The Commonwealth Medical College

Scranton, PA

Technical Report One
2012

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Structural Option

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Executive Summary

The purpose of Technical Report One is to fully understand the building the student is working on. A structural analysis will be done. It was found that TCMC is an ordinary steel moment frame building that will resist lateral loads with its rigid moment connections. No shear walls were used in the building. The west and the east wing had different foundations; the west wing is supported by a mat slab foundation and the east wing is supported by drilled caissons foundation. The main structural framing system is the W-shape A992 steel. The west wing uses a 7.5" thick concrete slab with steel deck and the east wing uses 5.25" thick concrete slab with steel deck.

Gravity loads in the building are found and calculated in this report. The design dead and live loads are found either by independent calculation or are given in the structural drawing, sheet S201A. This live load is compared to the minimum required load in ASCE 7-05. To further the analysis, spot checks were done on a typical column, beam, and slab. The column had failed, by only a small amount. This is because all the loads obtained are assumed to be more conservative. The beam and the slab are adequate.

Lateral Loads were calculated using ASCE 7-05 and ASCE 7-10. It was found that wind load in the end, controlled over seismic load. Wind load causes a base shear of 455 kips and an overturning moment of 22,272 kip-ft. Seismic however, only caused a base shear of 244kips and an overturning moment of 13,650 kip-ft. Base shear and overturning moments are not found on any of the drawings provided so it will not be compared to the actual designed base shear and overturning moment. Further lateral load analysis will be performed later on during this project.

The appendices section in this report shows all the hand calculations performed. Also, the appendices contain additional drawings and plans that will help you further understand the structure.

Building Introduction

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

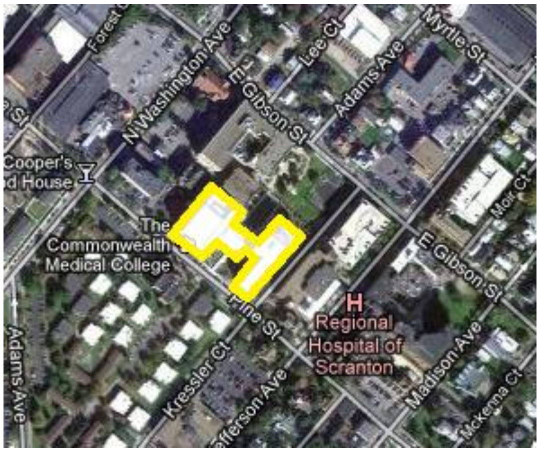


Figure 1 Aerial map from Google.com showing the location of the building site

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

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

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

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

Structural Overview

Design Codes

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

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

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

Additional Code Reference from American Concrete Institute:

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

Regulatory Guidelines and Standards

Accessibility	ICC/ANSI A117.1 1998

Material Properties

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

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

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

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

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

Foundations

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

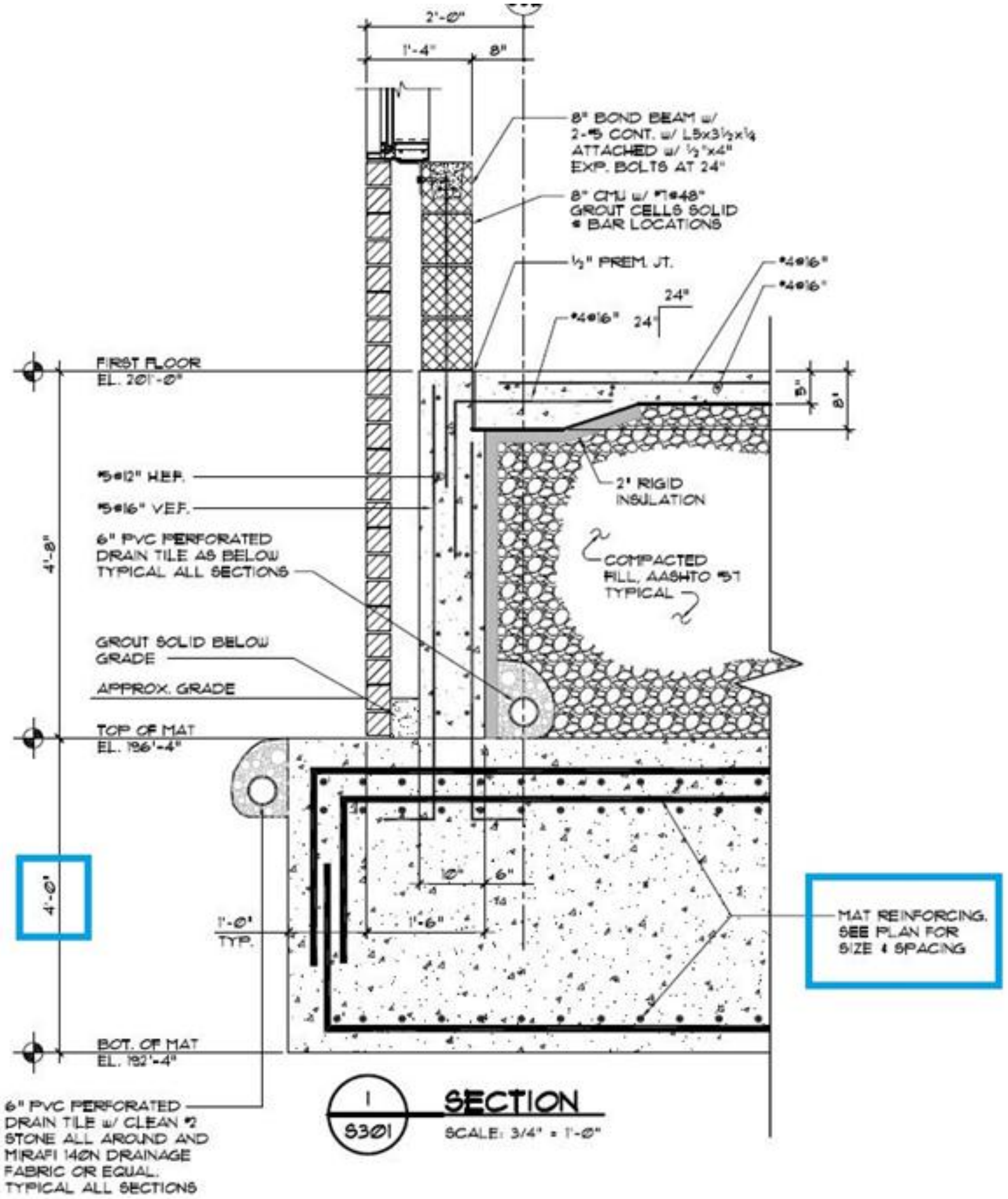


Figure 5 A typical Section cut showing the mat slab foundation. Courtesy of Highland Associates

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

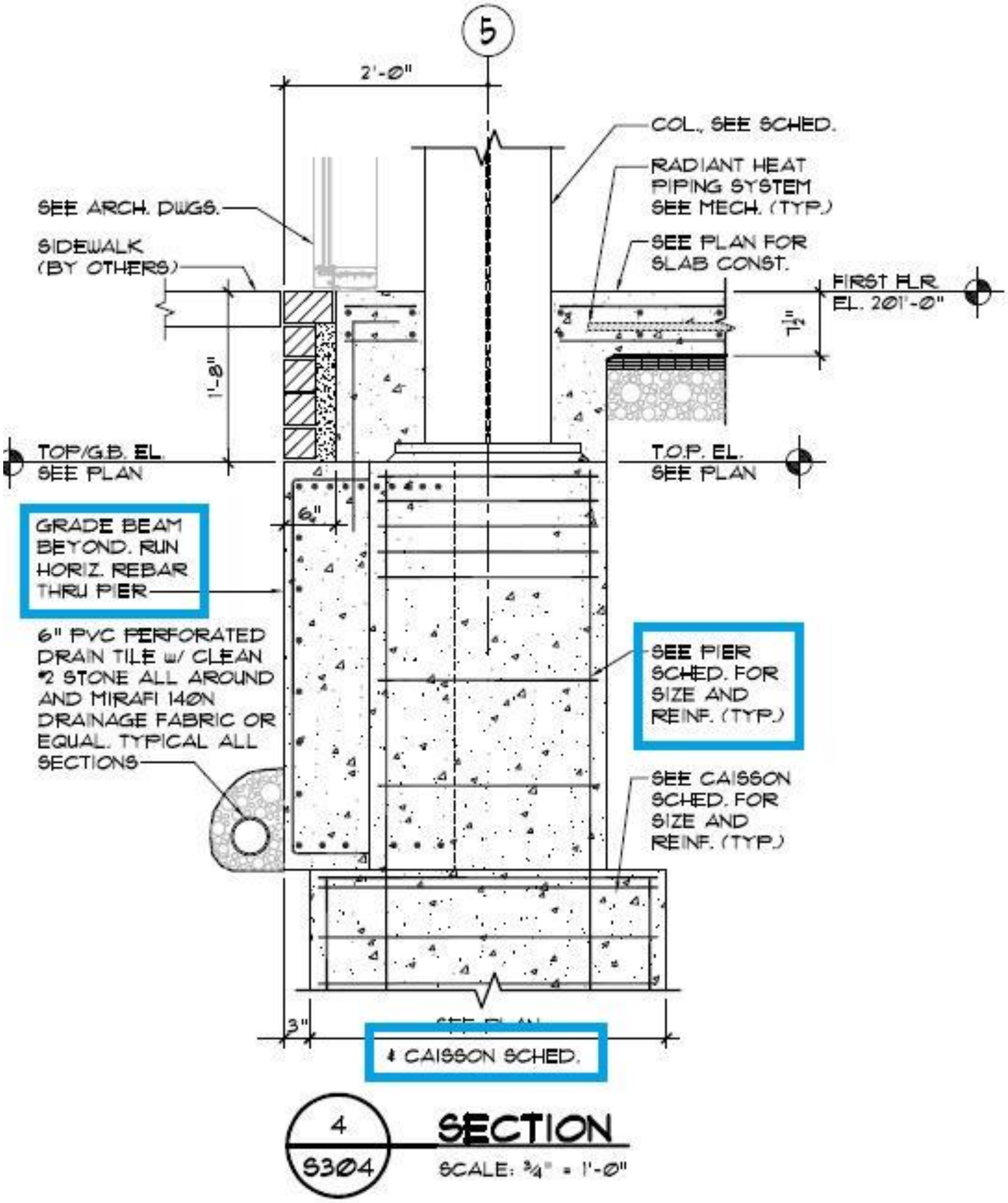


Figure 6 A section cut of a drilled caisson foundation. Courtesy of Highland Associates

Floor Systems

The existing floor system of the TCMC is held up by W-shaped steel columns and composite steel beams. Figure 14 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 14.

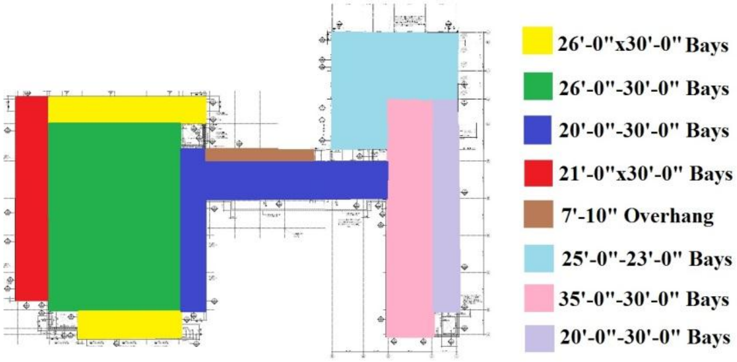
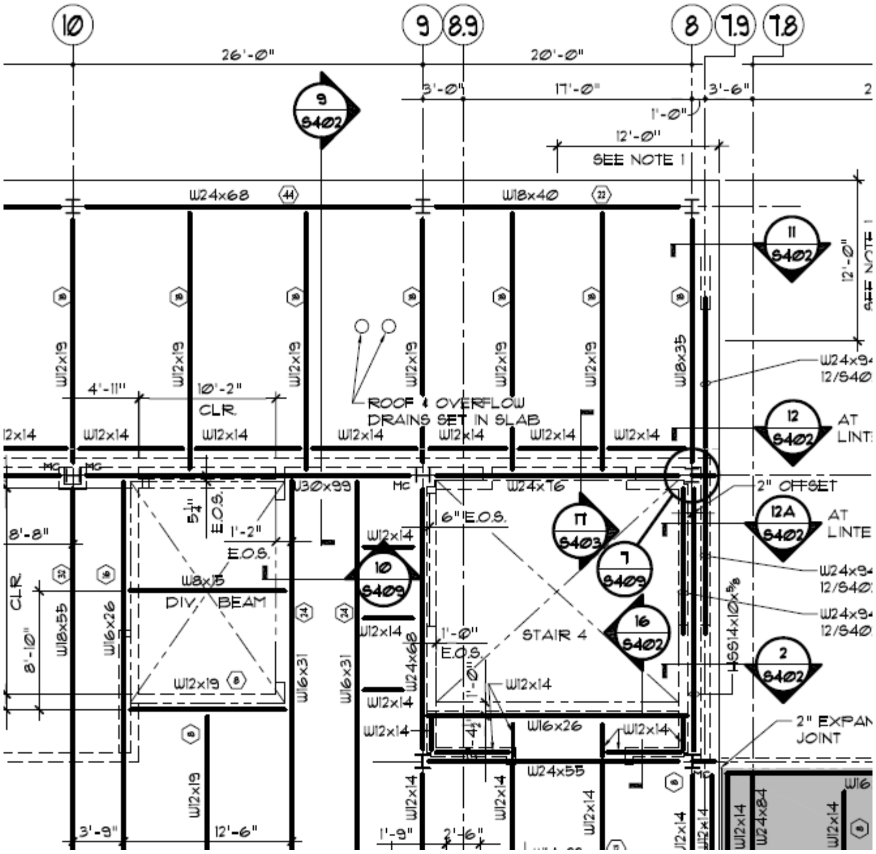


Figure 14 Different Bay sizes respective to their color

The floor is composite steel deck with concrete topping. The typical floor plan in the west wing is shown in Figure 15 along with two section cuts, Figures 16 and 17. 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".



