Technical Report 3

Nathan McGraw
Structural Option
Faculty Advisor: Dr. Richard Behr

November 16th, 2011

Inova Fairfax Hospital | South Patient Tower

Falls Church, VA

Table of Contents

Executive Summary:

The main purpose of this technical report is to evaluate the lateral system of the South Patient Tower (SPT). The analysis contained within this technical report started with the verification of the various loads (dead, live, and snow loads). Following the calculation of these loads, wind and seismic loads were obtained using the Main Wind Resisting System procedure and the Equivalent Lateral Force procedure given in ASCE 7-05. Once the wind loads were factored, a comparison was made to determine the controlling loads. It was determined that the lower floors in both the East-West and North-South direction are controlled by wind loads while the upper stories are controlled by seismic. In the East-West direction, the overturning moment is dictated by seismic, but the base shear is controlled by the wind loads. For the North-South direction, it was found that seismic controlled both the overturning moment as well as the base shear.

Next, a model was built of the South Patient Tower in ETABS. Two separate models were utilized in this technical report. The first model was constructed using rigid diaphragms to model the floor system and all of the gravity elements were also modeled to accurately represent the stiffness of the entire lateral system. The second model was built utilizing shell elements to represent the stiffness of the two-way concrete flat slab system. The latter model was done in order to better represent the stiffness of the entire structure as compared to the stiffness of just the lateral system and gravity members. Displacements, drifts, and torsional irregularities were carried out for both models, but due to time limitations, forces and spot checks were only completed for the rigid diaphragm model.

Upon completion of the models, modal information was obtained and checked against the assumed period used to calculate the seismic forces. Since the periods obtained were higher than C_uT_a, no changes were made to the seismic forces. In order to verify the accuracy of the models, the center of mass, center of rigidity, and shear forces were verified with hand calculations. Due to the complexity of the model, displacements and drifts could not be replicated by hand calculations. Spot checks were then performed on the members of the lateral resisting system including a shear wall and a moment frame column. Using the ETABS model, the shear capacities for the shear walls were checked against code. Interaction diagrams were produced for both the shear wall and column by hand to check the adequacy for these members to carry axial load and the lateral fore applied to each level. Using the displacements and relative drifts from ETABS, torsional irregularities were checked and accounted for in the modeling process. Drift was found to be very sensitive to the modeling method chosen and was excessive in some areas of the building during seismic loading for the rigid diaphragm model.

The governing load combinations were found and are given in this technical report. Once the overturning moments were calculated, the resisting moment was obtained using the building weight and the structure was deemed to be able to resist the overturning moment due to seismic loading.

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 an overall project cost of around $76 million. Standing at 175 ft, 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 (Provided by Turner Construction)

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 design features (see Figure 3). Inside the patient rooms, the use of low-VOC paints, building materials and furniture will lead to higher indoor air quality. Also, the use of low flow plumbing fixtures and sensors will reduce water consumption by up to 30%. Outside of the building, native drought resistant plants 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.

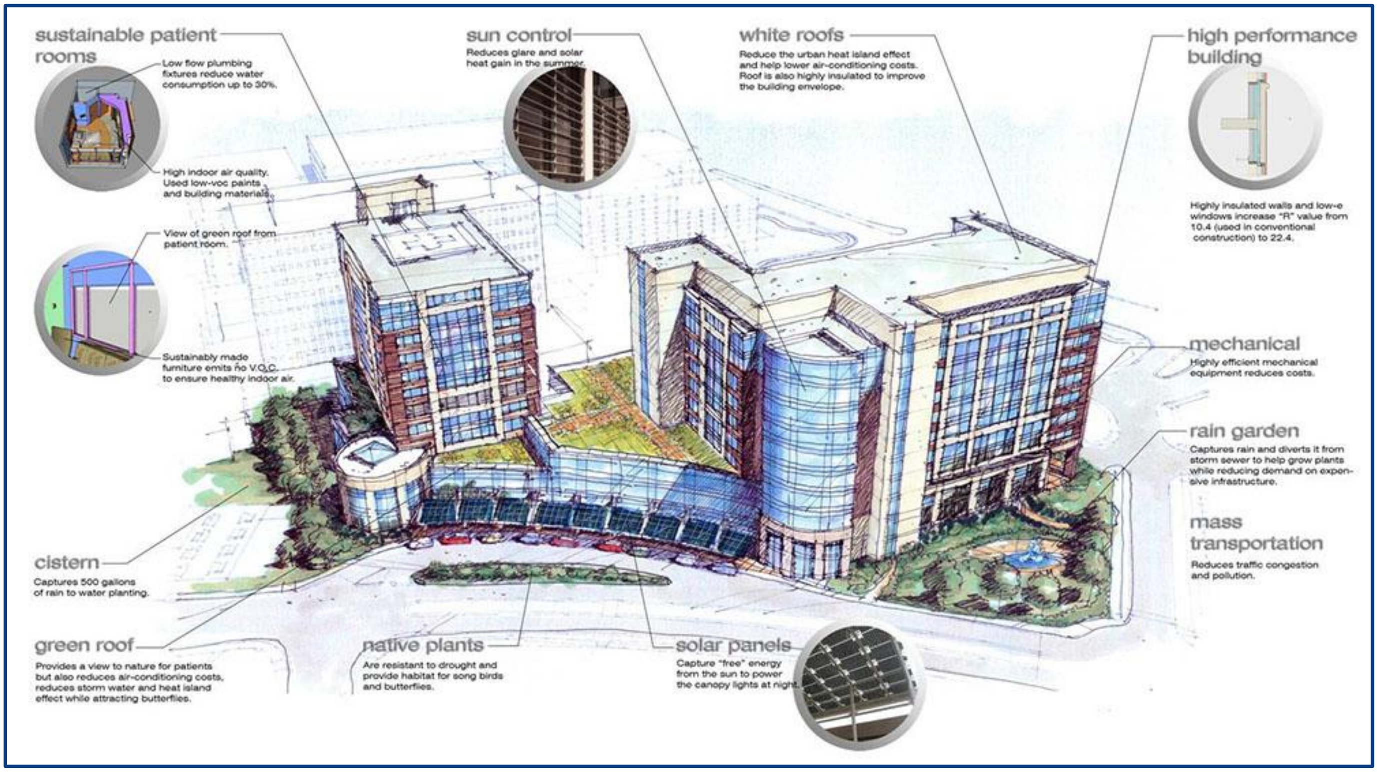


Figure 3:
Sustainability features (rendering provided by Wilmot/Sanz Architects)

Structural Overview:

Foundation:

Schnabel Engineering North performed the geotechnical studies for the South Patient Tower (SPT) 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 in. 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 in. 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.

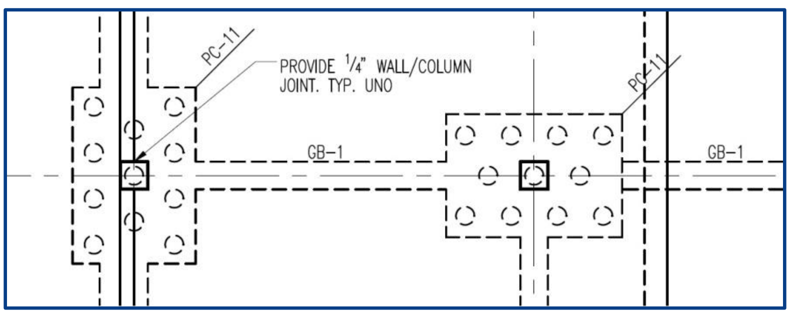


Figure 4:
Typical pile and pile cap

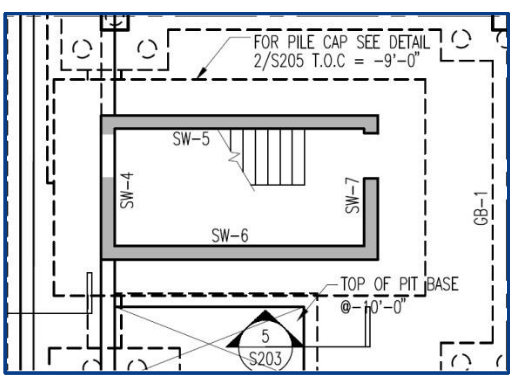


Figure 5:
Pile cap constructed around staircase

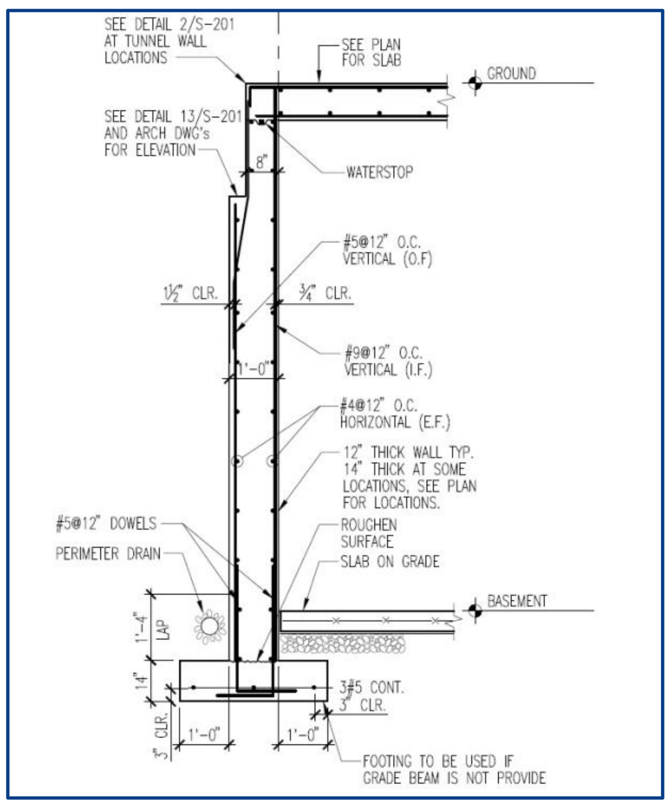


Figure 6:
Spread footing with basement wall

Floor System:

The elevated floors of the South Patient Tower are comprised of a 9 ½ in. two-way flat concrete 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 ft x10 ft x 6 in.

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 in. 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 increased to a 10 ½ in depth.

Reinforcement for the two-way slab system is comprised of both top and bottom steel. The typical bottom reinforcement consists of #5@12 in. o.c. each way (see Figures 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 on which direction the frame of interest is going). With a fairly simple column layout, the two-way slab system has a span of 29 ft in both directions for the most part.

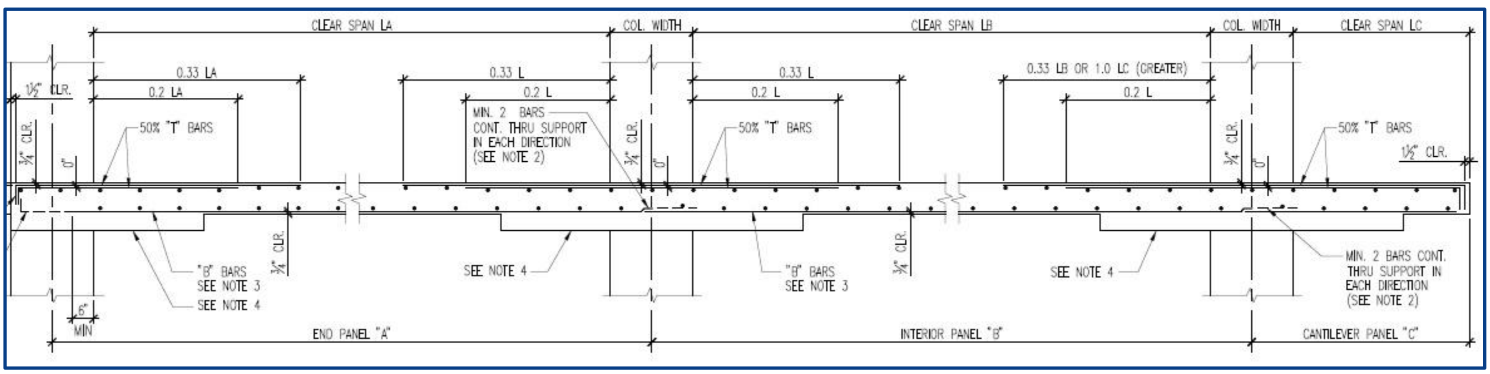


Figure 7:
Typical column strip reinforcement and placement

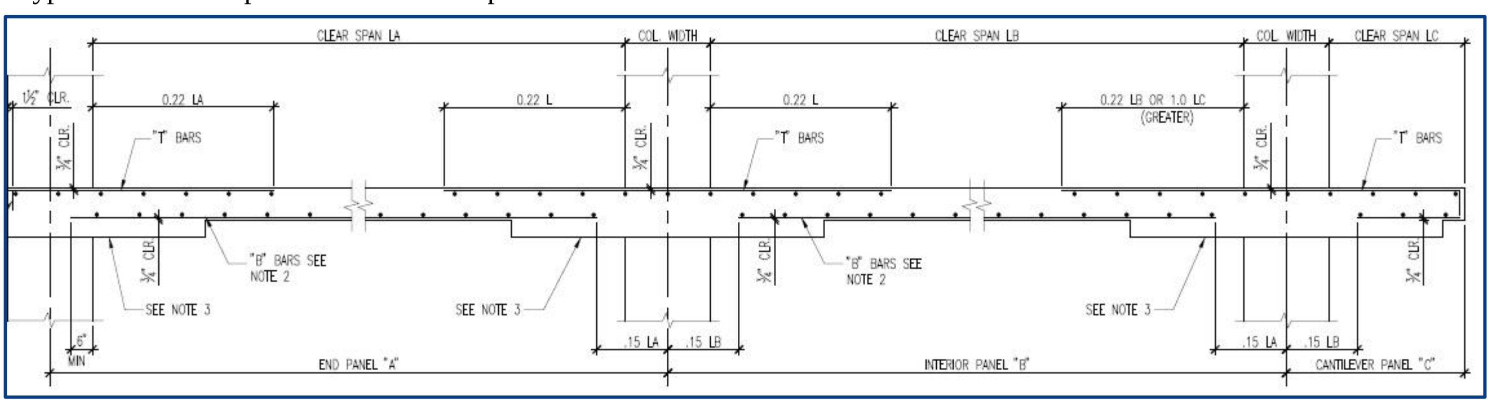


Figure 8:
Typical middle strip reinforcement and placement

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 ft x 29 ft with drop panels at every location (see Appendix F 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 in. x 30 in. in the basement level. The typical column size is 24 in. x 24 in. and 12 in. x 18 in. (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 5^{th} 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 longitudinal reinforcement bars within a column is 20, with the typical number being 4.

Lateral Systems:

Shear walls and ordinary moment resisting frames make up 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 while the moment frames are situated near the connection and at the far end of the structure (see Figure 9 located on the next page). The shear walls are 12 in. 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 ft. height to the top of the penthouse level.

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. The moment frames are mainly situated in the Y-Direction, and both the shear wall and moment frame notations can be seen in Figure 10 for future references throughout this report. After performing the analysis using ETABS, the displacements found in the Y-Direction were significantly smaller than the X-Direction. Due to the connection with the existing structure, the displacements in the Y-Direction are limited. This explains the need for most of the moment frames in that direction as well as the larger shear walls located near the connection point. Because most of the rigidity falls near the existing structure, the far end located furthest from the connection point could be of concern when dealing with displacements due to the lack of a lateral system in the X-Direction. Detailed elevations of the shear wall can be seen in Figure 11 depicting the various openings located in shear walls in both the X and Y direction.

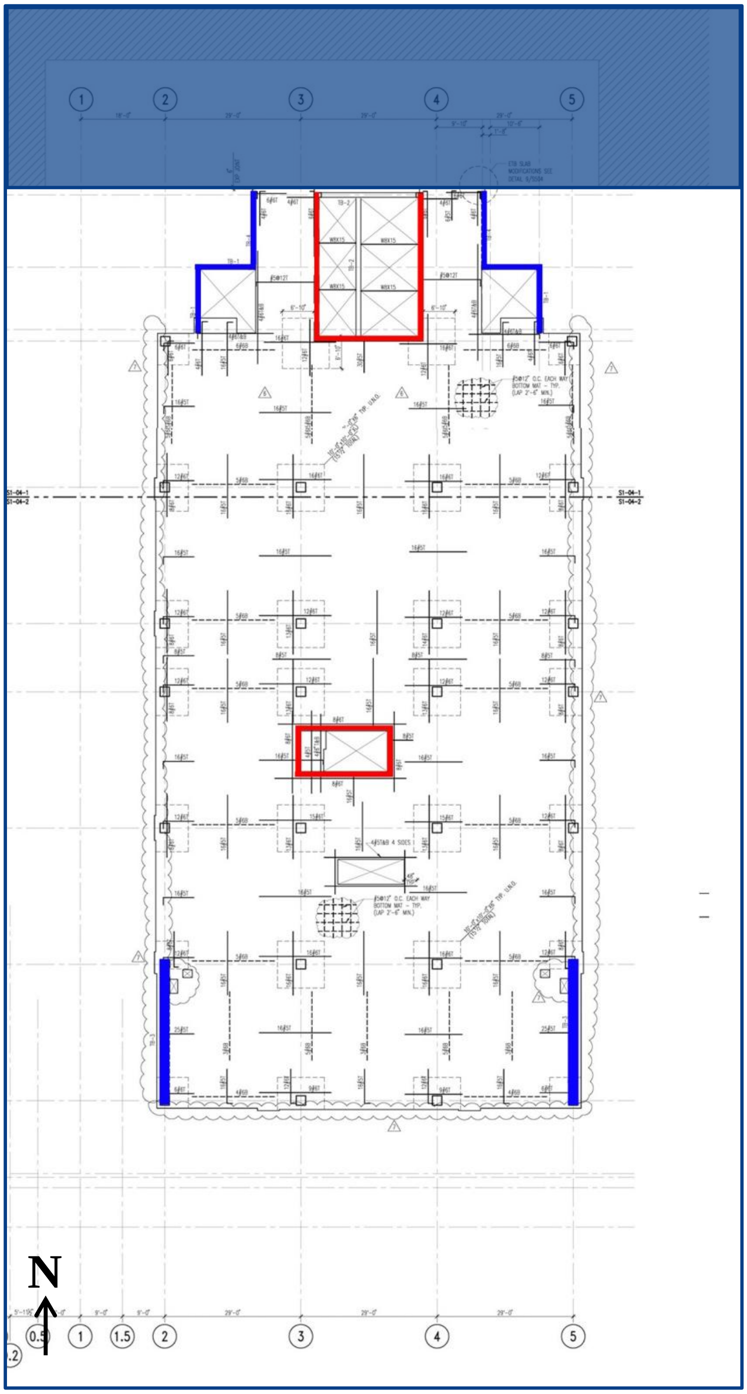


Figure 9:
Typical floor plan depicting the shear walls (shaded in red) and the moment frames (shaded in blue)

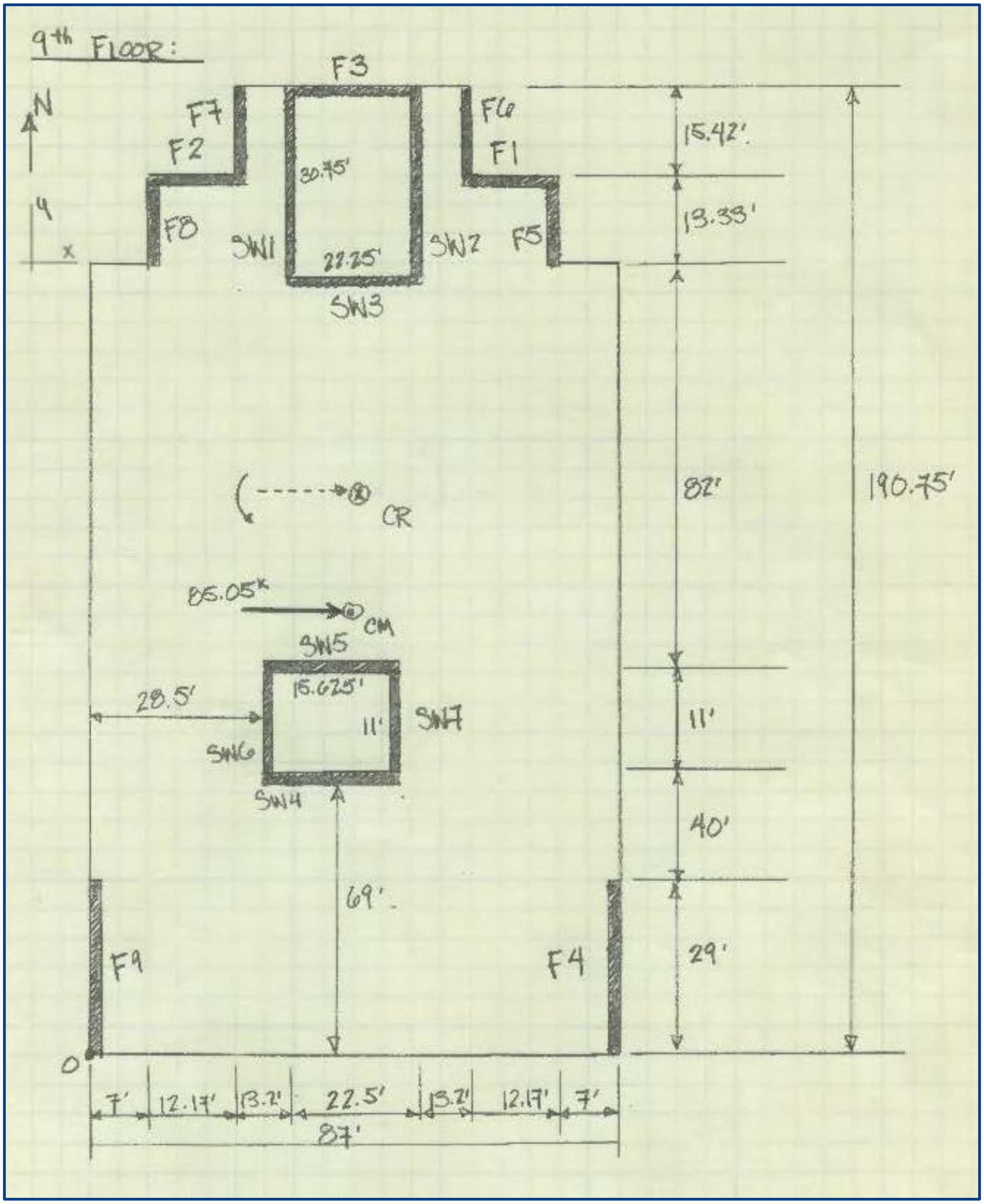


Figure 10:
Typical floor plan with shear walls and moment frames labeled for ease of reference
Reference location is taken from the bottom of Frame 9

