Technical Assignment 1

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Inova Fairfax Hospital | South Patient Tower

Falls Church, VA

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Executive Summary:

The following Technical Report summarizes the existing conditions and design concepts of the current structure of the South Patient Tower (SPT). All plans, schedules and photographs were provided by Turner Construction. To gain a further understanding of the structural system, the foundation, floor, framing, lateral and roof systems of the SPT was analyzed to determine how all of the systems work together as one structural system. This report includes research pertaining to the structural system as well as a comparison of the loads used in the design with the calculated loads.

Gravity loads were calculated for the various components of the structural system and included in this section is the total weight of the structure. Two main elements of the gravity system were checked, including an interior column and a slab panel. A typical basement column was checked for adequate compressive strength and a typical two-way slab panel from the 6th-11th floors was analyzed. Along with checking the strength and punching shear capacity of the slab, deflection calculations were performed to comply with serviceability criterion. All members checked for the gravity system were found to be adequate.

Lateral load calculations were performed in accordance with ASCE 7-05 procedures. A simplified building shape was used to determine the wind and seismic loads on the structure. From the calculations, the seismic loads were found to be within 2% of the design base shear listed on the structural drawings. The wind analysis was done in both directions and produced a base shear of 303.89 k and 791.93 k in the North-South and East-West wind direction respectively. Overturning moments were found to be 28,030 ft-k and 71,626 ft-k in the N-S and E-W direction. The seismic forces on the other hand produced a base shear of 693 k with an overturning moment of roughly 76,652 ft-k. Since the wind forces create the higher base shear and the seismic forces produce the higher overturning moment, both load cases must be accounted for and designed accordingly.

Also included in the report are appendices which contain all of the hand calculations, diagrams and tables, and structural plans that may be useful.

Building Introduction:

As an early phase in the Inova Fairfax Hospital Campus Development Plan, the South Patient Tower will be connected to the existing patient tower (see Figure 1) at all levels above grade including the penthouse. Construction started in the summer of 2010 and is expected to be completed by fall 2012 with and overall project cost of around \$76 million. Standing at 175’, the 236,000 ft^2 concrete structure consists of 12 stories above grade (excluding the penthouse) with an additional story below grade. A system of auger-cast piles and pile caps are used to support the structure with a soil bearing pressure of 3000 psf.

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

Along with the physical connection, the architecture of the South Patient Tower shares some similarities with the surrounding campus/hospital buildings. Wilmot/Sanz Architects designed the South Patient Tower as a continuation of the main architectural features of the existing patient tower building while at the same time displaying Inova’s commitment to sustainable and functional buildings. Consisting of 174 all-private intensive-care and medical/surgical patient rooms, the floor plans are situated so that the various intensive-care unit specialties correspond to the same level as that of the existing main hospital. In order to meet the patient’s specialized needs, workstations will be placed outside of the patient’s rooms to maintain privacy while being able to monitor the patients at the same time.

The façade is largely composed of a smooth finished precast concrete panel as well as a precast concrete panel with a thin brick face (see Figure 2). To add more architectural detail, thin brick soldier courses are used at every story level, starting with the 4th floor and continuing up the building to the 11th floor. The only tangent from the typical architectural pattern occurs on the 5th floor (main mechanical floor) where architectural louvers are used to allow air to exit the building. The first two levels are composed entirely of an aluminum curtain wall system which is also used for the majority of the building’s windows. The two main architectural features that stand out along the

Figure 2:
Exterior rendering showing the circular entrance and precast concrete façade

ground floor of the building are the large two-story rotunda and the canopy covering the main entrance which is constructed from 4 custom steel columns.

The South Patient Tower is attempting to achieve LEED Silver Certification by including numerous sustainable designs (see Figure 3). Inside the patient rooms, the use of low-VOC paints, building materials and furniture will lead to a higher indoor air quality. Also, the use of low flow plumbing fixtures and sensors will greatly reduce the water consumption by up to 30%. Outside of the building, native plants that are resistant to drought will surround the building. From the patient rooms, guests will be able to see the green roof and the water cisterns used to capture rain water.

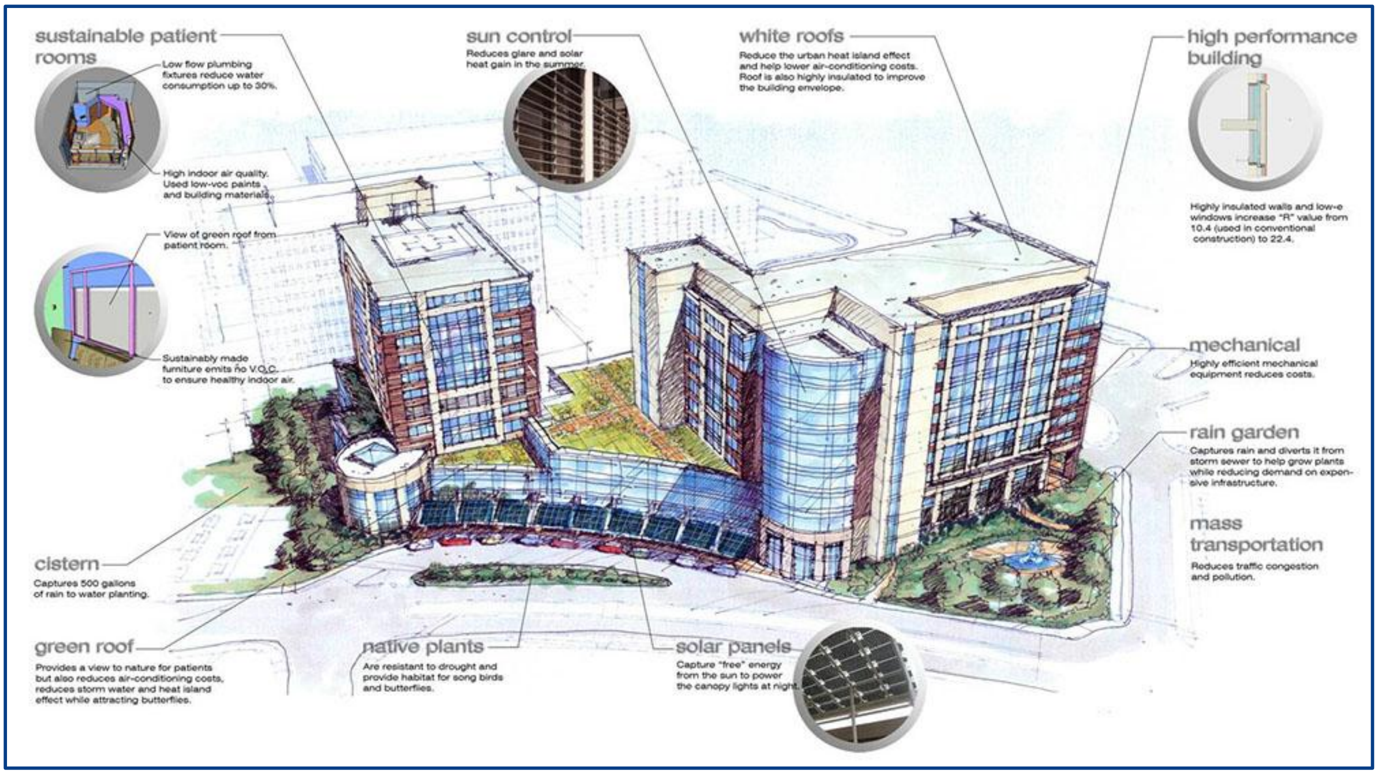


Figure 3:
Sustainability features (rendering provided by Wilmot/Sanz Architects)

Structural Overview:

Foundation:

Schnabel Engineering North performed the geotechnical studies for the South Patient Tower and provided the report in which they explain the site and below-grade conditions. The structural engineers of Cagley & Associates designed the foundation for an undisturbed soil net allowable bearing pressure of 3000 psf. Also given in the geotechnical report are lateral equivalent fluid pressures which are 60 psf/ft of depth for both the braced walls and cantilevered retaining walls. The sliding resistance (friction factor) was found to be 0.30.

In light of the soil conditions, the SPT utilizes a foundation with a system of 16" diameter auger-cast piles and pile caps on top of a slab on grade (see Figure 4). Due to higher stresses around the staircase and elevator pit, a large pile cap is situated around each of these areas to help alleviate the stresses on the slab (see Figure 5). The number of piles per pile cap varies throughout the foundation with the most common being 9 and 11.

Along with the 5" slab on grade, grade beams connect the piles within the foundation footprint. Along the perimeter of the foundation, the SPT makes use of spread and strip footings (see Figure 6). Since the foundation does not cover the entire area of the ground floor, some areas consist of piles and pile caps directly underneath the ground floor slab to support the main entrance and lobby space.

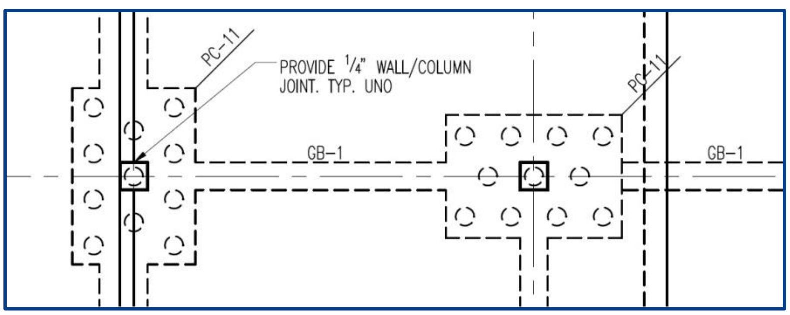


Figure 4:
Typical pile and pile cap

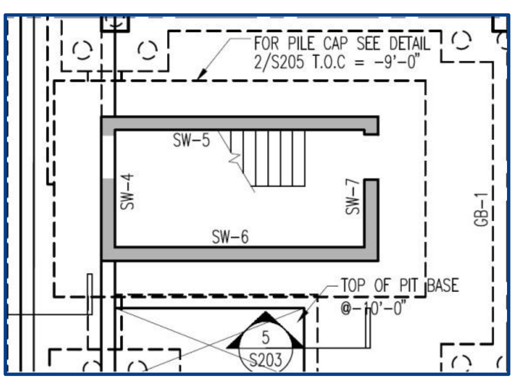


Figure 5:
Pile cap constructed around staircase

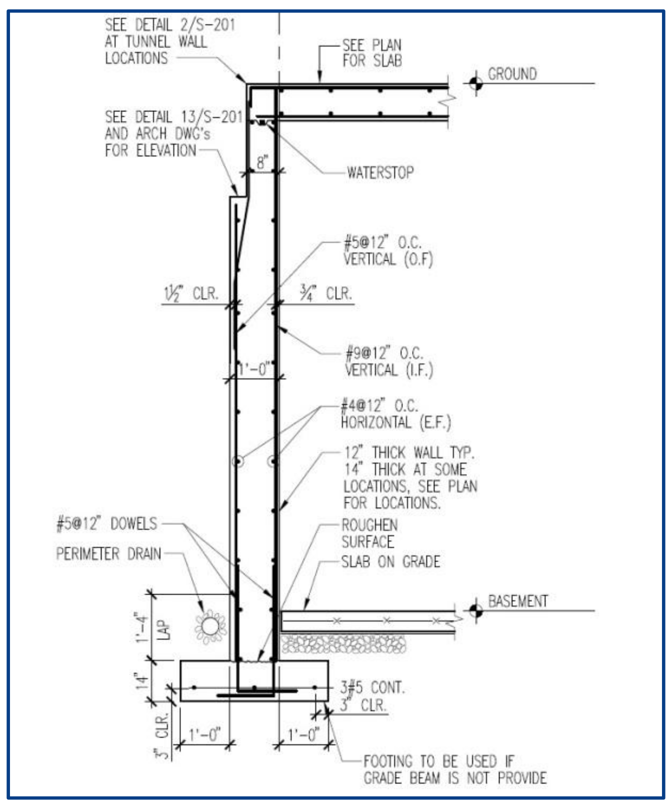


Figure 6:
Spread footing with basement wall

Floor System:

The typical floor construction for the South Patient Tower is comprised of a 9 ½" two-way flat slab. A drop panel is located at every column location in order to prevent punching shear as well as to increase the thickness of the slab to help with the moment carrying capacity of the slab near the columns. The typical size for the drop panel is 10'x10'x 6".

For the ground floor through the 4th floor, 5000 psi concrete is used for construction of the two-way slab while the upper floors use a 4000 psi concrete. The one exception to the 9 ½" slab is the mechanical floor (5th floor). Because of the higher load imposed by the mechanical equipment over the entire floor, the slab was designed accordingly and bumped up to 10 ½".

Reinforcement for the two-way slab system is comprised of both top and bottom steel. The typical bottom reinforcement consists of #5@12" o.c. each way (see Figure 7 and 8 for reinforcement details). Additional bottom reinforcement is listed on the drawings wherever needed as well as top reinforcement which is located in areas of negative moments (mainly around the columns and between column lines depending which direction the frame of interest is going). With a fairly simple column layout, the two-way slab system has a span of 29' in both directions for the most part.

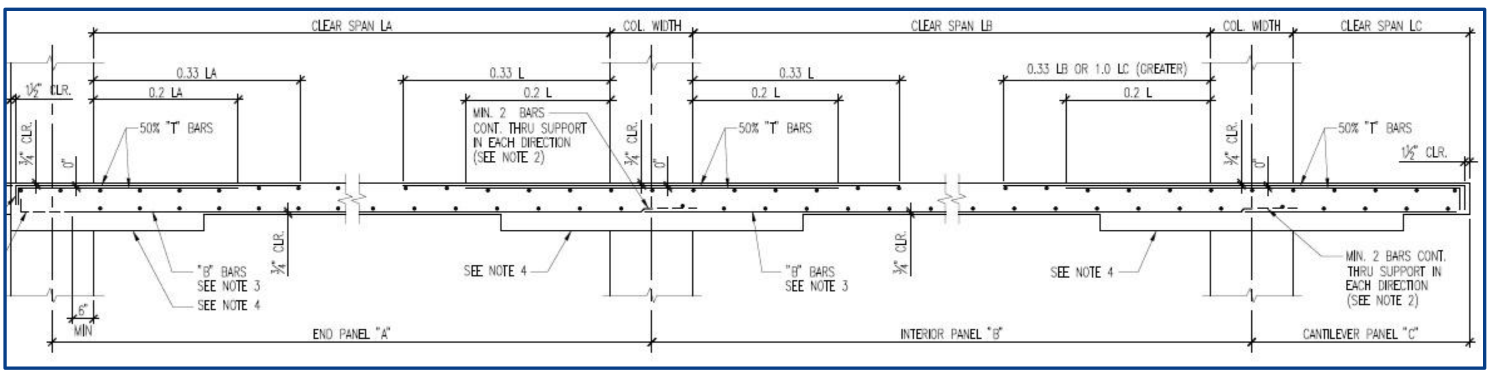


Figure 7:
Typical column strip reinforcement and placement

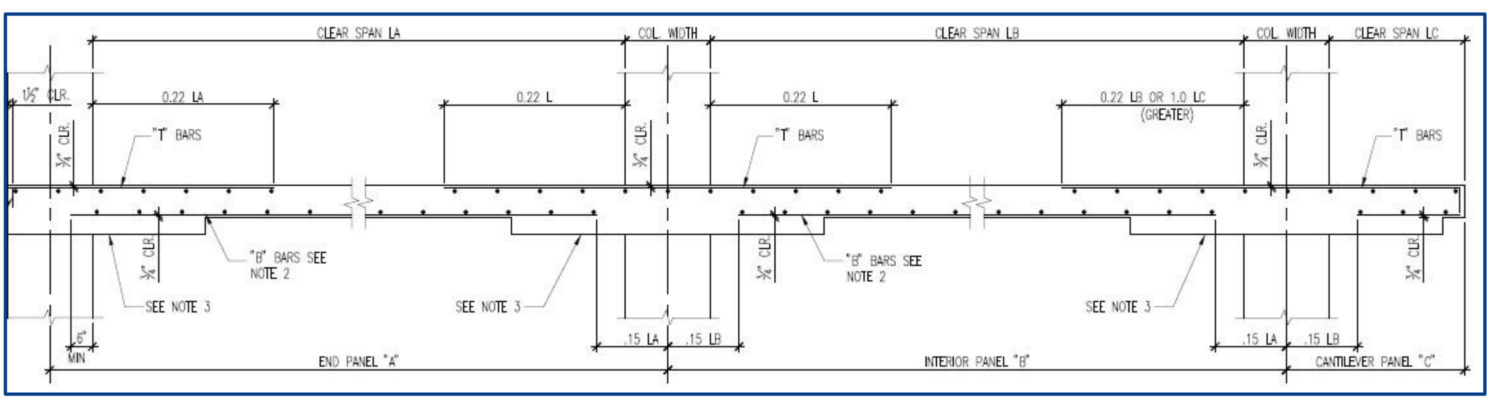


Figure 8:
Typical middle strip reinforcement and placement

Framing System:

As mentioned in the previous section, the columns follow a pretty regular pattern with a few exceptions. Typically the bay sizes are 29' x 29' with drop panels at every location (see Appendix D for typical floor plans). There are no interior beams but there are a few beams along the perimeter of the building towards the south end of the structure and near the connection to the existing hospital.

The columns are all cast-in-place concrete with the largest column being 30" x 30" in the basement level. The typical column size is 24" x 24" and 12" x 18" (rotated as required to fit the wall thickness). Because of the higher loads located in the columns towards the lower portions of the building, 7000 psi concrete is utilized up to the 5th floor level with the rest of the upper floor columns being 5000 psi concrete. Consisting of mainly #11 reinforcement bars with #4 stirrups, the maximum number of reinforcement bars around a column is 20 with the typical number being 4.

Lateral System:

Shear walls are the main lateral force resisting system in the South Patient Tower and are situated throughout the building to best resist the lateral forces in the building. Seven different walls make up the shear wall system which surrounds both the main staircase and the main elevator (see Figure 9 located on the next page). The shear walls are 12" thick and are composed of 5000 psi cast-in-place concrete. Most span from the basement level to the main roof line but the northern core around the elevator shaft extend up the entire 175' height to the top of the penthouse level. Also included in the main lateral force resisting system are ordinary moment frames

All of the shear walls are connected to the foundation with dowels to properly allow the loads to travel through the walls down to the foundation. These two shear wall cores along with the moment frames help resist lateral loads in both the North-South and East-West direction.

