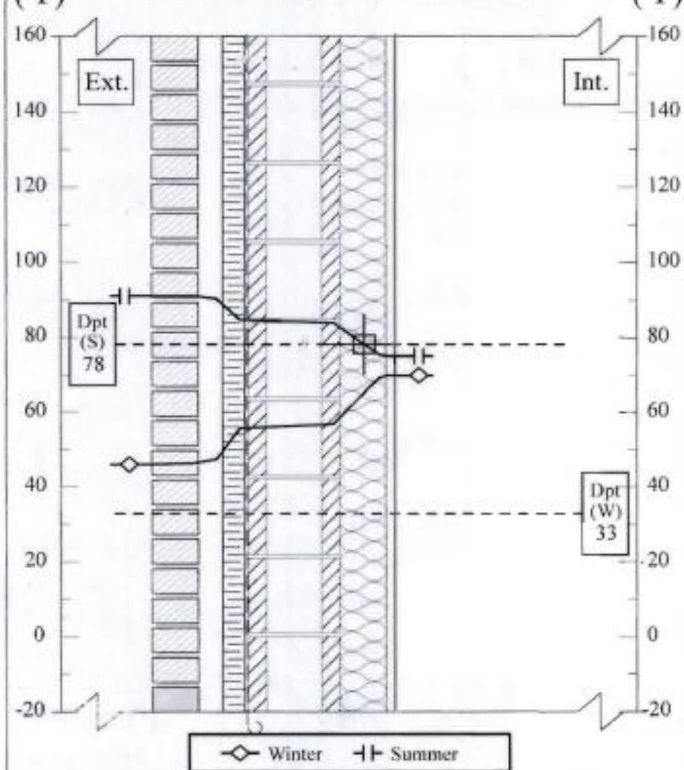
$$\phi V_n = \frac{2(16)^2(1.392)(8)}{\sqrt{(16)^2 + 12.96(4.42)^2}} = 252^k > 68^k \quad e = 4 + \frac{.83}{2} = 4.42$$

Appendix H: Façade Breadth

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	TCMC Wall Analysis
Number	
City	Miami, FL
Date	3/20/2013
Analysis by:	Xiao Ye Zheng
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ex
Temp (°F)	70	46	75	91
RH (%)	25	60	50	64
DPT (°F)	33	33	56	78

PENNSYLVANIA STATE UNIVERSITY

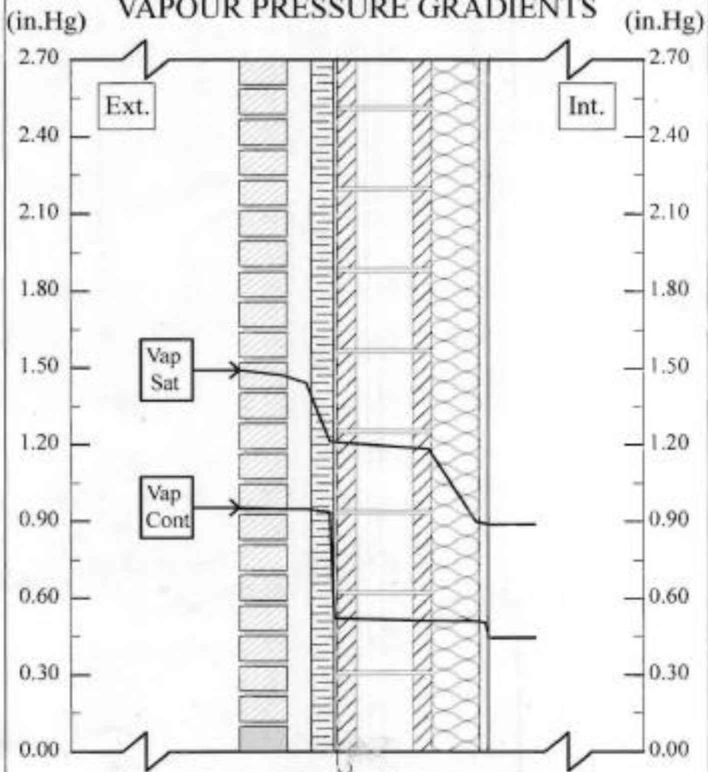
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 1680