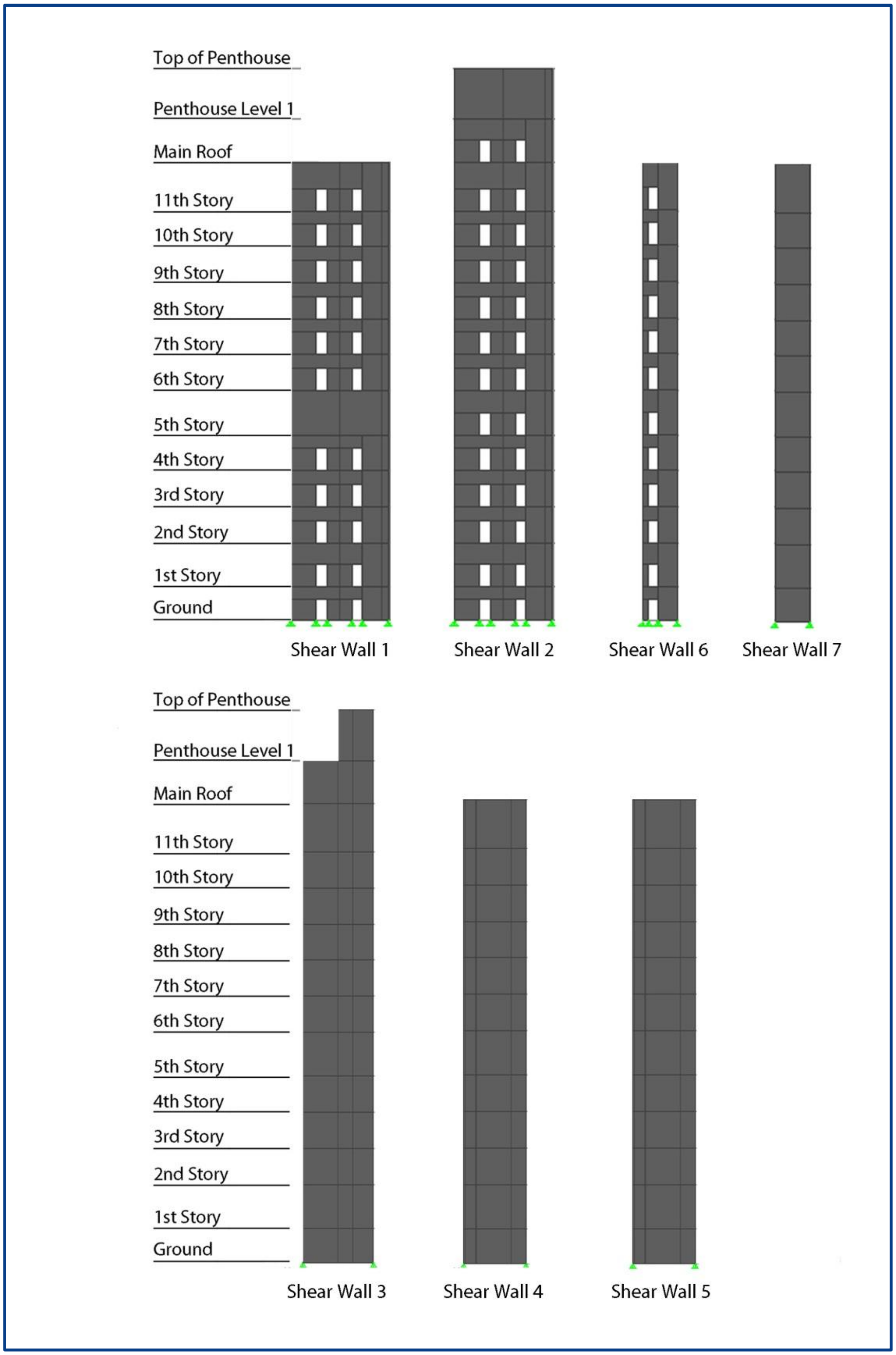


Figure 11:
Shear wall elevations with the upper half being the walls located in the Y-Direction and the lower half in the X-Direction

Roof System:

In general, there are three different main roof levels (see Figure 12). 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 with varying thickness.

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.

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 13). The slab thickness for the lower roofs (excluding the green roof) varies but is mainly 9 ½ in. while the main roof, which supports higher loads from the mechanical penthouse, is 12 in. thick.

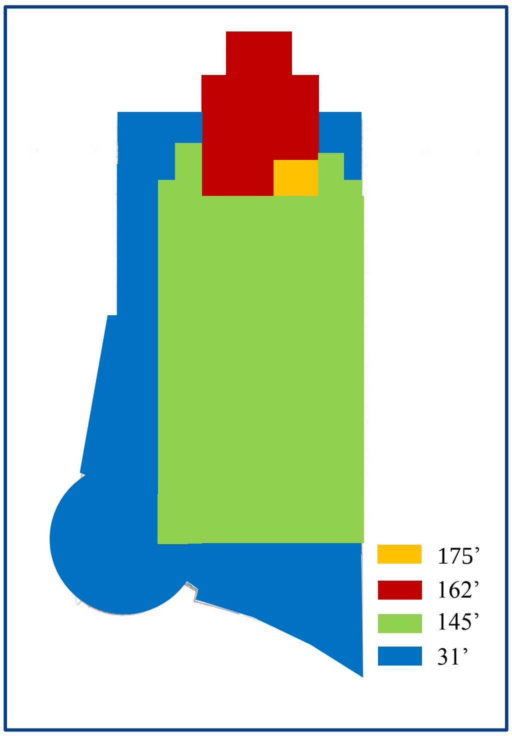


Figure 12:
Showing different roof heights in relation to 0’-0”

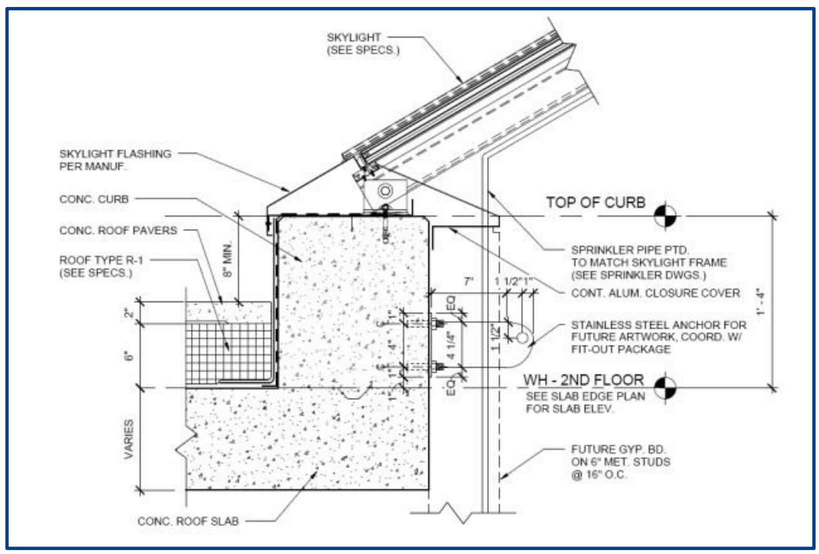


Figure 13:
Roof and skylight detail

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 (2010)

Materials Used:

The various kinds of materials and standards used for the construction of the South Patient Tower are listed in Figure 14a and 14b 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 ASTM 501	B (F_y = 35 ksi) 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 14a:
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 14b:
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.

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 15.

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

Figure 15:
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 in. concrete with a ½ in. thin brick face. To simplify calculating the weight of this system, a 6 in. concrete panel was assumed to account for both elements. Figure 16 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 39,000 K.

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 17). 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	25513	N/A
1st	25513	4393
2nd	11649	2418
3rd	17958	3902
4th	16571	3011
5th	16571	3285
6th	16571	3078
7th	16571	3011
8th	16571	3011
9th	16571	3011
10th	16571	3011
11th	16571	3066
Penthouse/Roof	16571	3831
		39026

Figure 16:
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 Canopie	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 17:
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 18. 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 Figure 19.

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 18:
Summary of roof snow load values

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

Figure 19:
Summary of roof snow drift calculations

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 and then applied to a lateral model of the structure created in ETABS. Hand calculations for both of these sections can be found in Appendices A (Wind) and B (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 due to the expansion joint that exists between the current structure and the existing portion. 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 20a and 20b). Using these two separate structures 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' but the structure was assumed to end at the main roof level (two levels below top of penthouse). Since the penthouse is roughly 15% of a typical floor plan and spans over to the existing portion of the hospital, it was concluded that the wind forces would be negligible and shared between the two buildings.

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.

For this technical report, load combinations were determined using Figure 6-9 of ASCE 7-05. The four different combinations were then broken up into the X and Y direction and then combined with the load combinations in Chapter 2 of ASCE 7-05. The wind load combinations broken up into the four different cases with accidental moments are summarized in Figure 21.

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 (unfactored) 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.

`

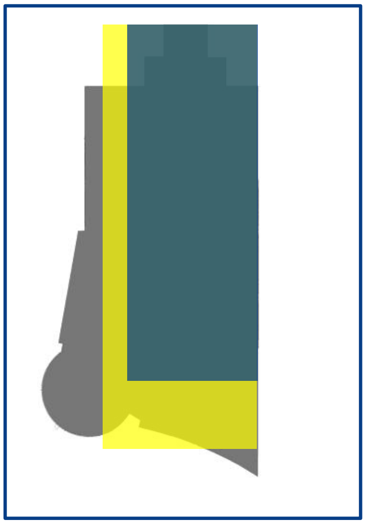


Figure 20a:
Plan view of the two separate wind towers

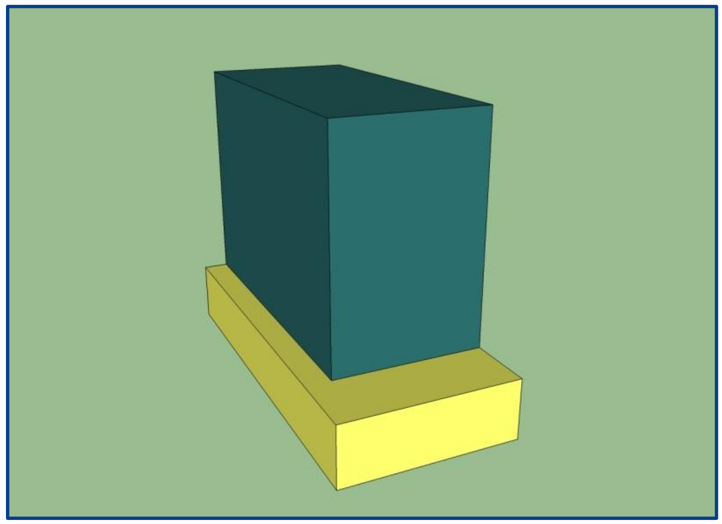


Figure 20b:
Perspective view of the two separate wind towers

Load Combinations for Serviceability (1.0 Wind)			
Wind	Case 1	$P_{WX} + P_{LX}$	
		$P_{WX} + P_{LY}$	
	Case 2	$0.75P_{WX} + 0.75P_{LX} + M_T$	$M_T = 0.75(P_{WX} + P_{LX})B_Xe_X$ $e_X = \pm 0.15B_X$
		$0.75P_{WY} + 0.75P_{LY} + M_T$	$M_T = 0.75(P_{WY} + P_{LY})B_Ye_Y$ $e_Y = \pm 0.15B_Y$
	Case 3	$0.75P_{WX} + 0.75P_{LX} + 0.75P_{WY} + 0.75P_{LY}$	
	Case 4	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T$	$M_T = 0.563(P_{WX} + P_{LX})B_Xe_X + 0.563(P_{WY} + PLY)B_Ye_Y$ $e_X = \pm 0.15B_X$ $e_Y = \pm 0.15B_Y$

Figure 21:
The four cases used for wind in determining displacements and drifts

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
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 22:
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	244.45	0.00
1st	10.83	5.42	568.58	7.00	735.00	18.70	236.68	202.56
2nd	24.83	7.00	735.00	5.67	595.35	20.44	217.98	507.49
3rd	36.17	5.67	595.35	5.67	510.00	18.12	197.54	655.24
4th	47.50	5.67	510.00	5.58	502.50	17.43	179.42	828.11
5th	58.67	5.58	502.50	7.13	641.70	20.58	161.99	1207.50
6th	72.93	7.13	641.70	5.62	505.80	21.32	141.41	1555.01
7th	84.17	5.62	505.80	5.67	509.85	19.43	120.09	1635.45
8th	95.50	5.67	509.85	5.67	509.85	19.96	100.66	1905.75
9th	106.83	5.67	509.85	5.67	510.30	20.38	80.70	2176.94
10th	118.17	5.67	510.30	5.67	509.85	20.78	60.32	2455.62
11th	129.50	5.67	509.85	7.67	689.85	25.02	39.54	3239.55
Roof	144.83	7.67	689.85	N/A	0.00	14.53	14.53	2104.13
					Total Base Shear =	244.45		
							Total Overturning Moment =	18,473.36 k-ft

Figure 23:
List of N-S direction wind forces

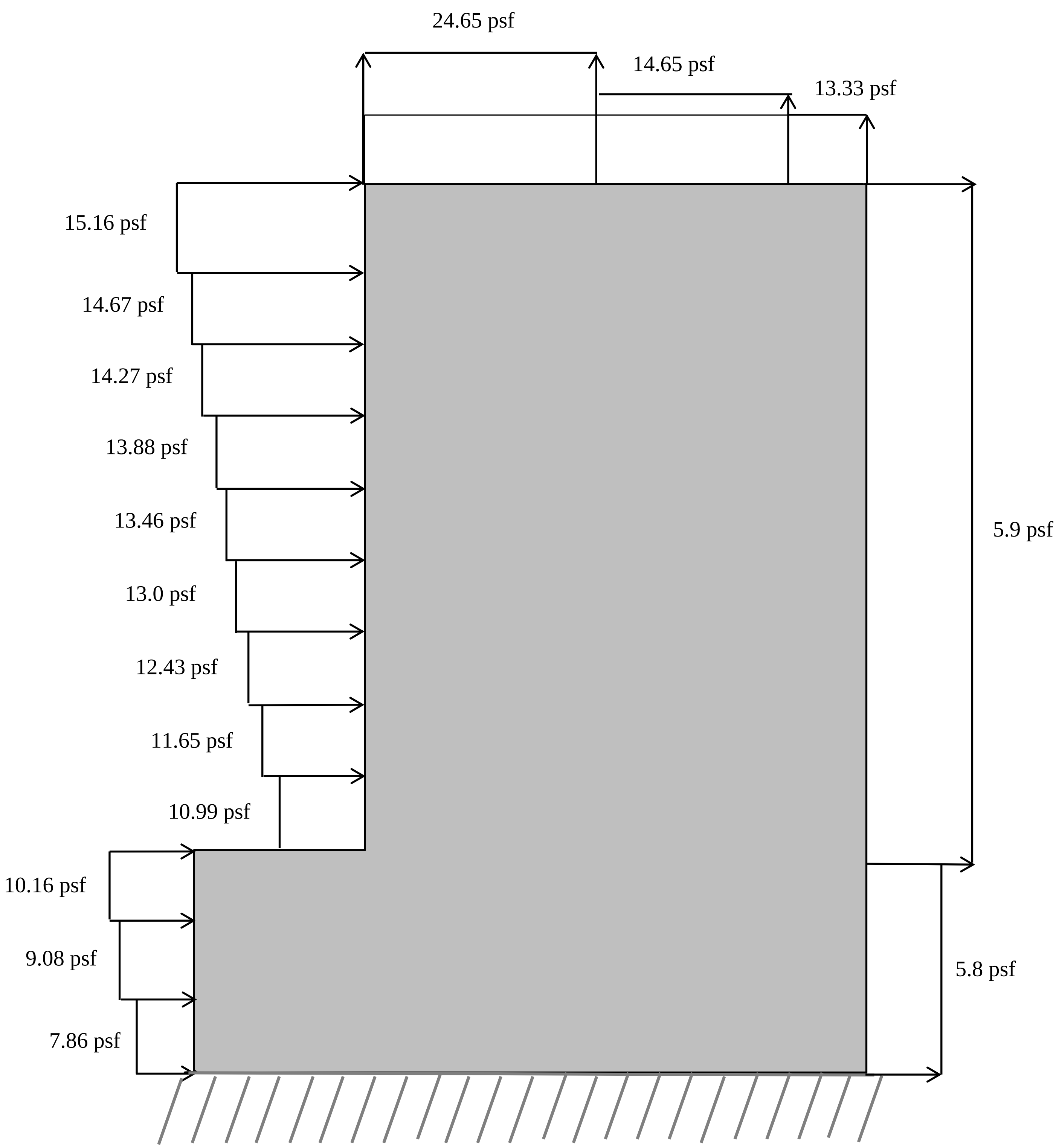


Figure 24a:
Diagram of N-S direction wind pressures

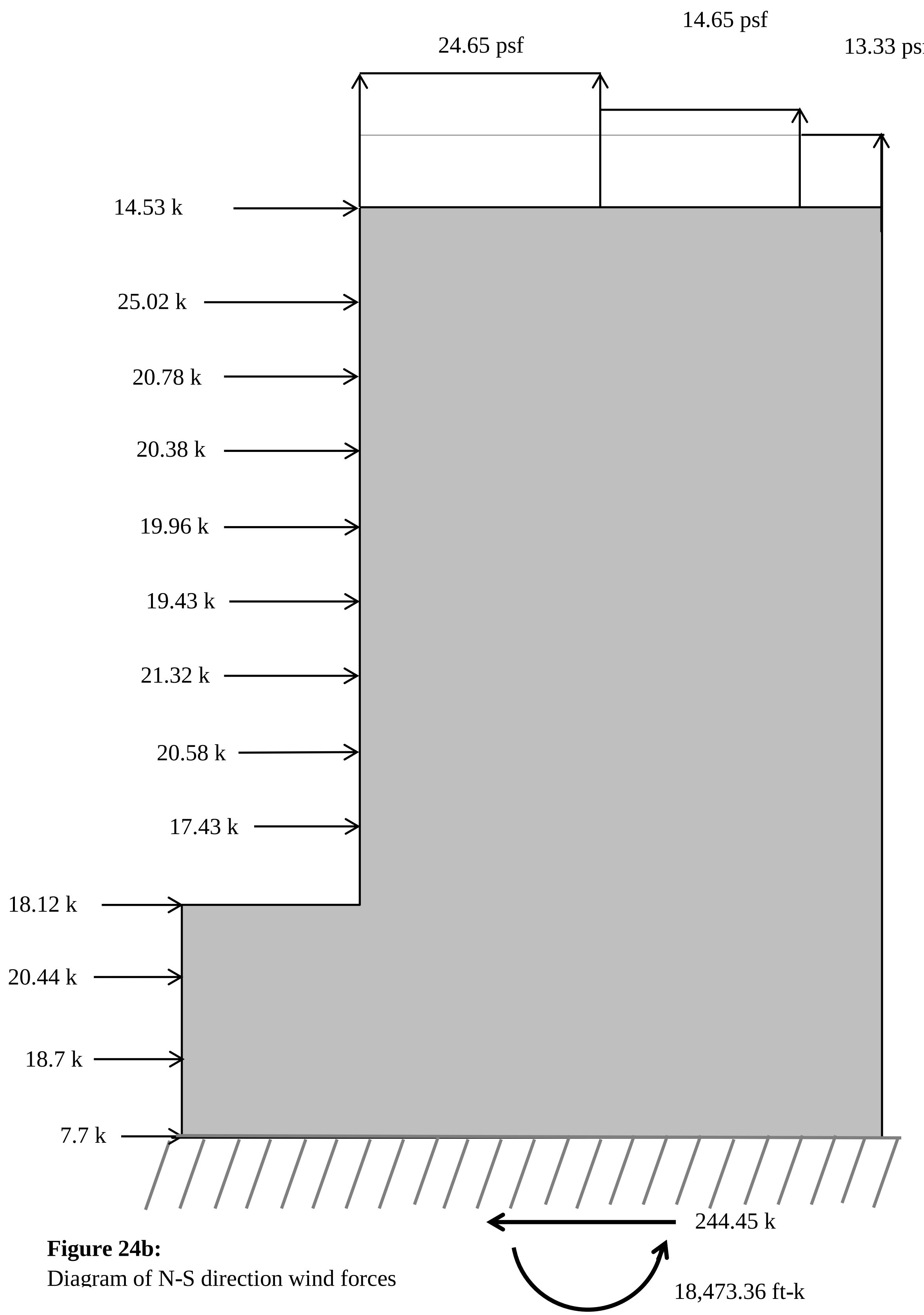


Figure 24b:
Diagram of N-S direction wind forces

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
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 25:
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	642.42	0.00
1st	10.83	5.42	1250.87	7.00	1617.00	53.16	620.09	575.77
2nd	24.83	7.00	1617.00	5.67	1309.77	57.23	566.93	1420.97
3rd	36.17	5.67	1309.77	5.67	1080.92	49.07	509.70	1774.84
4th	47.50	5.67	1080.92	5.58	1065.02	45.72	460.63	2172.07
5th	58.67	5.58	1065.02	7.13	1360.05	53.54	414.91	3141.15
6th	72.93	7.13	1360.05	5.62	1072.02	55.14	361.37	4021.21
7th	84.17	5.62	1072.02	5.67	1080.60	49.99	306.23	4207.29
8th	95.50	5.67	1080.60	5.67	1080.60	51.13	256.24	4883.29
9th	106.83	5.67	1080.60	5.67	1081.55	52.03	205.11	5558.62
10th	118.17	5.67	1081.55	5.67	1080.60	52.89	153.08	6249.54
11th	129.50	5.67	1080.60	7.67	1462.10	63.42	100.19	8212.81
Roof	144.83	7.67	1462.10	N/A	0.00	36.77	36.77	5325.66
					Total Base Shear =	642.42		
							Total Overturning Moment =	47,543.22 k-ft