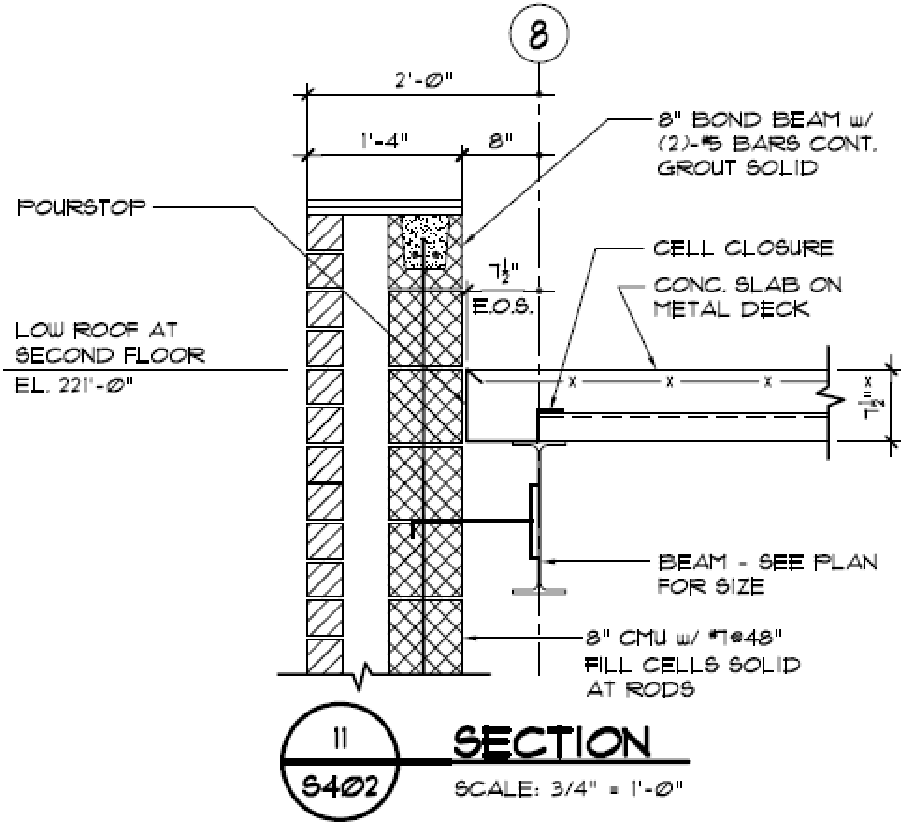


Figure 8 Section cut 11 from figure 7

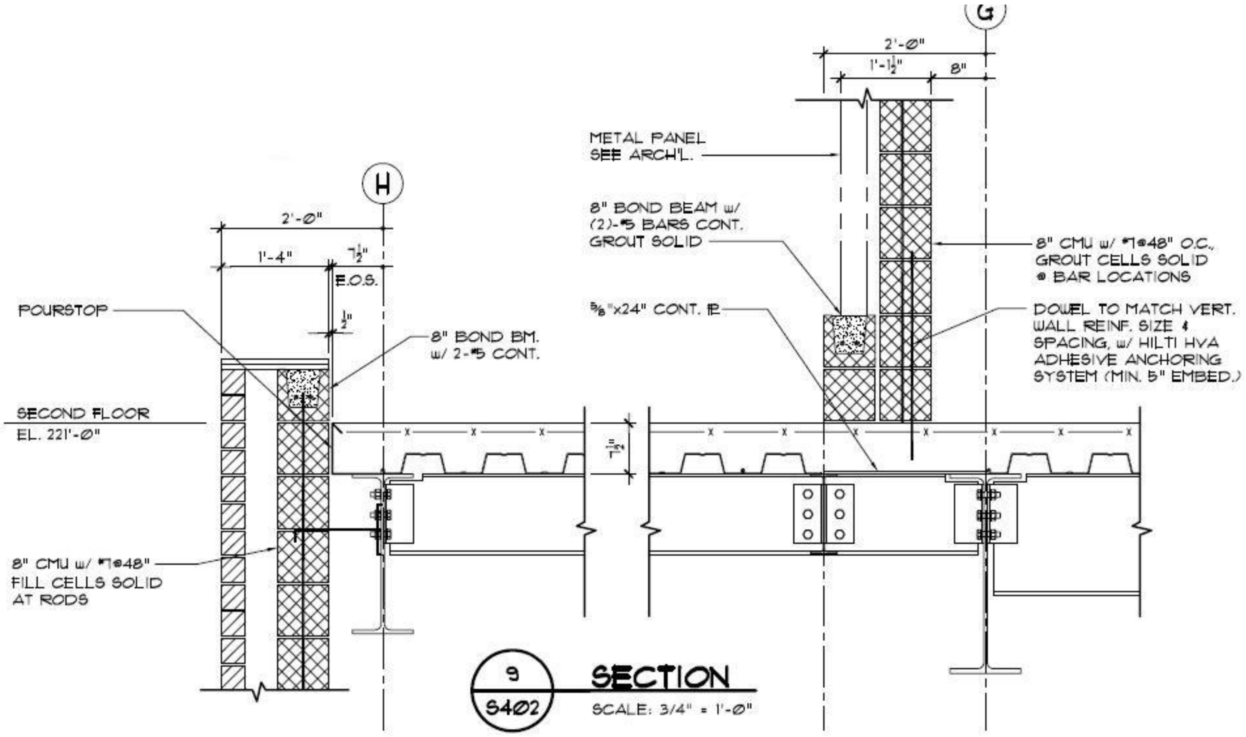


Figure 9 Section cut 9 from figure 7

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 7.1. 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 7.2 shows the only four frames that extend to the roof of the penthouse.

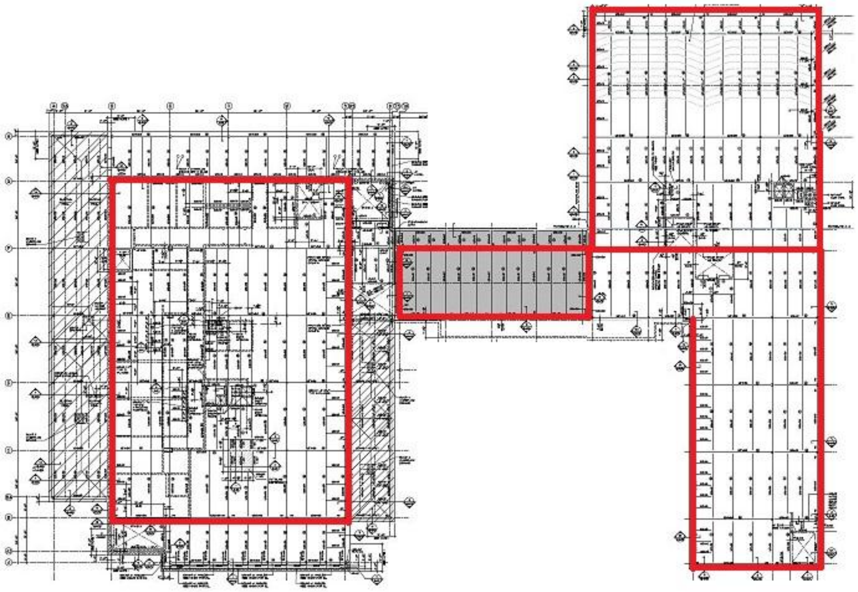


Figure 7.1 Locations of Moment Frames at TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

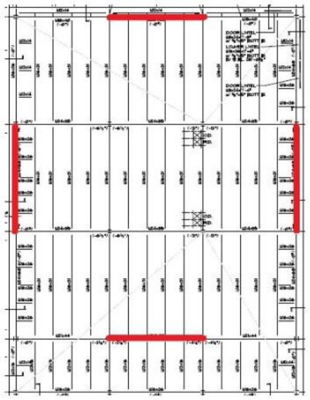


Figure 7.2 Locations of Moment Frames at the Penthouse of TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

Roof Systems

TCMC has over 9 different roof heights, as shown in figure 8, 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’.

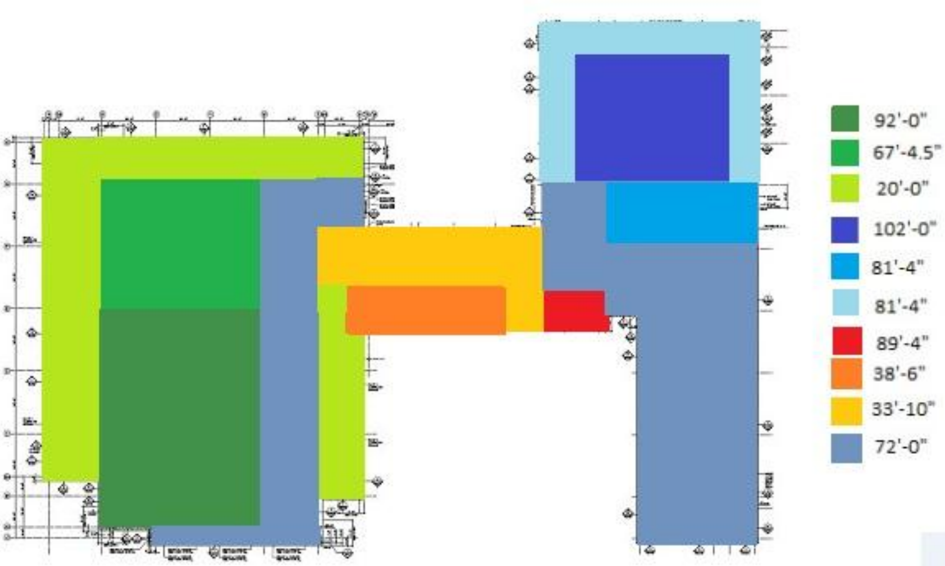


Figure 8 Plan showing the different roof heights; the darker, the higher.

The main roof is constructed of 1.5” type B wide rib, 22 gauge, painted roof deck supported by W-shape framing. A typical roof section cut is shown on figure 9. 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.