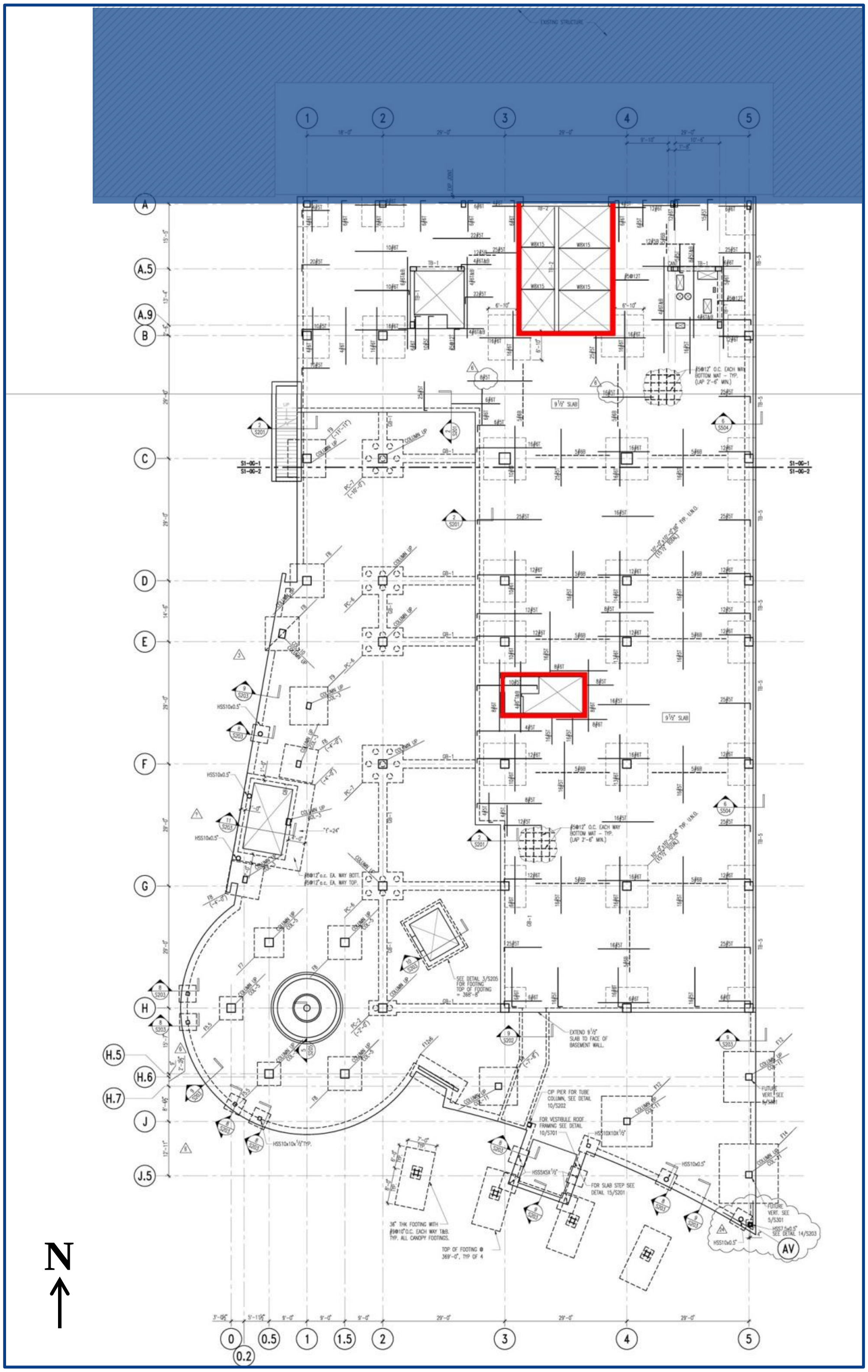


Figure 9:
Shear wall locations with existing building shaded in blue

Roof System:

In general, there are three different main roof levels (see Figure 10). The roofing system on the 11th floor is comprised mainly of Polyvinyl-Chloride (PVC) roofing situated on top of Composite Polyisocyanurate Board Insulation. This system rests on top of a concrete slab.

Highlighting the 11th floor roof is the pre-engineered aluminum helicopter landing system. Supporting the landing platform is a system of structural steel columns with vibration isolators (see Figure 11).

The main design features of the lower roof level (2nd floor) consist of a vegetated roof system, accent vegetation and concrete roof pavers. Also on the lower roof, a hexagonal skylight covers the circular rotunda (see Figure 12). The slab thickness for the lower roofs (excluding the green roof) is 9 ½" while the main roof, which supports higher loads from the mechanical penthouse, is 12" thick.

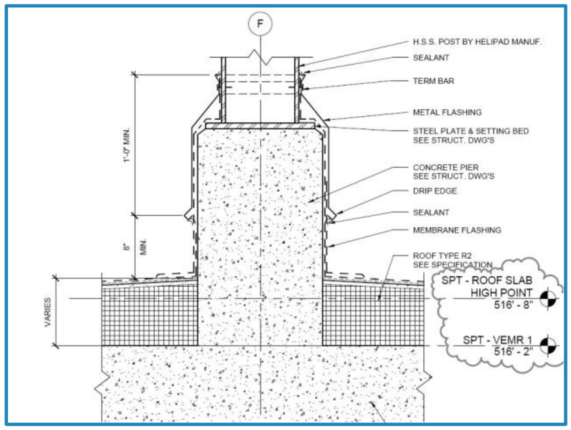


Figure 11:
Helipad support post

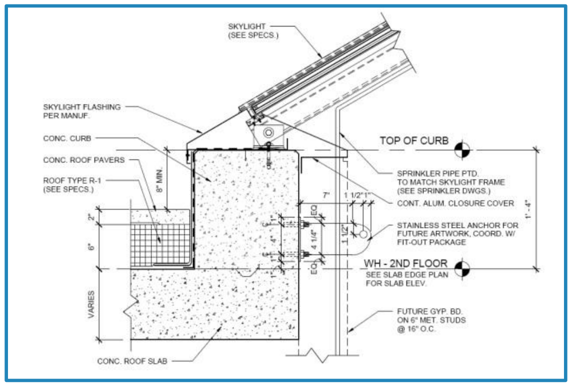


Figure 12:
Roof and skylight detail

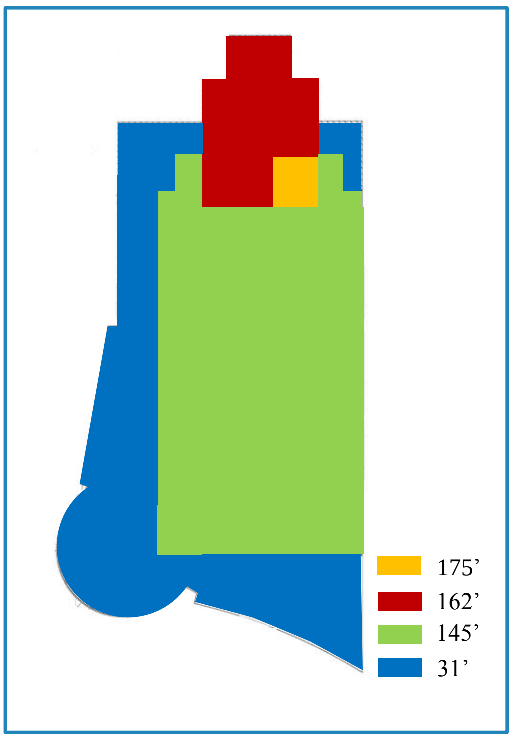


Figure 10:
Showing different roof heights in relation to 0'-0"

Design Codes:

According to Sheet S0-01, the original building was designed to comply with the following codes/standards:

- 2006 International Building Code (IBC 2006)
- 2006 Virginia Uniform Statewide Building Code (Supplement to 2006 IBC)
- Minimum Design Loads for Building and Other Structures (ASCE7-05)
- Building Code Requirements for Structural Concrete (ACI 318-05)
- American Concrete Institute Manual of Concrete Practice – Parts 1 through 5 (ACI)
- Manual of Standard Practice (Concrete Reinforcing Steel Institute)
- Manual of Steel Construction – Allowable Stress Design 9th Edition (American Institute of Steel Construction - AISC)
- Manual of Steel Construction, Volume II, Connections (ASD 9th Edition/LRFD 1st Edition – AISC)
- Detailing for Steel Construction (AISC)
- Structural Welding Code ANSI/DWS D1.1 (American Welding Society – AWS)
- Design Manual for Floor Decks and Roof Decks (Steel Deck Institute – SDI)
- Standard Specifications for Structural Concrete (ACI 301)

Thesis Codes and References:

- 2009 International Building Code
- ASCE 7-05
- ACI 318-08
- AISC Steel Manual - 14th Edition

Materials Used:

The various kinds of materials and standards used for the construction of the South Patient Tower are listed in Figure 12a and 12b on the following page. All information was derived from Sheet S0-01.

Concrete		
Usage	**Strength (psi)**	**Weight**
Piles	4000	Normal
Pile Caps	5000	Normal
Footings	3000	Normal
Grade Beams	3000	Normal
Foundation Walls	3000	Normal
Shear Walls	5000	Normal
Columns	5000/7000	Normal
Slabs-on-Grade	3500	Normal
Reinforced Slabs LG-L4	5000	Normal
Reinforced Beams LG-L4	5000	Normal
Reinforced Slabs L5-Roof	4000	Normal
Reinforced Beams L5-Roof	4000	Normal
Topping Slabs	3000	Lightweight
Concrete on Steel Deck	3000	Lightweight

Steel		
Type	**Standard**	**Grade**
Wide Flange Shapes and Tees	ASTM A992	50
Round Hollow Structural Shapes	ASTM A992	B (F_y = 35 ksi)
	ASTM 501	F_y = 36 ksi
Square or Rectangular Hollow Structural Shapes	ASTM A500	B (F_y = 46 ksi)
Other Structural Shapes and Plates	ASTM A36	N/A
High Strength Bolts	ASTM A325 N	N/A
Smooth and Threaded Rods	ASTM A572	N/A
Headed Shear Studs	ASTM A108	N/A
Welding Electrodes	AWS A5.1 or A5.5	E70xx
Galvanized Steel Floor Deck	ASTM A653 SS	33

Figure 12a:
Summary of materials used on the SPT project with design standards and strengths

Reinforcement	
Type	**Standard**
Deformed Reinforcing Bars	ASTM A615 (Grade 50)
Weldable Deformed Reinforcing Bars	ASTM A706
Welded Wire Fabric (WWF)	ASTM A185
Epoxy Coated Reinforcing Bars	ASTM A6775
Mechanical Connection Splices	DYIDAG, Lenton, or ACI 318 §12.14.3
Adhesive Reinforcing Bar Doweling Systems	ASTM A621

Miscellaneous	
Type	**Standard/Value**
Cement	ASTM C150 (Type I or II)
Blended Hydraulic Cement	ASTM C595
Aggregates	ASTM C33 (NW) ASTM C330 (LW)
Air Entraining Admixture	ASTM C260
Chemical Admixture	ASTM C494
Grout	ASTM C1107 (F'_c = 5000 psi)

Concrete Water Cementitious Ratio	
F'c @ 28 Days (psi)	**W/C (Max)**
$F'_c \leq 3500$	0.55
$3500 < F'_c < 5000$	0.50
$5000 \leq F'_c$	0.45

Figure 12b:
Summary of materials used on the SPT project with design standards and strengths

Gravity Loads:

As part of this technical report, the dead, live and snow loads have all been calculated and compared to the loads listed on the structural drawings. Following the determination of the various loads using ASCE 7-05, several gravity members part of the structural system were checked to verify their adequacy to carry the gravity loads. Detailed calculations for these members can be found in Appendix A.

Dead and Live Loads:

The structural drawings list the superimposed dead loads used by the structural engineers for the design of the gravity members which are summarized in Figure 13.

Superimposed Dead Loads	
Description	**Load**
Floors	20 psf
Standard Roof	20 psf
Main Roof	20 psf

Figure 13:
Summary of superimposed dead loads

Following the confirmation of the superimposed dead loads, these loads along with the weights of the slabs, columns, shear walls, roofs, façade and the drop panels were used to calculate the overall weight of the entire structure. The exterior walls are made up of 5 ½" concrete with a ½" thin brick face. To simplify calculating the weight of this system, a 6" concrete panel was assumed to account for both elements. Figure 14 on the following page shows the overall weight of each floor as well as the complete weight of the entire structure which was found to be approximately 38,600 k (detailed calculations of the weight can be found in Appendix C).

A comparison of the live loads used in the SPT and Table 4-1 in ASCE 7-05 resulted in very little differences except when it came to the loads used for the offices as well as the patient floors (see Figure 15). The offices were all designed for 60 + 20 psf partition loading, which is 10 psf over the value given in Table 4-1. This could be due to the fact that offices are located on floors with patient rooms and corridors which both have a total live load of 80 psf. To be conservative, the project engineer probably just used 80 psf to be on the safe side. One other difference in live load occurred with the patient floor levels. According to ASCE, the minimum live load for hospital patient floors is 40 psf + partitions. However, the engineers for the SPT used 60 psf + partitions. A possible explanation for the increased load could be attributed to the future needs of individualized patients. Because certain patients may need different equipment, the exact load is uncertain. Therefore, the more conservative value of 60 psf was chosen. Calculations involving the patient floors will use 60 psf + 20 psf for partitions for this report and future reports.

Live loads for both the café and the roof were not given, but a live load of 80 psf was assumed for the café. Since the main roof utilizes a helicopter landing system, the specification for the system indicated a minimum live load of 100 psf and therefore will be used. Because the green roof will be accessible, a live load of 100 psf will be used for the lower vegetated roofs.

Weight Per Level		
Level	**Area (ft^2)**	**Weight (kips)**
Ground	25512.5	N/A
1st	25512.5	4392.73
2nd	11649	2417.80
3rd	17958	3901.98
4th	16571	3010.72
5th	16571	3285.27
6th	16571	3078.14
7th	16571	3010.72
8th	16571	3010.72
9th	16571	3010.72
10th	16571	3010.72
11th	16571	3065.76
Penthouse/Roof	16571	3382.57
		38577.83

Figure 14:
Distribution of weight per floor level

Live Loads			
Space	**Design Live Load (psf)**	**ASCE 7-05 Live Load (psf)**	**Notes**
Assembly Areas	100 (U)	100	N/A
Corridors	100	100 (first floor) ; 80 psf above	Based on both "Corridors" and "Hospitals" Section
Patient Floors	60 + 20	60 + 20	Based on "Hospitals - Operating Rooms, Laboratories"
Lobbies	100	100	N/A
Marquess and Canopies	75	75	N/A
Mechanical Rooms	150 (U)	N/A	N/A
Offices	60 + 20	50 + 20	Office Load + Partition Load
Stairs and Exitways	100 (U)	100	N/A
Café	N/A	80	N/A
Roof	N/A	100	Based on Future Helicopter Landing System

Figure 15:
Comparison of live loads

Snow Loads:

Following the procedure outlined in Chapter 7 of ASCE 7-05 and using the snow load maps, the roof snow load and drift values were obtained. The factors used to calculate the flat roof snow load are summarized in Figure 16. A flat roof snow load of 21 psf was calculated which matched the structural drawings. Due to the different roof heights, drift was considered at multiple locations. A summary of the snow and drift calculations and results can be found in Appendix A.

Flat Roof Snow Load Calculations	
Variable	**Value**
Ground Snow Load - p_g (psf)	25
Exposure Factor - C_e	1
Temperature Factor - C_t	1
Importance Factor - I	1.2
Flat Roof Snow Load - p_f (psf)	**21**

Figure 16:
Summary of roof snow load values

Column C-3 Gravity Check:

Column C-3 was chosen because it is an interior column not part of the lateral force resisting system. Therefore lateral influences were not a factor in determining the adequacy of the column and second order affects could be neglected. The column falls along lines C and 3 in a 29'x29' bay and is a 30"x30" concrete column reinforced with (20) #11 vertical bars and #4 ties at 18" (see Figure 17). As the column travels up the building, the column changes size multiple times, ending up as a 24"x24" column at the main roof level. Loads were calculated at each level (13 levels total) and the final check for the column occurred at the ground level.

The live load from the roof was not allowed to be reduced as well as the mechanical floor (5th floor) and the ultrasound room. Other levels with 80 psf live loads were reduced accordingly. It was found that Column C-3 is more than adequate to carry the gravity loads. A detailed calculation and partial plan of the column can be found in Appendix A.

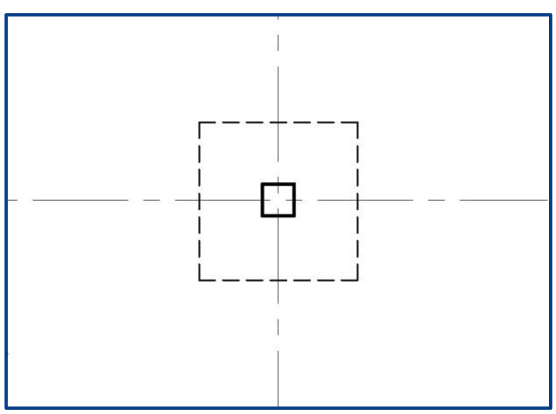


Figure 17:
Column C-3

Slab Gravity Check:

In the interest of performing a spot check on a slab that would relate to multiple floor levels, the slab chosen was a typical interior slab panel from the 6th-11th floors. The panel spans between column lines C and D. The Direct Design Method from ACI 318-08 was used to analyze and design the top and bottom reinforcement for the 9 ½” two-way flat slab system. Using Table 9.5(c) from ACI, the minimum thickness of the slab was found to be 9”. The structural engineers most likely used a thicker slab to compensate for the deflection criteria set forth by ACI 318-08. The frame analyzed is depicted in Figure 18 with the column and middle strip shown. Once the reinforcement was designed for the ultimate moment, a comparison was made to the structural drawings. The reinforcement calculated turned out to be quite comparable and almost exact to the engineer’s design. The deflection of the slab was then calculated for serviceability requirements laid out in ACI 318-08 Table 9.5(b). Both of the major deflection criterion (live load and total deflection after partitions) were met. The final check performed on the slab panel was shear. Both wide beam action and punching shear checks were completed with the two-way slab passing both checks. All calculations and results for the two-way flat slab system can be found in Appendix A.

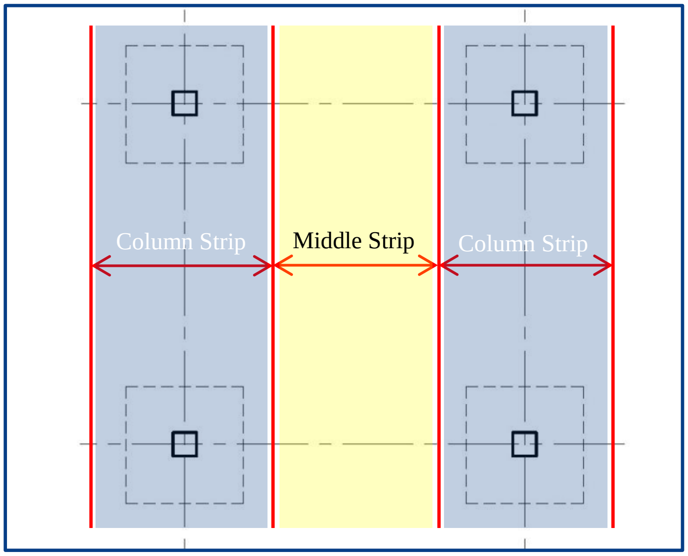


Figure 18:
Two-way slab system

Lateral Loads:

In order to obtain a better understanding of how the structural system of the SPT responds to lateral loads, both wind and seismic loads were calculated for this technical report. Hand calculations for both of these sections can be found in Appendices B (Wind) and C (Seismic).

Wind Loads:

Using the Method 2 procedure from Chapter 6 of ASCE 7-05 (Main Wind Force Resisting System – MWRFS), wind loads and pressures were found and applied to the building to find the story forces and eventually leading to the calculation of both the base shear and the overturning moment.