Figure 26:
List of E-W direction wind forces

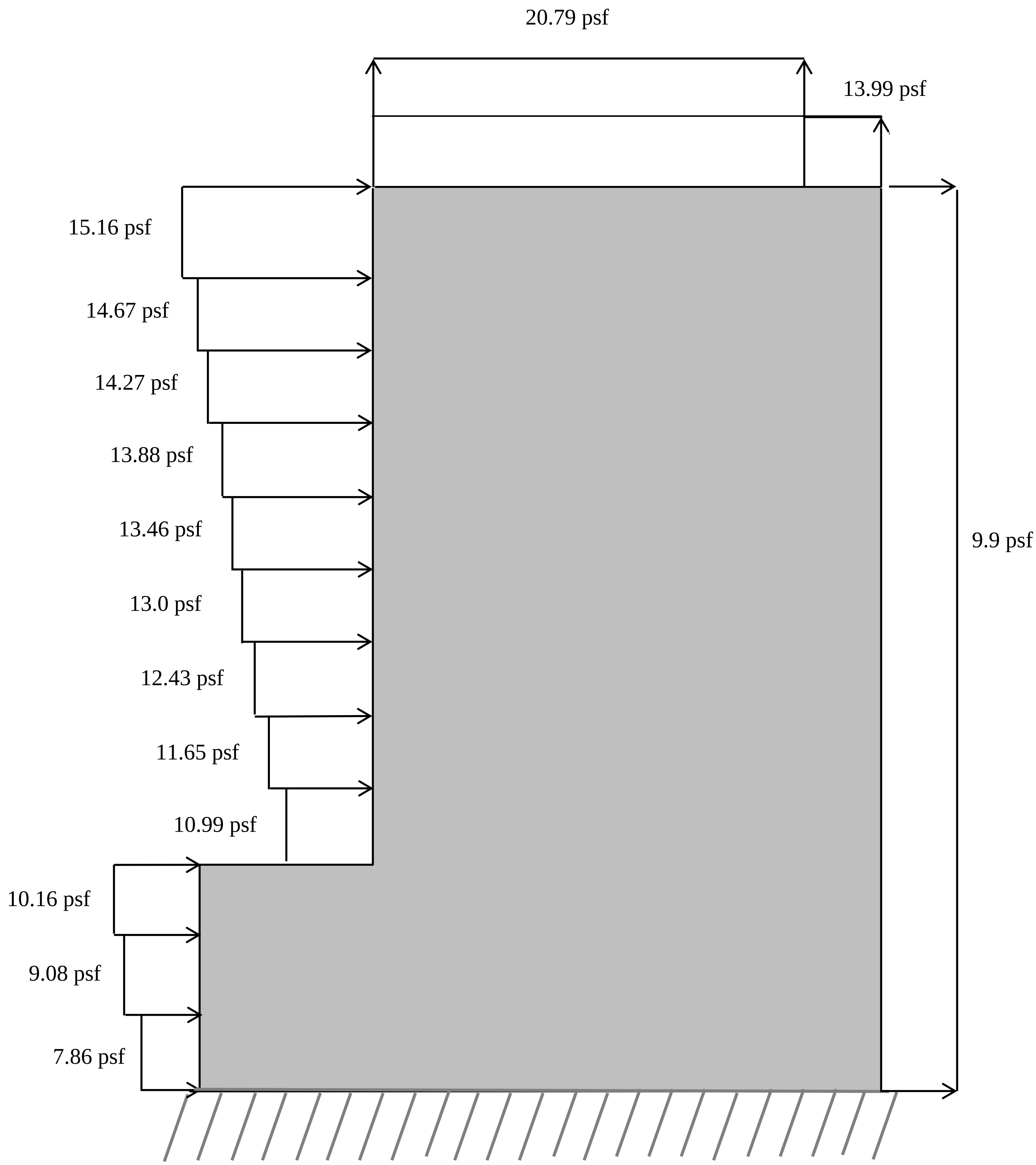


Figure 27a:
Diagram of E-W direction wind pressures

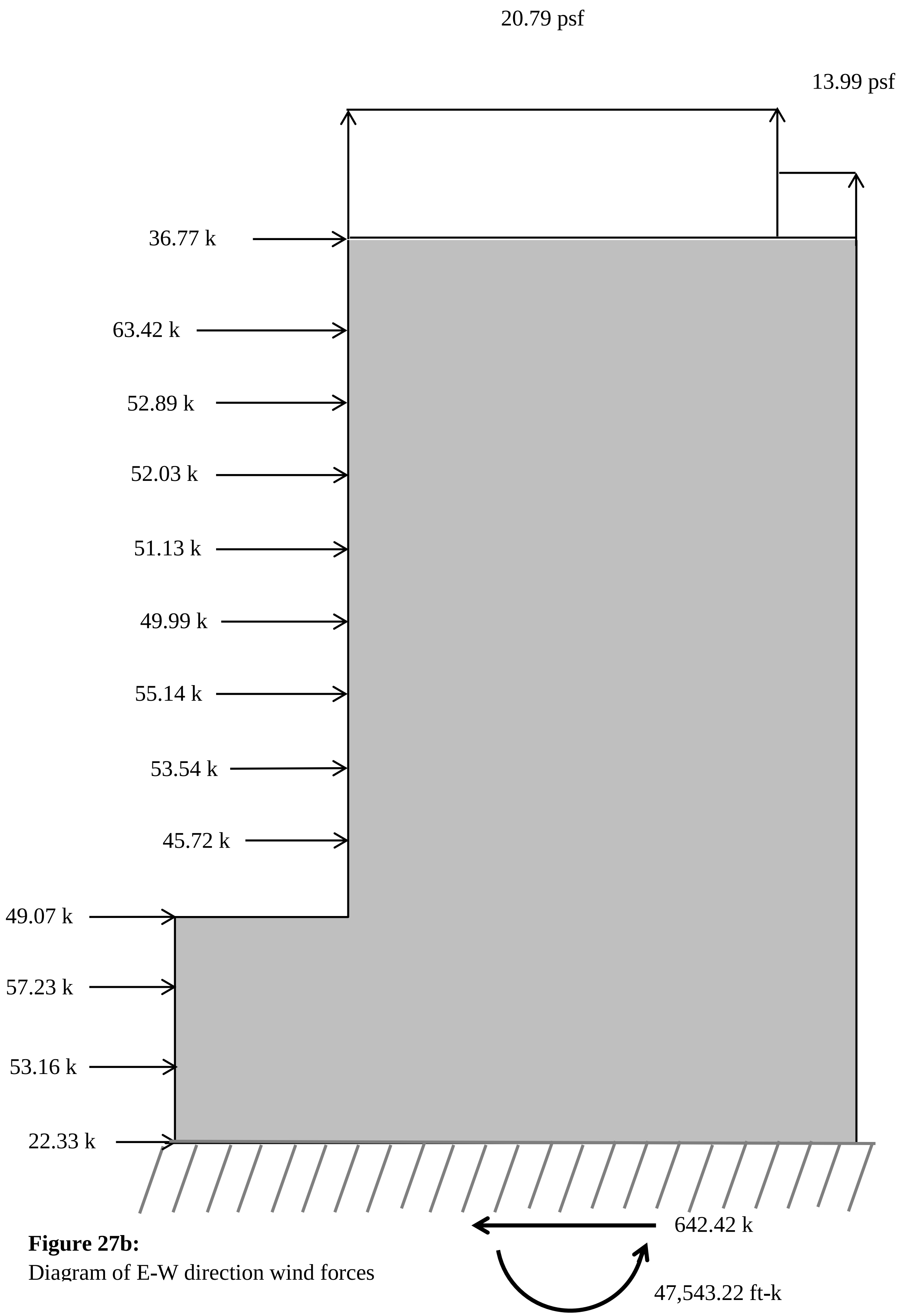


Figure 27b:
Diagram of E-W direction wind forces

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 since the expansion joint will separate the two buildings completely from each other. 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 747 k which only differed by about 6.7% from the structural drawings. This slight discrepancy is likely due to a difference in the calculated weight. One other difference that most likely caused the variation was that the structural drawings called out slightly different S_S and S_1 values. One 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. Also, since the Wind forces were obtained using the main roof level as the top (ignoring the penthouse in calculations), in order to accurately compare the two, the same level was used as the overall building height. Figures 28 and 29 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	3.86	692.50	41.13073686
2nd	2417.8	24.67	303505.33	0.0101	7.51	688.64	185.2779646
3rd	3902.0	36.00	866097.18	0.0287	21.43	681.13	771.6424501
4th	3010.7	47.33	1009605.78	0.0334	24.99	659.70	1182.676325
5th	3285.3	58.67	1522642.55	0.0504	37.68	634.71	2210.733348
6th	3078.1	72.67	1969868.32	0.0652	48.75	597.03	3542.578011
7th	3010.7	84.00	2397250.26	0.0794	59.33	548.28	4983.559489
8th	3010.7	95.33	2901211.23	0.0961	71.80	488.95	6844.963165
9th	3010.7	106.67	3436576.58	0.1138	85.05	417.15	9071.972736
10th	3010.7	118.00	4001651.25	0.1325	99.03	332.10	11686.0632
11th	3065.8	129.33	4678992.06	0.1550	115.80	233.07	14976.48054
Penthouse/Roof	3831.1	145.00	6947035.33	0.2301	171.93	117.27	24929.55332
						Base Shear =	**747.16 k**
						Total Overturning Moment =	**80,426.63 k-ft**

Figure 28:
List of seismic forces for both directions

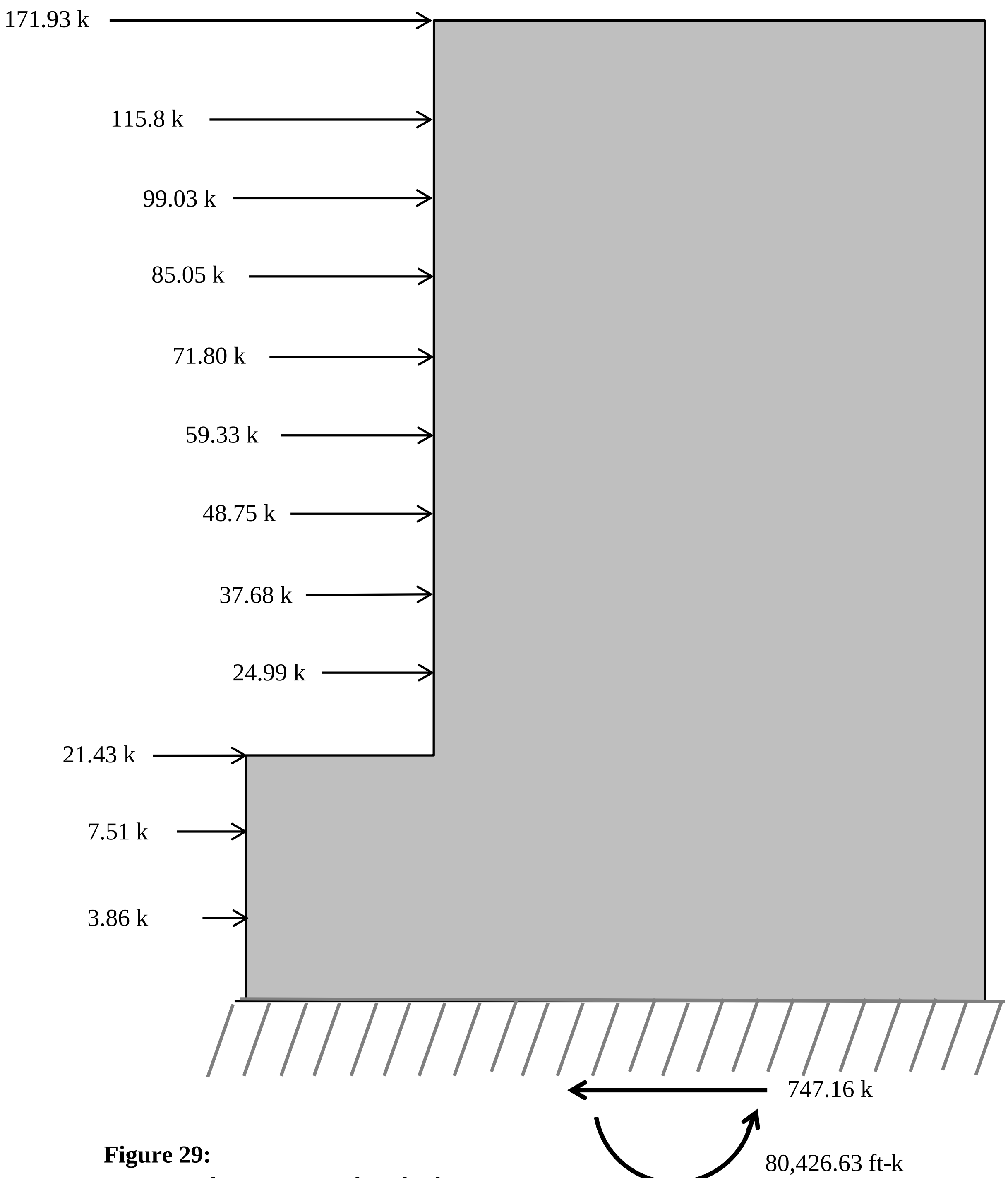


Figure 29:
Diagram of N-S/E-W earthquake forces

Accidental moments were also accounted and calculated for all seismic forces as per §12.824.2 of ASCE 7-05. The accidental torsion included for each story level is calculated by taking the story force and multiplying by an accidental eccentricity equal to 5% of the dimension of the building perpendicular to the direction the force is applied. The two earthquake load combinations (Figure 30) were then combined with the load combinations of Chapter 2 of ASCE 7-05. In order to account for the amplification factor for the accidental torsion, a value of 1.0 was assigned and the moments calculated (Figures 31 and 32). Once the analysis for both models was complete, a new amplification factor was calculated and applied to the structure. Further detail regarding amplification factor can be found in the Computer Modeling section. Once obtaining the accidental moments, the building was checked for horizontal irregularities which are discussed into further detail in the Computer Modeling Process section.

Load Combinations for Serviceability (1.0 Earthquake)		
Earthquake	Case 1	$1.0E_X + M_{ZX}$
	Case 2	$1.0E_Y + M_{ZY}$

Figure 30:
Serviceability combinations considering seismic loads

Rigid Diaphragm							
B_X	$5\%B_X$	A_{XY}	M_{ZY}	B_Y	$5\%B_Y$	A_{XX}	M_{ZX}
105	5.25	1.0	0	231	11.55	1.0	0.00
105	5.25	1.0	20.24	231	11.55	3.00	133.61
105	5.25	1.0	39.43	231	11.55	2.79	241.76
105	5.25	1.0	112.53	231	11.55	2.66	659.41
90	4.5	1.0	112.44	190.75	9.5375	2.61	622.49
90	4.5	1.0	169.57	190.75	9.5375	2.65	953.45
90	4.5	1.0	219.38	190.75	9.5375	2.66	1234.88
90	4.5	1.0	266.98	190.75	9.5375	2.65	1501.96
90	4.5	1.0	323.10	190.75	9.5375	2.65	1812.35
90	4.5	1.0	382.72	190.75	9.5375	2.63	2137.23
90	4.5	1.0	445.65	190.75	9.5375	2.62	2475.95
90	4.5	1.0	521.09	190.75	9.5375	2.61	2880.57
90	4.5	1.0	773.68	190.75	9.5375	2.59	4249.84
		$\sum M_{ZY}$ =	**3386.82**			**$\sum M_{ZX}$ =**	**18903.50**

Figure 31:
Calculated accidental moments for rigid diaphragm model

Two-Way Slab System - Shell Element							
B_X	$5\%B_X$	A_{XY}	M_{ZY}	B_Y	$5\%B_Y$	A_{XX}	M_{ZX}
105	5.25	1.0	0	231	11.55	1.00	0.00
105	5.25	1.0	20.24	231	11.55	2.97	132.32
105	5.25	1.0	39.43	231	11.55	2.76	239.18
105	5.25	1.0	112.53	231	11.55	2.66	659.57
90	4.5	1.0	112.44	190.75	9.5375	2.61	622.61
90	4.5	1.0	169.57	190.75	9.5375	2.61	938.44
90	4.5	1.0	219.38	190.75	9.5375	2.60	1207.66
90	4.5	1.0	266.98	190.75	9.5375	2.58	1458.11
90	4.5	1.0	323.10	190.75	9.5375	2.55	1746.41
90	4.5	1.0	382.72	190.75	9.5375	2.52	2044.03
90	4.5	1.0	445.65	190.75	9.5375	2.49	2349.78
90	4.5	1.0	521.09	190.75	9.5375	2.46	2712.73
90	4.5	1.0	773.68	190.75	9.5375	2.41	3958.88
		$\sum M_{ZY}$ =	**3386.82**			**$\sum M_{ZX}$ =**	**18069.73**

Figure 32:
Calculated accidental moments for shell element model

Lateral System Analysis:

In order to fully understand the behavior of the SPT under lateral loading (wind and seismic), two models were built in ETABS. The first model constructed included rigid diaphragms, while the second model consists of shell elements inserted to accurately model the behavior of the two-way slab. Attempts were made to verify all results using hand calculations, although due to the complexity of the lateral system, hand checks were not performed for displacements and drifts.

Computer Modeling Process:

Several assumptions were made while creating both of the lateral models that have an impact on the final results obtained from ETABS. According to §8.8.2 of ACI 318-05, the stiffness of the lateral resisting elements need to incorporate the cracking of the concrete. In order to achieve this, the code permits either a 50% factor to every gross section property for each concrete element or a certain percentage that is determined by the type of object (i.e. beam, column, wall, etc.). The former option was chosen for ease of modeling and was done by applying a 0.5 property modifier to various moments and shears based on the function/orientation of the member. One other modification to the material properties revolved around self-mass. In order to ease the modeling process, the mass for all of the elements for each floor was assigned to the diaphragm. Since the mass for each element was included in the diaphragm, the self-mass was turned off for each material property.

While looking at the structural drawings, it was determined to use pin connections for the base of the columns and shear walls. The reinforcement for the shear walls and columns did not increase approaching the foundation level, and actually slightly decreased in regard to the number of bars. Due to this, the connection to the foundation level for every member was assumed to be pinned.

In order to accurately model the connection of the cast-in-place beams and columns, every member had to include rigid-end offsets to move the location of the beam ends to the column face, and likewise for the columns. If not done, the connection of the beams and columns would be too rigid and not accurately predict the true nature of the connection. In regards for the shear walls, they were modeled as membranes, which carry shear in the line of direction but not in the direction perpendicular to the wall.

Although the purpose of this technical report is to analyze the lateral system of the SPT, the model includes the gravity framing members. This was done primarily just to have a complete model and possess no significant changes to the outcome of the lateral load analysis.