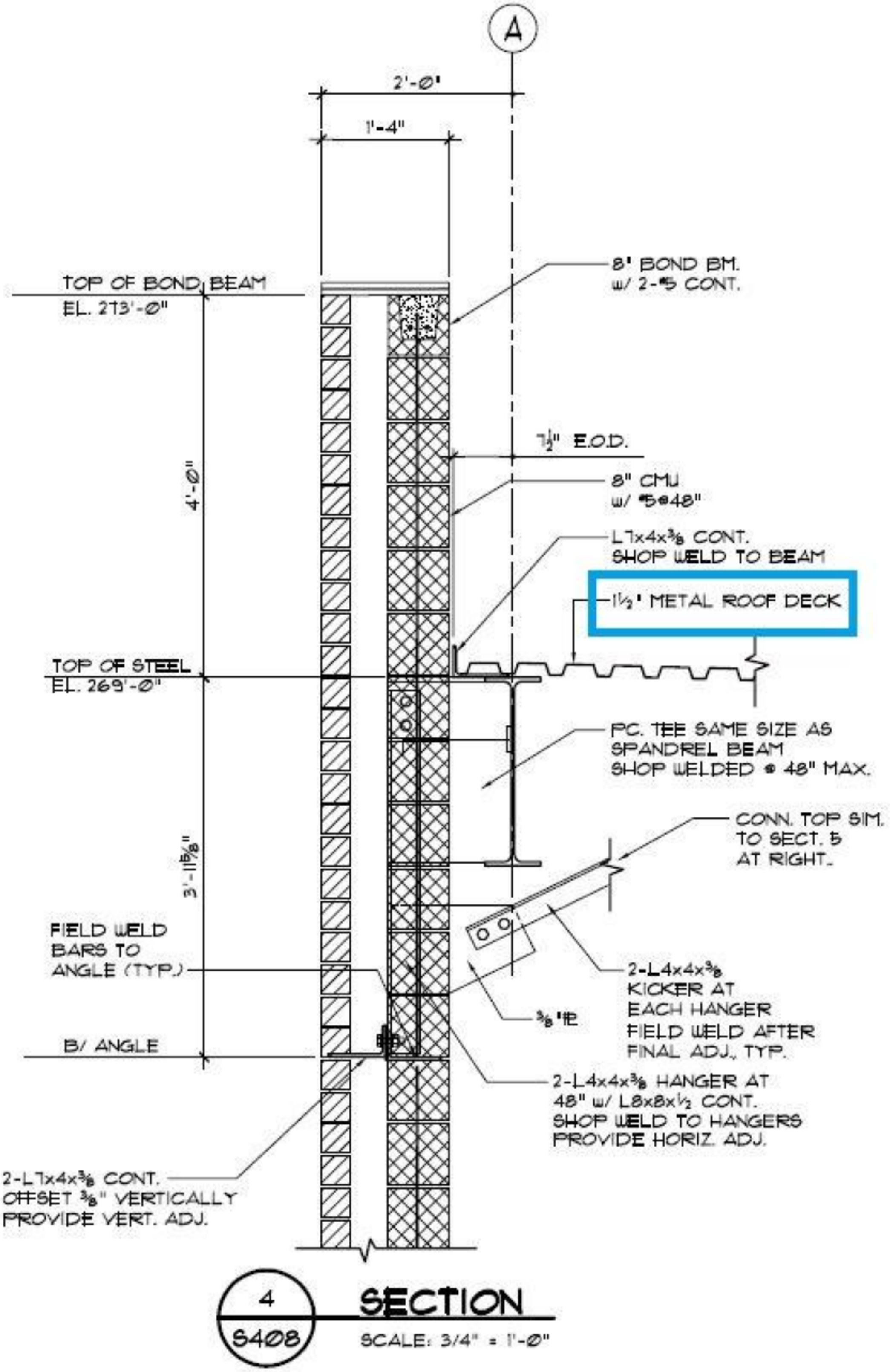


Figure 15 Typical roof section cut showing the roof deck

Gravity Loads

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

Dead and Live Loads

For the dead load calculations, the materials that have the most impact on the dead weight of the building were found and then calculated. The west wing primarily uses composite 3" steel deck with concrete slab that weighs 75 psf according to Vulcraft Steel Deck catalog. The east wing and the hallway use 2" steel deck, lightweight concrete, so it only weights 42 psf. Then W-shape Steel Beams and Columns are assumed as 15 psf that covers that whole entire building. The heaviest exterior wall is chosen and is assumed throughout the building at 1000plf. Then these weights are multiplied by the area or the length that they occupied in to get the weight in pounds. A sample of this calculation is shown for the 2nd floor of the TCMC in Figure 10 below. Doing this for every level, a weight in psf and lbs are both obtained. Then the total dead weight is found to be around 22,378 kips and will be used later in seismic calculations. A breakdown of the weight per Level is shown in Figure 11.

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

Figure 10 Total Weight per square foot of TCMC

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

Figure 11 Total Weights per Level of TCMC

The design live load for the TCMC can be found in the drawings on sheet S201A and S201B. A comparison of it to the minimum live load requirement from ASCE 7-05 can be seen on Figure 12. 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	ASCE 7-05 Live	Notes
	Load (psf)	Load (psf)	
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	ASCE 7-05 Live	Notes
	Load (psf)	Load (psf)	
Offices above 1st	65	50	Partitions and some heavier office equipment
Lobbies/ Corridors	100	100	
Corridors above 1st	80	80	
Stairs	100	100	
Classrooms	50	40	
Sorage above 1st	125	125	
Restrooms above 1st	75	N/A	
Auditorium	100	100	if seats are fixed, then only 60psf
Bookstore	150	N/A	
Lecture Halls	60	N/A	
Mechanical Room	150	N/A	
Library	75	N/A	
1st floor offices	65	50	
1st floor restrooms	75	N/A	
Roof	30	20	
Mechanical Roof	30	N/A	
1st floor storage	125	100	

Figure 12 Design live load is compared to ASCE 7-05, required live load

Snow Loads

The variables needed for snow load calculations are found on sheet S201B of the drawings. Figure 13 shows all the loads and variables that are from Sheet S201B of the structural drawing. Also, because of the many different roof heights, snow drifts can happen in over 10 different areas of the building. One of these areas is calculated and shown under Appendix A, snow load calculations. The result of that area is that the snow acuminated in the corner reached over 73 psf, more than double the amount compared to the regular flat roof amount of 30 psf. Snow drift is an important factor when designing TCMC.

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

Figure 13 Variable for snow load obtained from S201B

Gravity Check for Column E-10

The column E-10 is chosen because it represents a typical interior column within the building. Figure 21 shows the location of the column, within the red circle. The column is only 40' tall so it ends a little over the 3rd floor. After that, it is rigidly connected to another column directly above as it reach higher, all the way to the roof of the penthouse. The loads of each level are calculated, using the design dead and live load given in the structural drawings. These calculations are shown in Appendix A, under gravity load calculations. Under the conservative calculations, the column W12x136 used in the TCMC would have failed. The next W12 shape column, W12x152, meet the minimum requirement of the code with maximum allowable load at 1330 kips. The total load on the column calculated is 1259 kips.

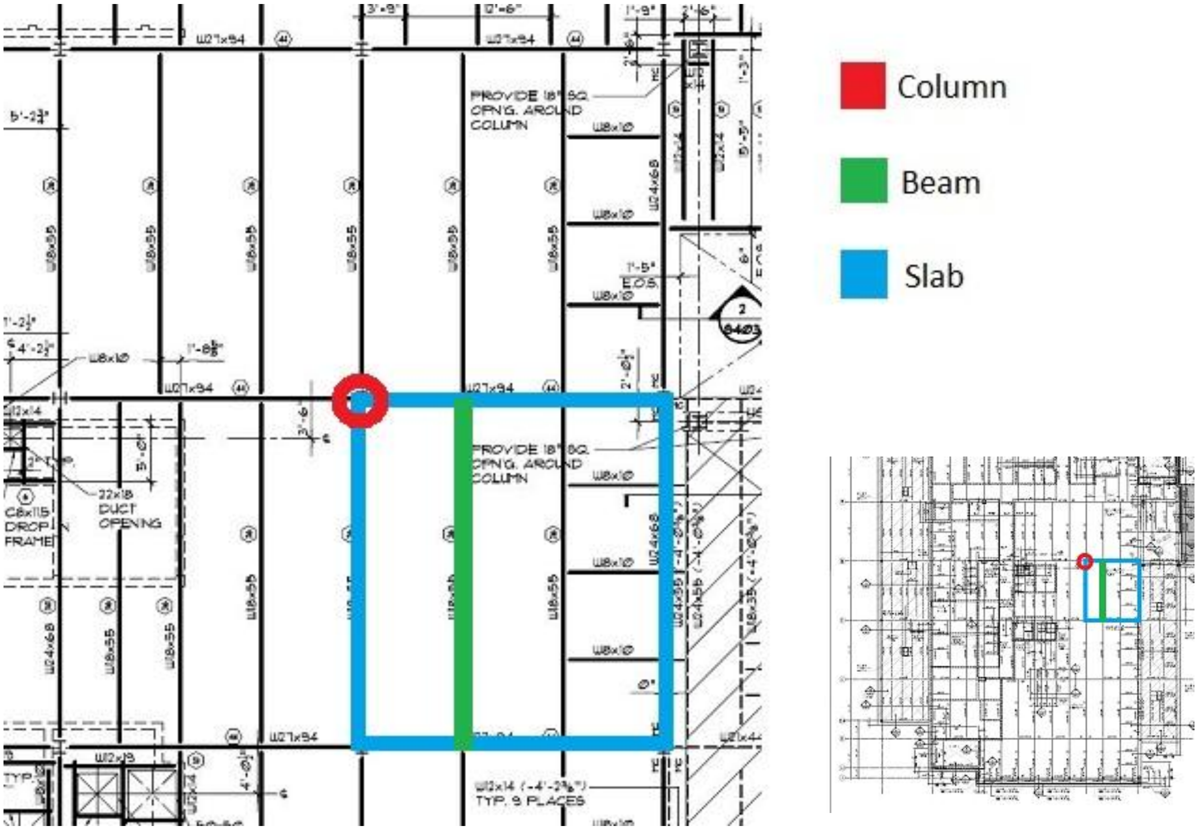


Figure 21 Images showing the column, beam and slab being considered for calculation. It is located on the second floor, in the middle of west wing.

Gravity Check for Beam between 10 and 9, spanning from D to E

The bean chosen represents a typical beam throughout the building. This is shown in the green line in figure 21. Its function is to carry the live and dead load from the floor above. The beam that TCMC used, W18x55 is more than enough to carry the load, under the calculations done in this report. In fact, a W18x40 would be the most economical in this case. But most likely, there was some load that hadn't been considered. The moment required is calculated to be 265 k-ft. A W18x40 can resist up to 294 k-ft. W18x55 resist up to 420 k-ft. The moment cause to the beam dominates over the shear this case. Lastly, live load deflection is calculated for serviceability and it just made it, deflecting 0.89 in while the maximum allowed is 1.00 in.

Gravity Check for Slab between D, E and 10, 9

The slab is made of 4.5" normal weight concrete with a 3" lok-floor 20 gauge composite floor deck and is the same throughout the whole west wing. This slab is also shown in figure 21 above, enclosed in the blue rectangle. Using Vulcraft Steel Deck catalog, the slab has been determined that it fits the all requirements, but barely made it. The catalog states the maximum allowable span is 8'-9" for a 1 span.

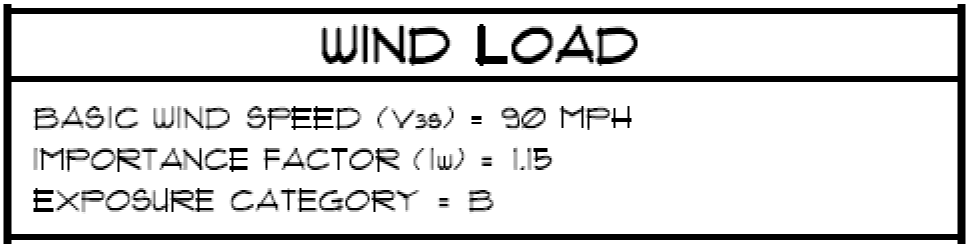
The slab we are considering now spans 8’-8”, so that is why it made it. However, this slab can be considered as a 3 span, which then, the maximum allowable span is increase to 11’-0”. The maximum live load it can carry is 279 psf, which is over the design factored live load of 160 psf.

Lateral Loads

Wind loads and seismic loads are always the primary types of lateral forces that buildings will face. Because of this, wind loads and seismic loads were calculated for the TCMC in this section. In the end, wind loads controls over seismic, with a base shear of 224 kip and an overturning moment of 22,727 kip-ft. This is close to twice the amount of loading compared to seismic forces.

Wind Loads

Using both ASCE 7-05 and ASCE 7-10 as guide, a full wind load calculation is done. Because the building is complex, a simplified, rectangular building is used in its place with dimensions similar to the original design. The calculation is done in a more conservative way. This full calculation can be found under Appendix B. The structure drawings, sheet S201B, provided the basic wind load variables needed; figure 22. With this, the pressures on each level of the building are found. Then finally, the shear forces and overturning moments are found. As shown in the calculations, wind pressure in the north-west direction controls the design. That is because the area facing north and south is a lot greater. The largest windward pressure the building face is 16.82 psf, including interior pressure. The largest base shear calculated is 454.7 kip and the largest overturning moment is 22,272 kip-ft. Figure 23 to 26 gives the detailed calculation of wind pressure and wind pressure along with pictures showing the forces.