	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Tem (°F)
1	brick, (vented), 4 in.	No Recor...	Generic...	3.50	0.64	46.3	91.0
2	cavity, 2 in.	No Recor...	Generic...	2.00	0.98	47.4	90.3
3	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	2.00	7.90	55.6	84.7
4	poly film, (6mil)	No Recor...	Generic...	0.01	0.12	55.7	84.7
5	block, 8 in.	No Recor...	Generic...	8.03	1.03	56.8	83.9
6	batt ins., 4 in.	No Recor...	Generic...	4.00	12.19	69.4	75.4
7	gypsum bd., 5/8 in., (#1)	No Recor...	Generic...	0.63	0.46	69.9	75.1
8	paint (#1), (12mil)	No Recor...	Generic...	0.01	0.12	70.0	75.0
	Total or (Layer 0)			20.18	23.44	(45.7)	(91.4

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name

Number

City Miami, FL

Date

Analysis by:

Wall Type Option

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext
Temp (°F)	–	–	75	91
RH (%)	–	–	50	64
DPT (°F)	–	–	56	78

PENNSYLVANIA STATE UNIVERSITY

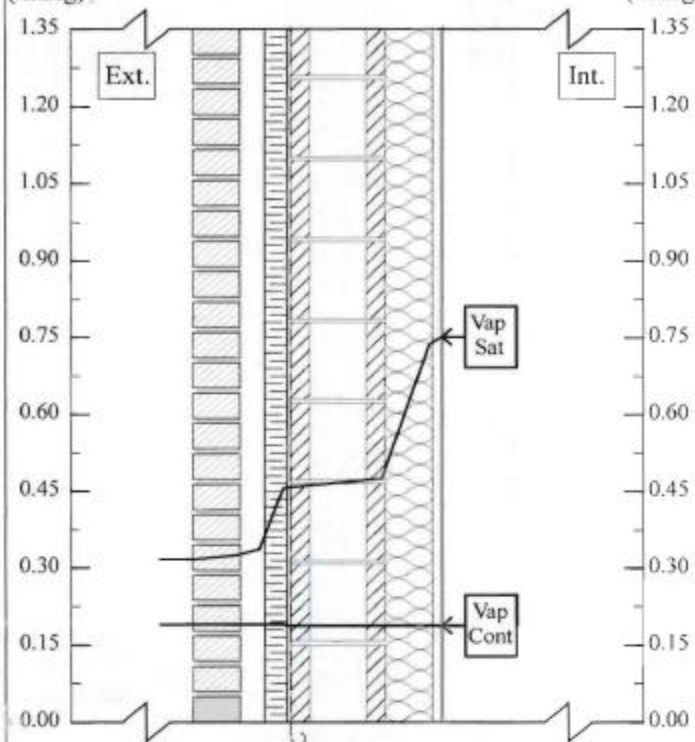
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 1680

	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCo (in.Hg
1	brick, (vented), 4 in.	No Recor...	Generic...	0.191	90.6	1.449	0.935
2	cavity, 2 in.	No Recor...	Generic...	0.016	89.9	1.419	0.935
3	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	0.515	84.5	1.195	0.922
4	poly film, (6mil)	No Recor...	Generic...	16.827	84.4	1.192	0.515
5	block, 8 in.	No Recor...	Generic...	0.418	83.7	1.166	0.505
6	batt ins., 4 in.	No Recor...	Generic...	0.040	75.4	0.887	0.504
7	gypsum bd., 5/8 in., (#1)	No Recor...	Generic...	0.229	75.1	0.878	0.498
8	paint (#1), (12mil)	No Recor...	Generic...	2.488	75.0	0.876	0.438
9							
10							
11							
12							
	TOTAL or (Layer 0)			20.810	(91.0)	(1.469)	(0.940

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name

Number

City Miami, FL

Date

Analysis by:

Wall Type Option

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext
Temp (°F)	70	46	–	–
RH (%)	25	60	–	–
DPT (°F)	33	33	–	–

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UNIVERSITY PARK, PA, USA, 1680