In order for Method 2 to be applied to the South Patient Tower, several simplifying assumptions had to be made. The main assumption involved in calculating the wind forces was ignoring the existing attached hospital. Also, because of the irregular shape of the first three levels of the SPT, the shape was transformed into a rectangle with the same area as the original footprint of the building. If the general shape for the third floor was used for the remaining upper portion of the building, the calculated forces would have been overestimated by a significant portion. To prevent this from happening, the tower itself was modeled with different proportions compared to the lower three levels (see Figure 19a and 19b). Using these two separate pieces allowed for a better estimation of the distribution of wind press and forces to each floor. Two different L/B values were used to obtain the leeward pressure. Because of the mechanical penthouse, the mean roof height used to calculate q_h was taken as the top of that structure which is at 175'.

The wind loads are collected by the components and cladding of the exterior of the building. The façade then transfers these wind forces to the slab system, which in turn sheds the load to the lateral force resisting system within the building and down to the foundation. The components and cladding pressures were not included in this technical report and therefore need to be addressed in technical report 3.

Most of the calculations for the wind section are achieved through the use of Microsoft Excel to simplify the process. The story forces at each level include both the windward and the leeward pressures. Internal pressures have been calculated but not included in the story forces due to the fact that they effectively cancel out. The following few pages contain figures and diagrams representing the pressures and forces for both the North-South and East-West directions. The base shear in the E-W direction was significantly higher than the N-S direction due to the slender nature of the building, and in turn the resulting moment also ended up being considerably greater.

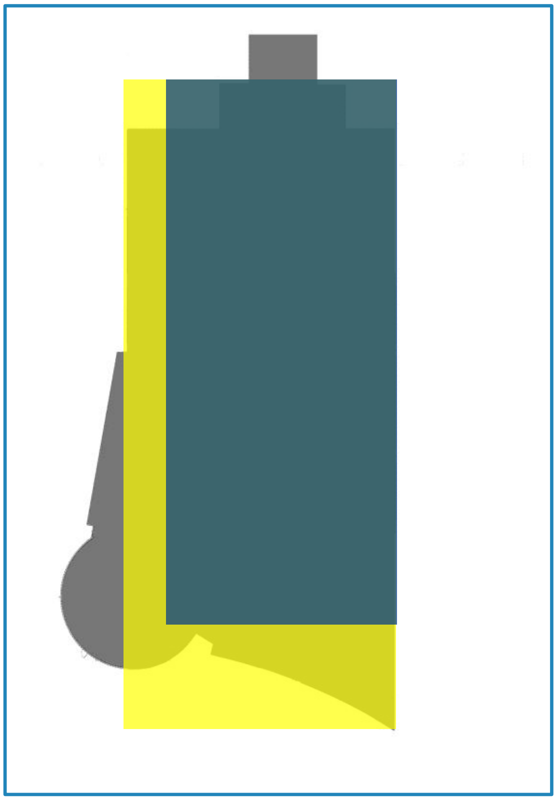


Figure 19a:
Plan view of the two separate wind towers

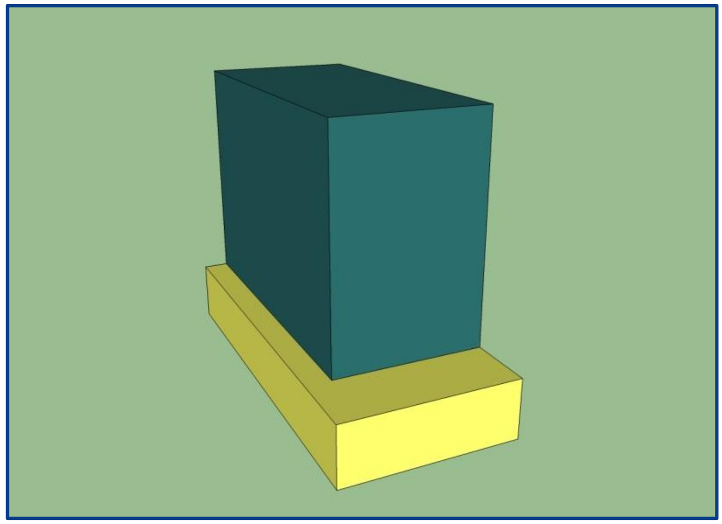


Figure 19b:
Perspective view of the two separate wind towers

Wind Pressures N-S Direction							
Wall Type	Floor	Distances (ft)	Wind Pressures (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(Gc_{pi})	(-)(Gc_{pi})	(+)(Gc_{pi})	(-)(Gc_{pi})
0' - 36.17'							
Windward Walls	Ground	0	7.86	4.23	-4.23	3.63	12.09
	1st	10.83	7.86	4.23	-4.23	3.63	12.09
	2nd	24.83	9.08	4.23	-4.23	4.85	13.31
	3rd	36.17	10.16	4.23	-4.23	5.93	14.39
Leeward Walls	All	All	-5.80	4.23	-4.23	-10.03	-1.57
Side Walls	All	All	-13.99	4.23	-4.23	-18.22	-9.76
36.17' - 175'							
Windward Walls	4th	47.50	10.99	4.23	-4.23	6.76	15.22
	5th	58.67	11.65	4.23	-4.23	7.42	15.88
	6th	72.93	12.43	4.23	-4.23	8.20	16.66
	7th	84.17	13.00	4.23	-4.23	8.77	17.23
	8th	95.50	13.46	4.23	-4.23	9.23	17.69
	9th	106.83	13.88	4.23	-4.23	9.65	18.11
	10th	118.17	14.27	4.23	-4.23	10.04	18.50
	11th	129.50	14.67	4.23	-4.23	10.44	18.90
	Penthouse	144.83	15.16	4.23	-4.23	10.93	19.39
	Roof	175.00	15.99	4.23	-4.23	11.76	20.22
Leeward Walls	All	All	-5.90	4.23	-4.23	-10.13	-1.67
Side Walls	All	All	-13.99	4.23	-4.23	-18.22	-9.76
Roof	N/A	0-87.5	-24.65	4.23	-4.23	-28.88	-20.42
	N/A	87.5-175	-14.65	4.23	-4.23	-18.88	-10.42
	N/A	175-350	-13.33	4.23	-4.23	-17.56	-9.10
	N/A	>350	-12.66	4.23	-4.23	-16.89	-8.43

Figure 20a:
List of N-S direction wind pressures

Wind Forces N-S Direction								
Floor Level	Elevation (ft)	Tributary Below		Tributary Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	5.42	568.58	7.77	303.89	0.00
1st	10.83	5.42	568.58	7.00	735.00	18.70	296.12	202.56
2nd	24.83	7.00	735.00	5.67	595.35	20.44	277.42	507.49
3rd	36.17	5.67	595.35	5.67	510.00	18.12	256.98	655.24
4th	47.50	5.67	510.00	5.58	502.50	17.43	238.86	828.11
5th	58.67	5.58	502.50	7.13	641.70	20.58	221.43	1207.50
6th	72.93	7.13	641.70	5.62	505.80	21.32	200.85	1555.01
7th	84.17	5.62	505.80	5.67	509.85	19.43	179.53	1635.45
8th	95.50	5.67	509.85	5.67	509.85	19.96	160.10	1905.75
9th	106.83	5.67	509.85	5.67	510.30	20.38	140.14	2176.94
10th	118.17	5.67	510.30	5.67	509.85	20.78	119.76	2455.62
11th	129.50	5.67	509.85	7.67	689.85	25.02	98.98	3239.55
Penthouse	144.83	7.67	689.85	15.09	1357.65	44.25	73.97	6408.32
Roof	175.00	15.09	1357.65	N/A	0.00	29.72	29.72	5200.82
					Total Base Shear =	303.89		
							Total Overturning Moment =	28,030.30 k-ft

Figure 21a:
List of N-S direction wind forces

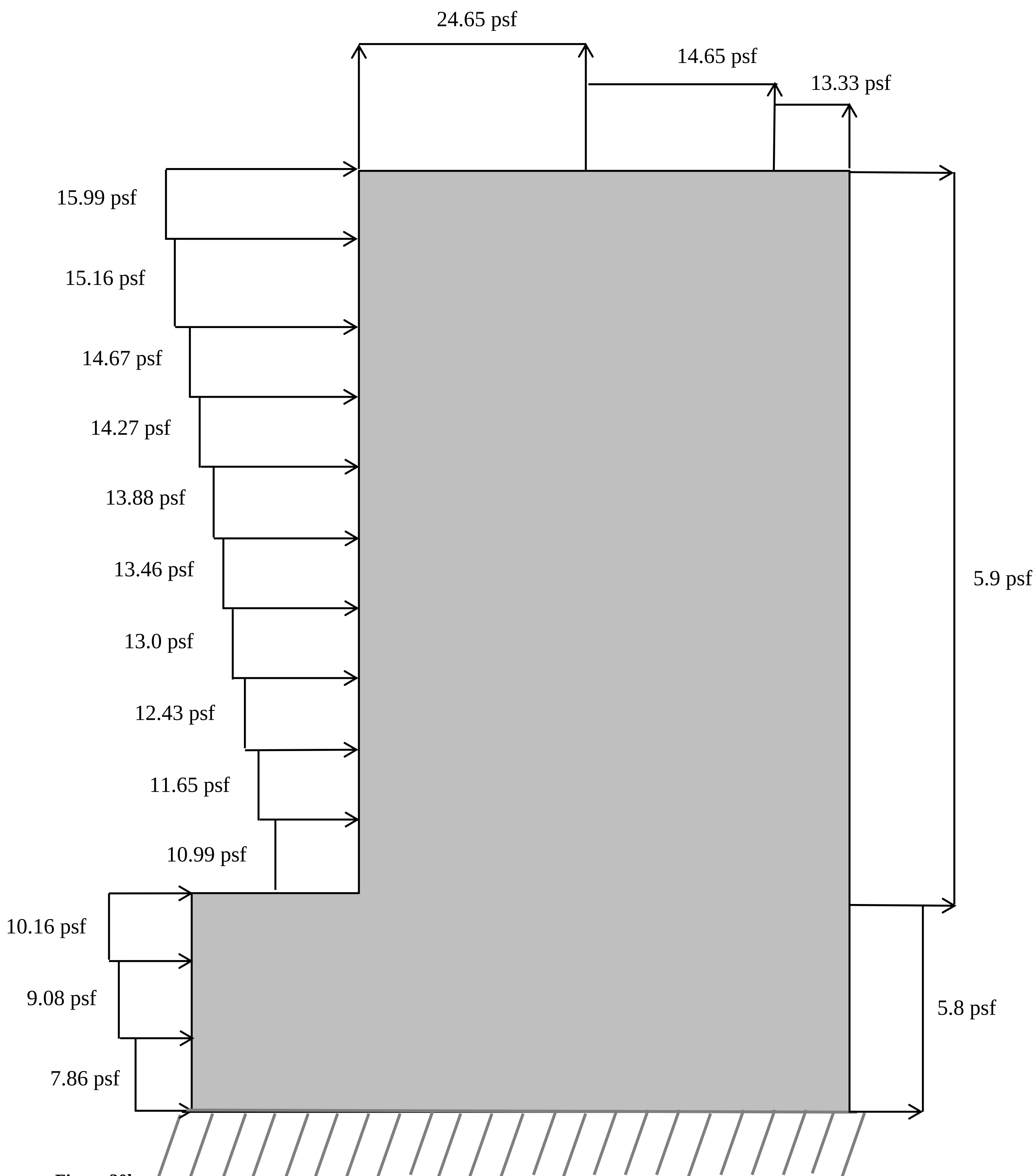


Figure 20b:
Diagram of N-S direction wind pressures

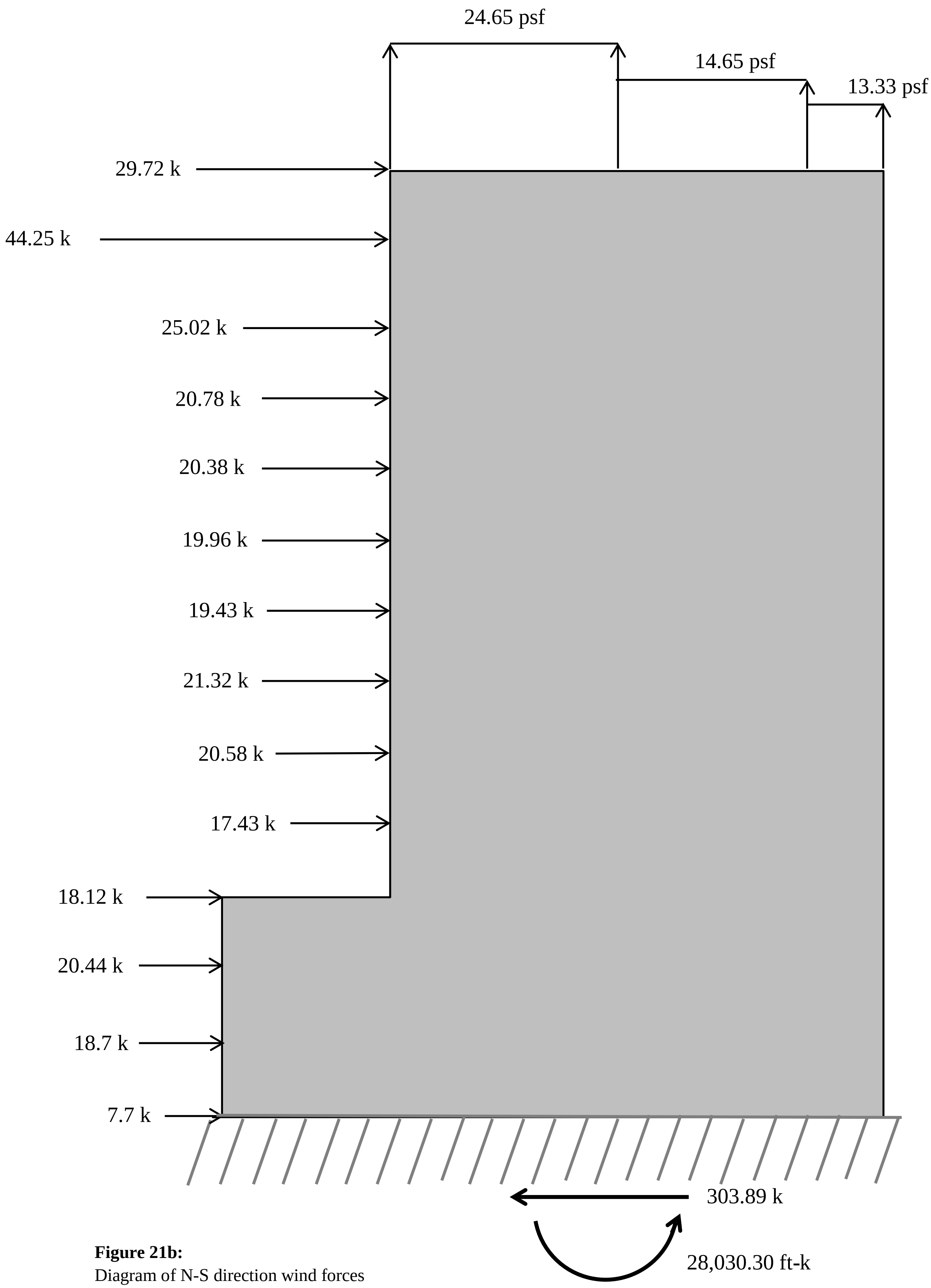


Figure 21b:
Diagram of N-S direction wind forces

Wind Pressures E-W Direction							
Wall Type	Floor	Distances (ft)	Wind Pressures (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(Gc_{pi})	(-)(Gc_{pi})	(+)(Gc_{pi})	(-)(Gc_{pi})
0' - 36.17'							
Windward Walls	Ground	0	7.86	4.23	-4.23	3.63	12.09
	1st	10.83	7.86	4.23	-4.23	3.63	12.09
	2nd	24.83	9.08	4.23	-4.23	4.85	13.31
	3rd	36.17	10.16	4.23	-4.23	5.93	14.39
Leeward Walls	All	All	-9.99	4.23	-4.23	-14.22	-5.76
Side Walls	All	All	-13.99	4.23	-4.23	-18.22	-9.76
36.17' - 175'							
Windward Walls	4th	47.50	10.99	4.23	-4.23	6.76	15.22
	5th	58.67	11.65	4.23	-4.23	7.42	15.88
	6th	72.93	12.43	4.23	-4.23	8.20	16.66
	7th	84.17	13.00	4.23	-4.23	8.77	17.23
	8th	95.50	13.46	4.23	-4.23	9.23	17.69
	9th	106.83	13.88	4.23	-4.23	9.65	18.11
	10th	118.17	14.27	4.23	-4.23	10.04	18.50
	11th	129.50	14.67	4.23	-4.23	10.44	18.90
	Penthouse	144.83	15.16	4.23	-4.23	10.93	19.39
	Roof	175.00	15.99	4.23	-4.23	11.76	20.22
Leeward Walls	All	All	-9.99	4.23	-4.23	-14.22	-5.76
Side Walls	All	All	-13.99	4.23	-4.23	-18.22	-9.76
Roof	N/A	0-87.5	-20.79	4.23	-4.23	-25.02	-16.56
	N/A	87.5-175	-13.99	4.23	-4.23	-18.22	-9.76
	N/A	175-350	-13.99	4.23	-4.23	-18.22	-9.76
	N/A	>350	-13.99	4.23	-4.23	-18.22	-9.76

Figure 22a:
List of E-W direction wind pressures

Wind Forces E-W Direction								
Floor Level	Elevation (ft)	Tributary Below		Tributary Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	5.42	1250.87	22.33	791.93	0.00
1st	10.83	5.42	1250.87	7.00	1617.00	53.16	769.61	575.77
2nd	24.83	7.00	1617.00	5.67	1309.77	57.23	716.44	1420.97
3rd	36.17	5.67	1309.77	5.67	1080.92	49.07	659.21	1774.84
4th	47.50	5.67	1080.92	5.58	1065.02	45.72	610.14	2172.07
5th	58.67	5.58	1065.02	7.13	1360.05	53.54	564.42	3141.15
6th	72.93	7.13	1360.05	5.62	1072.02	55.14	510.88	4021.21
7th	84.17	5.62	1072.02	5.67	1080.60	49.99	455.74	4207.29
8th	95.50	5.67	1080.60	5.67	1080.60	51.13	405.76	4883.29
9th	106.83	5.67	1080.60	5.67	1081.55	52.03	354.62	5558.62
10th	118.17	5.67	1081.55	5.67	1080.60	52.89	302.59	6249.54
11th	129.50	5.67	1080.60	7.67	1462.10	63.42	249.70	8212.81
Penthouse	144.83	7.67	1462.10	15.09	2877.46	111.53	186.28	16152.64
Roof	175.00	15.09	2877.46	N/A	0.00	74.76	74.76	13082.39
					Total Base Shear =	791.93		
							Total Overturning Moment =	71,625.68 k-ft