In total, two models were constructed for this technical assignment. The first model incorporates rigid diaphragms to represent the two-way flat slab system. Rigid diaphragms disregard the stiffness properties associated with the two-way flat slab system, and therefore do not participate in the process of resisting the lateral loads. In order to better account for the stiffness of the floor system, a second model was constructed and modeled the two-way concrete slab system as a shell element. By modeling the floor system as a shell element, the model integrates the stiffness properties of the slab and therefore can partake in the resistance to the lateral loads. By modeling the slab as a shell, the floor system is capable of taking both in-plane and out-of-plane shears and moments as compared to the membrane elements (shear walls), which only take in-plane shears.

The following figures (Figures 33-35) show perspective views of the lateral system as modeled in ETABS.

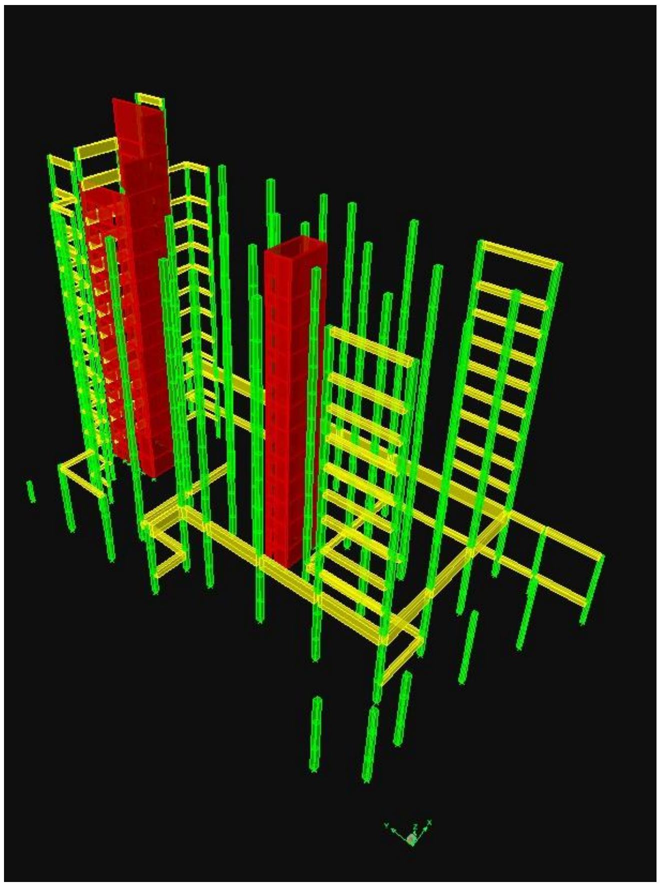


Figure 33:
ETABS modeling depicting the lateral resisting system and gravity members

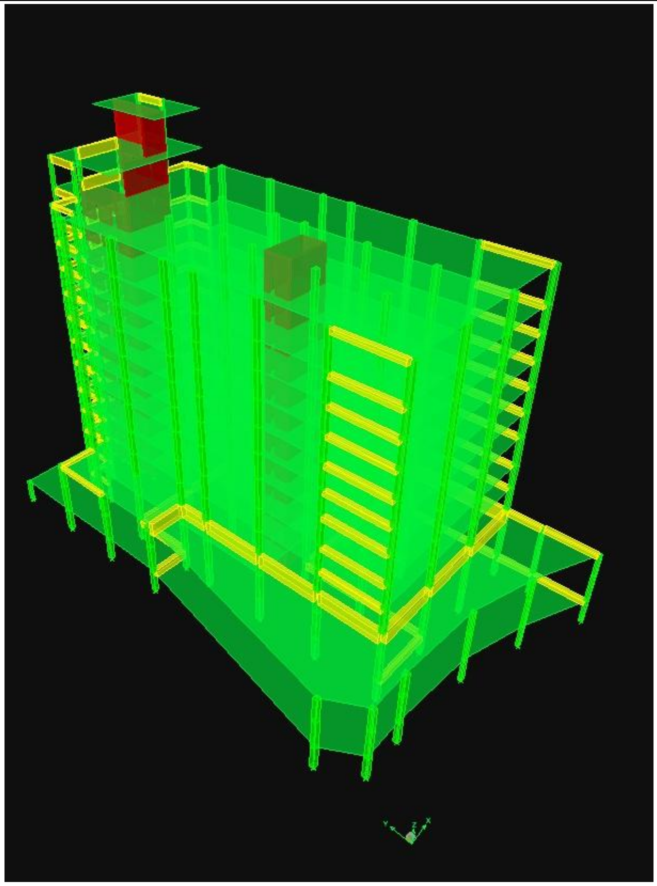


Figure 34:
ETABS model with the floor system modeled as a rigid diaphragm

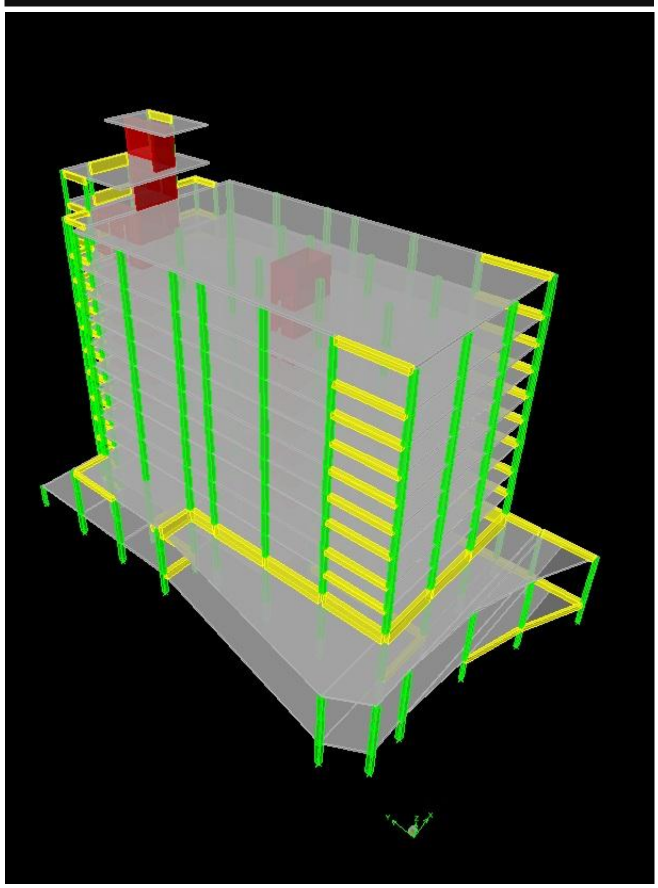


Figure 35:
ETABS model with the two-way concrete flat slab modeled as a shell element

Building Properties:

In order to check the ETABS model, the center of mass and the center of rigidity were calculated by hand and then compared to the values given by the rigid diaphragm model (Figure 36). The center of mass and center of rigidity hand calculations were done for a typical floor (9th Floor) and can be found in Appendix C.

Center of Mass and Center of Rigidity (ETABS)				
Story	**XCCM**	**YCCM**	**XCR**	**YCR**
MAIN ROOF	43.9	100.6	44.1	127.4
STORY11	43.7	96.3	44.3	126.7
STORY10	43.7	94.7	44.3	126.0
STORY9	43.6	93.9	44.3	125.1
STORY8	43.6	93.4	44.3	124.0
STORY7	43.6	93.0	44.3	122.7
STORY6	43.6	92.7	44.3	121.2
STORY5	43.6	92.5	44.4	119.0
STORY4	43.6	92.4	44.3	116.5
STORY3	42.4	90.5	44.1	114.0
STORY2	42.6	92.7	43.5	110.6
STORY1	41.5	91.4	42.9	105.1

Figure 36:
Center of mass and center of rigidity (ETABS) from point of origin in Figure 10

The center of mass was found by breaking up the slab into rectangles and calculating the weight for all of the lateral resisting elements including the slab. Because of the symmetry of the frames and for ease of calculation purposes, the frames were neglected in the center of mass calculations. The hand calculation for the center of mass produced almost identical numbers in the X-Direction, with a difference of 0.4 ft compared to the ETABS model. The Y-Direction varied slightly more, differing from the ETABS value by 1.6 ft due to ignoring the weight of the moment frames.

The center of rigidity, which is the location at which an applied load would cause no torsion in the floor diaphragm, was calculated by using the relative stiffness of the frames and the shear walls. To find the relative stiffness of each element that participates in the lateral resistance, a 1000 k load was applied at the center of rigidity on the main roof's rigid diaphragm in both the X and Y direction in ETABS. Taking advantage of pier and frame labeling, the shear forces in each of the walls and frames were then calculated for each floor. The total shear force in all of the members should add up to 1000 k (in the direction of interest) for each floor. However, as the forces traveled down the building, the sum of the shear forces for the members of interest started to vary slightly from the expected value. This can be attributed due to the inherent torsion that is created as the force moves down floor to floor since the center of rigidities and center of masses do not line up at the same location for each diaphragm. Another explanation that explains the somewhat creation of shear is shear reversal. This can happen when there is a sudden change in stiffness in the member and can also be contributed to the rigid diaphragm approximation. To correct this, the use of semi-rigid diaphragms to model the floor system should be utilized; however, due to time limitations, this was not feasible. Due to the symmetry of the structure/lateral system, the hand calculation for the center of rigidity produced relatively the same X location (differed by 0.2 ft). However, the Y-Coordinate for the center of rigidity done from hand calculations and that obtained from the ETABS model were off by about 10 ft. The difference in the two values is roughly 5.13% of the total dimension of the building, which is within a reasonable margin of error. Therefore, it seems as though the model can be considered accurate since the numbers could be replicated within a reasonable margin. The relative stiffness for each element in the X-Direction and the Y-Direction can be found in Figure 37.

Relative Stiffness in X-Direction - % of Total Direct Shear						
Story Level	SW 3	SW 4	SW 5	FRAME 1	FRAME 2	FRAME 3
Main Roof	66.4%	16.7%	12.4%	1.1%	1.1%	2.3%
11th	62.5%	18.8%	11.3%	1.5%	1.5%	4.4%
10th	62.2%	19.1%	11.6%	1.5%	1.5%	4.2%
9th	62.1%	18.8%	12.3%	1.4%	1.4%	4.1%
8th	61.8%	18.3%	13.2%	1.3%	1.4%	3.9%
7th	62.4%	17.2%	14.0%	1.3%	1.3%	3.7%
6th	67.1%	14.6%	14.6%	0.8%	0.8%	2.1%
5th	59.6%	16.3%	18.2%	1.2%	1.2%	3.6%
4th	71.1%	7.5%	15.2%	1.2%	1.3%	3.7%
3rd	68.0%	8.9%	18.2%	1.0%	1.0%	3.0%
2nd	68.1%	10.2%	19.4%	0.5%	0.5%	1.3%
1st	55.9%	19.3%	24.6%	0.0%	0.0%	0.2%

Relative Stiffness in Y-Direction - % of Total Direct Shear										
Story Level	SW 1	SW 2	SW 6	SW 7	FRAME 4	FRAME 5	FRAME 6	FRAME 7	FRAME 8	FRAME 9
Main Roof	34.1%	41.1%	5.1%	6.9%	4.3%	1.0%	1.0%	1.1%	1.0%	4.3%
11th	36.9%	34.3%	4.2%	8.3%	4.7%	1.5%	2.0%	2.0%	1.5%	4.7%
10th	36.3%	35.1%	4.3%	8.4%	4.7%	1.4%	1.8%	1.9%	1.4%	4.7%
9th	36.0%	35.8%	4.4%	8.6%	4.5%	1.4%	1.8%	1.8%	1.4%	4.4%
8th	35.5%	36.5%	4.7%	8.9%	4.2%	1.3%	1.7%	1.7%	1.3%	4.2%
7th	35.1%	37.2%	5.0%	8.7%	4.0%	1.2%	1.7%	1.6%	1.2%	4.2%
6th	50.3%	32.9%	3.1%	5.4%	2.7%	0.7%	0.9%	0.9%	0.7%	2.5%
5th	35.5%	39.2%	6.4%	11.0%	0.8%	1.1%	1.6%	1.5%	1.1%	1.8%
4th	26.5%	31.0%	4.1%	6.8%	16.0%	0.9%	0.6%	1.2%	0.9%	12.1%
3rd	21.4%	22.5%	2.9%	2.7%	32.7%	0.6%	0.0%	1.0%	0.7%	15.5%
2nd	38.2%	41.1%	7.3%	12.4%	0.0%	0.4%	0.0%	0.3%	0.3%	0.0%
1st	37.9%	37.6%	8.8%	15.7%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Figure 37:
Relative stiffness for the shear walls and frames in the X-Direction as well as the Y-Direction

Upon verifying the accuracy of the ETABS model, the amplification factor was then properly accounted for. Because of the large eccentricity (in the X-Direction) obtained from the difference in the Y-Coordinates of the center of mass and center of rigidity (~33 ft), torsion could pose a significant problem to this building. After running the models with the assumption that the amplification factor (A_X) equaled 1.0, a more accurate value for this amplification was obtained for both the rigid diaphragm model as well as the shell element model which can be found in Figure 38 and Figure 39 respectively.

Rigid Diaphragm				
	Level	$\delta_{X\,MAX}$	$\delta_{X\,MIN}$	A_{XX}
X-Direction Loading	Ground	0	0	1.00
	1st	0.087	-0.006	3.00
	2nd	0.454	-0.001	2.79
	3rd	0.602	0.013	2.66
	4th	0.960	0.030	2.61
	5th	1.413	0.033	2.65
	6th	2.033	0.046	2.66
	7th	2.579	0.059	2.65
	8th	3.148	0.077	2.65
	9th	3.730	0.100	2.63
	10th	4.317	0.127	2.62
	11th	4.905	0.157	2.61
	Penthouse/Roof	5.698	0.201	2.59

Figure 38:
Amplification factor for the rigid diaphragm model

Two-Way Slab System - Shell Element				
	Level	$\delta_{X\,MAX}$	$\delta_{X\,MIN}$	A_{XX}
X-Direction Loading	Ground	0	0	1.00
	1st	0.051	-0.002	2.97
	2nd	0.264	0.001	2.76
	3rd	0.356	0.008	2.66
	4th	0.551	0.017	2.61
	5th	0.779	0.024	2.61
	6th	1.083	0.037	2.60
	7th	1.335	0.051	2.58
	8th	1.585	0.069	2.55
	9th	1.828	0.091	2.52
	10th	2.061	0.117	2.49
	11th	2.283	0.145	2.46
	Penthouse/Roof	2.565	0.186	2.41

Figure 39:
Amplification factor for the shell element model

The Y-Direction was also checked for a new amplification factor, but due to the center of mass and center of rigidity lining up in the Y-Direction, the results obtained were less than 1.0. Therefore, the moments in the Y-Direction have an A_X of 1.0 (i.e. very little torsional issues in the Y-Direction). Once the new moments were obtained, both of the models were then checked for the torsional irregularities listed in Table 12.3-1 in ASCE 7-05. Type 1a and 1b were examined and they state that a torsional irregularity exists when the maximum story drift at one end of the structure is greater than 1.2 (Type 1a) or greater than 1.4 (Type 1b) times the average of the story drifts at the two ends of the structure. For both of the models, Extreme Torsional Irregularity (Type 1b) was found for every level of the building in the X-Direction with no torsional irregularity in the Y-Direction. The calculations for determining the irregularity in the X-Direction can be seen below in Figure 40 (Rigid Diaphragm Model) and Figure 41 (Shell Element Model).

Horizontal Irregularities (RD Model)					
		Torsional Irregularity			Horizontal Irregularity
		Δ_{max}	Δ_{min}	$\Delta_{max}/\Delta_{avg}$	
Case 1	1.0E_X + M_{ZX}	0	0	N/A	Torsional Irregularity 1b. Extreme Torsional Irregularity
		0.29	-0.06	2.48	
		1.15	-0.07	2.14	
		0.55	-0.06	2.24	
		1.17	-0.07	2.13	
		1.48	-0.14	2.20	
		2.02	-0.15	2.17	
		1.77	-0.12	2.15	
		1.84	-0.11	2.13	
		1.88	-0.10	2.11	
		1.89	-0.08	2.09	
		1.89	-0.07	2.08	
		2.55	-0.08	2.07	

Figure 40:
Torsional Irregularity calculations for rigid diaphragm model

Horizontal Irregularities (ShellModel)					
		Torsional Irregularity			Horizontal Irregularity
		Δ_{max}	Δ_{min}	$\Delta_{max}/\Delta_{avg}$	
Case 1	1.0E_X + M_{ZX}	0	0	N/A	Torsional Irregularity 1b. Extreme Torsional Irregularity
		0.17	-0.03	2.44	
		0.71	-0.06	2.18	
		0.30	-0.05	2.39	
		0.64	-0.05	2.18	
		0.75	-0.07	2.21	
		1.00	-0.09	2.19	
		0.83	-0.06	2.16	
		0.82	-0.05	2.12	
		0.79	-0.03	2.09	
		0.76	-0.01	2.04	
		0.72	0.00	2.01	
		0.91	-0.01	2.01	

Figure 41:
Torsional Irregularity calculations for shell element model

Upon verifying the horizontal irregularity, the periods for the structure were found for each model and compared (Figure 42). One note of interest is that when the two-way concrete flat slab is modeled as a shell element with the correct stiffness, the period decreases by almost a full second. Using the estimation of N/20 (for reinforced concrete shear walls) to approximate the period of the structure, the period obtained from both of the ETABS model is well above this value (13/65 = 0.65 sec). This can be contributed to the lack of a lateral system in the X-Direction at the southern end of the structure. Because of this, the southern end of the building is extremely flexible in this direction which causes the period to be higher than expected.

Modal Information		
	Rigid Diaphragm Model	Shell Element Model
T_x	2.943	2.081
T_y	2.112	1.595
T_z	1.729	1.413

Figure 42:
Modal information for the rigid diaphragm model and the shell element model

Comparison of Results and Hand Calculations:

Upon completing the models and verifying their accuracy, the controlling load combinations were found from §2.3.2 of ASCE 7-05:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. **1.2D + 1.6W + L + 0.5(Lr or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

The controlling load case for wind: 1.2D + 1.6W + L + 0.5(Lr or S or R)
The controlling load case for seismic: 1.2D + 1.0E + L + 0.2S

The wind forces are multiplied by 1.6 per Case 4 while the seismic loads receive a 1.0 factor due to Case 5. The controlling lateral load for each floor can be seen in Figure 43 with the majority of the lower floors being controlled by wind, while the upper floors are mainly controlled by the seismic lateral loads. This can be attributed to the relationship of height and mass for the earthquake loads. The main factor in determining which lateral load controls is based off of direct shear. Because the structure has extreme torsional rigidity in the X-Direction, torsional shears may change which load case controls each individual lateral element. A further investigation of torsional shears would be needed to confirm the controlling load case.

Controlling Loads			
	Level	Seismic	Wind
X-Direction	Ground	0.00	35.72
	1st	3.86	85.06
	2nd	7.51	91.56
	3rd	21.43	78.51
	4th	24.99	73.16
	5th	37.68	85.66
	6th	48.75	88.22
	7th	59.33	79.98
	8th	71.80	81.81
	9th	85.05	83.25
	10th	99.03	84.62
	11th	115.80	101.47
	Penthouse/Roof	171.93	58.83
Y-Direction	Ground	0.00	12.43
	1st	3.86	29.93
	2nd	7.51	32.70
	3rd	21.43	28.99
	4th	24.99	27.89
	5th	37.68	32.93
	6th	48.75	34.12
	7th	59.33	31.09
	8th	71.80	31.93
	9th	85.05	32.60
	10th	99.03	33.25
	11th	115.80	40.03
	Penthouse/Roof	171.93	23.25

Figure 43:
Controlling load cases broken up by floor (numbers in red indicate the lateral load that is controlled)

Next, displacements and interstory drifts were computed using the serviceability load combinations mentioned in the wind and earthquake sections respectively. Relative displacements and drifts as found in ETABS for both the rigid diaphragm model and the shell element model produced differing results. All of the wind drifts met the standard rule of thumb of H/400 for each model, where H is the story height. For the earthquake forces, the displacements had to be modified by a factor of C_d/I. For this structure, C_d is 4 and I is 1.5. The relative displacement allowed by ASCE 7-05 is $0.01h_{sx.}$ Sample drift calculations for a seismic and wind load in each direction can be found in Figure 44 and Figure 45 respectively. It should be noted that for the rigid diaphragm, many of the upper floors did not meet the maximum drift allowed by code for seismic. This again occurred in the weak spot (southern end) of the lateral force resisting system. Using the more accurate shell element model, the relative drifts were much smaller and passed the seismic code. It is of note that this is not an indication of failure since the drift calculations are part of the serviceability checks but could have a significant impact on the precast concrete panel façade.