	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCo (in.Hg
1	brick, (vented), 4 in.	No Recor...	Generic...	0.191	46.7	0.320	0.187
2	cavity, 2 in.	No Recor...	Generic...	0.016	47.7	0.332	0.187
3	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	0.515	55.7	0.448	0.187
4	poly film, (6mil)	No Recor...	Generic...	16.827	55.9	0.450	0.185
5	block, 8 in.	No Recor...	Generic...	0.418	56.9	0.468	0.185
6	batt ins., 4 in.	No Recor...	Generic...	0.040	69.4	0.725	0.185
7	gypsum bd., 5/8 in., (#1)	No Recor...	Generic...	0.229	69.9	0.737	0.185
8	paint (#1), (12mil)	No Recor...	Generic...	2.488	70.0	0.740	0.185
9							
10							
11							
12							
	TOTAL or (Layer 0)			20.810	(46.0)	(0.312)	(0.187

Appendix I: Solar Panel Breadth

Panasonic
ideas for life

HIT® Photovoltaic Module

HIT® Power 220A

VBHN220AA01

HIT Delivers More Real World Performance

19.8 % cell conversion efficiency

Hybrid cell produces the highest output on cloudy days

Highest warranted tolerance: -0/+10 %

Most PTC Watts: 204.4

Lowest temperature coefficient: -0.33%

Highest PTC/STC Ratio: 93%+

HIT® Power
Photovoltaic Module

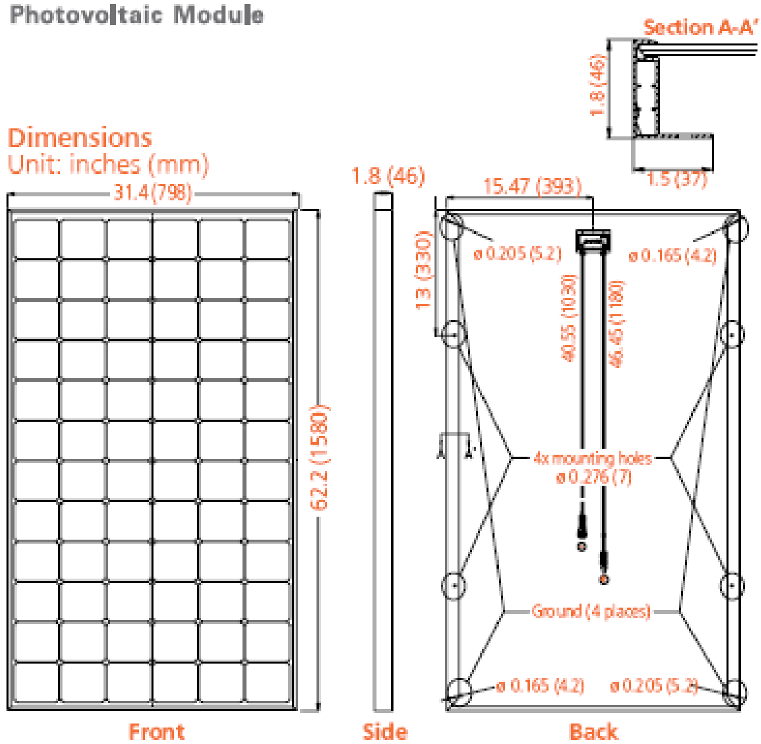
High Efficiency
HIT® Power solar panels are leaders in sunlight conversion efficiency. Obtain maximum power within a fixed amount of space. Save money using fewer system attachments and racking materials, and reduce costs by spending less time installing per Watt.

Power Guarantee
The power ratings for HIT Power panels guarantee customers receive 100% of the nameplate rated power (or more) at the time of purchase, enabling owners to generate more kWh per rated Watt, quicken investments returns, and help realize complete customer satisfaction.

Temperature Performance
As temperatures rise, HIT Power solar panels produce 10% or more electricity (kWh) than conventional crystalline silicon solar panels at the same temperature.

Valuable Features
The packing density of the panels reduces transportation, fuel, and storage costs per installed watt.

American Made Quality
Our silicon wafers located inside HIT solar panels are made in Oregon, and the panels are assembled in an ISO 9001 (quality), 14001 (environment), and 18001 (safety) certified factory. Unique eco-packing minimizes cardboard waste at the job site. The panels have a Limited 20-Year Power Output and 10-Year Product Workmanship Warranty.