WIND LOAD

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

Figure 22 Wind Load from sheet S201B

Wind Pressures N-S Direction							
Type	Floor	Distance	Wind Pressure	Internal Pressure		Net Pressure	
		(ft)	(psf)	(psf)		(psf)	
Windward Walls	Ground	0	9.33	3.62	-3.62	12.94	5.71
	2nd	21	9.33	3.62	-3.62	12.94	5.71
	3th	37	9.85	3.62	-3.62	13.47	6.23
	4th	53	11.10	3.62	-3.62	14.72	7.48
	Penthouse	69.5	11.89	3.62	-3.62	15.51	8.27
	Roof	93	13.20	3.62	-3.62	16.82	9.58
Leeward Walls	All	All	-6.60	3.62	-3.62	-2.98	-10.22
Side Walls	All	All	-11.55	3.62	-3.62	-7.93	-15.17
Roof	N/A	0-93	-14.85	3.62	-3.62	-11.23	-18.47
	N/A	93-186	-8.25	3.62	-3.62	-4.63	-11.87
	N/A	>186	-4.95	3.62	-3.62	-1.33	-8.57

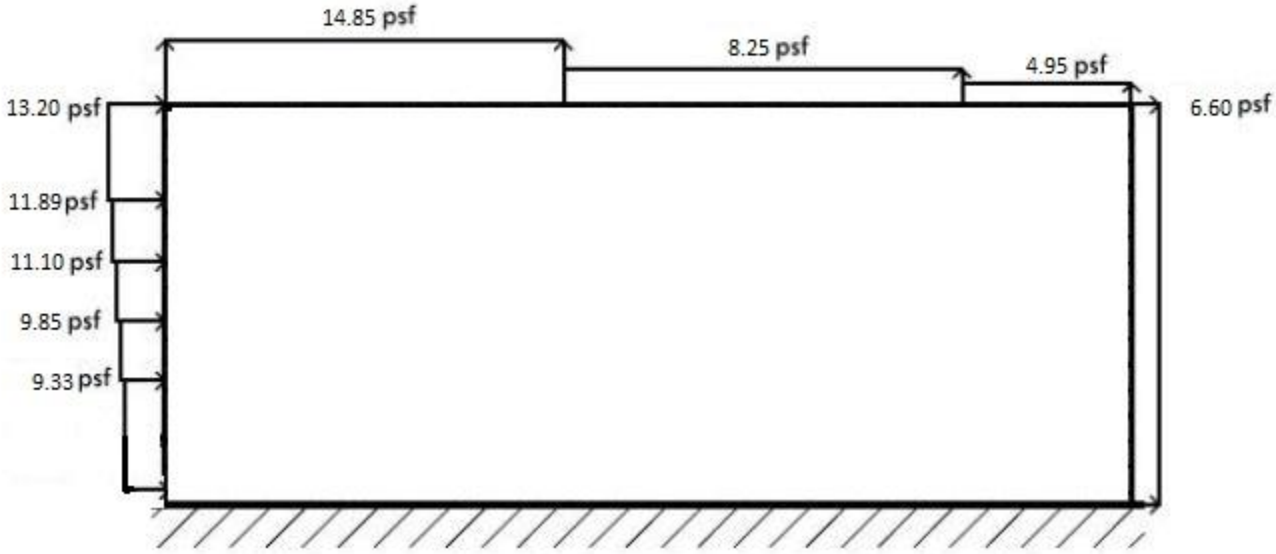


Figure 23 Wind Pressure acting on TCMC

Wind Forces N-S Direction								
Floor	Height	Trib Below		Trib Above		Story Force	Story Shear	Overturning
	(ft)	height (ft)	area (sf)	height (ft)	area (sf)	(k)	(k)	Moment (k-ft)
Ground	0	0	0	10	3400	44.0	454.7	0.0
2nd	20	10	3400	8	2720	79.2	410.6	1584.4
3th	36	8	2720	8	2720	73.3	331.4	2638.0
4th	52	8	2720	10	3400	90.1	258.2	4683.8
Penthouse	72	10	3400	10.5	3570	108.1	168.1	7781.6
Roof	93	10.5	3570	0	0	60.0	60.0	5584.3
Total						454.7	N/A	22272.1

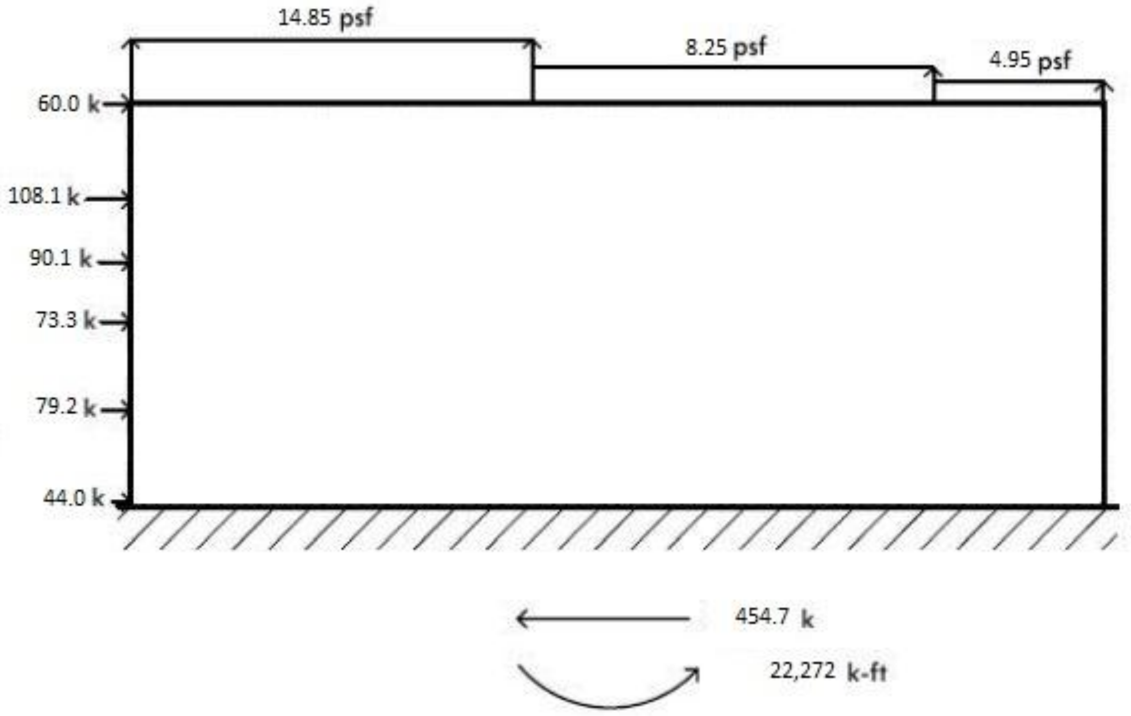


Figure 24 Wind Forces acting on TCMC

Wind Pressures E-W Direction

Type	Floor	Distance	Wind Pressure	Internal Pressure		Net Pressure	
		(ft)	(psf)	(psf)		(psf)	
Windward Walls	Ground	0	9.11	3.62	-3.62	12.73	5.49
	2nd	21	9.11	3.62	-3.62	12.73	5.49
	3th	37	9.62	3.62	-3.62	13.24	6.01
	4th	53	10.84	3.62	-3.62	14.46	7.23
	Penthouse	69.5	11.61	3.62	-3.62	15.23	7.99
	Roof	93	12.90	3.62	-3.62	16.51	9.28
Leeward Walls	All	All	-6.45	3.62	-3.62	-2.83	-10.07
Side Walls	All	All	-11.28	3.62	-3.62	-7.67	-14.90
Roof	N/A	0-93	-14.51	3.62	-3.62	-10.89	-18.13
	N/A	93-186	-8.06	3.62	-3.62	-4.44	-11.68
	N/A	>186	-4.84	3.62	-3.62	-1.22	-8.45

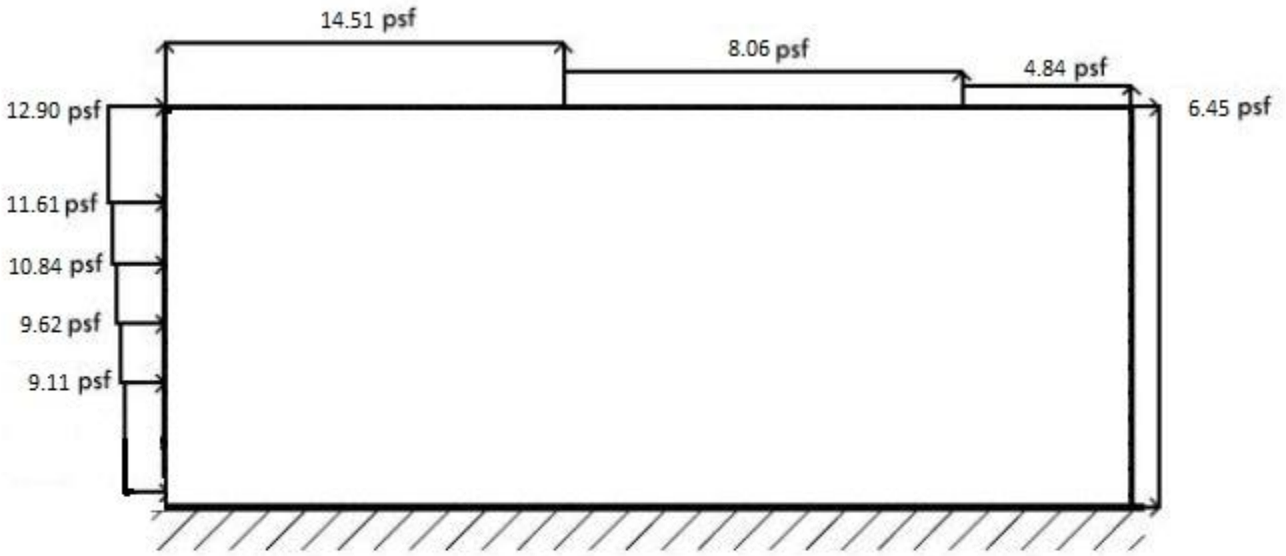


Figure 25 Wind Pressure acting on TCMC

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	2300	29.3	322.4	0.0
2nd	20	10	2300	8	1840	52.7	293.1	1053.9
3th	36	8	1840	8	1840	48.7	240.4	1754.3
4th	52	8	1840	10	2300	59.9	191.7	3113.2
Penthouse	72	10	2300	10.5	2415	71.8	131.8	5170.6
Roof	93	10.5	2415	0	0	39.9	60.0	3709.0
Total						302.3	N/A	14801.0

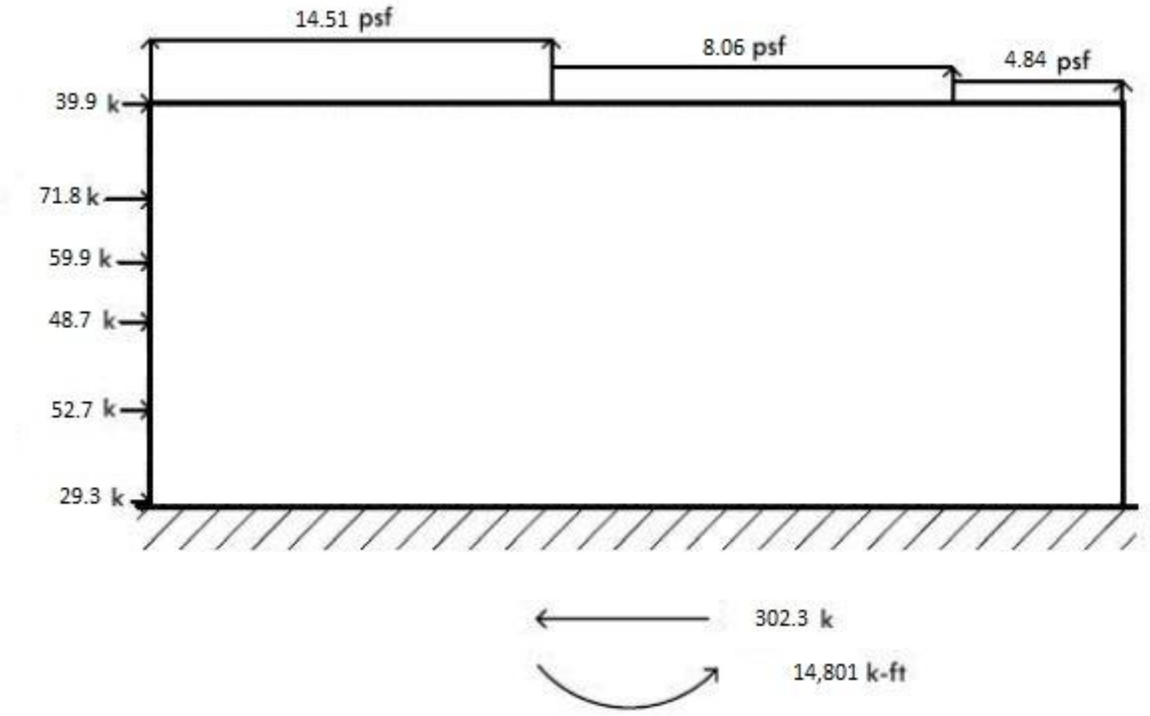
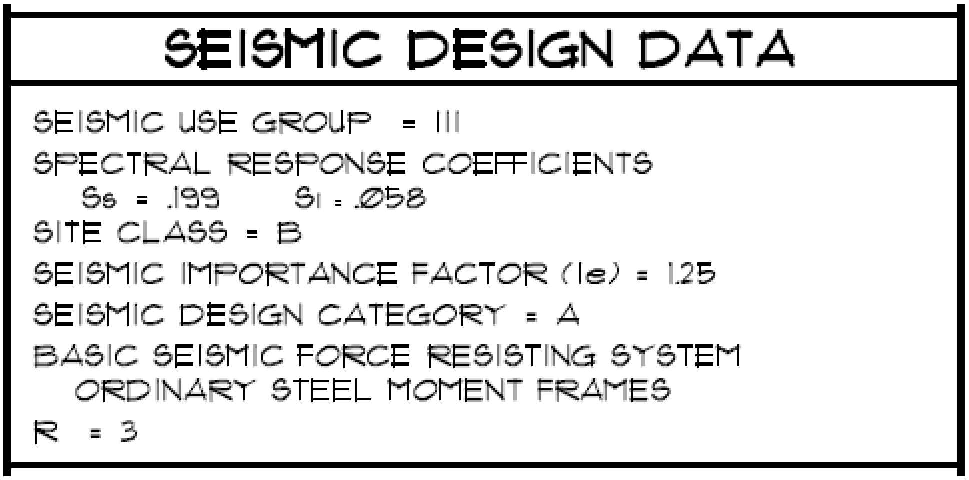


Figure 26 Wind Forces acting on TCMC

Seismic Loads

Seismic loads were calculated from ASCE 7-05, chapters 11 and 12. Sheet S201B in the structural drawings has a table with the seismic design data and from that, the other variables are easily calculated. Part of figure 27 is copied from S201B while the excel chart are the calculated variables.