Rigid Diaphragm Model												
Earthquake Serviceability					Displacements				Story Drifts		Allowable Story Drift ($\Delta_a = 0.010h_{sx}$)	
		Story Level	E (k)	M (ft-k)	δ_{XE}	δ_{YE}	$(C_d\delta_{XE})/I$	$(C_d\delta_{YE})/I$	Δ_X	Δ_Y	$\Delta_{a\ X/Y}$	$\Delta > \Delta_a$
Case 1	1.0Ex + Mzx	Ground	0.00	0.00	0	0	0	0	0	0	0	Yes
		1st	3.86	133.61	0.110	-0.031	0.29	-0.08	0.29	-0.08	1.28	Yes
		2nd	7.51	241.76	0.542	-0.142	1.45	-0.38	1.15	-0.30	1.68	Yes
		3rd	21.43	659.41	0.750	-0.193	2.00	-0.51	0.55	-0.14	1.36	Yes
		4th	24.99	622.49	1.189	-0.302	3.17	-0.81	1.17	-0.29	1.36	Yes
		5th	37.68	953.45	1.745	-0.443	4.65	-1.18	1.48	-0.38	1.36	No
		6th	48.75	1234.88	2.504	-0.630	6.68	-1.68	2.02	-0.50	1.68	No
		7th	59.33	1501.96	3.168	-0.795	8.45	-2.12	1.77	-0.44	1.36	No
		8th	71.80	1812.35	3.860	-0.965	10.29	-2.57	1.84	-0.45	1.36	No
		9th	85.05	2137.23	4.564	-1.137	12.17	-3.03	1.88	-0.46	1.36	No
		10th	99.03	2475.95	5.274	-1.310	14.06	-3.49	1.89	-0.46	1.36	No
		11th	115.80	2880.57	5.983	-1.481	15.95	-3.95	1.89	-0.46	1.36	No
		Penthouse/Roof	171.93	4249.84	6.938	-1.708	18.50	-4.55	2.55	-0.61	1.88	No
Two-Way Slab System - Shell Model												
Earthquake Serviceability					Displacements				Story Drifts		Allowable Story Drift ($\Delta_a = 0.010h_{sx}$)	
		Story Level	E (k)	M (ft-k)	δ_{XE}	δ_{YE}	$(C_d\delta_{XE})/I$	$(C_d\delta_{YE})/I$	Δ_X	Δ_Y	$\Delta_{a\ X/Y}$	$\Delta > \Delta_a$
Case 1	1.0Ex + Mzx	Ground	0.00	0.00	0	0	0	0	0	0.00	0	Yes
		1st	3.86	132.32	0.065	0.017	0.17	0.05	0.17	0.05	1.28	Yes
		2nd	7.51	239.18	0.332	0.082	0.88	0.22	0.71	0.17	1.68	Yes
		3rd	21.43	659.57	0.446	0.113	1.19	0.30	0.30	0.08	1.36	Yes
		4th	24.99	622.61	0.688	0.172	1.83	0.46	0.64	0.16	1.36	Yes
		5th	37.68	938.44	0.969	0.242	2.59	0.65	0.75	0.19	1.36	Yes
		6th	48.75	1207.66	1.346	0.338	3.59	0.90	1.00	0.26	1.68	Yes
		7th	59.33	1458.11	1.655	0.415	4.41	1.11	0.83	0.20	1.36	Yes
		8th	71.80	1746.41	1.962	0.489	5.23	1.30	0.82	0.20	1.36	Yes
		9th	85.05	2044.03	2.259	0.560	6.02	1.49	0.79	0.19	1.36	Yes
		10th	99.03	2349.78	2.542	0.627	6.78	1.67	0.76	0.18	1.36	Yes
		11th	115.80	2712.73	2.812	0.689	7.50	1.84	0.72	0.17	1.36	Yes
		Penthouse/Roof	171.93	3958.88	3.153	0.768	8.41	2.05	0.91	0.21	1.88	Yes

Figure 44:

Sample displacement and drift calculations for seismic loading (East-West Direction)

Rigid Diaphragm										
Wind Serviceability					Displacements/Story Drifts				Allowable Story Drift (Δ_a = L/400)	
		Story Level	$P_W + P_L$ (k)	M (ft-k)	δ_X	δ_Y	Δ_X	Δ_Y	Δ_a	$\Delta > \Delta_a$
Case 2	$0.75R_{WX} + 0.75R_X + M_T$ (+ex)	Ground	17.50	0.00	0	0	0	0	0	Yes
		1st	39.87	1381.50	0.057	-0.017	0.057	0.017	0.32	Yes
		2nd	42.92	1487.26	0.261	-0.071	0.204	0.055	0.42	Yes
		3rd	36.80	1275.21	0.358	-0.095	0.096	0.024	0.34	Yes
		4th	34.29	980.69	0.552	-0.145	0.195	0.050	0.34	Yes
		5th	40.16	1148.43	0.791	-0.207	0.239	0.062	0.34	Yes
		6th	41.36	1182.75	1.106	-0.287	0.316	0.079	0.42	Yes
		7th	37.49	1072.29	1.375	-0.355	0.268	0.068	0.34	Yes
		8th	38.35	1096.74	1.648	-0.423	0.273	0.068	0.34	Yes
		9th	39.02	1116.04	1.921	-0.490	0.273	0.068	0.34	Yes
		10th	39.67	1134.49	2.191	-0.557	0.270	0.066	0.34	Yes
		11th	47.57	1360.36	2.458	-0.622	0.267	0.065	0.34	Yes
		Roof	27.58	788.72	2.813	-0.707	0.356	0.085	0.47	Yes
	$0.75R_{WY} + 0.75R_Y + M_T$ (+ey)	Ground	5.83	0.00	0	0	0	0	0	Yes
		1st	14.03	220.89	0.004	0.007	0.004	0.007	0.32	Yes
		2nd	15.33	241.45	0.016	0.031	0.013	0.025	0.42	Yes
		3rd	13.59	214.04	0.021	0.043	0.004	0.012	0.34	Yes
		4th	13.07	176.48	0.032	0.067	0.011	0.024	0.34	Yes
		5th	15.44	208.37	0.045	0.096	0.014	0.028	0.34	Yes
		6th	15.99	215.87	0.063	0.133	0.018	0.037	0.42	Yes
		7th	14.57	196.73	0.078	0.165	0.015	0.032	0.34	Yes
		8th	14.97	202.10	0.093	0.198	0.015	0.033	0.34	Yes
		9th	15.29	206.35	0.108	0.232	0.015	0.033	0.34	Yes
		10th	15.59	210.40	0.122	0.265	0.014	0.033	0.34	Yes
		11th	18.77	253.33	0.136	0.298	0.014	0.033	0.34	Yes
		Roof	10.90	147.12	0.154	0.342	0.018	0.044	0.47	Yes
	$0.75R_{WX} + 0.75R_X + M_T$ (-ex)	Ground	17.50	0.00	0	0	0	0	0	Yes
		1st	39.87	-1381.50	0.012	0.001	0.012	0.001	0.32	Yes
		2nd	42.92	-1487.26	0.069	0.001	0.057	0.000	0.42	Yes
		3rd	36.80	-1275.21	0.098	0.001	0.029	0.000	0.34	Yes
		4th	34.29	-980.69	0.160	-0.002	0.063	0.002	0.34	Yes
		5th	40.16	-1148.43	0.240	-0.007	0.079	0.005	0.34	Yes
		6th	41.36	-1182.75	0.348	-0.014	0.108	0.007	0.42	Yes
		7th	37.49	-1072.29	0.444	-0.021	0.097	0.008	0.34	Yes
		8th	38.35	-1096.74	0.546	-0.030	0.101	0.009	0.34	Yes
		9th	39.02	-1116.04	0.649	-0.039	0.104	0.009	0.34	Yes
		10th	39.67	-1134.49	0.755	-0.049	0.106	0.010	0.34	Yes
		11th	47.57	-1360.36	0.862	-0.059	0.106	0.010	0.34	Yes
		Roof	27.58	-788.72	1.006	-0.073	0.144	0.014	0.47	Yes
	$0.75R_{WY} + 0.75R_Y + M_T$ (-ey)	Ground	5.83	0.00	0	0	0	0	0	Yes
		1st	14.03	-220.89	-0.005	0.007	0.005	0.007	0.32	Yes
		2nd	15.33	-241.45	-0.020	0.033	0.015	0.026	0.42	Yes
		3rd	13.59	-214.04	-0.027	0.046	0.007	0.013	0.34	Yes
		4th	13.07	-176.48	-0.040	0.071	0.013	0.025	0.34	Yes
		5th	15.44	-208.37	-0.055	0.100	0.016	0.029	0.34	Yes
		6th	15.99	-215.87	-0.076	0.138	0.020	0.038	0.42	Yes
		7th	14.57	-196.73	-0.092	0.171	0.017	0.033	0.34	Yes
		8th	14.97	-202.10	-0.109	0.205	0.017	0.034	0.34	Yes
		9th	15.29	-206.35	-0.126	0.239	0.017	0.034	0.34	Yes
		10th	15.59	-210.40	-0.142	0.274	0.016	0.034	0.34	Yes
		11th	18.77	-253.33	-0.158	0.308	0.016	0.034	0.34	Yes
		Roof	10.90	-147.12	-0.179	0.353	0.021	0.045	0.47	Yes

Figure 45:

Sample displacement and interstory drift calculation (Wind)

To finalize the structural analysis of the building, the overturning moment was checked against the resisting moment. The full calculations for the overturning moments for wind and seismic can be found in Appendix A and B respectively. It was found that the seismic overturning moment controlled the structure with a value of 80,500 ft-k. To determine the adequacy of the structure, the resisting moment was calculated using the weight of the building previously determined in the seismic section. Multiplying the weight by half of the least dimension of the building (moment arm) produced a resisting moment of 1,756,200 ft-k. The resisting moment was then checked with a factor of safety to assure that 2/3(M_R) was greater than M_o. Even with the additional factor of safety, the resisting moment capacity still exceeded the overturning moment by a significant portion. A further in depth investigation of the foundation will have to be performed in order to determine any areas of concern. However, at his stage, the foundation appears to be adequate for the overturning moments.

Finally, spot checks were performed for certain members of the later resisting system in order to determine their strength adequacy and then compared to values obtained either from ETABS or by hand calculations. Using the hand calculated center of mass and center of rigidity for a typical floor, the shears were split into direct and torsional shear components. Using the seismic story force for that level (the controlling load), the force was distributed to the shear walls acting in the direction of interest. The direct shears were calculated using the relative stiffness obtained using the ETABS model. Once the direct shears were distributed, the torsional shears were calculated (see Appendix C). Due to the large eccentricity that occurs with a loading in the X-Direction, the torsional shears for the seismic load in this direction produced relatively large numbers. After performing the calculations, it was discovered that Shear Wall 4 controlled in the X-Direction. Performing similar calculations in the Y-Direction produced slightly different values. Because of the small eccentricity associated with Y-Direction loading, the torsional shears for this case were non-substantial and did not affect the direct shears significantly. Shear Wall 1 controlled the design for Y-Direction loading.

To check the adequacy of the shear walls, the worst case wall for the 9th floor was chosen which turned out to be Shear Wall 4 with a total shear value of 48.5 k. All shear walls are provided with basic reinforcing of #4 rebar at 12" on center in each face, each way. An interaction diagram was constructed for this wall and the shear and moment calculated by hand was then plotted to check the acceptability of this wall.

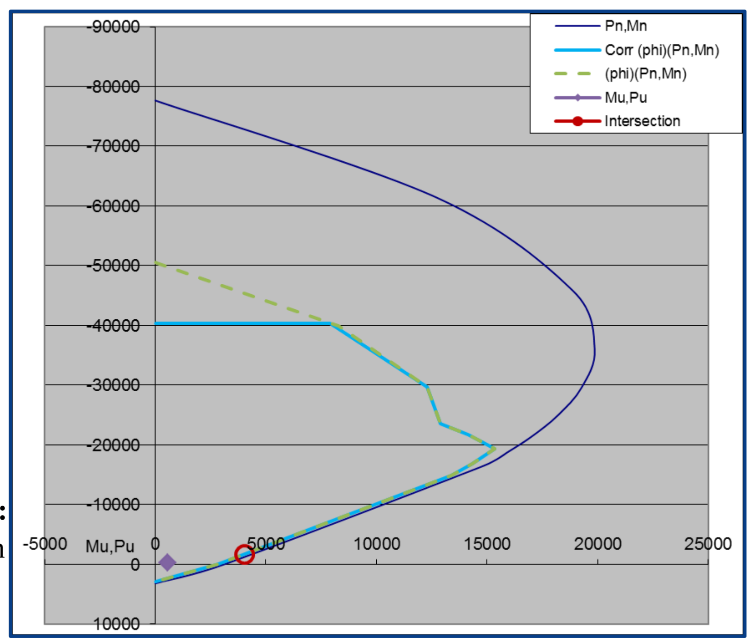


Figure 46: Interaction Diagram for SW4

As can be seen in Figure 46, the point is plotted and falls within the interaction diagram. Therefore the shear wall passes for both the axial load that accumulates traveling down the building as well as the moment that is caused from the lateral load applied to that floor. A spreadsheet containing all of the data and calculation for Shear Wall 4 can be found in Appendix C.

Finally, a column participating in the lateral system was analyzed to check the adequacy for this member to take both the gravity loads associated with the dead and live load of the structure as well as the lateral force from the seismic loading. The column chosen (G-2) is part of Frame 9 with (4) #11's with #4 ties, and the check was done for the 9th floor (8th floor column supporting the 9th floor). An interaction diagram was produced by hand by simply calculating the three main points to the diagram: the Pure Axial Strength, Pure Bending Strength, and Pure Tension points. Once the diagram was drawn, the axial load for this column was calculated using the controlling load combination previously mentioned in the Seismic section. The moment for column G-2 was taken out of ETABS due to the complexity in trying to solve for this value by hand. With these values, the point was plotted on the hand drawn interaction diagram and passed by a significant portion. Because this is a column supporting the 9th floor, the axial load is not as high as would be found with a basement column. However, in technical report 1, the basement column was found to pass the pure axial strength. In order to obtain a more accurate interaction diagram, the column was inserted into spColumn which produced the interaction diagram seen in Figure 47 (following page). The point was plotted and found to be adequate for both the axial and lateral loads associated with the 9th floor.

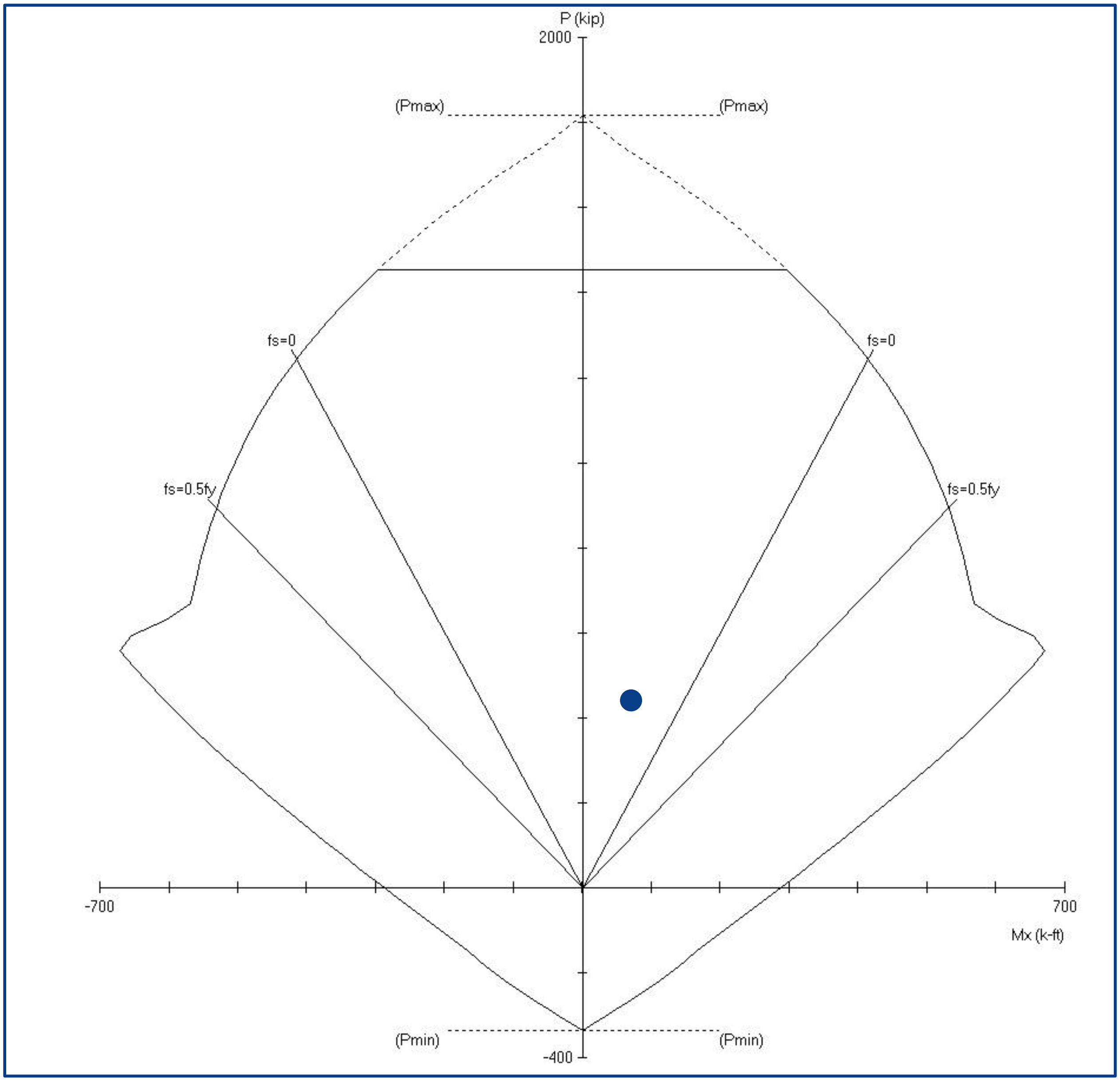


Figure 47:
Interaction Diagram for column G-2

Conclusion:

Upon thorough analysis, the lateral system of the South Patient Tower (SPT) was found to be sufficient to carry both the seismic and wind forces the structure is likely to experience throughout the building lifetime.

After factoring the seismic and wind loads according to the controlling load combinations, it was determined that the lower floors are controlled by wind loads while the upper levels are controlled by the seismic forces (due to mass and height relationship). Although certain members may be controlled by different combinations on each floor (due to torsional shears), the controlling load cases were determined strictly by direct shear and torsional shears were not evaluated for every member in this report and should be investigated in future reports. For overall base shear, seismic controlled in the North-South Direction by a factor of 3, while factored wind loads controlled the East-West Direction. In both directions, the seismic controlled the overturning moment.

Two models were built to fully encompass the structural behavior of the building. The first structural model consisted of rigid diaphragms. The second model constructed comprised shell elements for the floor system and was found to have a slightly higher stiffness than the rigid diaphragm model due to the increased rigidity from the two-way concrete slab system. The modal information for the two models produced an extremely different period in the X-Direction. The period for the shell element model decreased by 0.862 sec in the X-Direction, 0.517 sec in the Y-Direction, and 0.316 sec for the torsional period (Z).

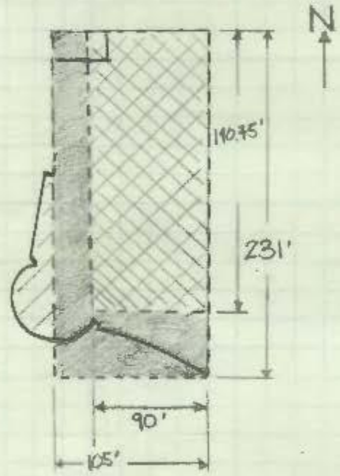
Displacements and drifts were calculated using both models with surprising conclusions. Both the rigid diaphragm model and the shell element model passed the standard rule of thumb for both the overall displacement of the building as well as interstory drifts for wind loads. However, the rigid diaphragm model did not meet ASCE 7-05 for seismic drifts. The drift produced by the rigid diaphragm model was roughly 2.5 in. while code limited the drift to 1.88 in. The weak area for the SPT is the southern end of the building located at the opposite end of the connection to the existing structure. The displacements near the connection where most of the rigidity is located produced values ten times less than the southern end. Using the more accurate shell element model, the drifts met code and were not of concern. In both models, seismic drifts controlled the overall structure. Upon further calculations, the horizontal plan experiences extreme torsional irregularities in the X-Direction.

After performing the spot checks for both the shear wall and column, it was determined by plotting the axial load and moments on the interaction diagrams that both of these members are sufficient and adequate to carry both the axial loads as well as the lateral forces associated with wind and seismic.