Figure 23a:
List of E-W direction wind forces

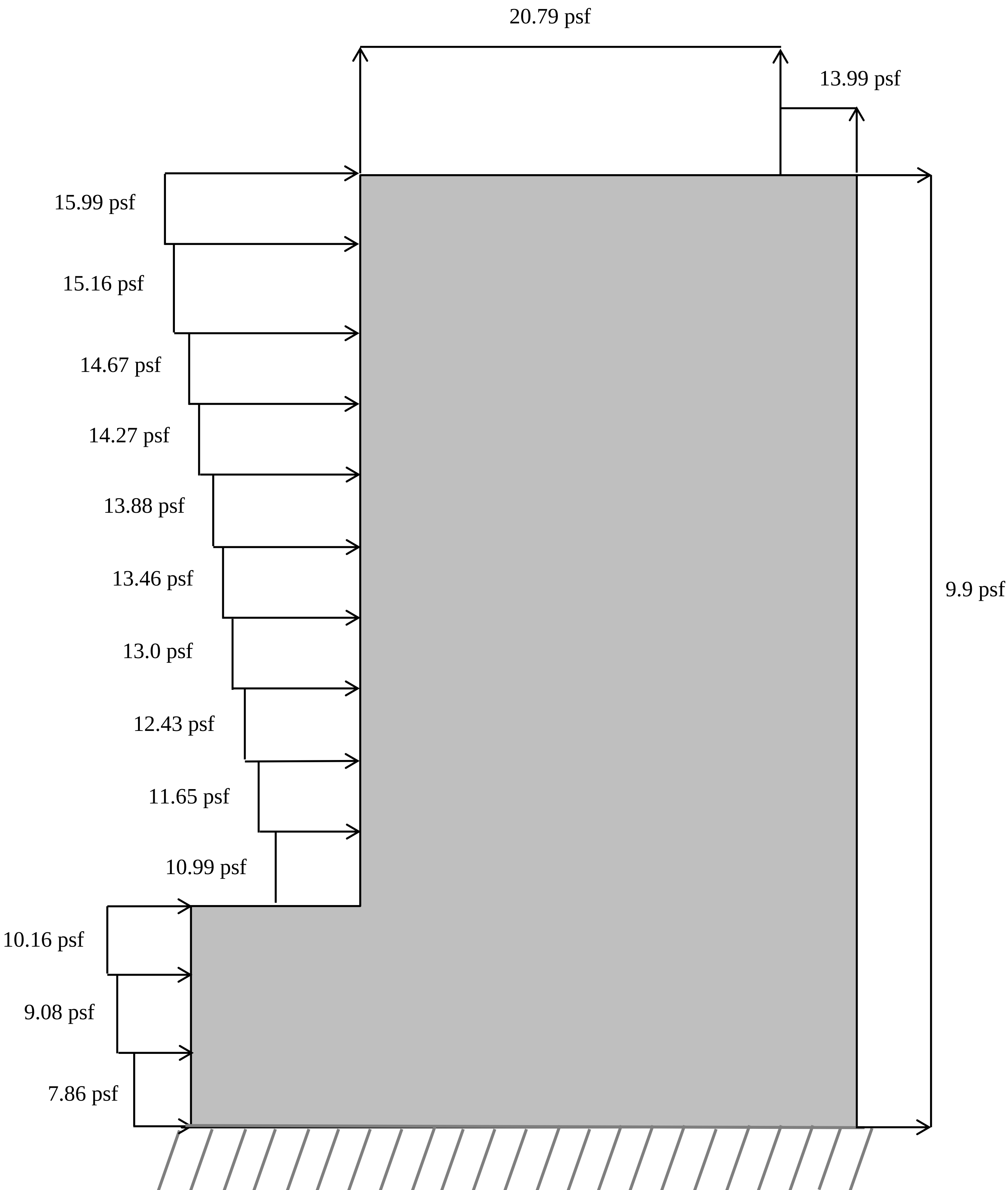


Figure 22b:
Diagram of E-W direction wind pressures

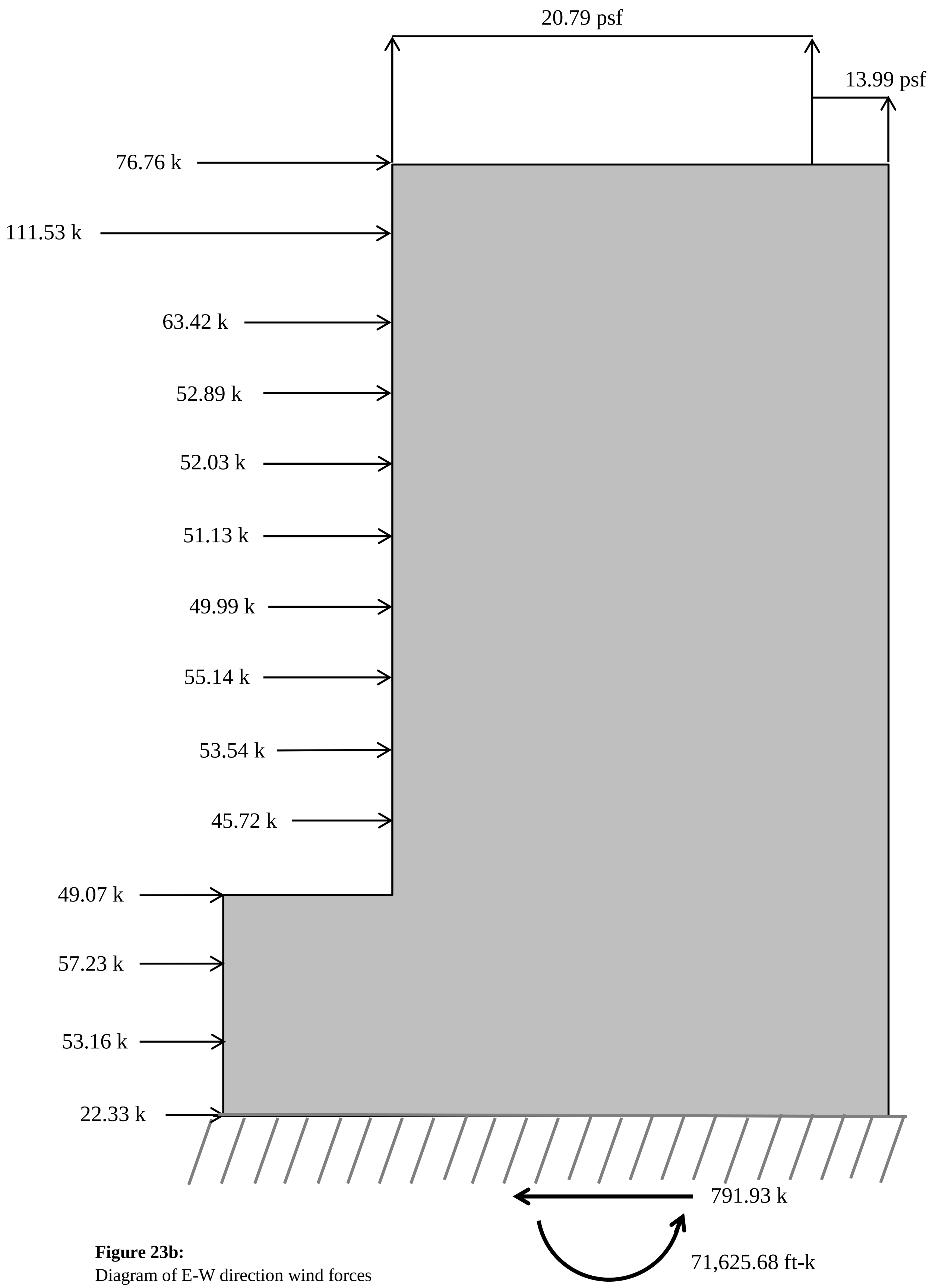


Figure 23b:
Diagram of E-W direction wind forces

Seismic Loads:

Using Chapters 11 and 12 of ASCE 7-05, the seismic loads were calculated with the Equivalent Lateral Force procedure. The approximate fundamental period for the structure was estimated using §12.8.2.1 and the "All other Structural Systems" category. The increased stiffness from the connected portion of the existing hospital was ignored in this study of the seismic loads. The movement of the loads due to seismic activity originates where most of the mass is locked, the two-way slab systems. The slabs then transfer the load to the shear walls and moment frames which in turn carry the forces down to the foundation.

The seismic loads generated a base shear of approximately 692.5 k which only differed by about 1.07% from the structural drawings. This slight discrepancy is likely due to a difference in the calculated weight. Once assumption made to simplify the seismic analysis revolved around the penthouse. Because the penthouse spans from both the existing hospital and the South Patient Tower, the penthouse was not included in the height of the overall structure. The main reason behind this thought process was that the story forces from the seismic loads will be shared between the buildings. The weight of the penthouse was included and lumped on the main roof level to increase the story forces seen by that level. Figures 24a and 24b list and display the story forces.

Seismic Forces N-S and E-W Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	N/A	0	0	0	0	692.50	0
1st	4392.7	10.67	155808.37	0.0052	3.57	692.50	38.07840844
2nd	2417.8	24.67	303505.33	0.0100	6.95	688.93	171.5284128
3rd	3902.0	36.00	866097.18	0.0287	19.84	681.98	714.3785554
4th	3010.7	47.33	1009605.78	0.0334	23.13	662.13	1094.909442
5th	3285.3	58.67	1522642.55	0.0504	34.89	639.00	2046.673942
6th	3078.1	72.67	1969868.32	0.0652	45.13	604.11	3279.681881
7th	3010.7	84.00	2397250.26	0.0793	54.93	558.98	4613.727547
8th	3010.7	95.33	2901211.23	0.0960	66.47	504.06	6336.995712
9th	3010.7	106.67	3436576.58	0.1137	78.74	437.58	8398.738013
10th	3010.7	118.00	4001651.25	0.1324	91.69	358.85	10818.83578
11th	3065.8	129.33	4678992.06	0.1548	107.20	267.16	13865.06993
Penthouse/Roof	3382.6	158.00	6981386.28	0.2310	159.96	159.96	25273.07233
						Base Shear =	692.5 k
						Total Overturning Moment =	76,651.69 k-ft

Figure 24a:
List of seismic forces

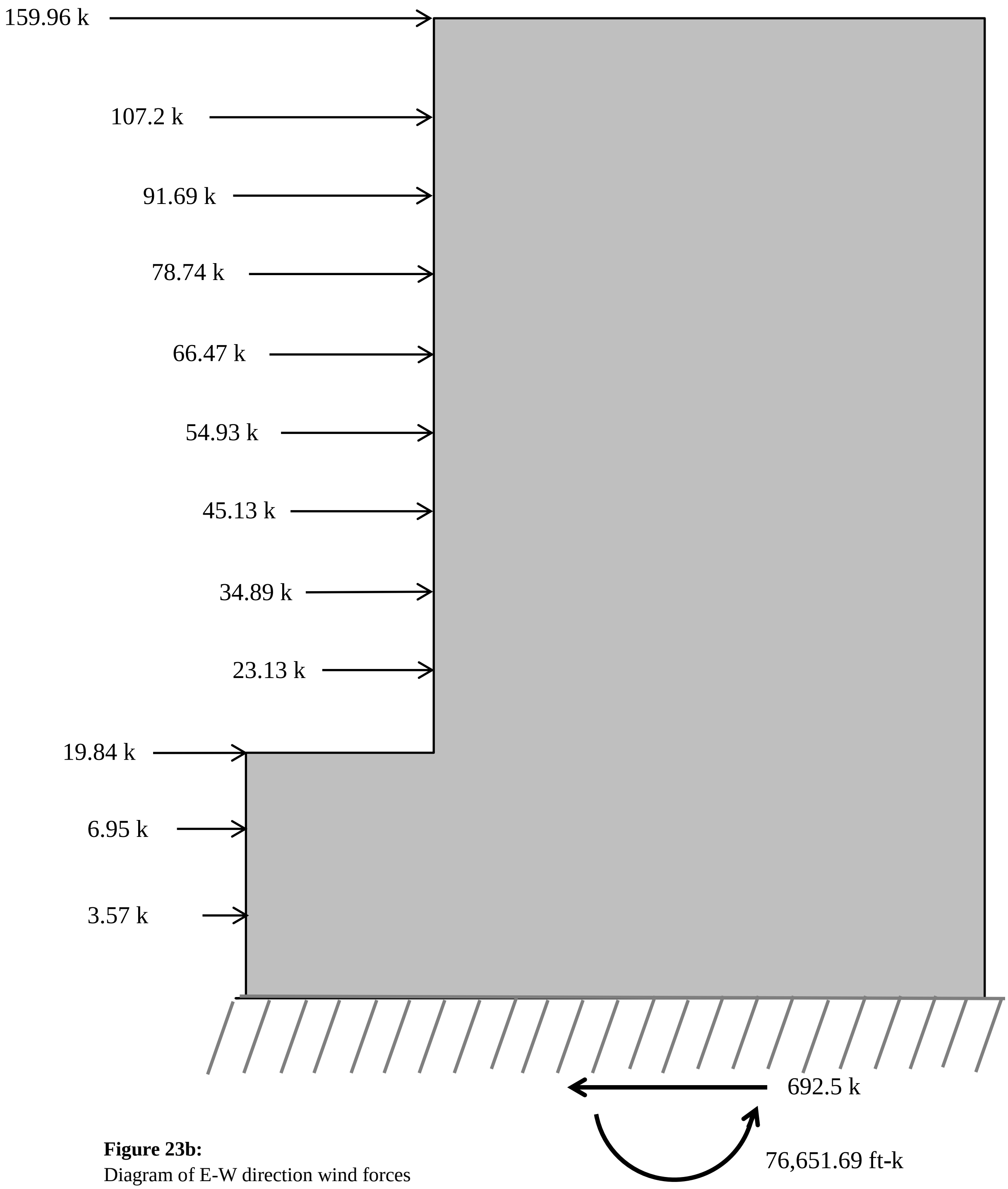


Figure 23b:
Diagram of E-W direction wind forces

Conclusion:

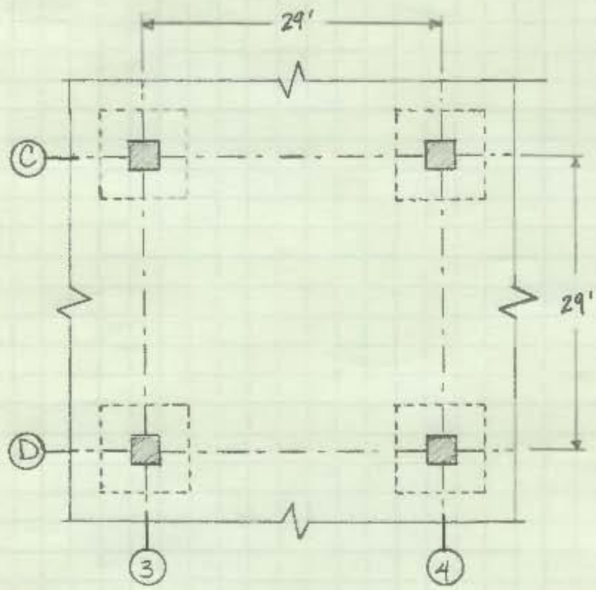
Technical Report 1 analyzed and discussed the findings from the study of the structural system of the South Patient Tower. By examining and analyzing each component, a greater understanding of the structural system as a whole was gained. Through spot checks, it was determined that the current structure for the SPT is adequate to carry all of the gravity loads down to the foundation. The specific gravity checks included a typical slab panel and column. The two-way slab also passed the deflection criterion set forth by ACI 318-08 as well as the shear checks and limitations.

Also included in this report was the determination of the various gravity loads and the lateral loads. Superimposed dead loads were used for the determination of the total building weight while the live loads were checked against ASCE 7-05 and differences were noted and explained.

Finally, wind and seismic loads were both calculated to see what kind of base shear and overturning moments these forces would produce. Many simplifications were made to the geometry of the building in order to use the procedures set forth in ASCE 7-05. Seismic loads were found to control in the North-South direction, but in the East-West direction, the wind base shear controlled and the seismic overturning moment controlled. Therefore, both of these lateral loads need to be accounted for and designed properly. The design seismic loads listed on the structural drawings were matched to within a reasonable percent difference. In the third technical report, it will be determined how the lateral force resisting members handle the later loads and distribute them down to the foundation.

Appendix A: Gravity Load Calculations

NATHAN MCGRAW | SPOTCHECK - SLAB (TECH1) | PAGE 1 OF 7



COLUMN SIZES = 24" x 24"

LIVE LOAD = 80 PSF

F_y = 60 KSI

F'_c = 4000 PSI

1 TYPICAL INTERIOR PANEL
FLOORS 6 - 10

USING ACI 318-08

TABLE 9.5 (c) : Minimum thickness of slabs without interior beams and with drop panels

For $f_y = 60{,}000$ psi $\rightarrow$ $\frac{l_n}{36} = \frac{(29' - 2') \times 12"}{36} = 9.0"$

*TO BE CONSERVATIVE, THE STRUCTURAL ENGINEERS USED A SLAB THICKNESS $t = 9.5"$. THE FOLLOWING CALCULATIONS WILL USE $t = 9.5"$ TO REMAIN CONSISTENT

DIRECT DESIGN METHOD:

1. 3 Continuous spans in each direction ✓
2. Panel ratio ≤ 2
   $\frac{l_2}{l_1} = \frac{29'}{29'} = 1.0 < 2$ ✓
3. $l_1 \geq 2/3\, l_2$ ✓
4. Can't have a column offset of more than 10% of length ✓

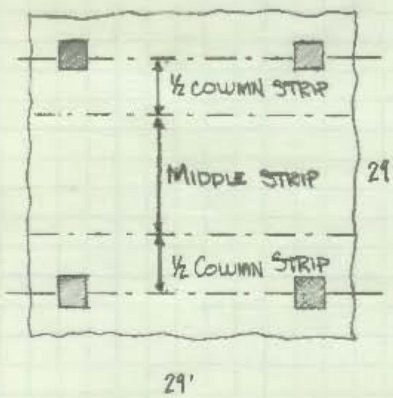
NATHAN MCGRAW | SPOT CHECK - SLAB (TECH 1) | PAGE 2 OF 7

5. $W_L \leq 2W_D$

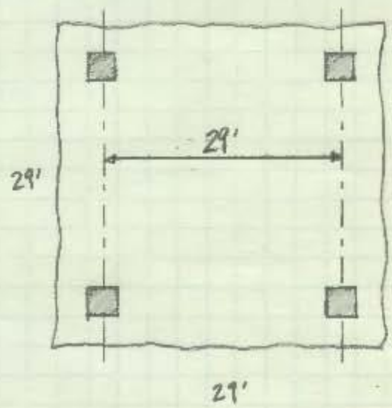
$W_D = (9.5''/12'')(150\ lb/ft^3) = 118.75$ psf
$W_{DSI} = 20$ psf

$W_L = 80\text{ psf} \leq 2(118.75 + 20) = 277.5\text{ psf} = W_D$ ✓

OK TO USE DIRECT DESIGN METHOD



2 FRAME A



3 FRAME B

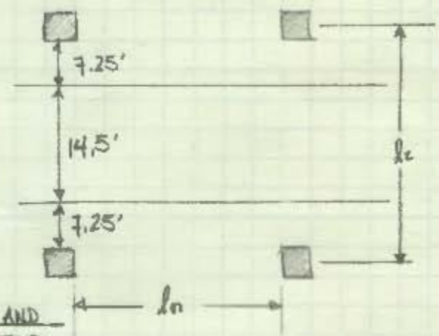
* SINCE FRAME A AND FRAME B HAVE THE SAME DIMENSION, ONLY NEED TO DESIGN ONE COLUMN STRIP AND ONE MOMENT STRIP FOR ONE FRAME

½ COLUMN STRIP $= (29'/4) = 7.25'$

MIDDLE STRIP $= (29'/2) = 14.5'$

MOMENT: $M_o = \dfrac{W_u l_2 l_n^2}{8}$

$W_u = 1.2D + 1.6L$
$W_u = 1.2(138.75\text{ psf}) + 1.6(80\text{ psf})$
$W_u = 294.5$ psf