SEISMIC DESIGN DATA

SEISMIC USE GROUP = III

SPECTRAL RESPONSE COEFFICIENTS

S_S = .199 S_1 = .058

SITE CLASS = B

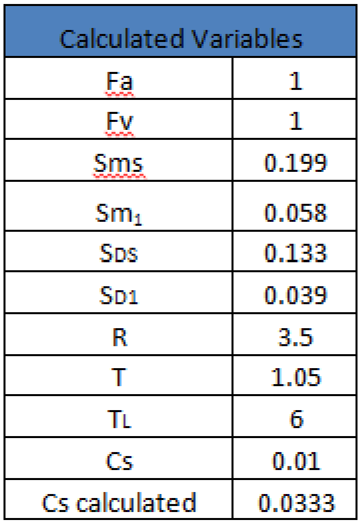
SEISMIC IMPORTANCE FACTOR (Ie) = 1.25

SEISMIC DESIGN CATEGORY = A

BASIC SEISMIC FORCE RESISTING SYSTEM

ORDINARY STEEL MOMENT FRAMES

R = 3



Calculated Variables	
Fa	1
Fv	1
Sms	0.199
Sm_1	0.058
S_{DS}	0.133
S_{D1}	0.039
R	3.5
T	1.05
T_L	6
Cs	0.01
Cs calculated	0.0333

Figure 27 Seismic Design Data and the Calculated Variables

With the information of figure 27 given, the base shear of 244 kip is found. The base shear is considered small because of the small Cs used. The initial calculation of the Cs is 0.0333 but the code allows it to not exceed 0.01, when $T<T_L$. After base shear is found, vertical distribution to all level is then calculated. Then we solved for the story shear and finally obtained the overturning moment of 13,650 k-ft. Figure 28 shows that table with the distribution of forces, along with a picture.

Vertical Distribution of Seismic Forces							
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (kips)	Story Shear (k)	Overturning Moment (k-ft)
Roof	93	1867	603893	0.252	56.46	56.46	5250.7
Penthouse	72	2163	504842	0.211	47.20	103.66	3398.3
4th	52	4286	660627	0.276	61.76	165.42	3211.7
3th	36	4286	413369	0.173	38.65	204.07	1391.3
2nd	20	4677	213197	0.089	19.93	224.00	398.6
1st	0	5099	0	0.000	0.00	224.00	0.0
Total			2395927.97	1.000	224.00	N/A	13650.6

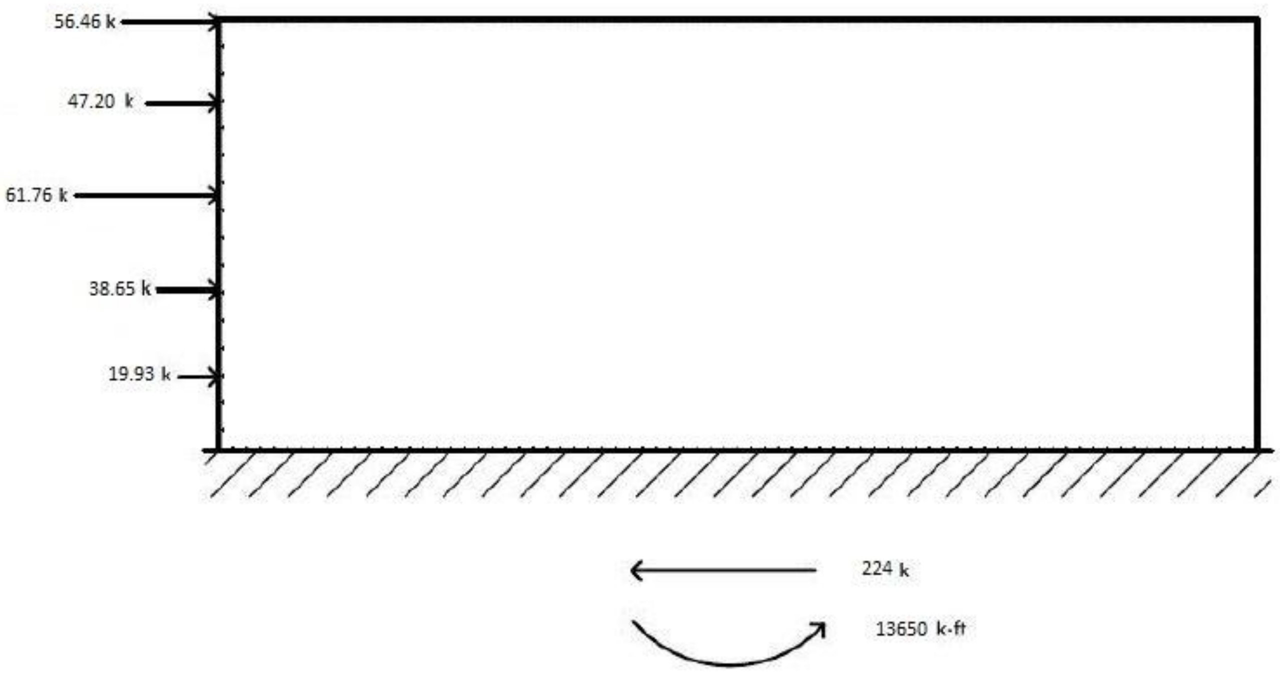


Figure 28 Seismic Forces acting on TCMC

Conclusions

All the information obtained in this Technical Report 1 is intended to fully understand the structure. Because TCMC has a unique shape, trying to understand it was a lot harder than compared to a regular rectangular structure. The foundations, floor systems, roof systems, framing systems, and lateral systems were all summarized here, along with diagrams that could further explain the system.

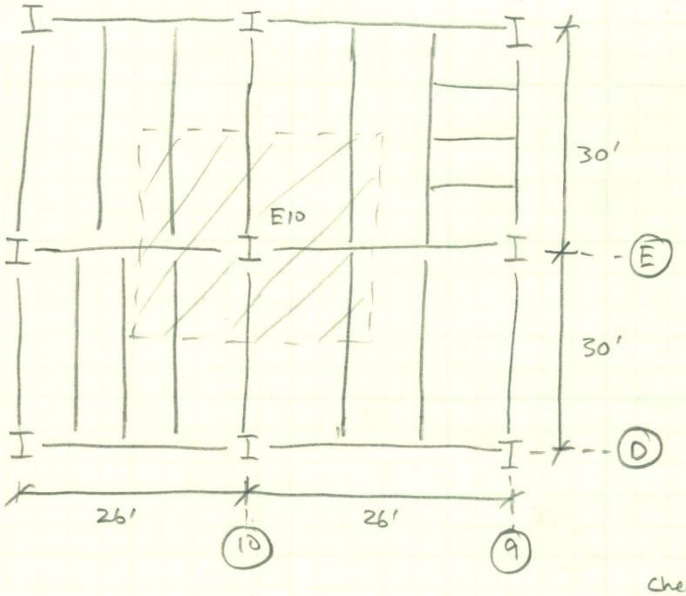
Gravity loads were determined, and a comparison is made to ASCE 7-05. Spot checks for a typical column, beam, and slab were done to see if they can adequate. Lateral Loads were also determined using ASCE 7-05 and ASCE 7-10. It was found that wind load in the end, dominated over seismic load. This is likely due to the lightweight of the building and the allowable use of a really small Cs when calculating the seismic base shear. Base shear and overturning moments are not found on any of the drawings provided so there is nothing to compared, when determining the accuracy of the calculations done in this report.

This report took a great amount of time and in the end, a much better understanding of TCMC allows the student to further experience the real feeling of working on a real life project.

Appendices

Spot Check Column | Tech 1 | pg 1 of 2

Column E10 is chosen for a spot test



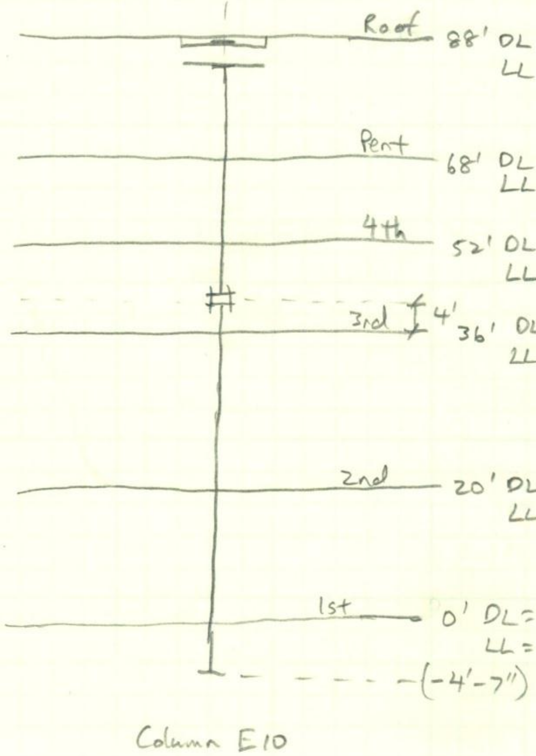
▨ $A_T = 30(26)$
$= 780\ ft^2$

design dead load is found on sheet S201A and S201B.

Live Load is assumed as 100 psf, heaviest in design criteria in TCMC.

Check to see if columns match. TCMC uses W12x136

Partial plan on first floor west wing.



Column E10

88' DL = 20psf
LL = 20 psf
$P_D = 780(20) = 15600$ lbs
$P_L = 780(20) = 15600$ lbs

68' DL = 193 psf
LL = 150 psf
$P_D = 780(193) = 150{,}540$ lbs
$P_L = 780(150) = 117{,}000$ lbs

52' DL = 93 psf
LL = 100 psf
$P_D = 780(93) = 72540$ lbs
$P_L = 780(100) = 78000$ lbs

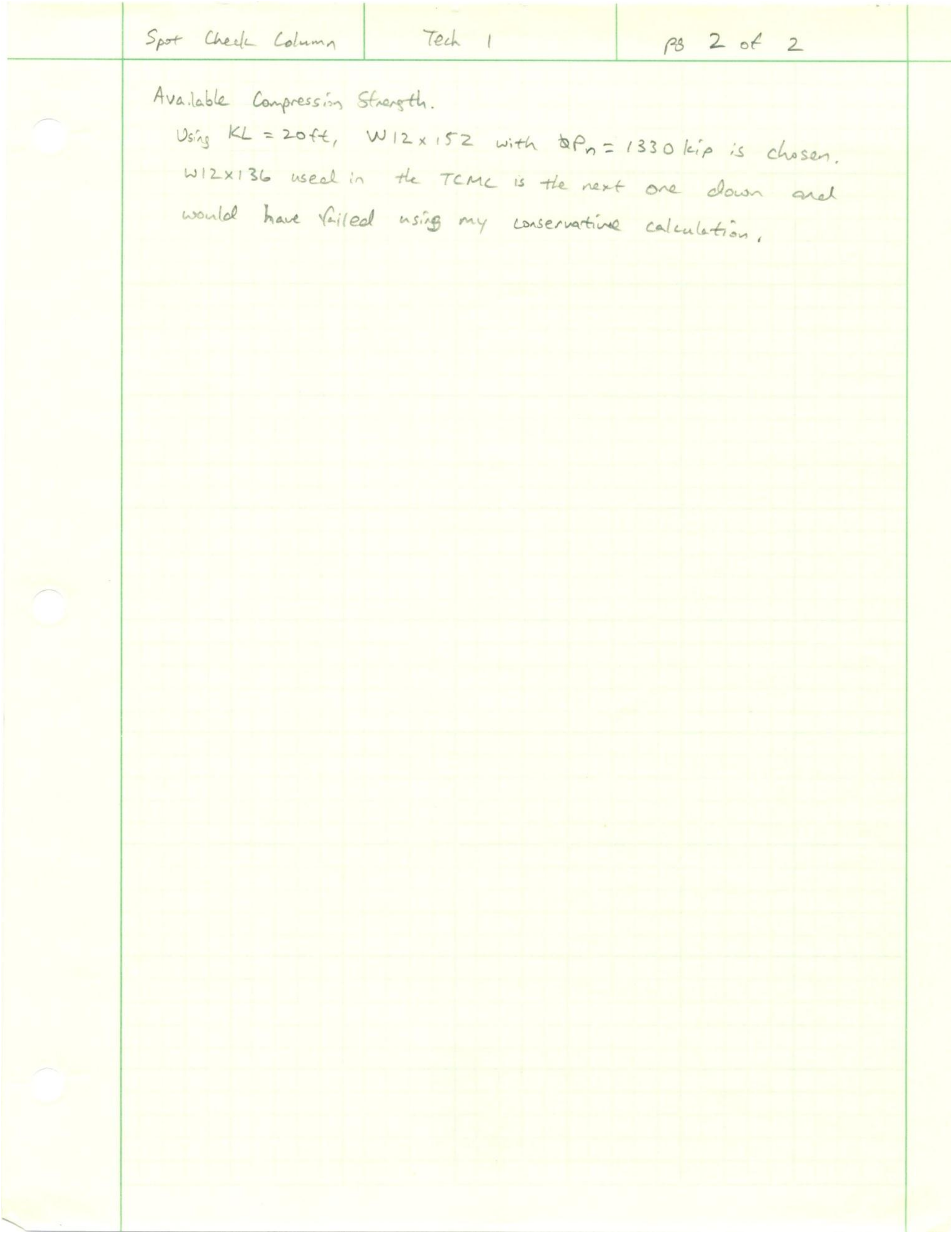
36' DL = 93 psf
LL = 100 psf
$P_D = 780(93) = 72540$ lbs
$P_L = 780(100) = 78000$ lbs