Appendix A: Wind Load Calculations

NATHAN MCGRAW | WIND LOAD CALCS (TECH 3) | PAGE 1 OF 7

SIMPLIFYING ASSUMPTIONS:



N-S DIRECTION WIND:

$0 \rightarrow 36.17'$
L = 231'
B = 105'

$36.17' \rightarrow 175'$
L = 190.75'
B = 90'

E-W DIRECTION WIND:

$0 \rightarrow 36.17'$
L = 105'
B = 231'

$36.17' \rightarrow 175'$
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:

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

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

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

EAST-WEST DIRECTION:

Shear Wall 3:

$h_i = 174'-4''$
$D_i = 25'$
$A_i = (174'-4'')(25') = 4358.25$

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

Shear Wall 5 & 6:

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

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

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

$$n_1 = \frac{385(1.2)^{0.5}}{174'-4''} = 2.43 > 1.0 \rightarrow \text{RIGID STRUCTURE}$$

NORTH-SOUTH DIRECTION

Shear Wall 1 and 2:

$h_i = 174'-4''$
$D_i = 30.75'$
$A_i = (174'-4'')(30.75') = 5360.75$

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

Shear Wall 4 and 7:

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

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

NATHAN MCGRAW | WIND LOAD CALCS (TECH 3) | PAGE 3 OF 7

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

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

$\therefore$ GUST EFFECT FACTOR MAY BE TAKEN AS 0.85 (§6.5.8.1)

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

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

EXTERNAL PRESSURE COEFFICIENTS:

WALLS:

WINDWARD $\rightarrow C_p = 0.8$

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 $\rightarrow C_p = -0.7$

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 = $(190.75')(90') = 17167.5$ ft² $\rightarrow$ Reduction factor = 0.8

DESIGN WIND PRESSURES:

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

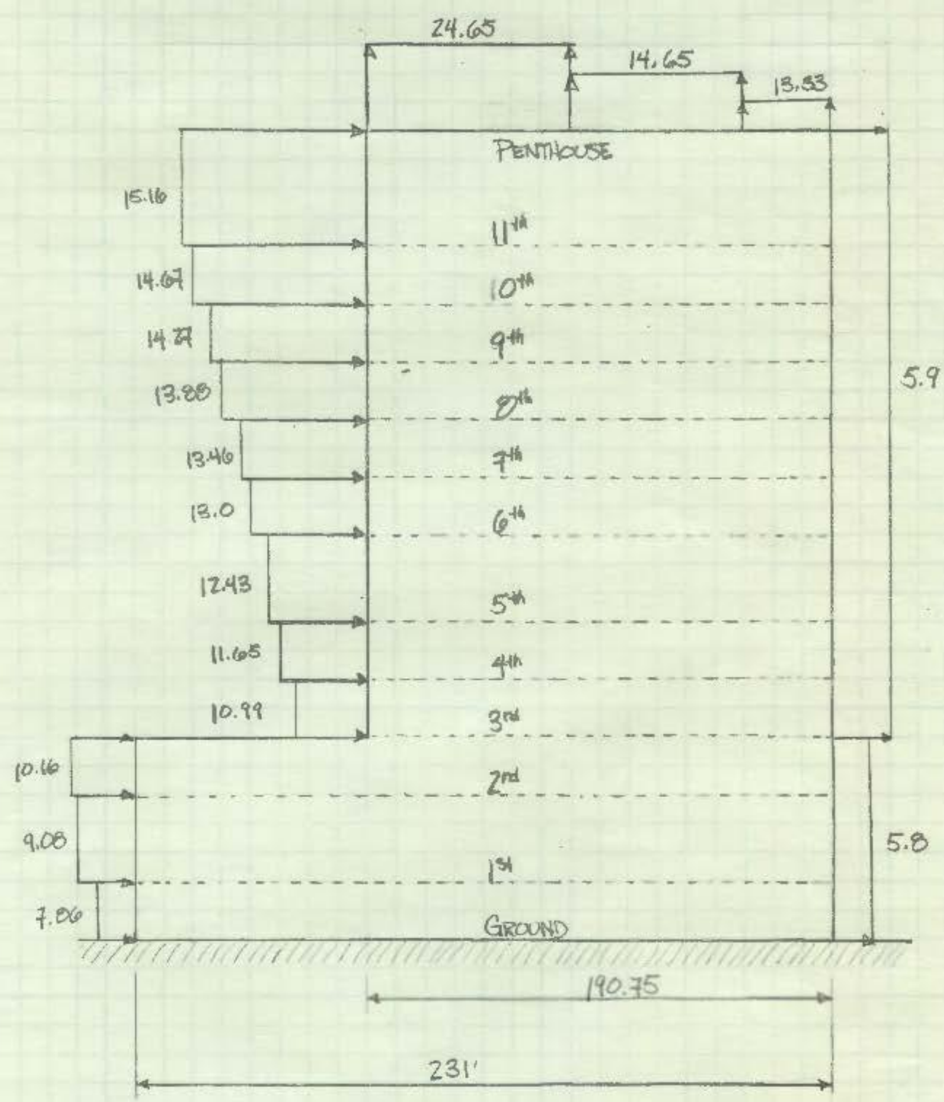
Leeward Walls, Side Walls, Roof: $p_h = q_h(GC_p - GC_{pi})$

SAMPLE CALC:

$p_z = (11.55)(0.85)(0.8) - (23.51)(\pm 0.18) = 7.86$ psf ± 4.23 psf

NATHAN McGRAW WIND LOAD CALCS (TECH 3) PAGE 4 OF 7

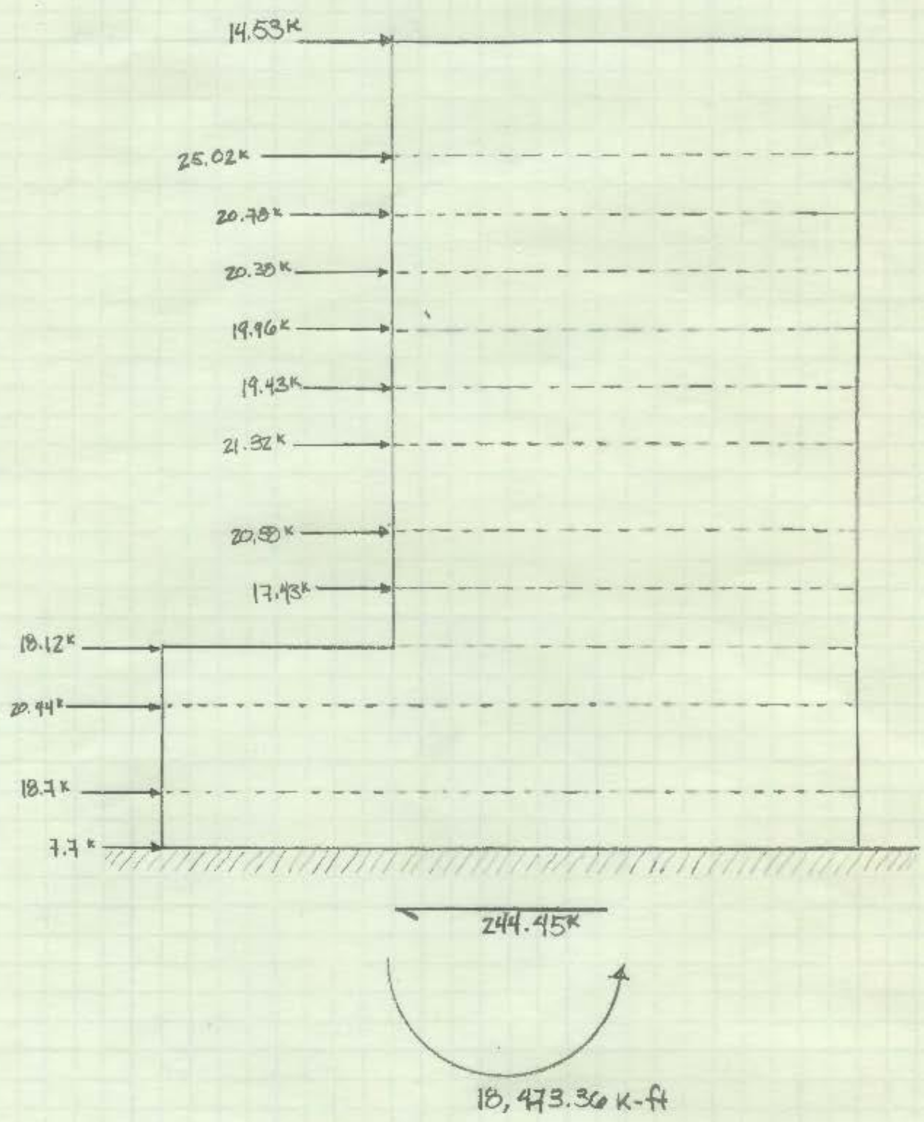
DIAGRAM OF N-S WIND PRESSURES:



* ALL WIND VALUES ARE IN PSF

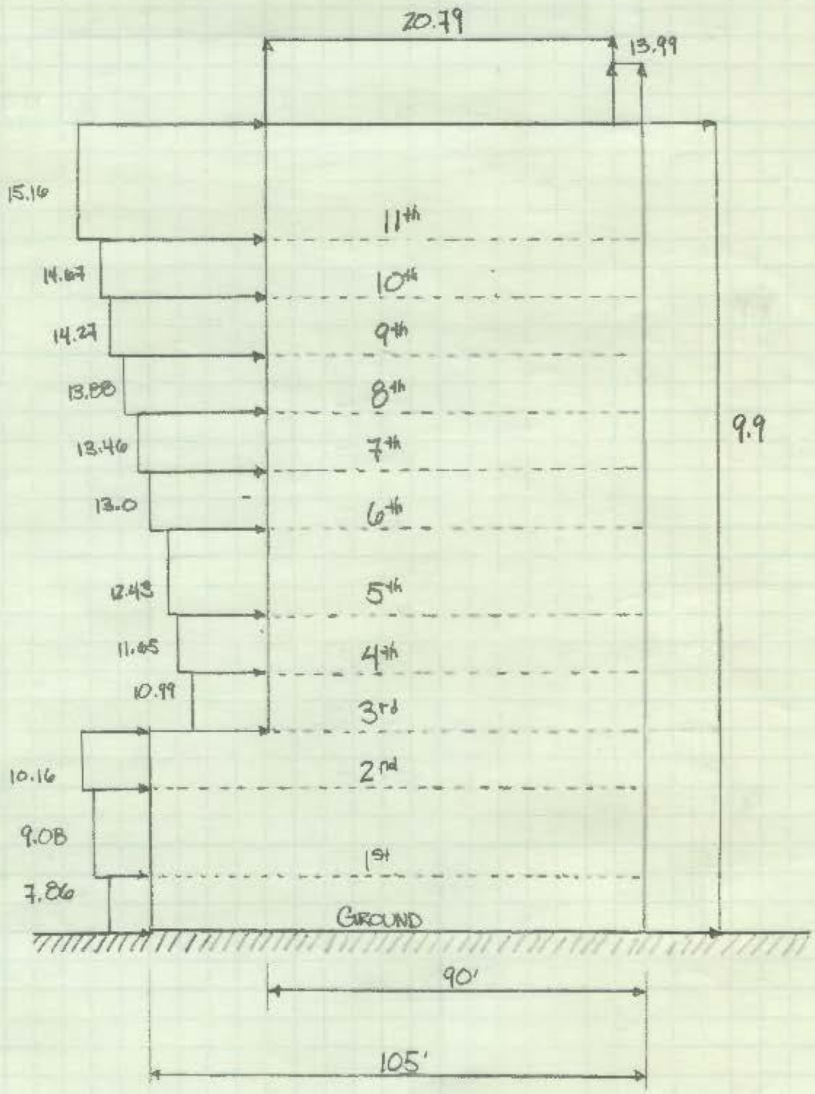
NATHAN McGRAW | WIND LOAD CALCS (TECH 3) | PAGE 5 OF 7

DIAGRAM OF N-S WIND FORCES:



NATHAN MCGRAW | WIND LOAD CALCS (TECH 3) | PAGE 6 OF 7

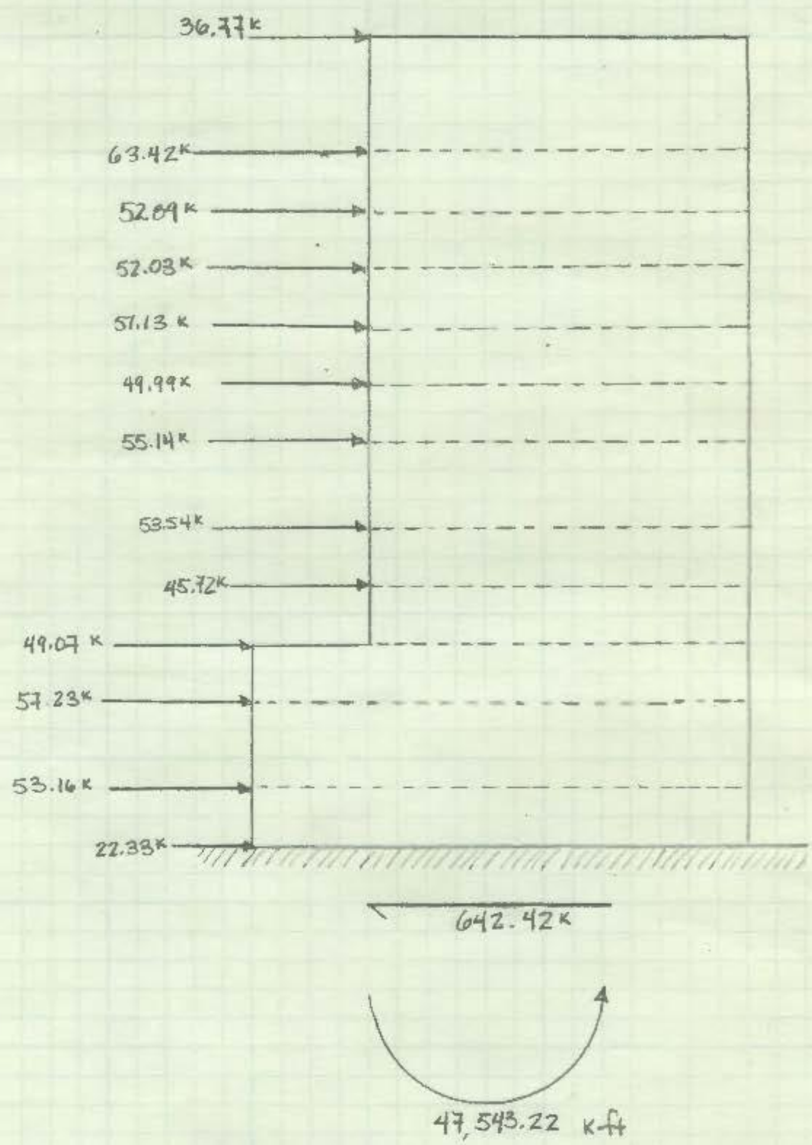
DIAGRAM OF E-W WIND PRESSURES:



* ALL WIND VALUES ARE IN PSF

NATHAN MCGRAW | WIND LOAD CALCS (TECH 3) | 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

Wind Forces N-S Direction

Floor Level	Elevation (ft)	Tributary Below Height (ft)	Tributary Below Area (ft^2)	Tributary Above Height (ft)	Tributary Above Area (ft^2)	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	Factored Forces (k)
Ground	0.00	N/A	0.00	5.42	568.58	7.77	244.45	0.00	12.43
1st	10.83	5.42	568.58	7.00	735.00	18.70	236.68	202.56	29.93
2nd	24.83	7.00	735.00	5.67	595.35	20.44	217.98	507.49	32.70
3rd	36.17	5.67	595.35	5.67	510.00	18.12	197.54	655.24	28.99
4th	47.50	5.67	510.00	5.58	502.50	17.43	179.42	828.11	27.89
5th	58.67	5.58	502.50	7.13	641.70	20.58	161.99	1207.50	32.93
6th	72.93	7.13	641.70	5.62	505.80	21.32	141.41	1555.01	34.12
7th	84.17	5.62	505.80	5.67	509.85	19.43	120.09	1635.45	31.09
8th	95.50	5.67	509.85	5.67	509.85	19.96	100.66	1905.75	31.93
9th	106.83	5.67	509.85	5.67	510.30	20.38	80.70	2176.94	32.60
10th	118.17	5.67	510.30	5.67	509.85	20.78	60.32	2455.62	33.25
11th	129.50	5.67	509.85	7.67	689.85	25.02	39.54	3239.55	40.03
Roof	144.83	7.67	689.85	N/A	0.00	14.53	14.53	2104.13	23.25
					Total Base Shear =	244.45			391.2 k
							Total Overturning Moment =	18,473.36 k-ft	29,557.38 k-ft

Wind Forces E-W Direction

Floor Level	Elevation (ft)	Tributary Below Height (ft)	Tributary Below Area (ft^2)	Tributary Above Height (ft)	Tributary Above Area (ft^2)	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	Factored Forces (k)
Ground	0.00	N/A	0.00	5.42	1250.87	22.33	642.42	0.00	35.72
1st	10.83	5.42	1250.87	7.00	1617.00	53.16	620.09	575.77	85.06
2nd	24.83	7.00	1617.00	5.67	1309.77	57.23	566.93	1420.97	91.56
3rd	36.17	5.67	1309.77	5.67	1080.92	49.07	509.70	1774.84	78.51
4th	47.50	5.67	1080.92	5.58	1065.02	45.72	460.63	2172.07	73.16
5th	58.67	5.58	1065.02	7.13	1360.05	53.54	414.91	3141.15	85.66
6th	72.93	7.13	1360.05	5.62	1072.02	55.14	361.37	4021.21	88.22
7th	84.17	5.62	1072.02	5.67	1080.60	49.99	306.23	4207.29	79.98
8th	95.50	5.67	1080.60	5.67	1080.60	51.13	256.24	4883.29	81.81
9th	106.83	5.67	1080.60	5.67	1081.55	52.03	205.11	5558.62	83.25
10th	118.17	5.67	1081.55	5.67	1080.60	52.89	153.08	6249.54	84.62
11th	129.50	5.67	1080.60	7.67	1462.10	63.42	100.19	8212.81	101.47
Roof	144.83	7.67	1462.10	N/A	0.00	36.77	36.77	5325.66	58.83
					Total Base Shear =	642.42			1027.87 k
							Total Overturning Moment =	47,543.22 k-ft	76,069.15 k-ft

Appendix B: Seismic Calculations

NATHAN McGRAW | SEISMIC LOAD CALCS (TECH 3) | PAGE 1 OF 5

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):

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{D1} = \frac{2}{3} S_{M1}$

ADJUSTED MAXIMUM CONSIDERED EQ (§ 11.4-3):

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

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

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

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

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

$R = 4\frac{1}{2}$ → Shear Wall-Frame Interactive System with Ordinary Reinforced Concrete Moment Frames and Ordinary Reinforced Concrete Shear Walls

$C_d = 4.0$ → USED TO GET CODE DISPLACEMENTS FROM ELASTIC MODEL

NATHAN McGRAW | SEISMIC LOAD CALCS (TECH 3) | 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"

$T_a = (0.02)(145')^{0.75}$

$T_a = 0.8357$ sec. $C_u = 1.7$ (TABLE 12.8-1)

SEISMIC RESPONSE COEFFICIENT : § 12.8.1.1

$T_L = 8$ sec. (FIGURE 22-15)

$T = C_u \cdot T_a$

$T = 1.7(0.8357) = 1.4207$

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

$$C_s = \min \begin{cases} \dfrac{(0.1643)}{(4.5/1.5)} = 0.0548 \\ \dfrac{(0.0816)}{(1.4207)(4.5/1.5)} = 0.019145 \\ \dfrac{(0.0816)(8)}{(1.4207)^2(4.5/1.5)} = 0.1078 \end{cases}$$

$\Rightarrow C_s$ 0.019145 > 0.01 ∴ OK ✓

ETABS PERIOD:

RIGID DIAPHRAGM MODEL:

$T_x = 2.943$ SEC

$T_y = 2.112$ SEC

$T_z = 1.729$ SEC

TWO-WAY SLAB SYSTEM MODELED AS SHELL ELEMENT:

$T_x = 2.0813$ SEC

$T_y = 1.5947$ SEC

$T_z = 1.4134$ SEC

* BECAUSE T_x AND T_y ARE BOTH GREATER THAN THE UPPER LIMIT $(C_u T_a)$, T OF 1.4207 SEC WAS USED TO CALCULATE THE FORCES IN BOTH THE X AND Y DIRECTIONS.