4 COLUMN AND MIDDLE STRIP

NATHAN MCGRAW | SPOT CHECK - SLAB (TECH 1) | PAGE 3 OF 7

$$M_o = \frac{(294.5\ psf)(29')(29' - 2')^2}{8} = 778.25\ ft\text{-}K$$

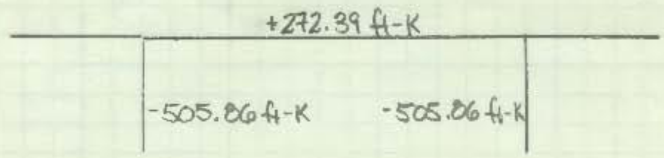
USING ACI 318-08 § 13.6.3.3: EXTERIOR EDGE FULL RESTRAINED

INTERIOR NEGATIVE FACTORED MOMENT = $0.65\ M_o$

POSITIVE FACTORED MOMENT = $0.35\ M_o$

INTERIOR NEGATIVE MOMENT = $0.65 M_o = 0.65(778.25) = 505.86$ ft-K

POSITIVE FACTORED MOMENT = $0.35(778.25) = 272.39$ ft-K



5 NEGATIVE AND POSITIVE MOMENTS

DISTRIBUTION OF MOMENTS: (ACI 318-08 § 13.6.4)

$\alpha_1 = 0 \Rightarrow$ No interior beams

$l_2/l_1 = 29'/29' = 1.0$

$\alpha_1 (l_2/l_1) = 0$

$\beta_T = 0 \Rightarrow$ No edge beams

1) NEGATIVE MOMENT @ INTERIOR SUPPORT = 75%

2) POSITIVE MOMENT OF INTERIOR PANEL = 60%

NATHAN MCGRAW | SPOT CHECK - SLAB (TECH 1) | PAGE 4 OF 7

-505.86 → 75% to column strip = -379.40 ft-K → 100% to slab
-505.86 → 25% to middle strip = -126.46 ft-K

+272.39 → 60% to column strip = 163.43 ft-K → 100% to slab
+272.39 → 40% to middle strip = 108.96 ft-K

SUMMARY:

FRAME B	TOTAL WIDTH = 29'	COLUMN STRIP = 14.5'	MIDDLE STRIP = 14.5'
TOTAL MOMENT	-505.86	+272.39	-505.86
MOMENT IN COLUMN STRIP SLAB	-379.40	+163.43	-379.40
MOMENT IN MIDDLE STRIP SLAB	-126.46	+108.96	-126.46

NATHAN MCGRAW | SPOT CHECK - SLAB (TECH 1) | PAGE 5 OF 7

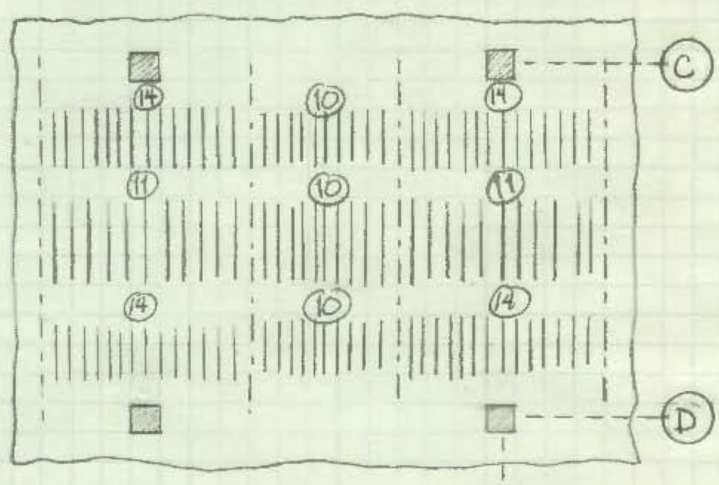
MIDDLE STRIP

DESCRIPTION:	INTERIOR SPAN M^-	M^+
1) MOMENT M_u (K-ft)	-126.46	+108.96
2) WIDTH OF COLUMN STRIP	174"	174"
3) EFFECTIVE DEPTH	8.375"	8.375"
4) $M_n = M_u/\phi$	-140.51	121.07
5) $R = \frac{M_n \times 12000}{bd^2}$	138.16	119.04
6) ρ (TABLE A.5a NILSON)	0.00235	0.00202
7) $A_s = \rho bd$	3.42	2.94
8) $A_{s,min} = 0.0018bt$	2.9754	2.9754
9) $N = \frac{\text{Larger of 7 or 8}}{0.44}$	7.77 = 8	6.76 = 7
10) $N_{MIN} = \frac{\text{width of strip}}{2t}$	9.16 = 10	9.16 = 10

REINFORCEMENT DESIGN AND DISTRIBUTION: ASSUMING #6 BARS

COLUMN STRIP — INTERIOR SPAN

DESCRIPTION:	M^-	M^+
1) MOMENT M_u (K-ft)	-379.4	$+163.43$
2) WIDTH OF COLUMN STRIP	$14.5' \times 12" = 174"$	174"
3) EFFECTIVE DEPTH	$15.5" - 0.75" - \frac{1}{2}(5/8") = 14.4375"$	8.375
4) $M_n = M_u/\phi = 0.9$	-421.56	$+181.59$
5) $R = \frac{M_n \times 12000}{bd^2}$	139.48	178.55
6) ρ (TABLE A.5a Nilson)	0.00237	0.00306
7) $A_s = \rho bd$	5.9537 in²	4.45 in²
8) $A_{s,min} = 0.0018bt$	2.9754	2.9754
9) $N = \frac{\text{Larger of 7 or 8}}{0.44}$ (↳ Area of #6)	13.53 = 14	10.1 = 11
10) $N_{MIN} = \frac{\text{width of strip}}{2t}$	9.16 = 10	9.16 = 10



6 REINFORCEMENT DETAIL

4

REINFORCEMENT COMPARISON WITH STRUCTURAL DRAWINGS:

COLUMN STRIP:

$M^- \rightarrow (14)\#6 = (14)(0.44\ in^2) = 6.16\ in^2$

A_s provided $= (14)\#6 = (14)(0.44\ in^2) = 6.16\ in^2 > 5.95\ in^2$ ✓

* SOME COLUMN STRIPS PROVIDE (16)#6 WHICH IS OK

$M^+ \rightarrow (11)\#6 = (11)(0.44\ in^2) = 4.84\ in^2$

A_s provided $= \#5 @ 12"$ o.c. $= (14.5')(0.31\ in^2/ft) = 4.495\ in^2 > 4.45\ in^2$ ✓

MIDDLE STRIP:

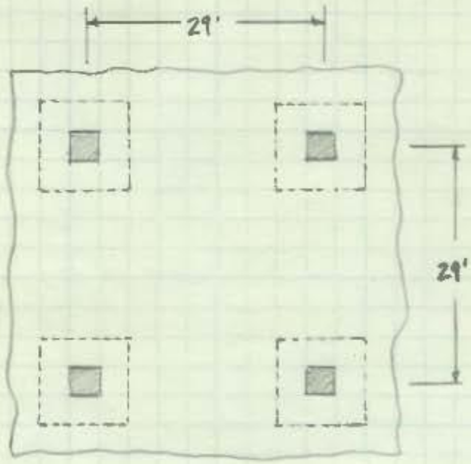
$M^- \rightarrow (10)\#6 = (10)(0.44\ in^2) = 4.4\ in^2$

A_s provided $= (16)\#5 = (16)(0.31\ in^2) = 4.96\ in^2 > 3.42\ in^2$ ✓

$M^+ \rightarrow (10)\#6 = (10)(0.44\ in^2) = 4.4\ in^2$

A_s provided $= \#5 @ 12" = (14.5')(0.31\ in^2/ft) = 4.495\ in^2 > 2.9754\ in^2$ ✓

NATHAN MCGRAW | (TECH 1) SPOT CHECK - DEFLECTION | PAGE 1 OF 2



$t = 9.5"$

ASSUMED: (Weighted Average)

- 67.5 % of moment to column strip
- 32.5 % of moment to middle strip

DL = 138.75 psf
LL = 80 psf

(1) TYPICAL INTERIOR PANEL

IMMEDIATE DEFLECTION DUE TO TOTAL DEAD LOAD:

COLUMN STRIP:

$W_D = (138.75\text{ psf})(29')(0.675) = 2.716\text{ K/ft}$

$I_g = \dfrac{(14.5' \times 12")(9.5")^3}{12} = 12431.9375\text{ in}^4$

$E_c = 57000\sqrt{4000\text{ psi}} = 3605\text{ ksi}$

$\Delta_{D(max)} = \dfrac{0.0026(2.716)(29')^4(12")^3}{(3605)(12431.9375)} = 0.1926"$

MIDDLE STRIP

$W_D = (138.75\text{ psf})(29')(0.325) = 1.308\text{ K/ft}$

$I_g = \dfrac{(14.5' \times 12")(9.5")^3}{12} = 12431.9375\text{ in}^4$

$\Delta_{D(max)} = \dfrac{0.0026(1.308)(29')^4(12")^3}{(3605)(12431.9375)} = 0.0927"$

TOTAL IMMEDIATE Δ DUE TO $DL_{TOTAL} = 0.1926" + 0.0927" = 0.285"$

NATHAN M^cGRAW | (TECH 1) SPOT CHECK - DEFLECTION | PAGE 2 OF 2

IMMEDIATE DEFLECTION DUE TO TOTAL LIVE LOAD:

COLUMN STRIP:

$W_L = (80\text{psf})(29')(0.675) = 1.566\ \text{k/ft}$

$\Delta_{L(MAX)} = \dfrac{0.0048(1.566)(29')^4(12")^3}{(3605)(12431.9375)} = 0.205"$

MIDDLE STRIP:

$W_L = (80\text{ psf})(29')(0.325) = 0.754\ \text{k/ft}$

$\Delta_{L(MAX)} = \dfrac{0.0048(0.754)(29')^4(12")^3}{(3605)(12431.9375)} = 0.0987"$

TOTAL IMMEDIATE Δ DUE TO LL_{TOTAL} = $0.205" + 0.0987" = 0.304"$

ADDITIONAL DL Δ AFTER A LONG TIME DUE TO DL_{TOTAL}

* ASSUME $\lambda = 3.0$

$\Delta_{D(MAX)} = (3.0)[0.285 + 0.25(0.304)] = 1.083"$

CHECK DEFLECTIONS WITH ACI318-08: TABLE 9.5(b)

LIVE LOAD: $l/360$ (Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections)

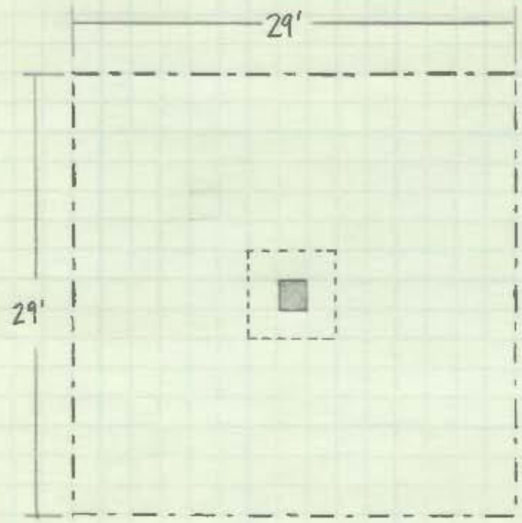
$\dfrac{l}{360} = \dfrac{(29' \times 12")}{360} = 0.967" > 0.304"$ ✓

TOTAL DEFLECTION AFTER PARTITIONS:

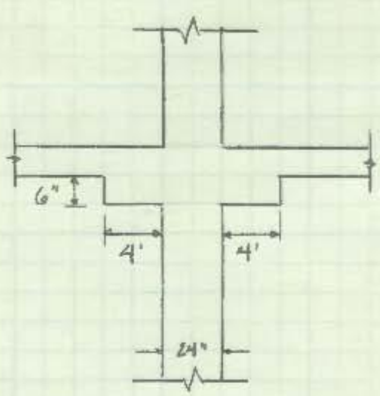
Δ_{max} for partitions $= 0.1(0.285) + 0.304 + 1.083" = 1.4155"$

ACI 318-08 → $\dfrac{l}{240} = \dfrac{(29' \times 12")}{240} = 1.45" > 1.4155"$ ✓

NATHAN McGRAW | SPOT CHECK - SHEAR (TECH 1) | PAGE 1 OF 3

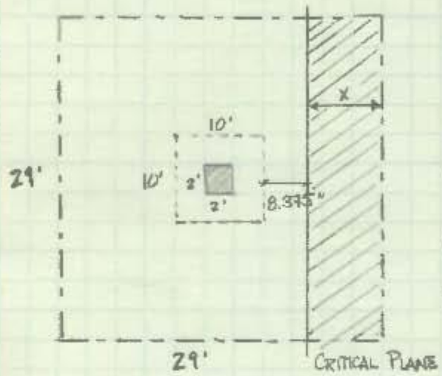


1 PLAN VIEW OF TYPICAL INTERIOR COLUMN



2 TYPICAL COLUMN ELEVATION

WIDE BEAM ACTION:



$d = 8.375"$

$x = 29/2 - 5' - (8.375"/12") = 8.8'$

$W_u = 1.2(138.75\ psf) + 1.6(80\ psf)$

$W_u = 294.5\ psf$

3 WIDE BEAM ACTION

$V_n = V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}\,(29' \times 12")(8.375") = 368.66^k$

$\phi V_n = 368.66^k (0.75) = 276.49^k$

$V_u = W_u \times 8.8' \times 29'$

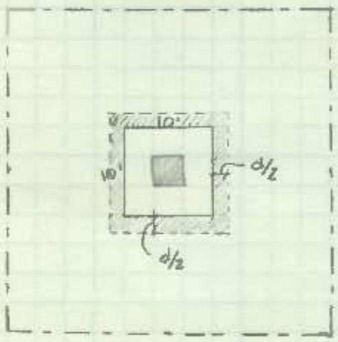
$V_u = (0.2945)(8.8)(29)$

$V_u = 75.16^k$

$\phi V_n = 276.49^k > V_u = 75.16^k$ ✓

NATHAN M^cGRAW | SPOT CHECK - SHEAR (TECH 1) | PAGE 3 OF 3

PUNCHING SHEAR:



$d/2 = 8.375/2 = 4.1875"$

$b_o = 2(120" + 8.375 + 120" + 8.375") = 513.5"$

$b_o/d = \frac{513.5"}{8.375"} = 61.31$

$120" + 8.375" = 128.375" = 10.70'$

4 PUNCHING SHEAR

$V_c = (\alpha_s / b_o/d + 2)\sqrt{f'_c}\, b_o d$ $\alpha_s = 40 \rightarrow$ interior column

$V_c = (40/61.31 + 2)\sqrt{4000}(513.5)(8.375) = 721.43^k$

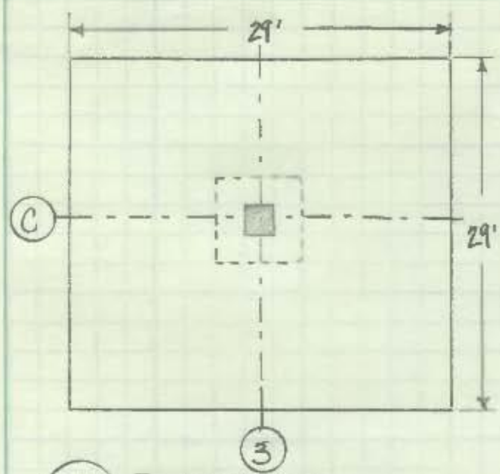
$\phi V_c = 0.75(721.43^k) = 541.1^k$

$V_u = W_u$ AREA

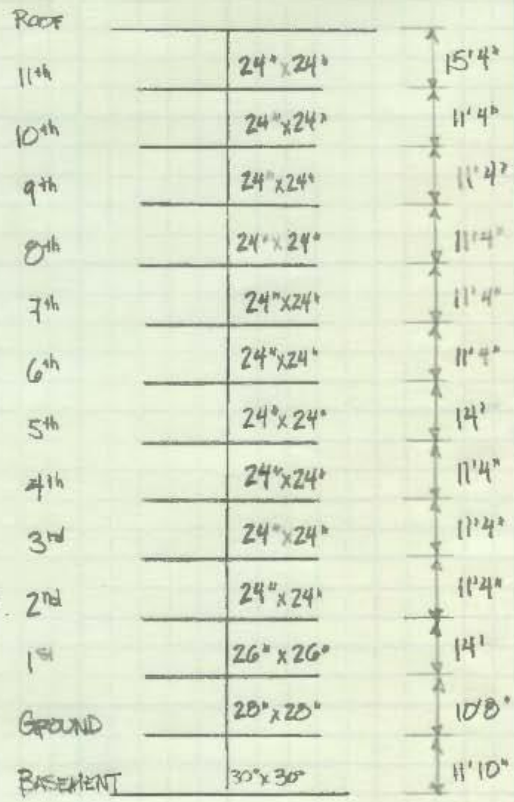
$V_u = (0.2945)(29' \times 29' - 10.70' \times 10.70')$

$V_u = 213.97^k < \phi V_c = 541.1^k$ ✓

NATHAN McGRAW | SPOT CHECK - COLUMN (TECH1) | PAGE 1 OF 3



1 PARTIAL PLAN OF COLUMN C3



2 ELEVATION

TRIBUTARY AREA = 29' x 29' = 841 ft^2
INFLUENCE AREA = 4 x 841 ft^2 = 3364 ft^2
LOADS:

ROOF:

LL = 100 psf (Unreducible)
DL = 152.9375 psf
S = 21 psf
SDL = 20 psf
P_{DL} DROP PANELS = 7.5^k

TYPICAL FLOOR LOAD:

LL = | 80 psf corridors
min | 60 + 20 psf patient floors
= 80 psf

$LL_{MECHANICAL\ FLOOR}$ = 150 psf (Unreducible)
SDL = 20 psf
DL = (9.5"/12")(150 lb/ft^3) = 118.75 psf
P_{DL} DROP PANELS = (50 ft^3)(150 lb/ft^3) = 7.5^k
$DL_{MECHANICAL\ FLOOR}$ = (10.5/12)(150) = 131.25
$LL_{ULTRASOUND}$ = 150 psf (Unreducible)
$K_{LL}A_T$ > 400 # ⇒ USE LIVE LOAD REDUCTION

NATHAN McGRAW | SPOT CHECK COLUMN (TECH 1) | PAGE 2 OF 3

COLUMN LOADS :

ROOF: $P_D = (841)(152.9375 + 20)/1000 = 145.4^K + 7.5^K = 152.9^K$
$P_L = (841)(100)/1000 = 84.1^K$

11th: $P_D = [(841)(20 + 118.75) + (24\times24/144)(150)(15.333)]/1000 + 7.5 = 133.4^K$
$LL_r = 0.25 + \frac{15}{\sqrt{3364}} = 0.509$

$P_L = 0.509(841)(80)/1000 = 34.22^K$

10th: $P_D = [(841)(20 + 118.75) + (24\times24/144)(150)(11.333)]/1000 + 7.5 = 131.0^K$
$LL_r = 0.25 + \frac{15}{\sqrt{2\times3364}} = 0.433$

$P_L = 0.433(841)(80)/1000 = 29.12^K$

9th: $P_D = 131.0^K$
$LL_r = 0.25 + \frac{15}{\sqrt{3\times3364}} = 0.399 < 0.4 \Rightarrow$ USE 0.4