20' DL = 93 psf
LL = 100 psf
$P_D = 780(93) = 72540$ lbs
$P_L = 780(100) = 78000$ lbs

0' DL = 93 psf
LL = 100 psf
$P_D = 780(93) = 72540$ lbs
$P_L = 780(100) = 78000$ lbs

$P_{D\,total} = 456.3\ k$
$P_{L\,total} = 444.6\ k$

$P_{Total} = 1.2(456.3) + 1.6(444.6) = \boxed{1259\ k}$



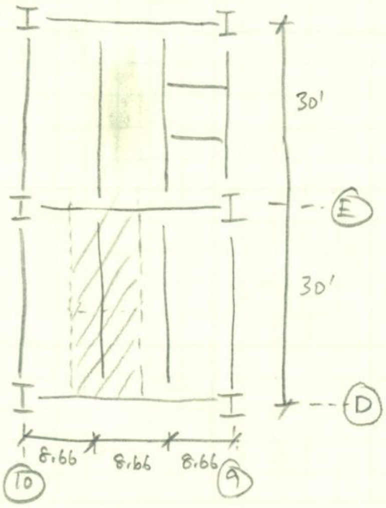
Spot Check Column   Tech 1   pg 2 of 2

Available Compression Strength.

Using KL = 20ft, W12x152 with $\phi P_n = 1330$ kip is chosen. W12x136 used in the TCMC is the next one down and would have failed using my conservative calculation.

Spot Check Beam | Tech 1 | Page 1 of 1

Check Beam Between ⑩ and ⑨ spanning from Ⓓ to Ⓔ



The beam is subject to uniform loads by the floor slab.

TCMC used W18x55 as the beam. See if it works.

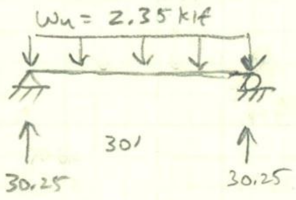
A_T for point load, $= 30(8.66) = 260\ \text{ft}^2$

* Assume Dead and Live Load is the same as calculated for column load.

$w_d = 8.66(93) = 805.4$ plf

$w_L = 8.66(100) = 866.6$ plf

$w_u = 1.2(805.4) + 1.6(866.6) = 2353$ plf



$M_{u,max} = \frac{wl^2}{8} = 265$ k-ft

$R = V_{max} = 30.25$ k

Use Zx table on Steel manual

Most economical

W18x40 with $\phi M_{px} = 294$ k.ft $>$ 265 k.ft

$\phi V_{px} = 169$ k $\gg$ 30.25 k

Deflection check: $\Delta_{LL} \leq L/360$

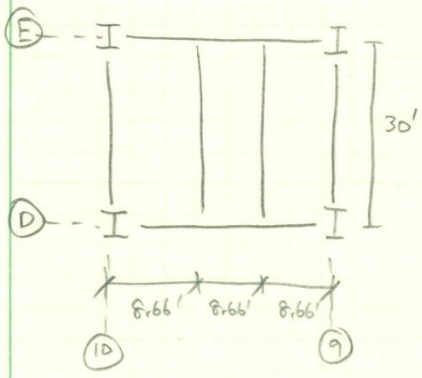
$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(.866)(30)^4(12)^3}{384(29{,}000)(612)} = 0.89 \text{ in}$$

$$L/360 = \frac{30(12)}{360} = 1 \text{ in} > 0.89 \text{ in} \quad ✓$$

So $\boxed{\text{W18x40}}$ is the most economical. TCMC uses a 18x55 beam so there could be a load that wasn't considered here.

Spot Check Slab | Tech 1 | Page 1 of 1

Check Slab between Ⓓ, Ⓔ and ⑩, ⑨



The slab is made of 4½ NWC with a 3" lok-floor 20 gauge composite floor deck.

Vulcraft Steel Deck catalog will be used to check if the slab meets the criteria.

Total slab thickness = 7.5"

Self D_L = 75 psf

The slab span 8'-8" just met the maximum span allowed ≤ 8'-9"

The slab is more of a 3 span case so,

8'-8" ≤ 11'-0" ✓ good.

At span 8'-8", the slab can carry a maximum load of 279 psf

$$\frac{231-303}{6''} = \frac{231-x}{4''}$$

$x = 279$ psf.

Dead Load = 93 psf

Live Load = 100 psf

$= 1.6(100) = 160$ psf < 279 psf

so the slab used in TCMC had met every requirement and barely made it using my calculation. Just a little more load and the slab would have failed.

Snow Load Calcs | Tech 1 | pg 1 of 2

Given in Sheet S201B

$P_g = 35$psf

$P_f = 30$ psf

$C_e = 1.0$

$I_s = 1.1$

$C_T = 1.0$

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

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

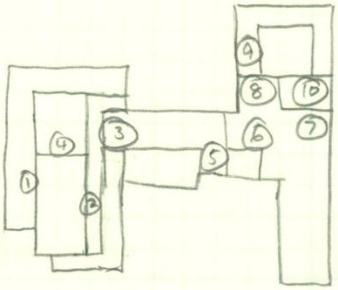
$= 26.95$, we will use 30psf as designed.

Snow Density

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

$= 0.13(35) + 14 = 18.55$

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

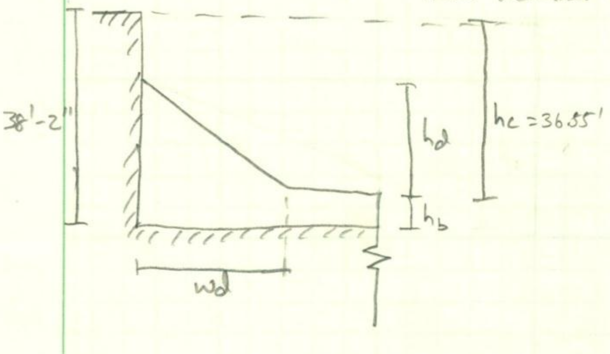


Since roof height varies, snow drift must be considered. Snow drifts can happen in many places. One of the case will be selected and calculated.

Ⓧ important drift locations.

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

From location ③ to ②



$h_c = 38.17 - 1.62 = 36.55'$

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

so drift will be considered

Snow Load Calcs | Tech 1 | pg 1 of 2

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

$l_u = 43$ ft in this case

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

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

$$= 2.29\text{ ft} < 36.55' \checkmark$$

$$w_d = 4(2.29) = 9.15\text{ ft}$$

Leeward $l_u = 26$ ft

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

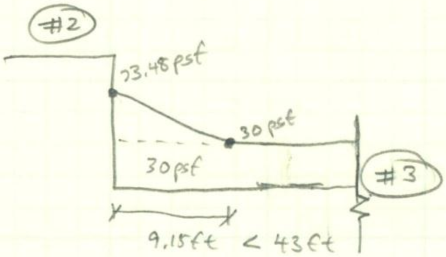
$$= 1.70 < 36.55$$

$$w_d = 4(1.7) = 6.8\text{ ft}$$

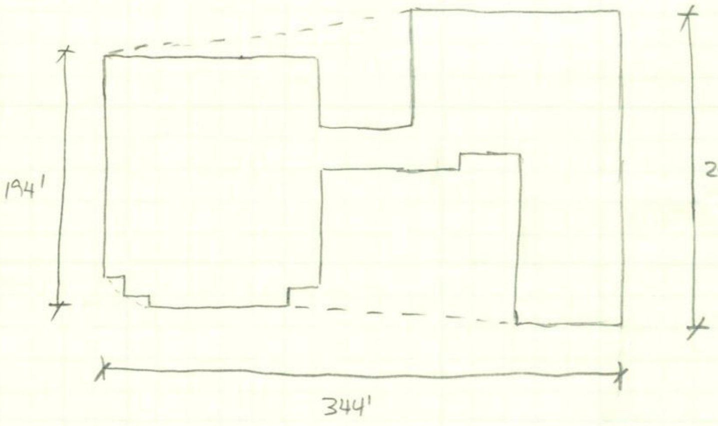
Windward case controls.

$$p_d = 2.29(18.55) = 42.48\text{ psf}$$

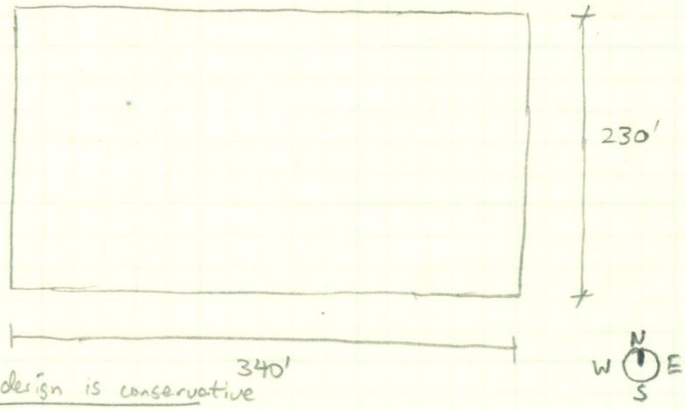
$43.48 + 30 = 73.48$ psf = highest snow load



Wind Design | Tech 1 | pg 1 of 5



Simplified Building



The building is changed to a rectangle for easier calculation, 230' is chosen for the depth (B) and 340' is chosen for the length (L)

This design is conservative

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

$I_W = 1,15$ = Buildings with a capacity > 500 for college

Exposure Category = B Urban surroundings

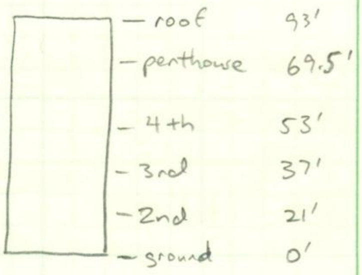
$K_d = 0,85$ for buildings main wind force resisting system.