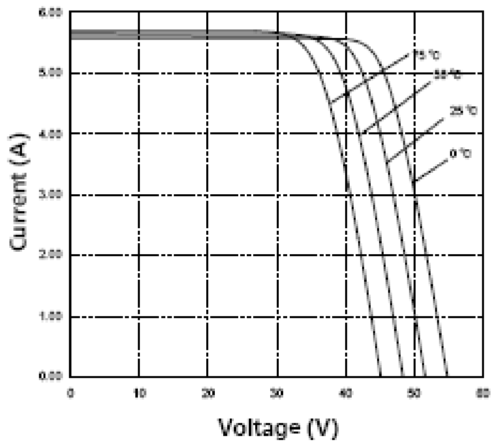


HIT® Power 220A

Electrical Specifications

Model	HIT Power 220A or VBHN220AA01
Rated Power (Pmax)[1]	220 W
Maximum Power Voltage (Vpm)	42.7 V
Maximum Power Current (Ipm)	5.17 A
Open Circuit Voltage (Voc)	52.3 V
Short Circuit Current (Isc)	5.65 A
Temperature Coefficient (Pmax)	-0.336%/ °C
Temperature Coefficient (Voc)	-0.145 V/ °C
Temperature Coefficient (Isc)	1.98 mA/ °C
NOCT	114.8°F (46°C)
CEC PTC Rating	204.4 W
Cell Efficiency	19.8%
Module Efficiency	17.4%
Watts per Ft.²	16.22 W
Maximum System Voltage	600 V
Series Fuse Rating	15 A
Warranted Tolerance (-/+)	-0% / +10%

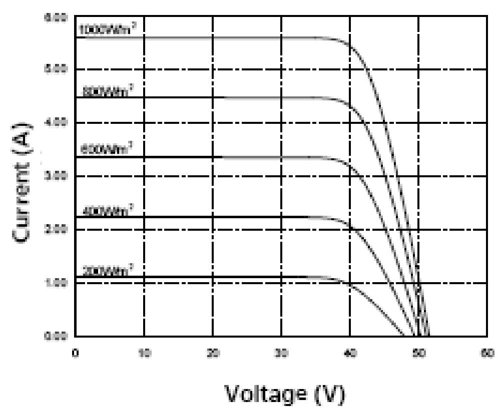
Dependence on Temperature



Mechanical Specifications

Internal Bypass Diodes	3 Bypass Diodes
Module Area	13.56 Ft² (1.26m²)
Weight	35.3 Lbs. (16kg)
Dimensions LxWxH	62.2x31.4x1.8 in. (1580x798x46 mm)
Cable Length +Male/-Female	46.45/40.55 in. (1180/1030 mm)
Cable Size / Type	No. 12 AWG / PV Cable
Connector Type[3]	Multi-Contact® Type IV (MC4™)
Static Wind / Snow Load	60PSF (2880Pa) / 39PSF (1867Pa)
Pallet Dimensions LxWxH	63.2x32x72.8 in. (1607x815x1850 mm)
Quantity per Pallet / Pallet Weight	35 pcs./1322.7 Lbs (600 kg)
Quantity per 53' Trailer	980 pcs.

Dependence on Irradiance



Operating Conditions & Safety Ratings

Ambient Operating Temperature[2]	-4°F to 115°F (-20°C to 46°C)
Hail Safety Impact Velocity	1" hailstone (25mm) at 52 mph (23m/s)
Fire Safety Classification	Class C
Safety & Rating Certifications	UL 1703, cUL, CEC
Limited Warranty	10 Years Workmanship, 20 Years Power Output

[1] STC: Cell temp. 25°C, AM1.5, 1000W/m²
[2] Monthly average low and high of the installation site.
Note: Specifications and information above may change without notice.
[3] Safety locking clip (PV-SSH4) is not supplied with the module.

"HIT" is a registered trademark of Panasonic Group. The name "HIT" comes from "Heterojunction with intrinsic Thin-layer" which is an original technology of Panasonic Group.

⚠ **CAUTION!** Please read the installation manual carefully before using the products.