$P_L = 0.4(841)(80)/1000 = 26.9^K$

8th: $P_D = 131.0^K$
$P_L = 26.9^K$

7th: $P_D = 131.0^K$
$P_L = 26.9^K$

6th: $P_D = 131.0^K$
$P_L = 26.9^K$

5th: $P_D = [(841)(20 + 131.25) + (24\times24/144)(150)(14')]/1000 + 7.5 = 143.1^K$
$P_L = (150)(841)/1000 = 126.15^K$

4th: $P_D = 131.0^K$
$P_L = 26.9^K$

3rd: $P_D = 131.0^K$
$P_L = 26.9^K$

2nd: $P_D = 131.0^K$
$P_L = 26.9^K$

1st: $P_D = [(841)(20 + 131.25) + (26\times26/144)(150)(14')]/1000 + 7.5 = 144.6^K$
$P_L = 26.9^K$

NATHAN McGRAW | SPOT-CHECK-COLUMN (TECH 1) | PAGE 3 OF 3

GROUND: $P_D = [(84)(20 + 113.25) + (28 \times 28/144)(150)(10.667')]/1000 + 7.5 = 128.3^K$

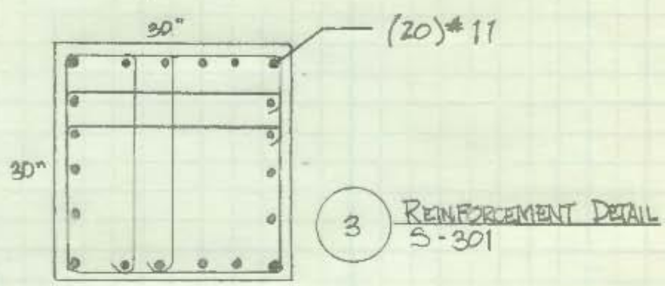
$P_L = 126.15^K$

$P_{D\ TOTAL} = 1750.3^K$
$P_{L\ TOTAL} = 530.84^K$
$P_{ROOF\ LIVE} = 84.1^K$

$P_U = 1.2(1750.3^K) + 1.6(530.84^K) + 0.5(84.1^K)$

$P_U = 2991.75^K$

CHECK COLUMN REINFORCING:



$A_s = (20)(1.56\ in^2) = 31.2\ in^2$

* COLUMN IS CHECKED FOR PURE COMPRESSION BECAUSE IT IS AN INTERIOR COLUMN

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

$\phi P_o = 0.65[(0.85)(7)(30" \times 30" - 31.2) + (31.2)(60)]$

$\phi P_o = 4576.9^K$

PURE COMPRESSION LIMITED BY α

$\phi P_n = \alpha \phi P_o = 0.8(4576.9^K) = 3661.5^K > 2991.75^K = P_U$ ✓

$\rho = \frac{31.2}{30" \times 30"} = 0.035 > 0.01$ ✓ (MINIMUM REINFORCEMENT ACI318-08)

NATHAN MCGRAW | SNOW LOAD CALCS (TECH 1) | PAGE 1 OF 2

GROUND SNOW LOAD → FROM FIGURE 7-1 → $P_g = 25$ psf

FLAT ROOF SNOW LOADS:

$$\boxed{p_f = 0.7 C_e C_t I p_g}$$

EXPOSURE FACTOR (C_e) → FROM TABLE 7-2 → Exposure B → $C_e = 1.0$

TEMPERATURE FACTOR (C_t) → FROM TABLE 7-3 → $C_t = 1.0$

IMPORTANCE FACTOR (I) → FROM TABLE 7-4 → Category IV → $I = 1.2$

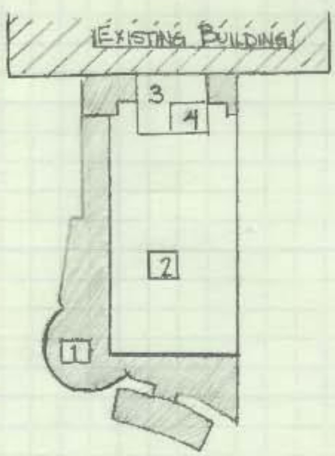
FLAT ROOF SNOW LOAD:

$$p_f = 0.7(1.0)(1.0)(1.2)(25\text{ psf}) = \boxed{21\text{ psf}}$$

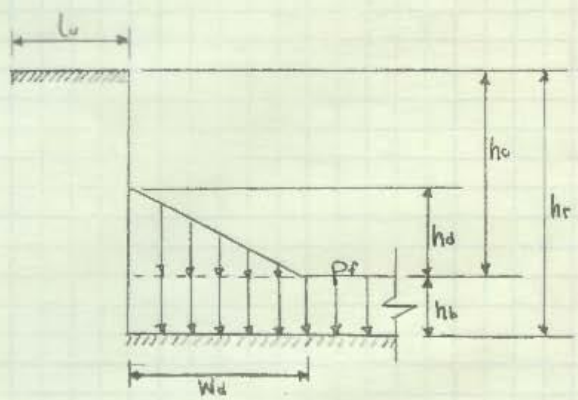
DRIFTS ON LOWER ROOFS:

Roof Heights:

Level 1 = 30'-10"
Level 2 = 144'-10"
Level 3 = 162'-4"
Level 4 = 174'-4"



PLAN VIEW



ELEVATION VIEW

NATHAN McGRAW | SNOW LOAD CALCS (TECH 1) | PAGE 2 OF 2

DRIFT CALCULATION : Level 2 → 1

$h_r = 144'\text{-}10'' - 30'\text{-}10'' = 114'$

$h_b = P_f/\gamma = \dfrac{21 \text{ psf}}{17.25} = 1.22'$

$\gamma = 0.13 p_g + 14 = 0.13(25) + 14 = 17.25 < 30 \text{ pcf} \quad \therefore \text{OK} \checkmark$

$h_c = h_r - h_b = 114' - 1.22' = 112.78'$

$\dfrac{h_c}{h_b} = \dfrac{112.78}{1.22} = 92.4 > 0.2 \quad \therefore$ MUST CALCULATE DRIFT

WINDWARD:

$l_u =$ lower roof $= 39'\text{-}10''$

$h_d = 0.75\left[0.43 \sqrt[3]{39.83'} \sqrt[4]{25+10} - 1.5\right] = 1.55' < h_c \quad \therefore \text{OK} \checkmark$

$W_d = 4h_d = 4(1.55') = 6.2'$

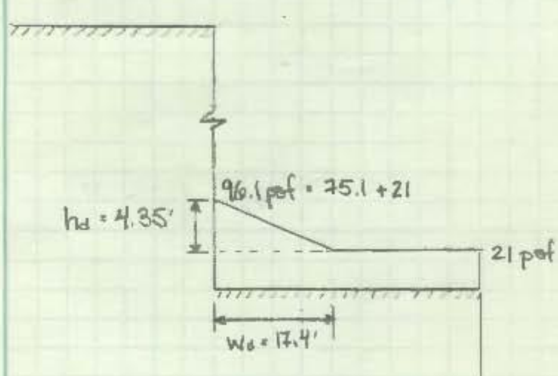
LEEWARD:

$l_u =$ upper roof $= 175'\text{-}4''$

$h_d = 0.43 \sqrt[3]{175'\text{-}4''} \sqrt[4]{25+10} - 1.5 = 4.35' < h_c \quad \therefore \text{OK} \checkmark$

$W_d = 4h_d = 4(4.35') = 17.4'$ → CONTROLS

$p_d = h_d \gamma = (4.35')(17.25) = 75.1 \text{ psf}$



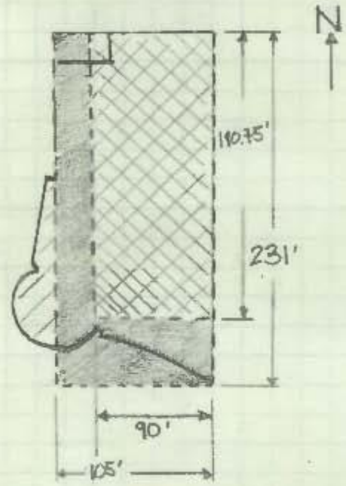
* CALCULATED WORST CASE SCENERIO (LARGEST l_u ON ROOF LEVEL 2)
OTHER DRIFT CALCULATIONS PREFORMED USING EXCEL

Snow Drift Load Calculations								
Roof Levels	Windward				Leeward			
	L_u (ft)	h_d (ft)	p_d (psf)	w_d (ft)	L_u (ft)	h_d (ft)	p_d (psf)	w_d (ft)
1 and 2	39.83	1.55	26.80	6.22	175.33	4.35	75.10	17.42
2 and 3	159.5	3.13	53.98	12.52	46.33	2.26	38.92	9.03
2 and 4	159.5	3.13	53.98	12.52	31.33	1.80	31.00	7.19
1 and 3	37.33	1.50	25.82	5.99	50.17	2.36	40.67	9.43
3 and 4	19.33	0.98	16.91	3.92	30.83	1.78	30.70	7.12

Appendix B: Wind Load Calculations

NATHAN McGRAW | WIND LOAD CALCS (TECH 1) | PAGE 1 OF 7

SIMPLIFYING ASSUMPTIONS:



N-S DIRECTION WIND:

0 → 36.17'
L = 231'
B = 105'

36.17' → 175'
L = 190.75'
B = 90'

E-W DIRECTION WIND:

0 → 36.17'
L = 105'
B = 231'

36.17' → 175'
L = 90'
B = 190.75'

USE METHOD 2 SINCE BUILDING WITH SIMPLIFYING ASSUMPTIONS MEETS CRITERIA OF 6.5.1 AND 6.5.2

BASIC WIND SPEED: $V = 90$ mph (FIGURE 6-1C)

WIND DIRECTIONALITY FACTOR: $K_d = 0.85$ (TABLE 6-4)

OCCUPANCY CATEGORY: TYPE IV (TABLE 6-1)

IMPORTANCE FACTOR: $I = 1.15$ (TABLE 6-1)

EXPOSURE CATEGORY: B - Urban/Suburban (§ 6.5.6.3)

TOPOGRAPHIC FACTOR: $K_{zt} = 1.0$ (§ 6.5.7)

VELOCITY PRESSURE COEFFICIENTS: Varies with height → See Excel Spreadsheet (TABLE 6-3)

VELOCITY PRESSURES: $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (§ 6.5.10)

GUST EFFECT FACTOR:

$$n_1 = \frac{385 (C_w)^{0.5}}{H}$$

$$C_w = \frac{100}{A_B} \sum \left(\frac{H}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83 (h_i / D_i)^2\right]}$$

$A_B = (105') \times (231') = 24255 \text{ ft}^2$
$H = 174'\text{-}4''$

EAST - WEST DIRECTION:

Shear Wall 3:

$h_i = 174'\text{-}4''$
$D_i = 25'$
$A_i = (174'\text{-}4'')(25') = 4358.25$

$$\left(\frac{174.33'}{174.33'}\right)^2 \frac{4358.25}{1+0.83\left(174.33/25'\right)^2} = 105.38$$

Shear Wall 5 & 6:

$h_i = 145'$
$D_i = 20'$
$A_i = (145')(20') = 2900$

$$\left(\frac{174.33'}{145}\right)^2 \frac{2900}{1+0.83\left(145/20\right)^2} = 93.93$$

$$C_W = \left(\frac{100}{24255}\right)\left(105.38 + 2(93.93)\right) = 1.20$$

$$n_i = \frac{385(1.2)^{0.5}}{174'\text{-}4''} = 2.43 > 1.0 \rightarrow \text{RIGID STRUCTURE}$$

NORTH - SOUTH DIRECTION

Shear Wall 1 and 2:

$h_i = 174'\text{-}4''$
$D_i = 30.75'$
$A_i = (174'\text{-}4'')(30.75') = 5360.75$

$$\left(\frac{174.33'}{174.33'}\right)^2 \frac{5360.75}{1+0.83\left(174.33/30.75\right)^2} = 193.69$$

Shear Wall 4 and 7:

$h_i = 145'$
$D_i = 10'$
$A_i = (145')(10') = 1450$

$$\left(\frac{174.33}{145}\right)^2 \frac{1450}{1+0.83\left(145/10'\right)^2} = 11.94$$

$$C_W = \left(\frac{100}{24255}\right)(2\times 193.69 + 2\times 11.94) = 1.69$$

$$n_1 = \frac{385(1.69)^{0.5}}{174'-4''} = 2.88 > 1.0 \rightarrow \text{RIGID STRUCTURE}$$

∴ GUST EFFECT FACTOR MAY BE TAKEN AS 0.85 (§6.5.8.1)

ENCLOSURE CLASSIFICATION: Enclosed Building → $GC_{pi} = \pm 0.18$ (FIGURE 6-5)

DESIGN WIND PRESSURES: $p = qGC_p - q_i(GC_{pi})$

EXTERNAL PRESSURE COEFFICIENTS:

WALLS:

WINDWARD → $C_p = 0.8$

LEEWARD:

N-S:

$0 \rightarrow 36.17'$: $L/B = 231/105 = 2.2 \rightarrow C_p = -0.29$

$36.17' \rightarrow 175'$: $L/B = 190.75/90 = 2.1 \rightarrow C_p = -0.295$

E-W:

$0 \rightarrow 36.17'$: $L/B = 105/231 = 0.45 \rightarrow C_p = -0.5$

$36.17' \rightarrow 175'$: $L/B = 90/190.75 = 0.47 \rightarrow C_p = -0.5$

SIDE → $C_p = -0.7$

ROOF: $\theta = 0°$ (Only considered wind for taller tower)

N-S: $h/L = \frac{175'}{190.75'} = 0.917$

E-W: $h/L = \frac{175'}{90'} = 1.9 > 1.0 \rightarrow$

	C_p
$0 \rightarrow h/2$	$-1.3^{**}, -0.18$
$> h/2$	$-0.7, -0.18$

** Value may be reduced:
Roof Area $= (190.75')(90') = 17167.5\ ft^2$ → Reduction factor = 0.8

DESIGN WIND PRESSURES:

Windward Walls: $p_z = q_z GC_p - q_h(GC_{pi})$

Leeward Walls, Side Walls, Roof: $p_h = q_h(GC_p - GC_{pi})$

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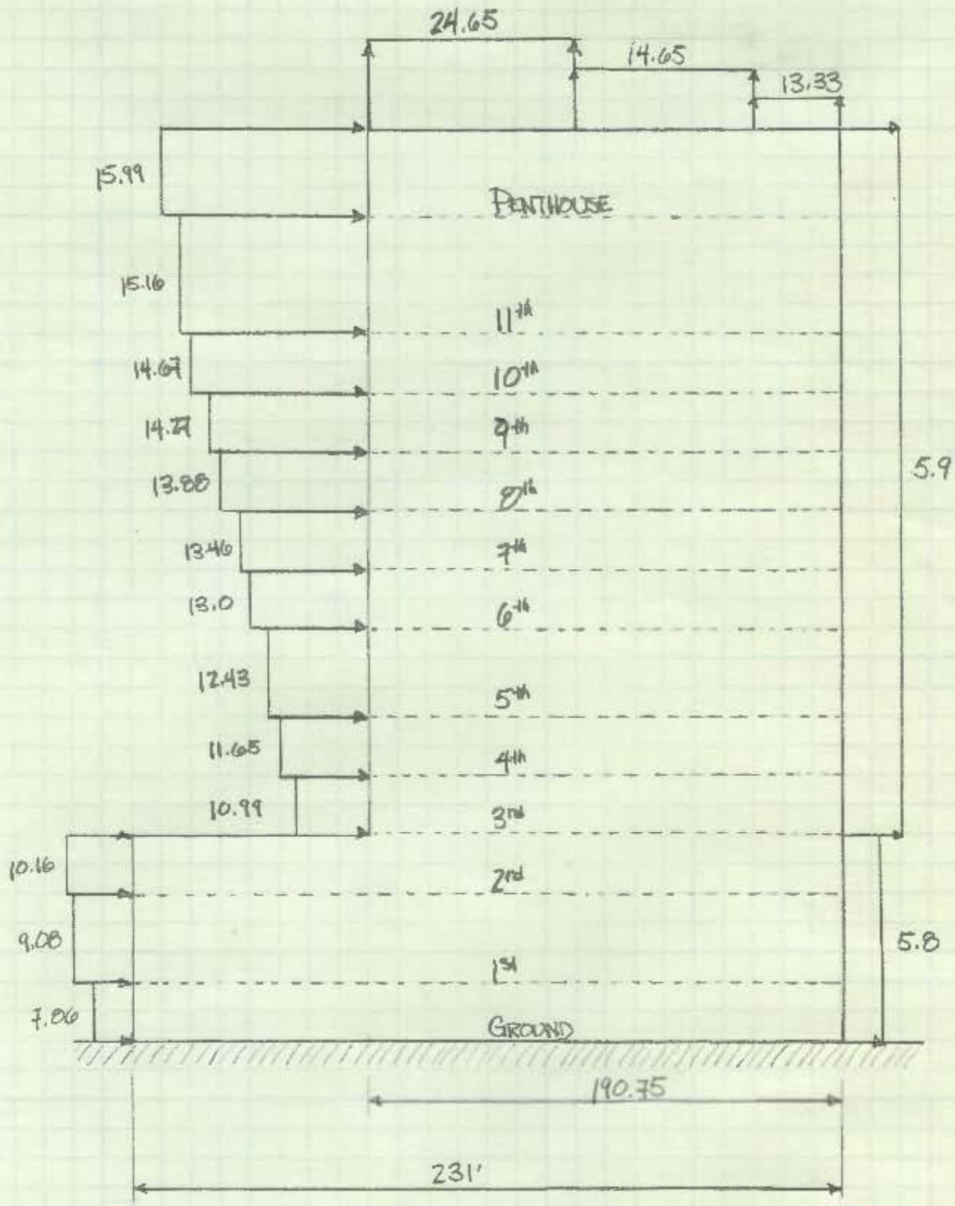
Diagram of N-S Wind Pressures:



* All wind values are in PSF

NATHAN M^cGRAW | WIND LOAD CALCS (TECH 1) | PAGE 5 OF 7

DIAGRAM OF N-S WIND FORCES:

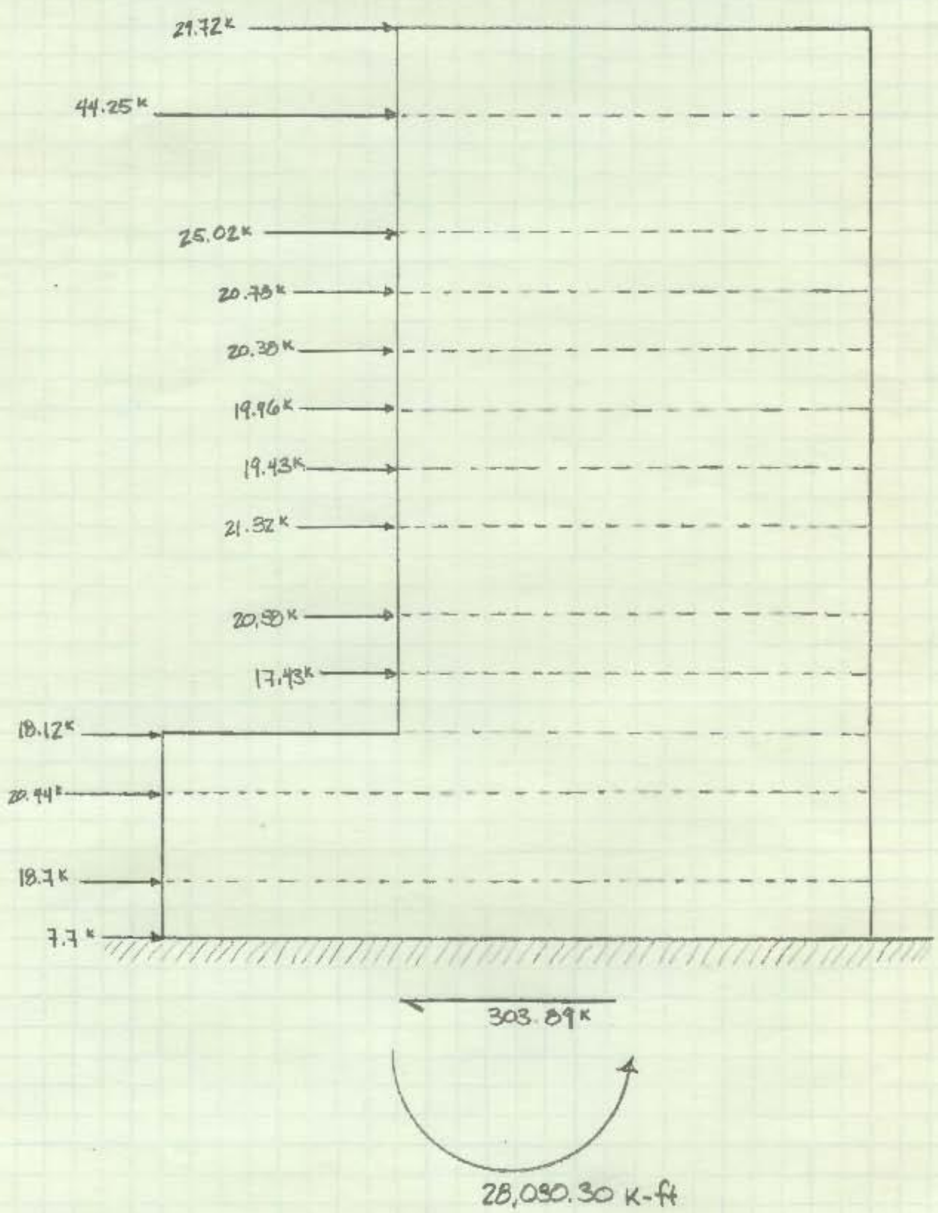


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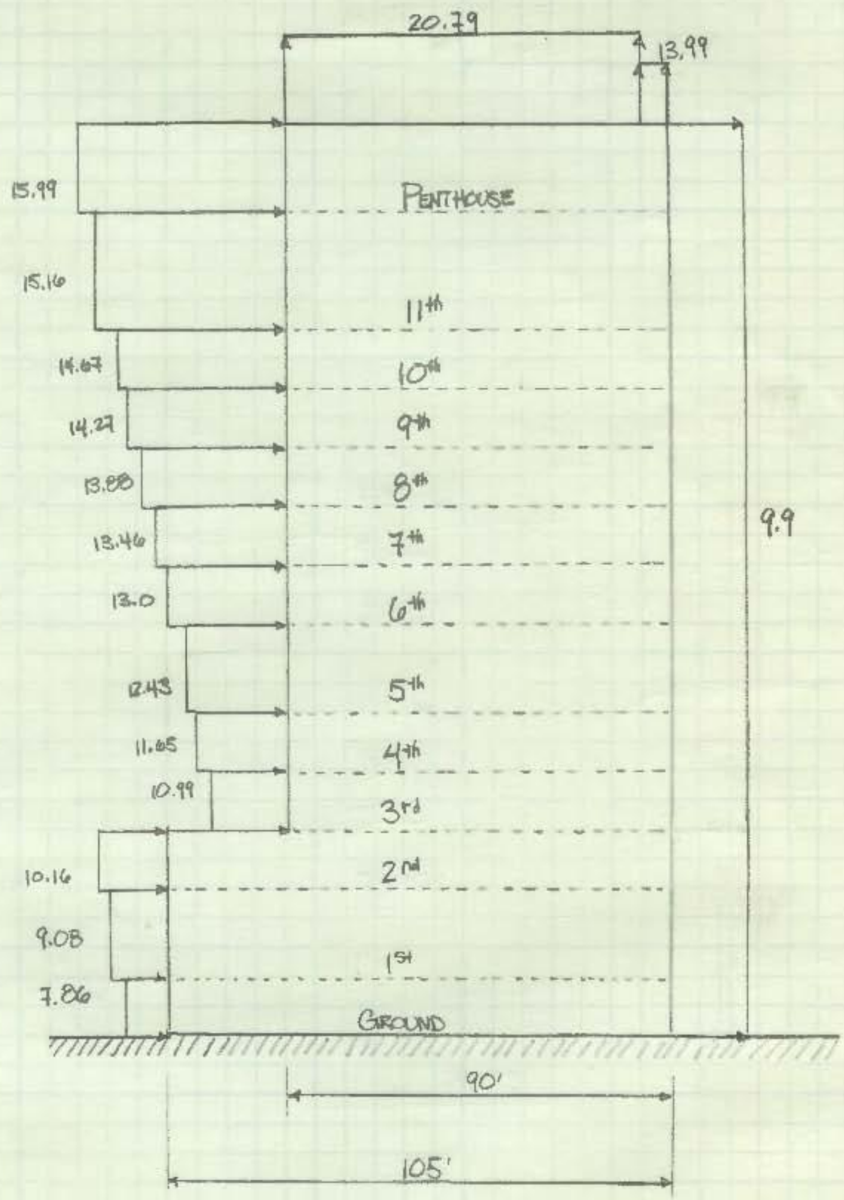
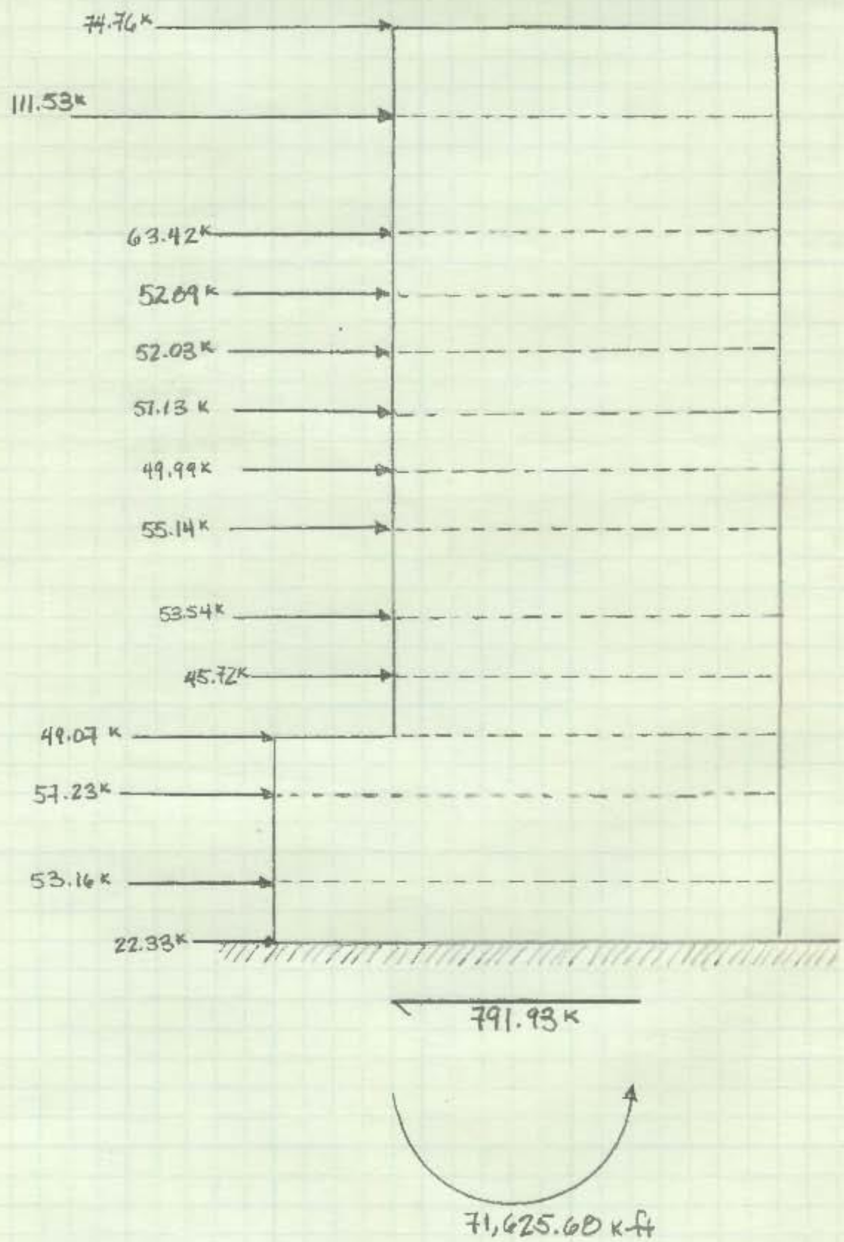
DIAGRAM OF E-W WIND PRESSURES:



* ALL WIND VALUES ARE IN PSF

NATHAN McGRAW | WIND LOAD CALCS (TECH 1) | PAGE 7 OF 7

DIAGRAM OF E-W WIND FORCES:



Building Dimensions		
Height Level	**N-S Wind**	**E-W Wind**
0' - 36.17'		
B (ft)	105	231
L (ft)	231	105
h (ft)	Not Used	Not Used
36.17' - 175'		
B (ft)	90	190.75
L (ft)	190.75	90
h (ft)	175	175

General Wind Load Design Criteria		
Design Wind Speed	90 mph	ASCE 7-05 (Fig. 6-1C)
Directionality Factor (K_d)	0.85	ASCE 7-05 (Table 6-4)
Importance Factor (I_w)	1.15	ASCE 7-05 (Table 6-1)
Exposure Category	B	ASCE 7-05 (§ 6.5.6.3)
Topographic Factor (K_{zt})	1	ASCE 7-05 (§ 6.5.7)
Internal Pressure Coefficient (GC_{pi})	± 0.18	ASCE 7-05 (Fig. 6-5)

Velocity Pressure Coefficients (K_z) and Velocity Pressures (q_z)			
Level	**Elevation (ft)**	**K_z**	**q_z (psf)**
Ground	0.0	0.57	11.55
1st	10.83	0.57	11.55
2nd	24.83	0.659	13.36
3rd	36.17	0.737	14.94
4th	47.50	0.7975	16.16
5th	58.67	0.845	17.13
6th	72.93	0.902	18.28
7th	84.17	0.943	19.11
8th	95.50	0.9765	19.79
9th	106.83	1.007	20.41
10th	118.17	1.035	20.98
11th	129.5	1.064	21.57
Penthouse	144.83	1.10	22.30
Roof	175.00	1.16	23.51

External Pressure Coefficients (C_P)		
Description	**N-S Wind**	**E-W Wind**
0' - 36.17'		
L/B	2.2	0.45
Windward Walls	0.8	
Leeward Walls	-0.29	-0.5
Side Walls	-0.7	
h/L	Not Used	Not Used
Roof - 0 to h/2		
Roof - h/2 to h		
Roof - h to 2h		
Roof - > 2h		
36.17' - 175'		
L/B	2.12	0.472
Windward Walls	0.8	
Leeward Walls	-0.295	-0.5
Side Walls	-0.7	
h/L	0.917	1.9
Roof - 0 to 87.5'	-1.2336	-1.04
Roof - 87.5' to 175'	-0.7332	-0.7
Roof - 175' to 350'	-0.6668	-0.7
Roof - > 350'	-0.6336	-0.7

Appendix C: Seismic Load Calculations

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SITE CLASS: D (Given on Sheet S0-01)

MAPPED SHORT PERIOD SPECTRAL RESPONSE ACCELERATION: $S_S = 0.154$
MAPPED 1-SECOND PERIOD SPECTRAL RESPONSE ACCELERATION: $S_1 = 0.051$
(* USED USGS WEB APPLICATION TO OBTAIN THESE VALUES)

IMPORTANCE FACTOR: Category IV (TABLE 1.1) → SEISMIC IMPORTANCE FACTOR = 1.5 (TABLE 11.5.1)

SITE COEFFICIENT, F_a: $F_a = 1.6$ (TABLE 11.4-1)

SITE COEFFICIENT, F_v: $F_v = 2.4$

DESIGN SPECTRAL ACCELERATION PARAMETERS (§ 11.4-4):

$S_{DS} = 2/3\, S_{MS}$

$S_{D1} = 2/3\, S_{M1}$

ADJUSTED MAXIMUM CONSIDERED EQ (§ 11.4-3):

$S_{MS} = F_a S_S = (1.6)(0.154) = 0.2464$

$S_{M1} = F_v S_1 = (2.4)(0.051) = 0.1224$

$S_{DS} = 2/3\,(0.2464) = 0.1643$

$S_{D1} = 2/3\,(0.1224) = 0.0816$

SEISMIC DESIGN CATEGORY:

Short Period Response → SDC = A (TABLE 11.6-1)

1-Second Period Response → SDC = C (TABLE 11.6-2)

* SINCE DIFFERENT SEISMIC DESIGN CATEGORIES, DESIGN TO WORST CASE SEISMIC DESIGN CATEGORY = C

PERMITTED ANALYTICAL PROCEDURE: Equivalent Lateral Force Analysis permitted (TABLE 12.6-1)

RESPONSE MODIFICATION COEFFICIENT: TABLE 12.2-1

R = 4½ → Shear Wall-Frame Interactive System with Ordinary Reinforced Concrete Moment Frames and Ordinary Reinforced Concrete Shear Walls

NATHAN McGRAW | SEISMIC LOAD CALCS (TECH 1) | PAGE 2 OF 5

APPROXIMATE FUNDAMENTAL PERIODS: § 12.8.2.1 AND TABLE 12.8-2

$T_a = C_t h_n^x$

$C_t = 0.02$
$x = 0.75$ "ALL OTHER STRUCTURAL SYSTEMS"

$T_a = (0.02)(158')^{0.75}$

$T_a = 0.8913$ sec. $C_u = 1.7$ (TABLE 12.8-1)

SEISMIC RESPONSE COEFFICIENT : § 12.8.1.1

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{(R/I)} \\ \dfrac{S_{D1}}{T(R/I)} \\ \dfrac{S_{D1} T_L}{T^2 (R/I)} \end{cases} \geq 0.01$$

$T_L = 8$ sec. (FIGURE 22-15)
$T = C_u \cdot T_a$
$T = 1.7(0.8913) = 1.5152$

$$C_s = \min \begin{cases} \dfrac{(0.1643)}{(4.5/1.5)} = 0.0548 \\ \dfrac{(0.0816)}{(1.5152)(4.5/1.5)} = 0.01795 \\ \dfrac{(0.0816)(8)}{(1.5152)^2(4.5/1.5)} = 0.0948 \end{cases}$$

$\Rightarrow C_s = 0.01795 > 0.01$ ∴ OK ✓

WEIGHT CALCULATIONS:

FACADE:

5 1/2" Concrete + 1/2" Thin Brick Face = $(6''/12'')(150\ lb/ft^3)$ = 75 psf

2" Air space ⇒ 0 psf

4" Glass Fiber Insulation with Vapor Barrier = 1 1/2 psf x 4" = 6 psf

3 5/8" Metal Studs = 1 psf

FACADE TOTAL WEIGHT = 75 + 6 + 1 = 82 psf

MAIN ROOF:

12" Concrete = $(12''/12'')(150\ lb/ft^3)$ = 150 psf

Roof Membrane = 2 psf

5/8" Roof Board = 1 1/2 psf x 5/8" = 0.9375 psf

MAIN ROOF TOTAL WEIGHT = 150 + 2 + 0.9375 = 152.9375 psf

Typical Roof:

9 1/2" Concrete = $(9.5''/12'')(150\ lb/ft^3)$ = 118.75 psf

6" Rigid Insulation = 1 1/2 psf x 6" = 9 psf

Roof Membrane = 2 psf

TYPICAL ROOF TOTAL WEIGHT = 118.75 + 9 + 2 = 129.75 psf

VEGETATED ROOF SYSTEM:

EXTRUDED-Polystyrene Board Insulation = $(1.8\ lb/ft^3)(6''/12'')$ = 0.9 psf

Roof Pavers = 25 psf

VEGETATED SYSTEM = 30 psf

VEGETATED ROOF TOTAL WEIGHT = 0.9 + 25 + 30 = 55.9 psf

* EXCEL CONTAINS TOTAL BUILDING WEIGHT WITH FLOOR BY FLOOR BREAKDOWN

Column Weights										
Level	24"x24"	30"x30"	26"x26"	12"x18"	12"x24"	28"x28"	18"x18"	18"x24"	Volume (ft^2)	Weight (kips)
1st - Below	26	0	0	8	1	4	1	1	535.15	80.27
1st - Top	26	0	1	8	1	3	1	1		
2nd - Below	26	0	1	8	1	3	1	1	548.20	82.23
2nd - Top	26	0	1	8	1	3	1	1		
3rd - Below	26	0	1	8	1	3	1	1	449.02	67.35
3rd - Top	25	0	0	6	0	2	0	0		
4th - Below	25	0	0	6	0	2	0	0	407.58	61.14
4th - Top	25	0	0	6	0	2	0	0		
5th - Below	25	0	0	6	0	2	0	0	455.56	68.33
5th - Top	25	0	0	6	0	2	0	0		
6th - Below	25	0	0	6	0	2	0	0	455.56	68.33
6th - Top	25	0	0	6	0	2	0	0		
7th - Below	25	0	0	6	0	2	0	0	407.58	61.14
7th - Top	25	0	0	6	0	2	0	0		
8th - Below	25	0	0	6	0	2	0	0	407.58	61.14
8th - Top	25	0	0	6	0	2	0	0		
9th - Below	25	0	0	6	0	2	0	0	407.58	61.14
9th - Top	25	0	0	6	0	2	0	0		
10th - Below	25	0	0	6	0	2	0	0	407.58	61.14
10th - Top	25	0	0	6	0	2	0	0		
11th - Below	25	0	0	6	0	2	0	0	479.51	71.93
11th - Top	25	0	0	6	0	2	0	0		
Penthouse/Roof Below	25	0	0	6	0	2	0	0	137.87	20.68
										764.81

Facade Weights						
Level	Tributary Height (ft)	Length (ft)	With (ft)	Total Perimeter (ft)	Area (ft^2)	Weight (kips)
1st	12.333	231	105	336	4144.0	339.8
2nd	12.667	231	105	336	4256.1	349.0
3rd	11.333	231	105	336	3807.9	312.2
4th	11.333	190.75	90	280.75	3181.7	260.9
5th	12.667	190.75	90	280.75	3556.3	291.6
6th	12.667	190.75	90	280.75	3556.3	291.6
7th	11.333	190.75	90	280.75	3181.7	260.9
8th	11.333	190.75	90	280.75	3181.7	260.9
9th	11.333	190.75	90	280.75	3181.7	260.9
10th	11.333	190.75	90	280.75	3181.7	260.9
11th	11.333	190.75	90	280.75	3181.7	260.9
Penthouse/Roof	7.667	190.75	90	280.75	2152.5	176.5
						3326.2