$K_{zt} = 1.0$

K_z = varies depending on which story

$GC_{pi} = \pm 0,18$ Enclosed building; Fully

Wind Design Tech 1 pg 2 of 5



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

$= 0.00256(.99)(1.0)(.85)(90)^2(1.15)$

$q_{z_r} = 20.1$ psf for roof

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

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

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

$q_{z_4} = 16.9$ psf for 4th story floor

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

$q_{z_3} = 15.0$ psf for 3th story floor

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

$q_{z_2} = 14.2$ psf for 2nd story floor

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

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

Finding Gust Effect Factor

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

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

g_Q and $g_v = 3.4$

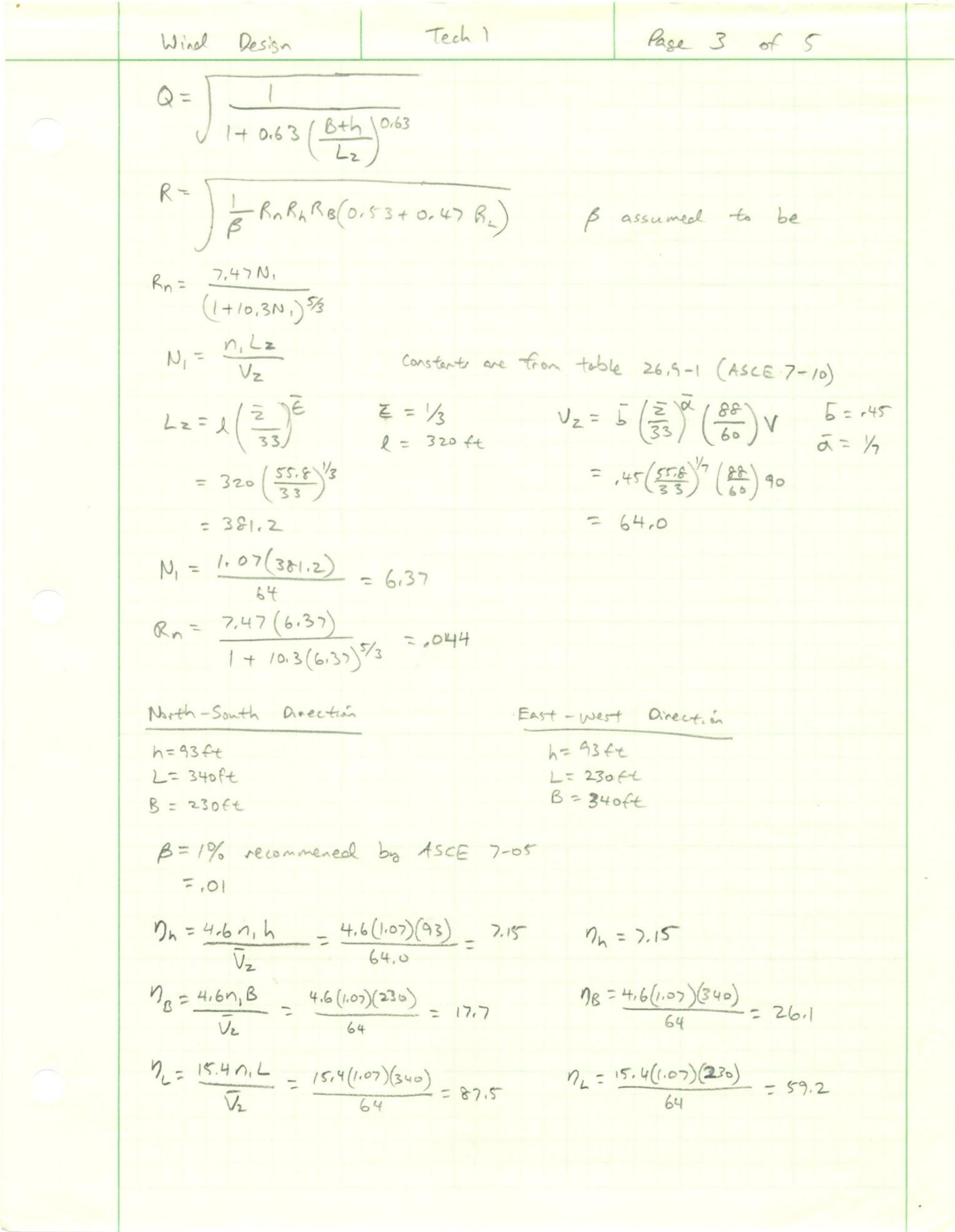
$n_1 = \frac{100}{H} = \frac{100}{93} = 1.07$ average value C26.9-6 ASCE7-10

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

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

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

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



Wind Design | Tech 1 | Page 3 of 5

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

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

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

$\bar{b} = .45$

$\bar{\alpha} = 1/7$

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

$$= 381.2$$

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

$$= 64.0$$

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

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

North-South Direction

h = 93 ft

L = 340 ft

B = 230 ft

East-West Direction

h = 93 ft

L = 230 ft

B = 340 ft

$\beta = 1\%$ recommened by ASCE 7-05

$= .01$

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{4.6(1.07)(93)}{64.0} = 7.15$$ $\eta_h = 7.15$

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6(1.07)(230)}{64} = 17.7$$ $$\eta_B = \frac{4.6(1.07)(340)}{64} = 26.1$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(1.07)(340)}{64} = 87.5$$ $$\eta_L = \frac{15.4(1.07)(230)}{64} = 59.2$$

Wind Design | Tech 1 | Page 4 of 5

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

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

$$R_B = \frac{1}{17.7} - \frac{1}{2(17.7)^2}\left(1-e^{-2(17.7)}\right) = .055 \qquad R_B = \frac{1}{26.1} - \frac{1}{2(26.1)^2}\left(1-e^{-2(26.1)}\right) = .038$$

$$R_L = \frac{1}{87.5} - \frac{1}{2(87.5)^2}\left(1-e^{-2(87.5)}\right) = .011 \qquad R_L = \frac{1}{59.2} - \frac{1}{2(59.2)^2}\left(1-e^{-2(59.2)}\right) = .017$$

$$R = \sqrt{\frac{1}{0.01}(.044)(.13)(.055)\left(0.53+0.47(.011)\right)} \qquad R = \sqrt{\frac{1}{.01}(0.044)(.13)(.038)\left(0.53+0.47(.017)\right)}$$

$$R = .130 \qquad R = .108$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{230+93}{381.2}\right)^{0.63}}} \qquad Q = \sqrt{\frac{1}{1+0.63\left(\frac{340+93}{381.2}\right)^{0.63}}}$$

$$Q = .800 \qquad Q = .771$$

$$G_f = 0.925\left(\frac{1+1.7(.275)\sqrt{(3.4)^2(.8)^2+(4.32)^2(.130)^2}}{1+1.7(3.4)(.275)}\right)$$

$$\boxed{G_f = .821}$$

$$G_f = 0.925\left(\frac{1+1.7(.275)\sqrt{(3.4)^2(.771)^2+(4.32)^2(.108)^2}}{1+1.7(3.4)(.275)}\right)$$

$$\boxed{G_f = .802}$$

Design Wind Pressures.

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

Wall

Windward $C_p = 0.8$

Side Wall $C_p = -0.7$

Leeward $C_p \Rightarrow L/B = 340/230 = 1.48$ | $L/B = 230/340 = .68$

$C_p = -0.4$ by interbolation | $C_p = -0.5$

Roof

$\theta = 0°$ $h/L = 93/340 = .28$ | $93/230 = .41$

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

$2h = 186$

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

For Both

$C_p = -0.9$ for 0 to h

$C_p = -0.5$ for h to 2h

$C_p = -0.3$ for >2h

Wind Design | Tech 1 | Page 5 of 5

N-S

$p = 20.1(.821)(0.8) - 20.1(\pm .18) = 9.58$ or 16.82

13.2 ± 3.618

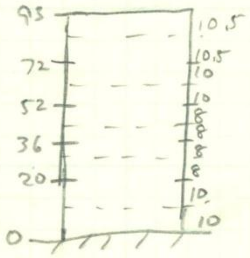
for Roof Wind Load.

See Excel for rest of calculations

E-W

$p = 20.1(.802)(0.8) - 20.1(\pm .18) = 9.28$

Wind Forces



Seismic Analysis | Tech 1 | B 1 of 2

Given on Sheet S201B

Using ASCE 7-05 to do calculations.

Seismic Use Group = III

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

Site Class = B

$I_e = 1.25$

Seismic Design Category = A

Ordinary Steel Moment Frames.

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

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

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

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

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

$\Omega = 3$

$C_d = 3$

Equivalent Lateral Force / Base Shear

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

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

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

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

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

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

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

$$C_s > 0.01$$

Use $C_s = .01$ for calculations

$V = (.01)(22{,}378\,k)$

$\boxed{V = 224 \text{ kip}}$

If using $C_s = .0333$ (conservative)

$V = .0333(22{,}378)$

$V = 745\,k$

Seismic Analysis | Tech 1 | pg 2 of 2

Vertical Distribution of Seismic Forces.

$$F_x = C_{ux} V$$

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

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

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

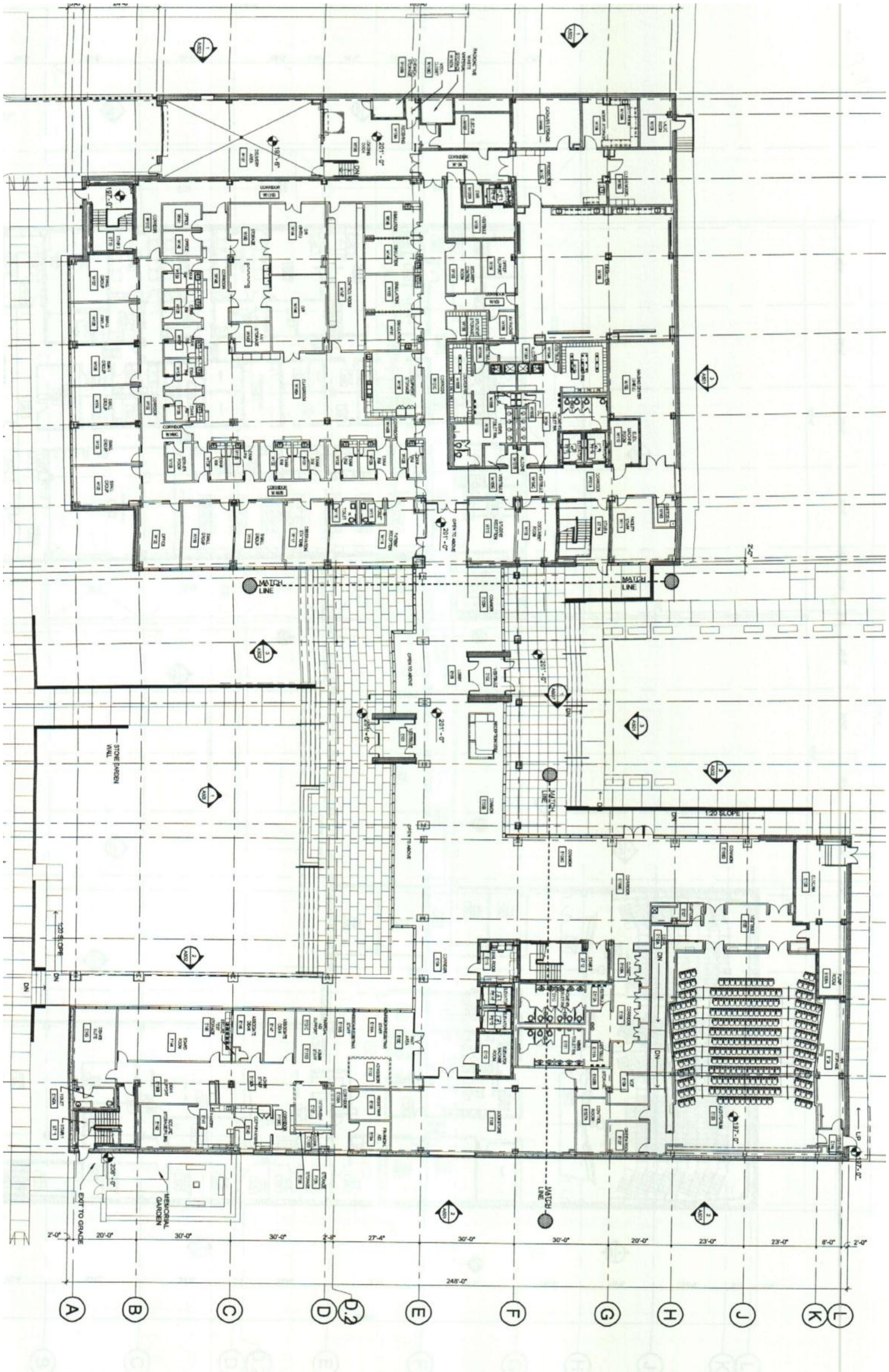
$$C_{rf} = 0.252$$

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

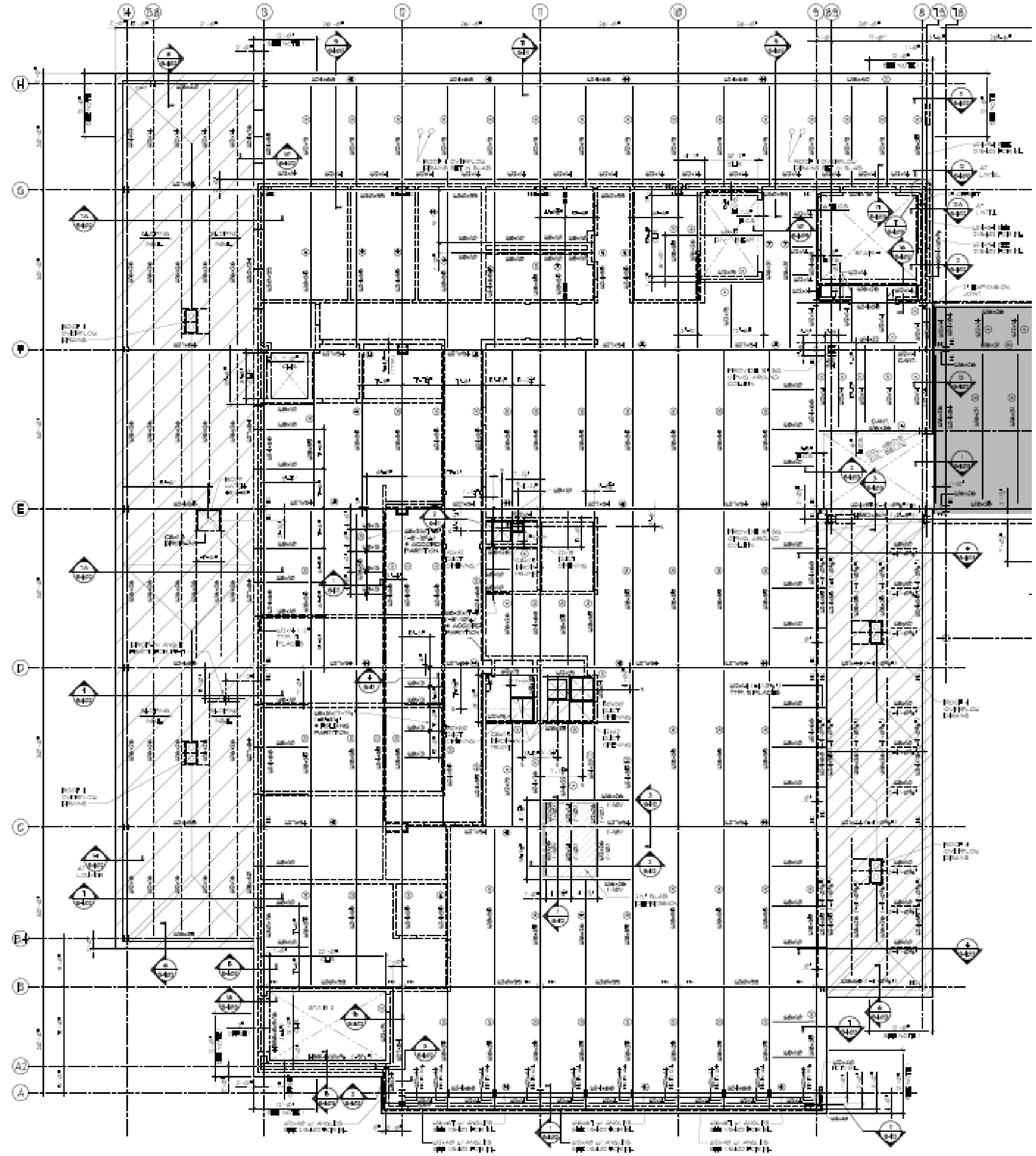
$$F_x = 0.252(224)$$

$$F_r = 56.5 \text{ k}$$

See rest on excel.