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Unit of SANYO North America Corporation

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Cupertino, CA 95014
Phone 408-861-8424
Fax 408-861-3990
http://www.panasonic.com/solar

Panasonic®

SB 3300 / 3800 / 3800/V

Powerful
- Efficiency up to 95.6 %
- OptiCool active temperature management
- The best tracking efficiency with OptiTrac MPP tracking

Safe
- Galvanic isolation
- Integrated ESS DC load-disconnecting unit
- Rated nominal power at temperatures up to 45 °C

Flexible
- For indoor and outdoor installation
- Suitable for generator grounding

SUNNY BOY 3300 / 3800

The generalist

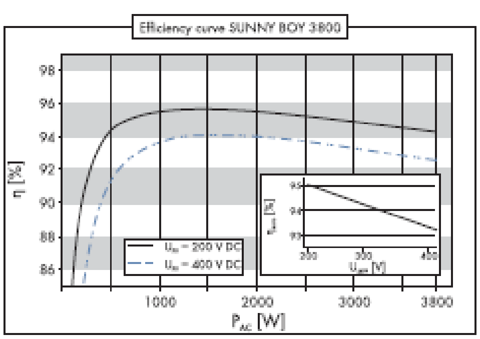
It is robust, easy-to-handle, and, thanks to its galvanic isolation, used in all kinds of AC grids: the Sunny Boy 3300 / 3800. Due to its suitability for generator grounding, it can be combined with all module types. The generously-proportioned die-cast aluminum housing together with the OptiCool active cooling system guarantee the highest yields and a long service life, even under extreme conditions.

Technical Data
SUNNY BOY 3300 / 3800 / 3800/V

	SB 3300	SB 3800	SB 3800/V*
Input (DC)			
Max. DC power	3820 W	4040 W	4040 W
Max. DC voltage	500 V	500 V	500 V
PV-voltage range, MPPT	200 V – 400 V	200 V – 400 V	200 V – 400 V
Max. input current	20 A	20 A	20 A
Number of MPP trackers	1	1	1
Max. number of strings (parallel)	3	3	3
Output (AC)			
Nominal AC ouput	3300 W	3800 W	3680 W
Max. AC power	3600 W	3800 W	3680 W
Max. output current	18 A	18 A	16 A
Nominal AC voltage / range	220 V – 240 V / 180 V – 260 V	220 V – 240 V / 180 V – 260 V	220 V – 240 V / 180 V – 260 V
AC grid frequency (self-adjusting) / range	50 Hz / 60 Hz / ±4.5 Hz	50 Hz / 60 Hz / ± 4.5 Hz	50 Hz / 60 Hz / ± 4.5 Hz
Phase shift (cos φ)	1	1	1
AC connection	single-phase	single-phase	single-phase
Efficiency			
Max. efficiency / Euro-Eta	95.2 % / 94.4 %	95.6 % / 94.7 %	95.6 % / 94.7 %
Protection devices			
DC reverse polarity protection	●	●	●
ESS DC load-disconnecting switch	●	●	●
AC short-circuit protection	●	●	●
Ground fault monitoring	●	●	●
Grid monitoring (SMA Grid Guard)	●	●	●
Galvanically isolated	●	●	●
General Data			
Dimensions (W / H / D) in mm	450 / 352 / 236	450 / 352 / 236	450 / 352 / 236
Weight	38 kg	38 kg	38 kg
Operating temperature range	–25 °C ... +60 °C	–25 °C ... +60 °C	–25 °C ... +60 °C
Noise emission (typical)	≤ 40 dB(A)	≤ 42 dB(A)	≤ 42 dB(A)
Consumption: operating (standby) / night	< 7 W / 0.1 W	< 7 W / 0.1 W	< 7 W / 0.1 W
Topology	LF transformer	LF transformer	LF transformer
Cooling concept	OptiCool	OptiCool	OptiCool
Mounting location: indoors / outdoors (IP65)	●/●	●/●	●/●
Features			
DC connection: MC3 / MC4 / Tyco	●/○/○	●/○/○	●/○/○
AC connection: plug connector	●	●	●
LCD	●	●	●
Interfaces: Bluetooth / RS485	○/○	○/○	○/○
Warranty: 5 years / 10 years	●/○	●/○	●/○
Certificates and approvals	www.SMA.de	www.SMA.de	www.SMA.de
Certificate number (please include when ordering)	–	–	V0153

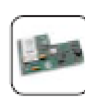
● Standard ○ Optional

Data at nominal conditions – Last update: March 2009
*Version for country requirements in accordance with EN 50438 with I_{AC} = 16 A



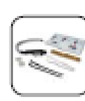
Accessories

RS485 interface of type 485PB-NR



Bluetooth Piggy-Back

Grounding-Kit "Positive" ESHV-P-NR



Grounding-Kit "Negative" ESHV-P-NR