Drop Panel Weights			
Level	**Number**	**Area (ft^2)**	**Weight (kips)**
1st	19.5	975	146.25
2nd	12	600	90
3rd	30.5	1525	228.75
4th	18.5	925	138.75
5th	18.5	925	138.75
6th	18.5	925	138.75
7th	18.5	925	138.75
8th	18.5	925	138.75
9th	18.5	925	138.75
10th	18.5	925	138.75
11th	18.5	925	138.75
Penthouse/Roof	20	1000	150
			1725

Shear Wall Weights		
Level	**Volume (ft^3)**	**Weight (kips)**
1st	1819.2	272.9
2nd	1868.4	280.3
3rd	1671.6	250.7
4th	1671.6	250.7
5th	1868.4	280.3
6th	1868.4	280.3
7th	1671.6	250.7
8th	1671.6	250.7
9th	1671.6	250.7
10th	1671.6	250.7
11th	1966.6	295.0
Penthouse/Roof	1130.9	169.6
		3082.7

Superimposed Dead Load			
Level	**Slab Area (ft^2)**	**Roof Area (ft^2)**	**Weight (kips)**
Ground	25611	0	N/A
1st	25611	0	512
2nd	11649	0	233
3rd	16571	9040	512
4th	16571	0	331
5th	16571	0	331
6th	16571	0	331
7th	16571	0	331
8th	16571	0	331
9th	16571	0	331
10th	16571	0	331
11th	16571	0	331
Penthouse/Roof	0	16571	331
			4240

Slab Weights			
Level	**Slab Area (ft^2)**	**Roof Area (ft^2)**	**Weight (kips)**
Ground	25611	0	N/A
1st	25611	0	3041
2nd	11649	0	1383
3rd	16571	9040	2531
4th	16571	0	1968
5th	16571	0	2175
6th	16571	0	1968
7th	16571	0	1968
8th	16571	0	1968
9th	16571	0	1968
10th	16571	0	1968
11th	16571	0	1968
Penthouse/Roof	0	16571	2534
			25439

Weight Per Level		
Level	**Area (ft^2)**	**Weight (kips)**
Ground	25512.5	N/A
1st	25512.5	4392.73
2nd	11649	2417.80
3rd	17958	3901.98
4th	16571	3010.72
5th	16571	3285.27
6th	16571	3078.14
7th	16571	3010.72
8th	16571	3010.72
9th	16571	3010.72
10th	16571	3010.72
11th	16571	3065.76
Penthouse/Roof	16571	3382.57
		38577.83

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BASE SHEAR:

$V = C_s W$

W = weight of building (Calculated in spreadsheet)

$C_{s\,N\text{-}S} = C_{s\,E\text{-}W} = 0.01795$

$V = (0.01795)(38{,}577.83^K)$

$\boxed{V = 692.5^K}$

(STRUCTURAL DRAWINGS = 700^K → 1.07% DIFFERENCE ✓

STORY FORCES:

$F_x = C_{vx} V$

$C_{vx} = \dfrac{w_x h_x^k}{\sum w_i h_i^k}$

w = weight of each story
h = height of story above grade
$K = 1 + \dfrac{T - 0.5}{2}$ $(1 \le K \le 2)$

$K = 1 + \dfrac{1.5152 - 0.5}{2} = 1.5076$

* CALCULATED THE STORY FORCES AND OVERTURNING MOMENTS IN AN EXCEL SPREADSHEET

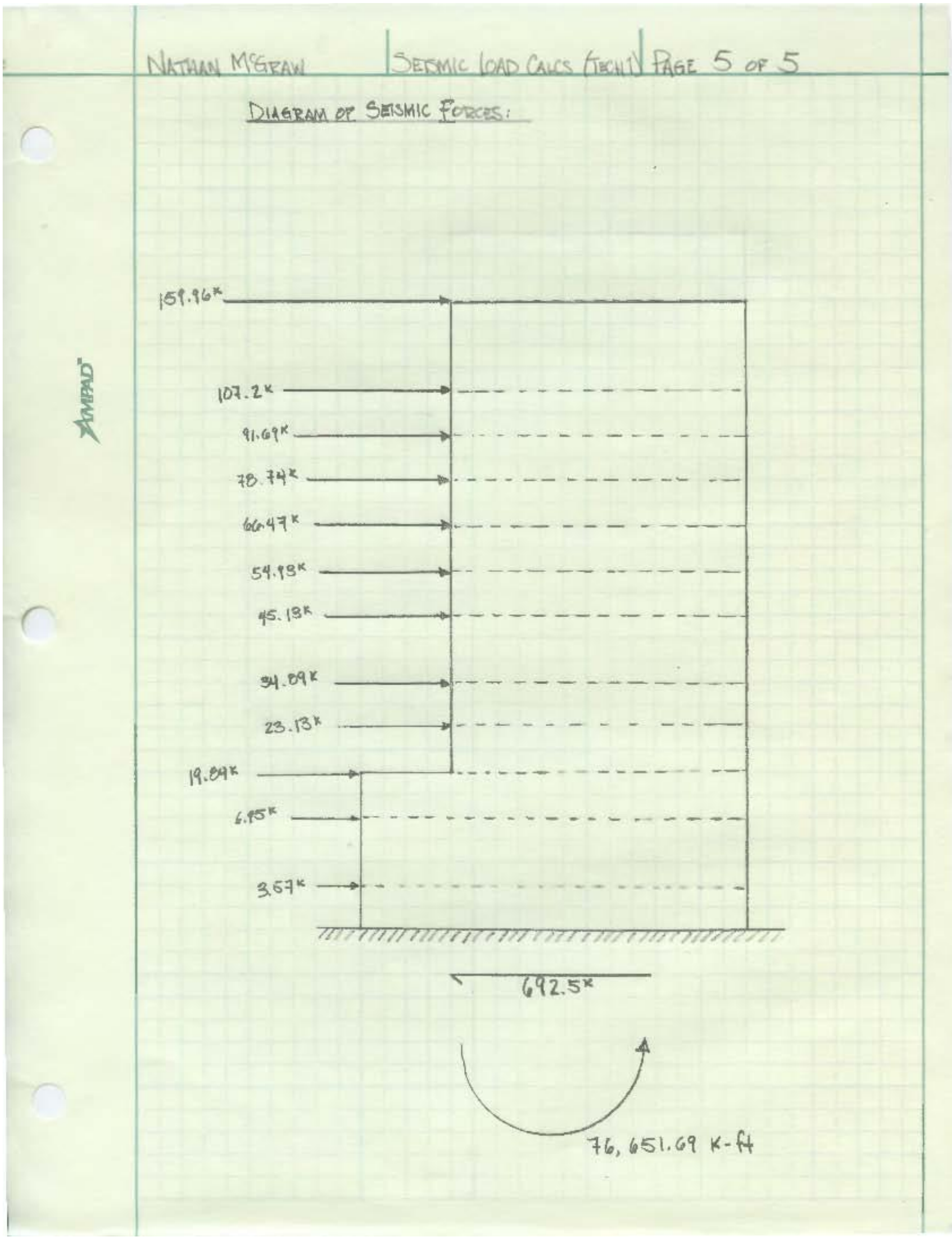
NATHAN MCGRAW
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DIAGRAM OF SEISMIC FORCES:
159.96k
107.2k
91.69k
78.74k
66.47k
54.93k
45.13k
34.89k
23.13k
19.84k
6.95k
3.57k
692.5k
76,651.69 k-ft

Appendix D: Typical Plans

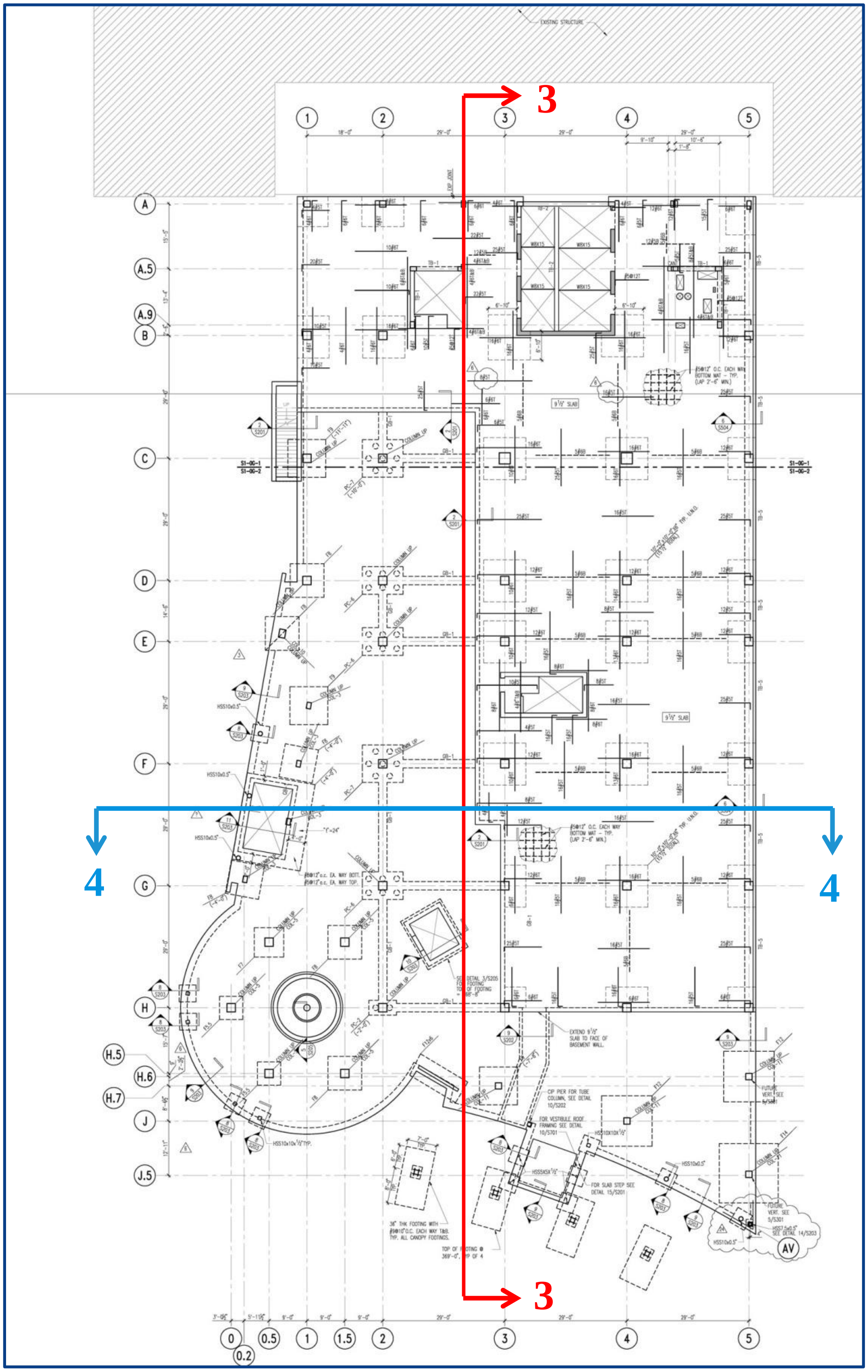

Figure 1:
Ground floor plan (See following figures for sections indicated on the plan)

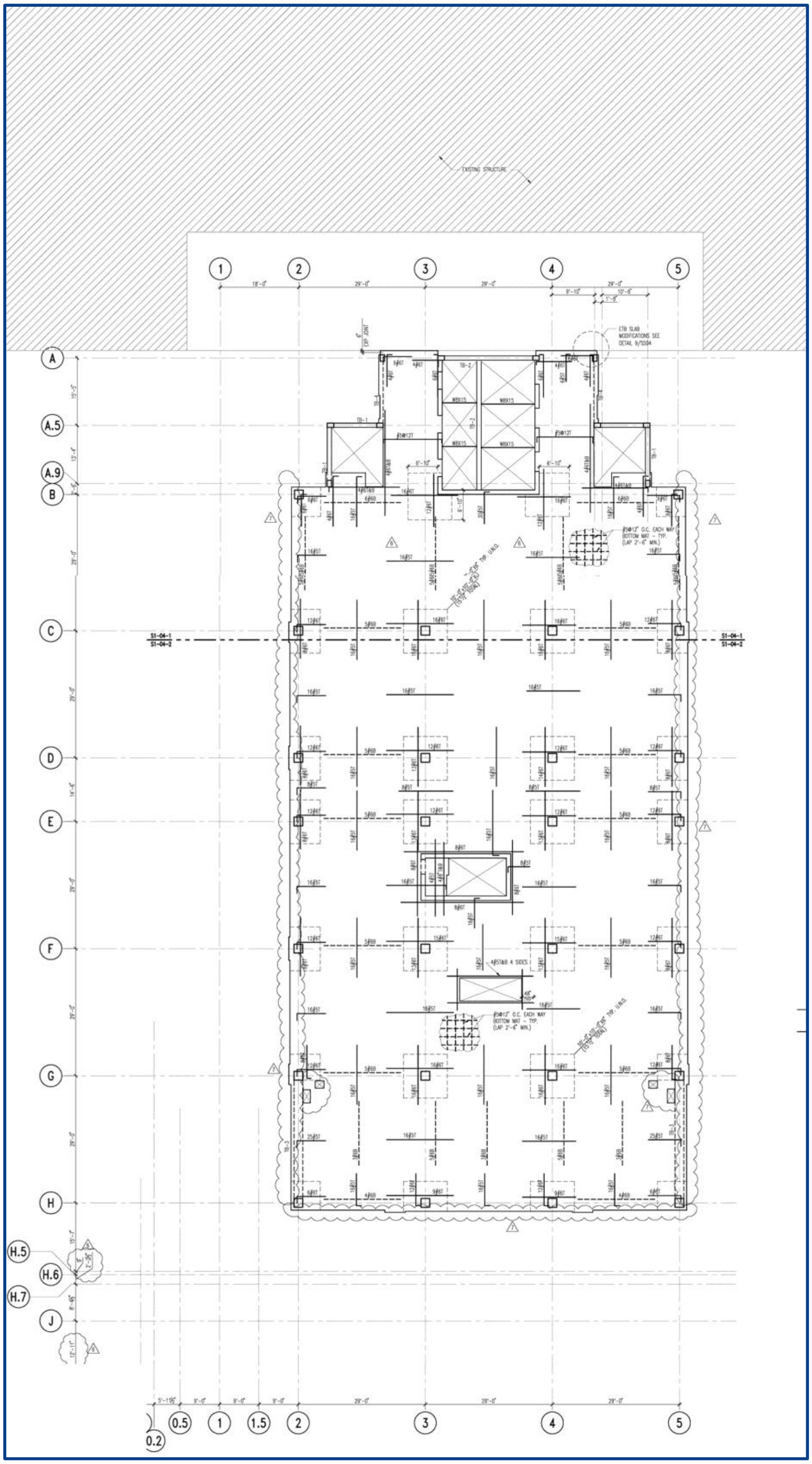

Figure 2:
Typical floor plan (6^{th} – 11^{th})

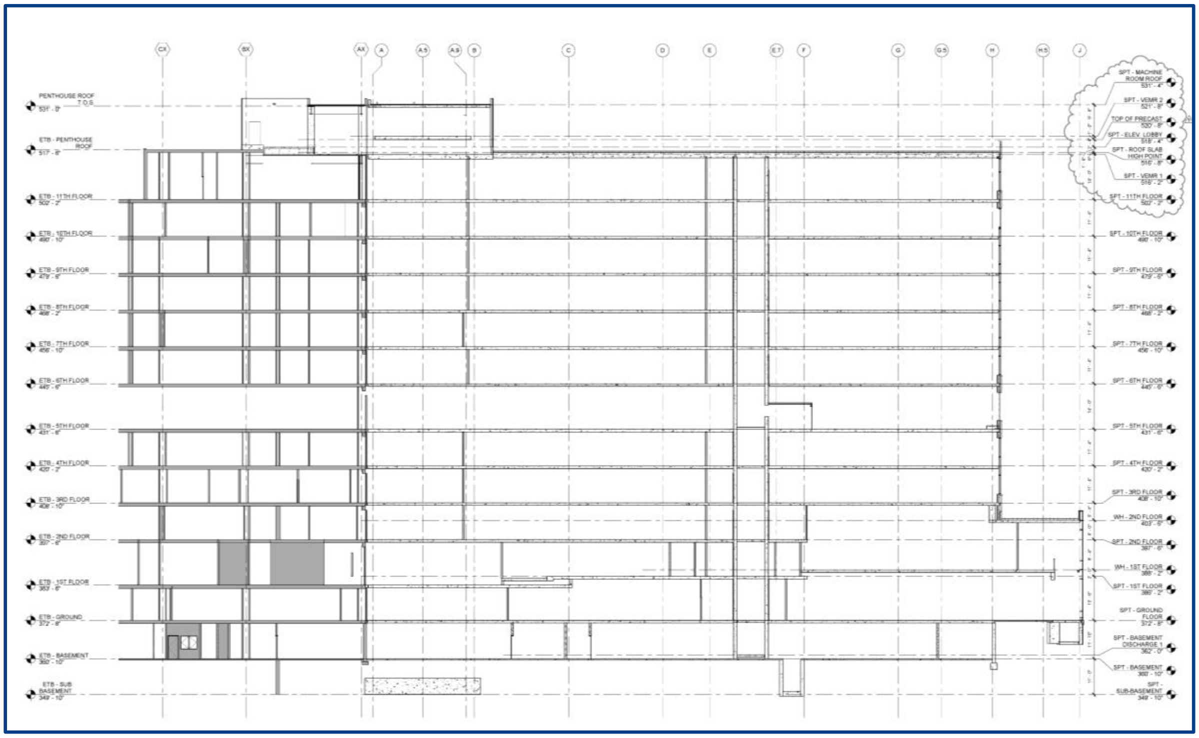

Figure 3:
North – South section cut

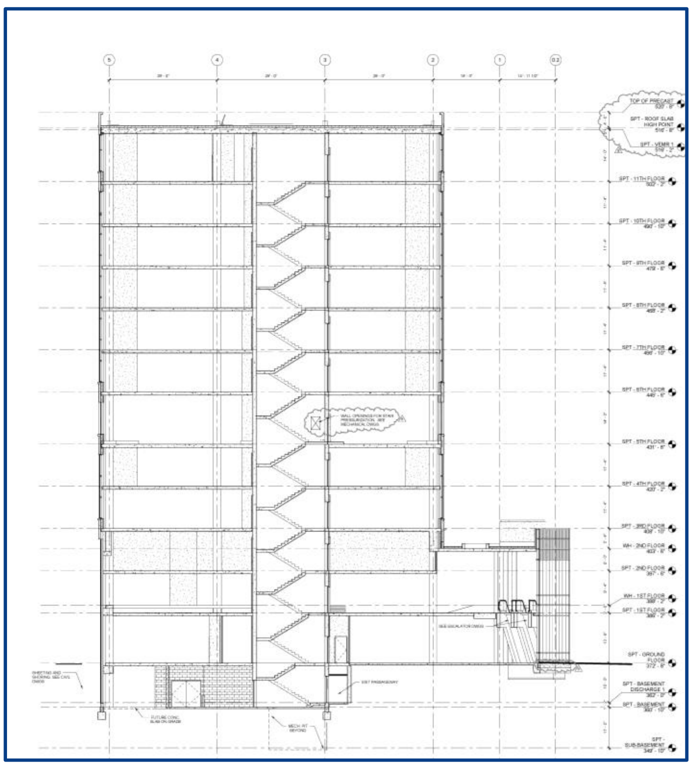

Figure 4:
East – West section cut