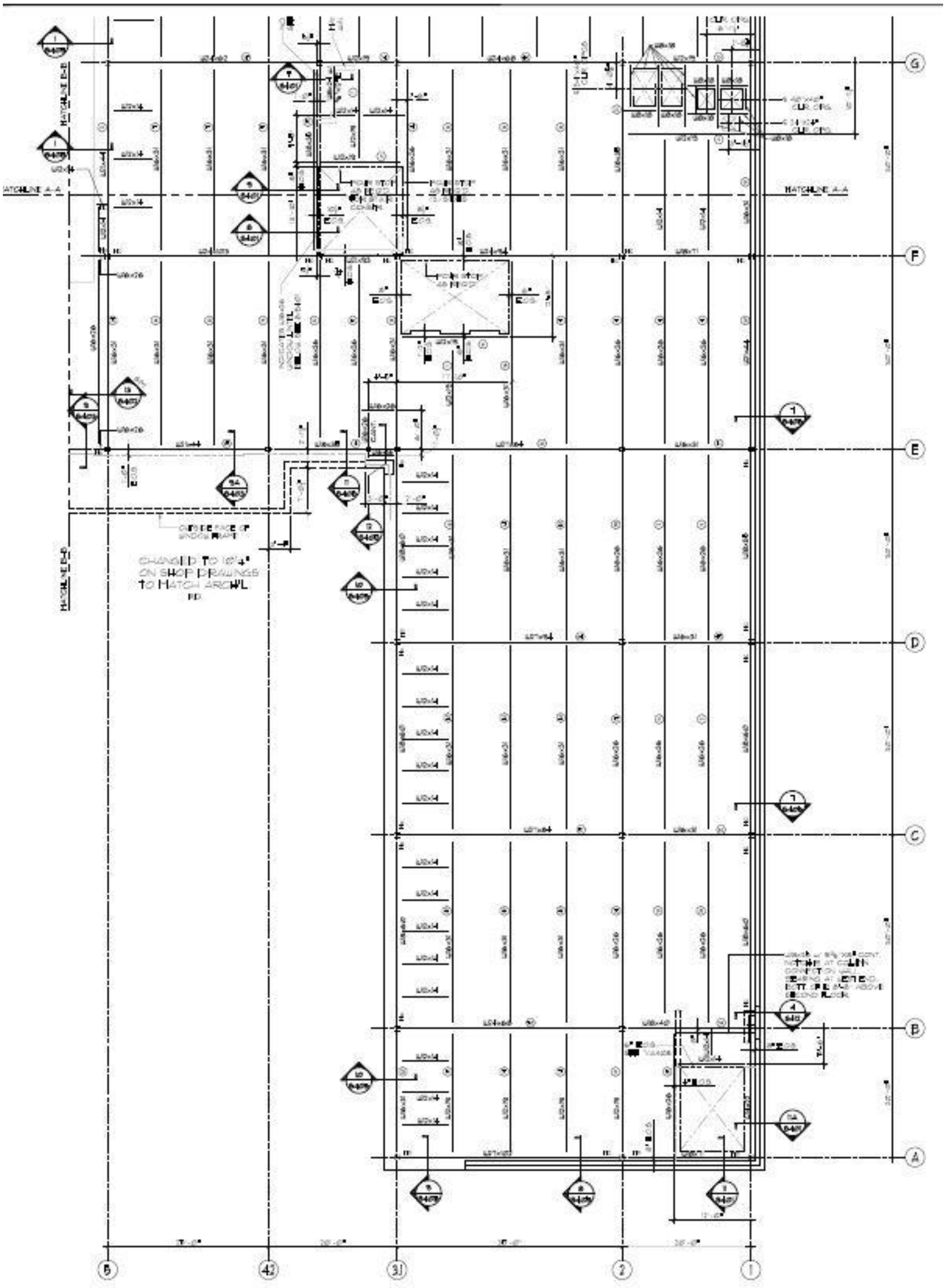
Floor Plan of the 1st Floor <------N



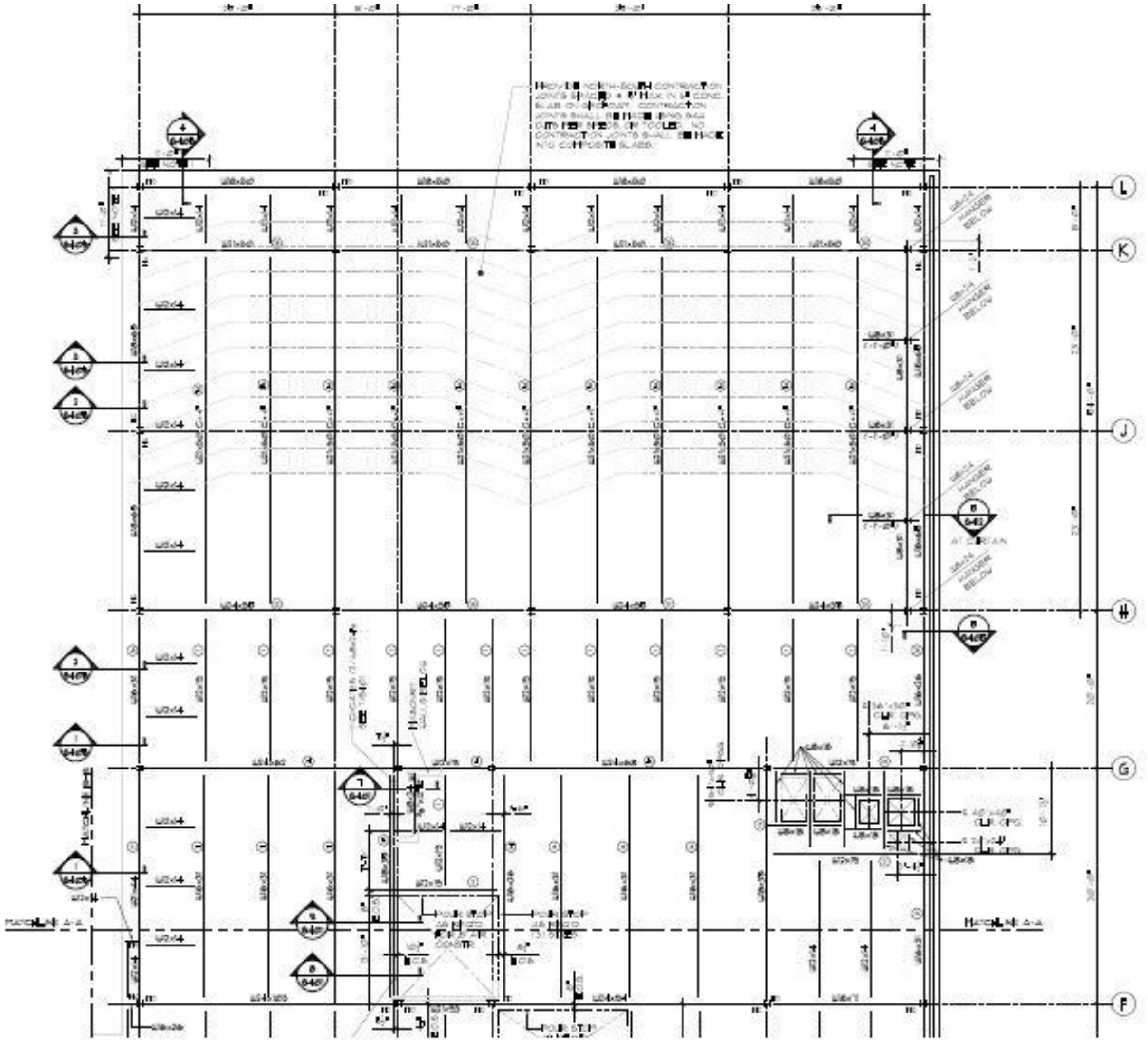
2^{nd} Story frame, west wing

General Building Data

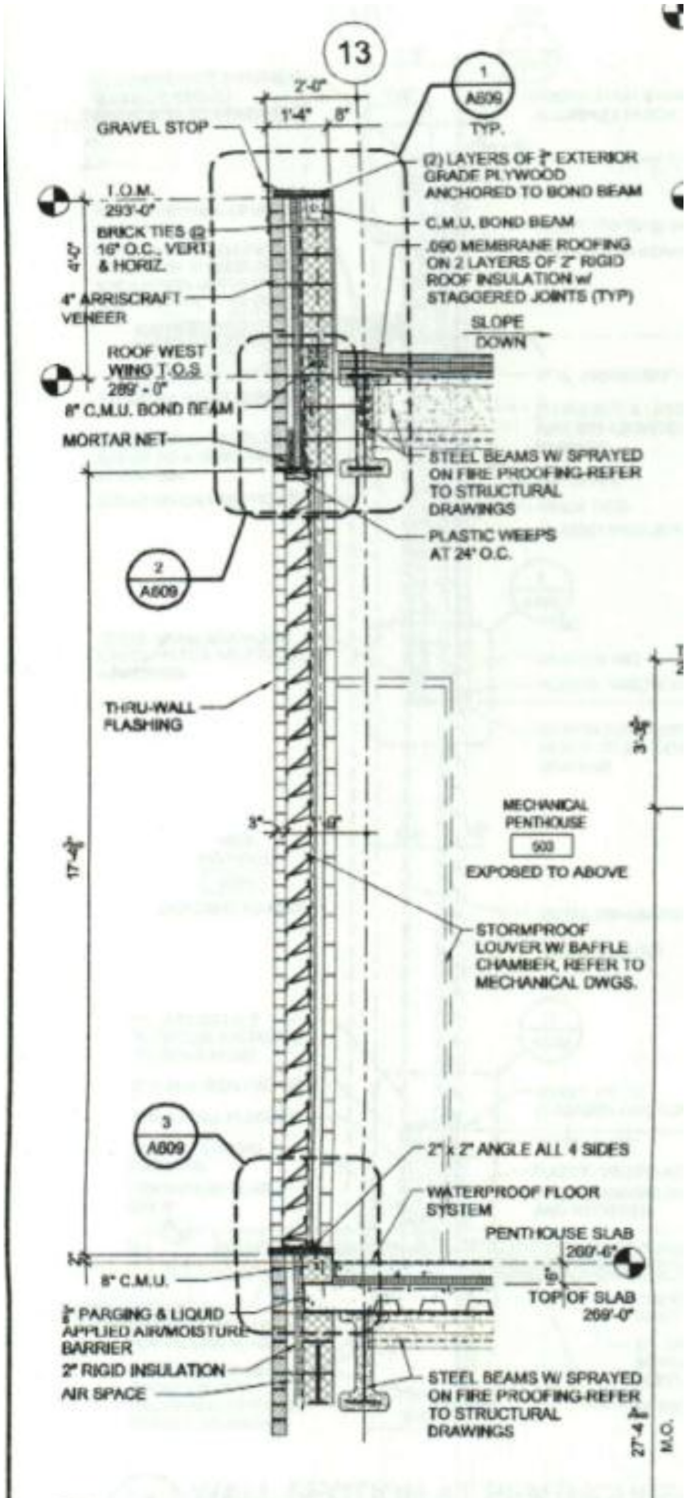
1. Building name: The Commonwealth Medical College
2. Location and site: 525 Pine St, Scranton, PA 18509



2nd Story frame, east wing (south side)



2nd Story frame, east wing (north side)



3 WALL SECTION @ BRICK VENEER

3/8" = 1'-0"