NATHAN MCGRAW | SEISMIC LOAD CALCS (TECH 3) | PAGE 3 OF 5

WEIGHT CALCULATIONS:

FACADE:

5 1/2" Concrete + 1/2" Thin Brick Face = (6"/12")(150 lb/ft³) = 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/ft³) = 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 1/2"/12")(150 lb/ft³) = 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/ft³)(6"/12") = 0.9 psf

Roof Pavers = 25 psf

VEGATED SYSTEM = 30 psf

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

* EXCEL CONTAINS TOTAL BUILDING WEIGHT WITH FLOOR BY FLOOR BREAKDOWN

AMPAD

NATHAN MCGRAW | SEISMIC LOAD CALCS (TECH 3) | PAGE 4 OF 5

BASE SHEAR:

$V = C_s W$

W = weight of building (Calculated in spreadsheet)

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

$V = (0.019145)(39{,}026.339^k)$

$\boxed{V = 747.16^k}$

(STRUCTURAL DRAWINGS = 700^k → 6.74% DIFFERENCE ✓

* DIFFENCE DUE TO DIFFERENT WEIGHT CALCS AND DIFFENT S_s AND S_1 VALUES

STORY FORCES:

$F_x = C_{vx} V$

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

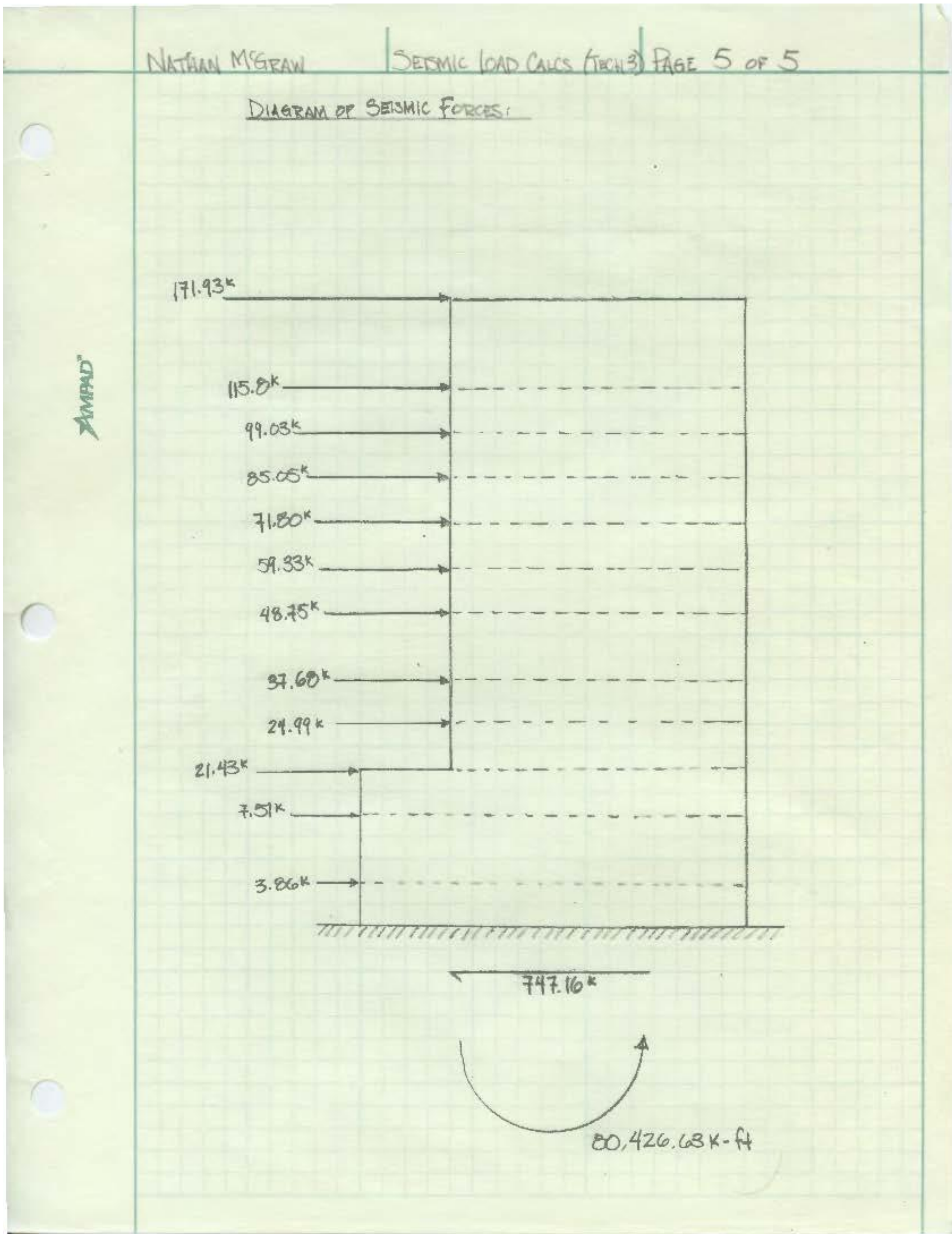
w = weight of each story

h = height of story above grade

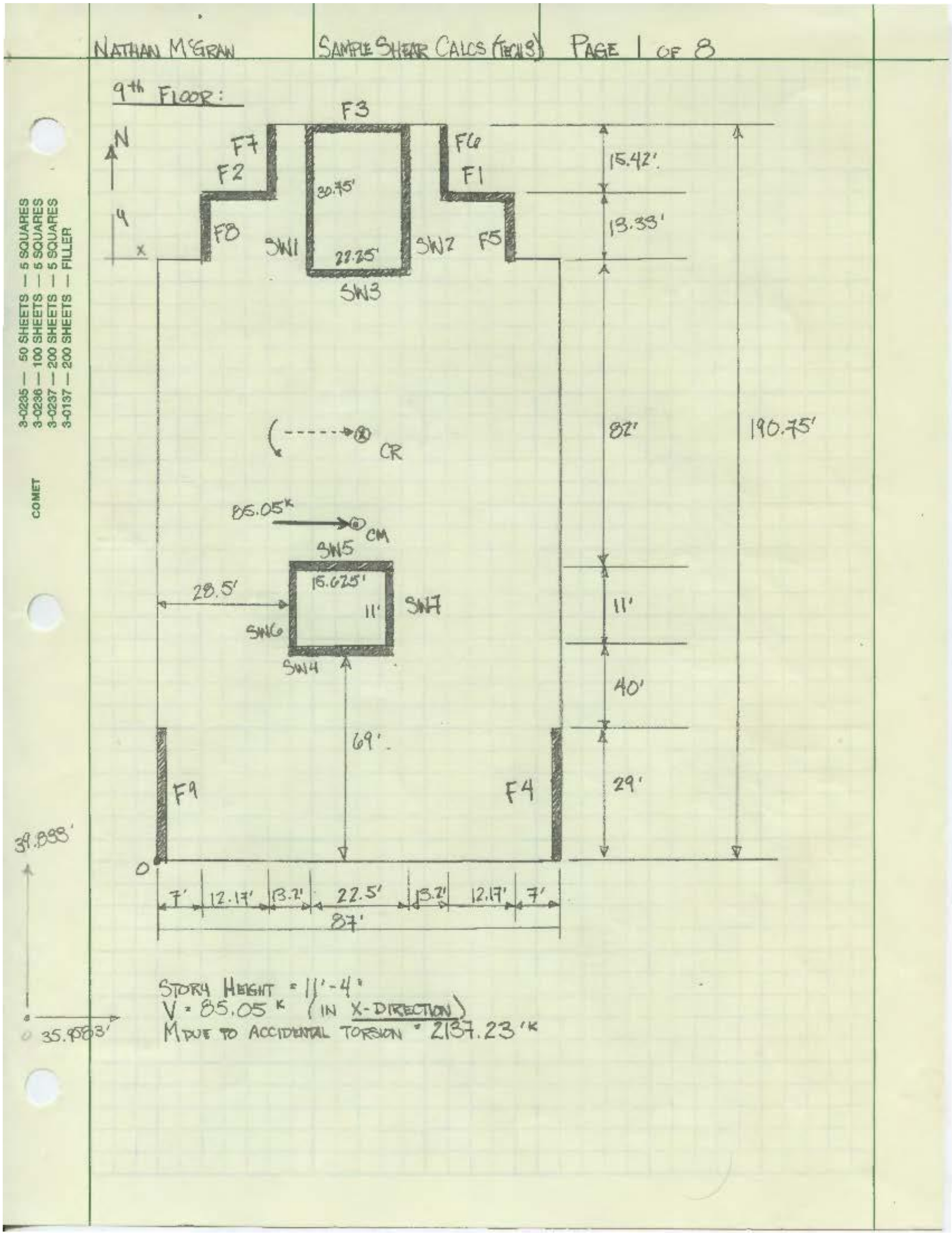
$K = 1 + \dfrac{\sqrt{T} - 0.5}{2}$ $(1 \le K \le 2)$

$K = 1 + \dfrac{1.4207 - 0.5}{2} = 1.5076$

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


NATHAN MCGRAW
SEISMIC LOAD CALCS (TECH 3) PAGE 5 OF 5
DIAGRAM OF SEISMIC FORCES:
171.93k
115.8k
99.03k
85.05k
71.80k
59.33k
48.75k
37.68k
24.99k
21.43k
7.51k
3.86k
747.16k
80,426.63k-ft

Appendix C: Spot Checks



NATHAN McGRAW | SAMPLE SHEAR CALCS (TECH 3) | PAGE 2 OF 8

CENTER OF MASS: (IGNORING FRAMES FOR EASE OF CALCULATION AND SYMMETRY)

SLAB:

IN X-DIRECTION = 87/2 = 43.5' (DUE TO SYMMETRY)

IN Y-DIRECTION:

$(162')(87')(9.5/12)(150) = 1673662.5$ @ $y = 81'$

$(73')(13.33')(9.5/12)(150) = 115554.4375$ @ $y = 168.665'$

$(48.9')(15.42')(9.5/12)(150) = 89542.0125$ @ $y = 183.04'$

WALLS:

SW1: $(30.75)(11.333')(12/12)(150) = 52273.4625$ @ $x = 32.37'$; $y = 175.375'$

SW2: 52273.4625 @ $x = 54.87'$; $y = 175.375'$

SW3: $(22.5')(11.333')(12/12)(150) = 38248.875$ @ $x = 43.62'$, $y = 160'$

SW4: $(15.625')(11.333')(12/12)(150) = 26561.71875$ @ $x = 36.325$, $y = 69'$

SW5: 26561.71875 @ $x = 36.325$; $y = 80'$

SW6: $(11')(11.333')(12/12)(150) = 18699.45$ @ $x = 28.5'$, $y = 74.5'$

SW7: 18699.45 @ $x = 44.125'$, $y = 74.5'$

X-DIRECTION: $\Sigma M = 2112077.088$

$$\bar{x} = \frac{[(1673662.5)(43.5) + (115554.4375)(43.5) + (89542.0125)(43.5) + (52273.4625)(32.37) + (52273.4625)(54.87') + (38248.875)(43.62) + (26561.71875)(36.325) + (26561.71875)(36.325) + (18699.45)(28.5) + (18699.45)(44.125)]}{2112077.088}$$

$\boxed{\bar{x} = 43.2'}$ (ETABS ⇒ 43.6')

$$\bar{y} = \frac{[(1673662.5)(81) + (115554.4375)(168.665) + (89542.0125)(183.04) + (52273.4625)(175.375) + (52273.4625)(175.375) + (38248.875)(160) + (26561.71875)(69) + (26561.71875)(80') + (18699.45)(74.5) + (18699.45)(74.5)]}{2112077.088}$$

$\boxed{\bar{y} = 95.95'}$ (ETABS ⇒ 93.9')

NATHAN MCGRAW | SAMPLE SHEAR CALCS (TECH 3) | PAGE 3 OF 8

CENTER OF RIGIDITY:

$R_{SW1} = 0.360$
$R_{SW2} = 0.358$
$R_{SW3} = 0.621$
$R_{SW4} = 0.188$
$R_{SW5} = 0.123$
$R_{SW6} = 0.044$
$R_{SW7} = 0.086$
$R_{F1} = 0.014$
$R_{F2} = 0.014$
$R_{F3} = 0.041$
$R_{F4} = 0.045$
$R_{F5} = 0.014$
$R_{F6} = 0.018$
$R_{F7} = 0.018$
$R_{F8} = 0.014$
$R_{F9} = 0.044$

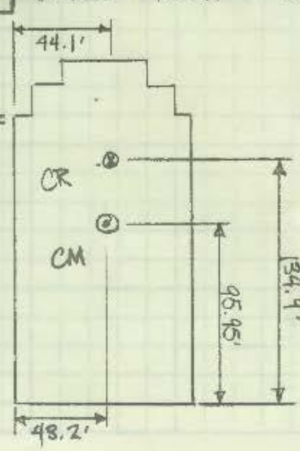
$$X_r = \frac{[R_{F4}(87') + R_{SW6}(28.5') + R_{SW7}(44.125') + R_{F8}(7') + R_{F7}(19.17') + R_{SW1}(32.37') + R_{SW2}(54.87') + R_{F6}(67.83') + R_{F5}(80')]}{R_{F9} + R_{F4} + R_{SW6} + R_{SW7} + R_{F8} + R_{F7} + R_{SW1} + R_{SW2} + R_{F6} + R_{F5}}$$

$\boxed{\bar{X} = 44.1'}$ (ETABS → 44.3')

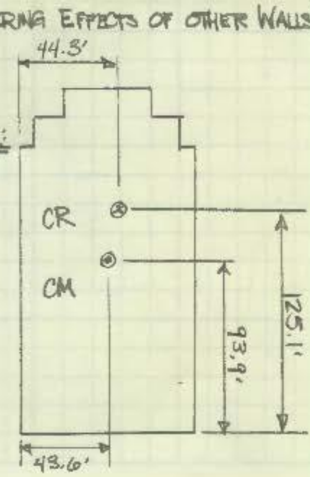
$$y_r = \frac{[R_{SW4}(69') + R_{SW5}(80) + R_{SW3}(160) + R_{F1}(175.33') + R_{F2}(175.33') + R_{F3}(190.75')]}{R_{SW4} + R_{SW5} + R_{SW3} + R_{F1} + R_{F2} + R_{F3}}$$

$\boxed{y_r = 134.9'}$ (ETABS → 125.1' - * ETABS CONSIDERING EFFECTS OF OTHER WALLS)

HAND CALCULATIONS:



ETABS MODEL:



NATHAN MCGRAW | SAMPLE SHEAR CALCS (TECH3) | PAGE 4 OF 8

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

TORSIONAL RIGIDITY:

$$J = \Sigma R_i d_i^2$$

$$J = R_{SW1}(11.73)^2 + R_{SW2}(10.53')^2 + R_{SW3}(25.1')^2 + R_{SW4}(65.9')^2 + R_{SW5}(54.9')^2 + R_{SW6}(15.6')^2 + R_{SW7}(0.025')^2 + R_{F1}(40.43')^2 + R_{F2}(40.43')^2 + R_{F3}(55.85')^2 + R_{F4}(42.9')^2 + R_{F5}(35.9')^2 + R_{F6}(23.73')^2 + R_{F7}(24.93')^2 + R_{F8}(37.1')^2 + R_{F9}(44.1')^2$$

$$J = 2079 \quad (K/IN)\,FT^2$$

DIRECT SHEAR:

$$V_{SW4} = 0.188(85.05^K) = 16.0^K \; (\leftarrow)$$
$$V_{SW5} = 0.123(85.05^K) = 10.5^K \; (\leftarrow)$$
$$V_{SW3} = 0.621(85.05^K) = 52.8^K \; (\leftarrow)$$
$$V_{F3} = 0.041(85.05^K) = 3.5^K \; (\leftarrow)$$
$$V_{F1} = 0.014(85.05^K) = 1.2^K \; (\leftarrow)$$
$$V_{F2} = 0.014(85.05^K) = 1.2^K \; (\leftarrow)$$

CHECK: 85.2^K (OFF DUE TO ROUNDING)

TORSIONAL SHEARS IN WALLS

$$V_{SW4} = \frac{[(85.05^K)(38.95) + 2137.23'K](65.9)(0.188)}{2079} = 32.5^K$$

$$V_{SW5} = \frac{[(85.05)(38.95) + 2137.23'K](54.9)(0.123)}{2079} = 17.7^K$$

$$V_{SW3} = \frac{[(85.05)(38.95) + 2137.23'K](25.1')(0.621)}{2079} = 40.9^K$$

$$V_{F3} = \frac{(5449.9275)(55.85)(0.041)}{2079} = 6.0^K$$

$$V_{F2} = \frac{(5449.9275)(40.43)(0.014)}{2079} = 1.5^K$$

$$V_{F1} = \frac{(5449.9275)(40.43)(0.014)}{2079} = 1.5^K$$

$$V_{SW1} = \frac{(5449.9275)(11.73)(0.360)}{2079} = 11.1^K$$

$$V_{SW2} = \frac{(5449.9275)(10.53)(0.358)}{2079} = 9.9^K$$

$$V_{SW6} = \frac{(5449.9275)(15.6')(0.044)}{2079} = 1.8^K$$

NATHAN MCGRAW SAMPLE SHEAR CALCS (TECH3) PAGE 5 OF 8

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

$$V_{SW7} = \frac{(5449.9275)(0.025')(0.018)}{2079} = 0^K$$

$$V_{F4} = \frac{(5449.9275)(42.9')(0.045)}{2079} = 5.1^K$$

$$V_{F5} = \frac{(5449.9275)(35.9')(0.014)}{2079} = 1.3^K$$

$$V_{F6} = \frac{(5449.9275)(23.73')(0.018)}{2079} = 1.1^K$$

$$V_{F7} = \frac{(5449.9275)(24.93')(0.018)}{2079} = 1.2^K$$

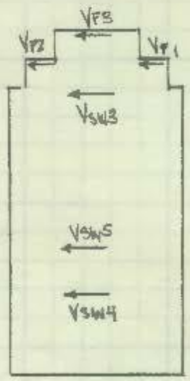
$$V_{F8} = \frac{(5449.9275)(37.1')(0.014)}{2079} = 1.4^K$$

$$V_{F9} = \frac{(5449.9275)(44.1')(0.044)}{2099} = 5.1^K$$

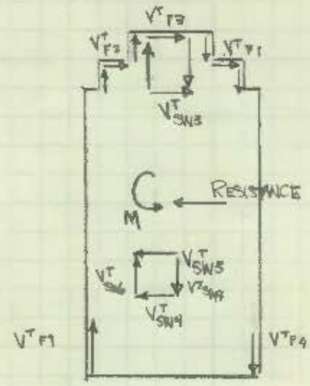
3-0236 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

NATHAN McGRAW | SAMPLE SHEAR CALCS (TECH 3) | PAGE 6 OF 8

DIRECT SHEAR:



TORSIONAL SHEAR:



TOTAL SHEAR:

$V_{SW4} = 16^K + 32.5^K = 48.5^K$ $(\leftarrow)$ → WORST CASE FOR X-DIRECTION LOADING

$V_{SW5} = 10.5^K + 17.7^K = 28.2^K$ $(\leftarrow)$

$V_{SW3} = 52.8^K - 40.9^K = 11.1^K$ $(\leftarrow)$

$V_{F3} = 3.5^K - 6.0^K = -2.5^K$ $(\rightarrow)$

$V_{F1} = 1.2^K - 1.5^K = -0.3^K$ $(\rightarrow)$

$V_{F2} = 1.2^K - 1.5^K = -0.3^K$ $(\rightarrow)$

CHECK $= 84.7^K$

NATHAN McGRAW | SAMPLE SHEAR CALCS (TECH3) | PAGE 7 OF 8

$V = 85.05^K$ (IN y-DIRECTION)

$M_{DUE\ TO\ ACCIDENTAL\ TORSION} = 382.72^K$

DIRECT SHEAR:

$V_{SW1} = (0.360)(85.05) = 30.6^K$

$V_{SW2} = (0.358)(85.05) = 30.4^K$

$V_{SW6} = (0.044)(85.05) = 3.7^K$

$V_{SW7} = (0.086)(85.05) = 7.3^K$

$V_{F4} = (0.045)(85.05) = 3.8^K$

$V_{F5} = (0.014)(85.05) = 1.2^K$

$V_{F6} = (0.018)(85.05) = 1.5^K$

$V_{F7} = (0.018)(85.05) = 1.5^K$

$V_{F8} = (0.014)(85.05) = 1.2^K$

$V_{F9} = (0.044)(85.05) = 3.7^K$

CHECK $= 84.9^K$

TORSIONAL SHEAR IN WALLS:

$$V_{SW1} = \frac{[(85.05^K)(0.9') + 382.72](11.73)(0.360)}{2079} = 0.9^K$$

$$V_{SW2} = \frac{(459.265)(10.53)(0.358)}{2079} = 0.8^K$$

$$V_{SW6} = \frac{(459.265)(15.6)(0.044)}{2079} = 0.2^K$$

$$V_{SW7} = \frac{(459.265)(0.025)(0.086)}{2079} = 0^K$$

$$V_{F4} = \frac{(459.265)(42.9)(0.045)}{2079} = 0.4^K$$

$$V_{F5} = \frac{(459.265)(35.9)(0.014)}{2079} = 0.1^K$$

$$V_{F6} = \frac{(459.265)(23.73)(0.018)}{2079} = 0.1^K$$

$$V_{F7} = \frac{(459.265)(24.93)(0.018)}{2079} = 0.1^K$$

$$V_{F8} = \frac{(459.265)(37.1)(0.014)}{2079} = 0.1^K$$

$$V_{F9} = \frac{(459.265)(44.1)(0.044)}{2079} = 0.4^K$$

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

NATHAN MGRAW | SAMPLE SHEAR CALCS (TECH 3) | PAGE 8 OF 8

* SINCE e IS EXTREMELY SMALL FOR 4-DIRECTION LOADING, OTHER WALLS NOT CHECKED FOR TORSIONAL SHEAR ISSUES

TOTAL SHEAR:

$V_{SW1} = 30.6^k + 0.9^k = 31.5^k$ → WORST CASE FOR 4-DIRECTION LOADING

$V_{SW2} = 30.4^k - 0.8^k = 29.6^k$

$V_{SW6} = 3.7^k + 0.2^k = 3.9^k$

$V_{SW7} = 7.3^k - 0.0^k = 7.3^k$

$V_{F4} = 3.8^k - 0.4^k = 3.4^k$

$V_{F5} = 1.2^k - 0.1^k = 1.1^k$

$V_{F6} = 1.5^k - 0.1^k = 1.4^k$

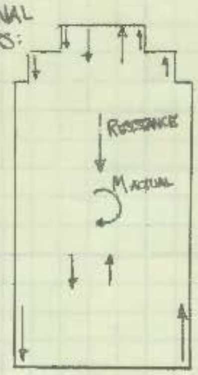
$V_{F7} = 1.5^k + 0.1^k = 1.6^k$

$V_{F8} = 1.2^k + 0.1^k = 1.3^k$

$V_{F9} = 3.7^k + 0.4^k = 4.1^k$

CHECK: $85.2^k \approx 85^k$

TORSIONAL SHEARS:

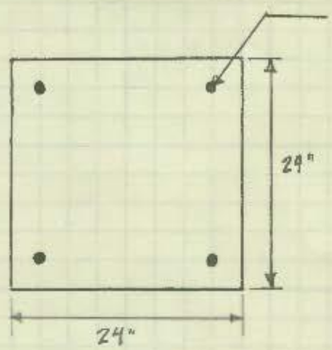


3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

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

NATHAN MCGRAW | COLUMN SPOT CHECK (TECH 3) | PAGE 1 OF 5

COLUMN G-2 CHECK:



$f'_c = 5000$ psi
$f_y = 60$ ksi

AXIAL STRENGTH:

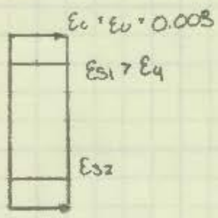
$P_o = 0.85 f'_c A_c + A_s f_y$

$A_s = (4)(1.56\ \text{in}^2) = 6.24\ \text{in}^2$

$P_o = 0.85(5)[(24)(24) - 6.24] + (6.24)(60)$

$P_o = 2795.88$

$P_o = 2796^k$



$\varepsilon_y = 60/29000 = 0.00207$

* COMPRESSION-CONTROLLED SECTION SINCE IN PURE COMPRESSION
$\therefore \phi = 0.65$

$\phi P_n = 0.65(2796) = 1817.3$

$\boxed{\phi P_o = 1817^k}$

NATHAN McGRAW | COLUMN SPOT CHECK (TECH3) | PAGE 2 OF 5

PURE BENDING STRENGTH: $d = 1.5 + 0.5 + \frac{1}{2}d_b = 2.7"$

$\varepsilon_c = \varepsilon_u = 0.003$

$\varepsilon_{s1} = \frac{0.003}{c}(c - 2.7)$

$\varepsilon_{s2} > \varepsilon_y$

$0.85(4)$

$(2)(1.56)(0.003/c)(c-2.7)29000$

$0.85(5)(24)(0.80c)$

$2(1.56)(60) = 187.2$

$T = C$

$187.2 = 271.44\,(c - 2.7/c) + 81.6c$

$187.2c = 271.44c - 732.9 + 81.6c^2$

$0 = -732.9 + 84.24c + 81.6c^2$

$c = 2.53"$

$\varepsilon_{s1} = \frac{0.003}{2.53}(2.53 - 2.7) = -0.000202$

$f_{s1} = \varepsilon_{s1}E_s = -0.000202(29000) = -5.85$ KSI

$\varepsilon_{s2} = \frac{0.003}{2.53}(2.53 - 21.3) = -0.022 < -\varepsilon_y$

$f_{s2} = -f_y = -60$ KSI

$0.85(5)$

$\beta_1 c$

$0.85(5)(24)(0.8)(2.53) = 206.45^k$

$2(1.56)(5.85) = 18.24^k$

$(2)(1.56)(60) = 187.2^k$

$M_o = 206.45\,(24/2 - 0.8 \times 2.53/2) - 18.24\,(24/2 - 2.7) - 187.2\,(24/2 - 21.3)$

$M_o = 3839.8$ K-IN

$M_o = 320$ K-FT

$\phi = 0.9$ FOR TENSION CONTROLLED SECTION

$\phi M_o = (0.9)(320) = 288\,'^k$

$\boxed{\phi M_o = 288\,'^k}$

NATHAN MCGRAW | COLUMN SPOT CHECK (TECH3) | PAGE 3 OF 5

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

PURE TENSION:

$T_o = (4)(1.56)(60) = -374.4^{k}$

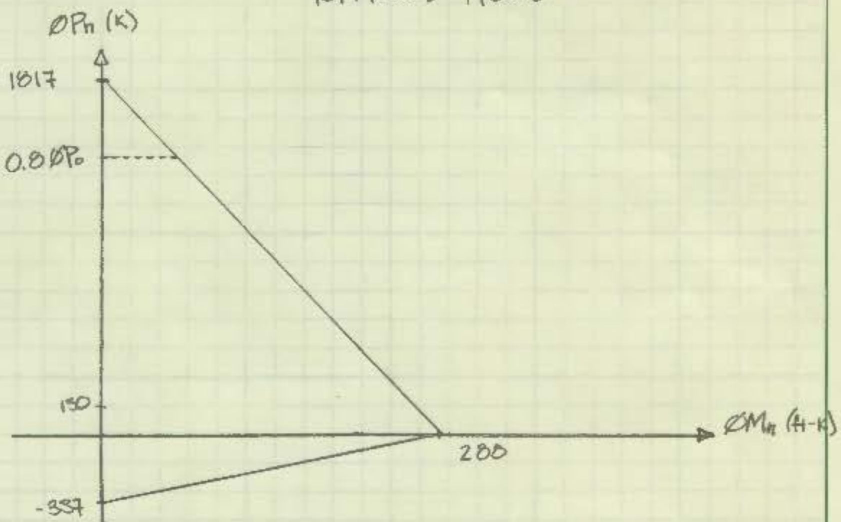
$\phi = 0.9$

$\phi T_o = (0.9)(-374.4) = -337^{k}$

$\boxed{\phi T_o = -337^{k}}$

INTERACTION DIAGRAM:

$1817(0.8) = 1453.6^{k}$



NATHAN McGRAW | COLUMN SPOT CHECK (TECH 3) | PAGE 4 OF 5

DETERMINING AXIAL LOAD (P):

* USING LOADS LISTED IN TECH 1.

ROOF:

LL = 100 PSF (unreducible)
DL = 152.9375 PSF
S = 21 PSF
P_{DL} DROP PANELS = 7.5^K

TYPICAL FLOOR LOAD:

LL = 80 PSF ; MAX 60 + 20 PSF = 80 PSF

SDL = 20 PSF
DL = 118.75 PSF
P_{DL} DROP PANEL = 7.5^K
$K_{LL}A_T$ > 400 SQ FT FOR LL REDUCTION

COLUMN G-2:

TRIBUTARY AREA = $(29')(29'/2) = 420.5\ FT^2$
INFLUENCE AREA = $(420.5\ FT^2)(2) = 841\ FT^2$

ROOF: $P_D = \dfrac{(420.5)(152.9375+20)}{1000} = 72.7^K + 7.5^K = 80.2^K$

$P_L = \dfrac{(420.5)(100)}{1000} = 42.05^K$

11^{th} FLOOR: $P_D = \dfrac{\left[(420.5)(20+118.75) + (24\times24/144)(150)(15.333')\right]}{1000} + 7.5^K = 75^K$

$LL_r = 0.25 + \dfrac{15}{\sqrt{841}} = 0.77$

$P_L = \dfrac{0.77(420.5)(80)}{1000} = 25.8^K$

10^{th} FLOOR: $P_D = \dfrac{\left[(420.5)(20+118.75) + (24\times24/144)(150)(11.333')\right]}{1000} + 7.5^K = 72.6^K$

$LL_r = 0.25 + \dfrac{15}{\sqrt{(2)(841)}} = 0.62$

$P_L = \dfrac{0.62(420.5)(80)}{1000} = 20.9^K$

9^{th} FLOOR: $P_D = 72.6^K$

$LL_r = 0.25 + \dfrac{15}{\sqrt{(3)(841)}} = 0.55$

$P_L = \dfrac{0.55(420.5)(80)}{1000} = 18.5^K$

NATHAN McGRAW | COLUMN SPOT CHECK (TECH 3) | PAGE 5 OF 5

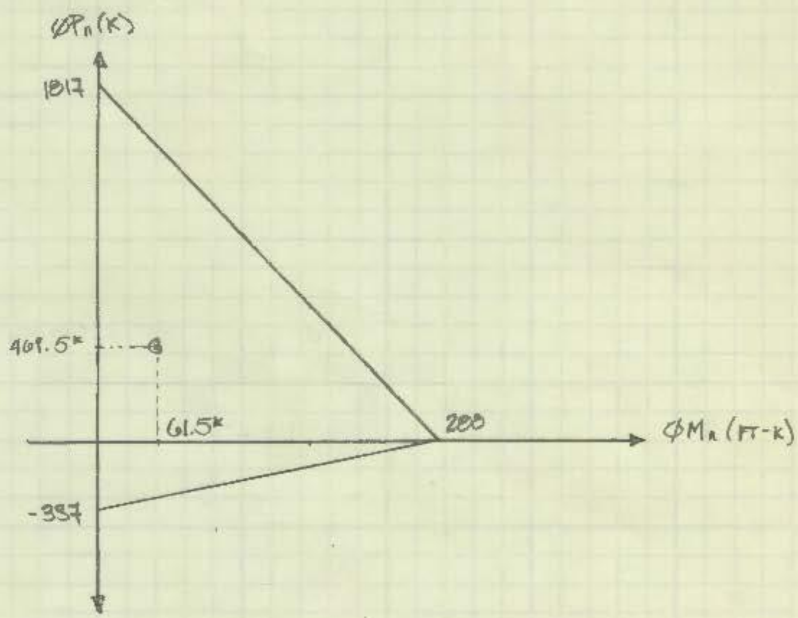
P_D TOTAL = 300.4^k
P_L TOTAL = 107.25
$S = (420.5)(21\,psf)/1000 = 8.8^k$

LC = 1.2D + 1.0E + L + 0.2S

$P_U = 1.2(300.4) + 1.0(107.25) + 0.2(8.83) = 469.5^k$

$\boxed{P_U = 469.5^k \quad M_U = 61.5\,'^k}$ (OBTAINED FROM ETABS) → EARTHQUAKE IN Y-DIRECTION

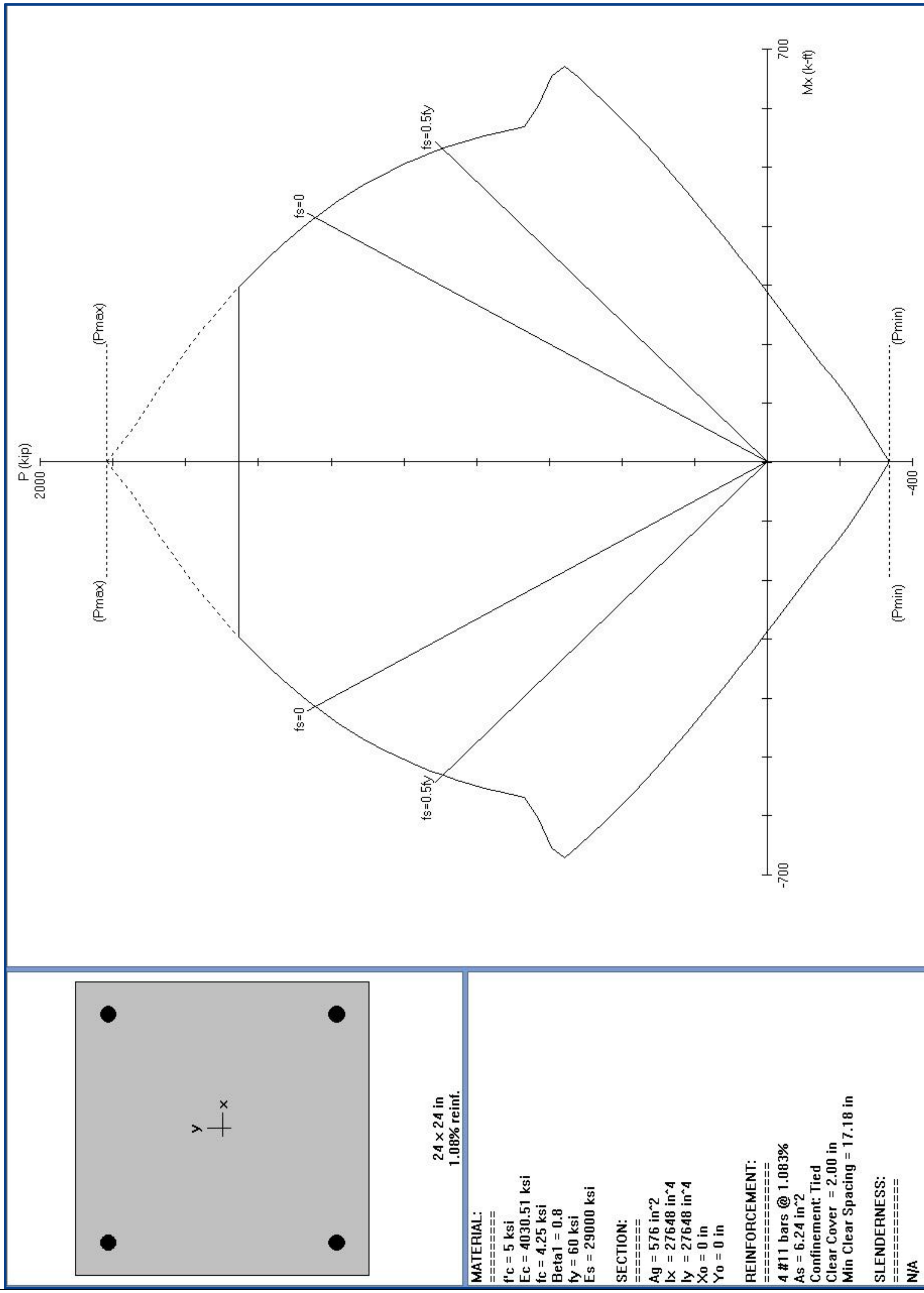
INTERACTION DIAGRAM



* SINCE (M_U, P_U) POINT FALLS WITHIN INTERACTION DIAGRAM,

$\boxed{\text{COLUMN IS ADEQUATE}}$

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

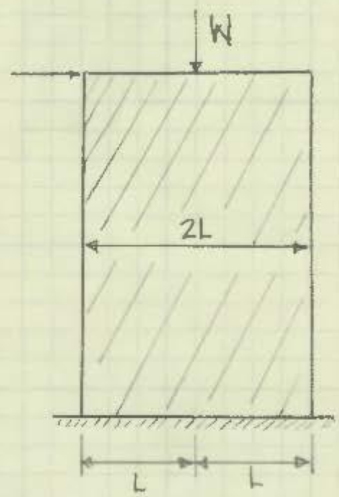

P (kip)
2000
(Pmax)
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
(Pmin)
(Pmin)
-400
700
-700
Mx (k-ft)
24 x 24 in
1.08% reinf.
MATERIAL:
f'c = 5 ksi
Ec = 4030.51 ksi
fc = 4.25 ksi
Beta1 = 0.8
fy = 60 ksi
Es = 29000 ksi
SECTION:
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
4 #11 bars @ 1.083%
As = 6.24 in^2
Confinement: Tied
Clear Cover = 2.00 in
Min Clear Spacing = 17.18 in
SLENDERNESS:
N/A

NATHAN M'GRAW | OVERTURNING M (TECH 3) | PAGE 1 OF 1

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

CONTROLLING BASE OVERTURNING MOMENT:

$M_{EQ} = 80,426.63 \text{ K-FT}$



$M_{RESISTING} = \frac{1}{2} \left| \begin{matrix} B \\ L \end{matrix} \right|_{MIN} \times W = \frac{1}{2} \left| \begin{matrix} 90' \\ 190.75' \end{matrix} \right|_{MIN} \cdot (39026.3^{K})$

$M_{RESISTING} = \frac{1}{2}(90)(39026.3^{K}) = \boxed{1756183.5 \text{ K-FT}}$

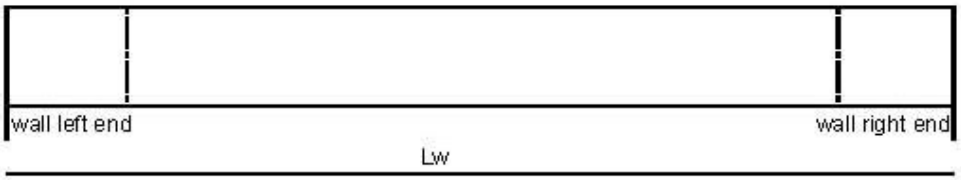
$\frac{2}{3} M_R > M_o$

$\frac{2}{3}(1756183.5) = 1170789 \text{ K-FT} > M_o = 80,426.63 \text{ K-FT} \therefore OK \checkmark$

Shear Wall Calculations

Material

f'c =	5.0 Ksi	- concrete strength
fy =	60 Ksi	- steel reinforcement yield strength
Es=	29000 Ksi	



Wall

Lw =	188 in	- wall lentgh
tw =	12 in	- wall thickness
hw =	136 in	- wall height
cw =	0.75 in	- concrete cover @ wall

Reinforcement

Vertical | # curtains: 2

bar size	spacing	db	As	# bars/curtain	As total	actual spacing	Max Spacing	
- Lw								
#4	12 in	0.5	0.2	15	3	11.59	18	Meet max spacing
- wall left end (vertical)								
#4	0.75 in	0.5	0.2	1	0.2	1 in	18	Meet max spacing
- wall right end (vertical)								
#4	0.75 in	0.5	0.2	1	0.2	1 in	18	Meet max spacing
		total # bars/curtain		17				

As=	6.8	ρ_ℓ	0.30%	Meet min reinf
Ac=	2250	ρ minum ACI 11.9.9.4	0.27%	

Horizontal | # curtains: 2

bar size	spacing	db	As	Max Spacing	
- wall (horizontal)					
#4	12 in	0.5	0.2	18	Meet max spacing

ρt	0.28%	Meet min reinf
ρ minum ACI 11.9.9.2	0.25%	

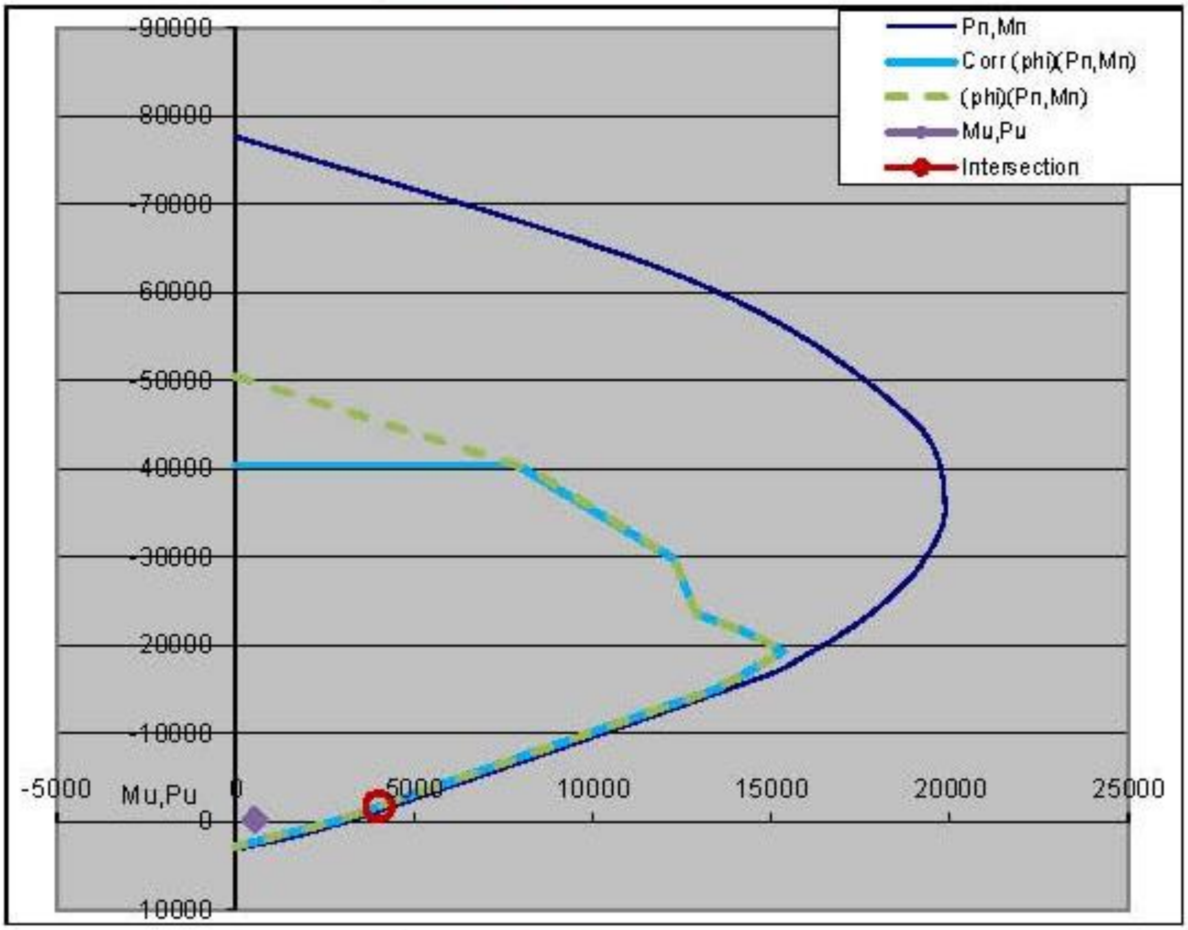
Loads

Mu=	6596000 in-lb
Vu=	48500 lb
Pu=	234.5 kip

1/2

Results

Shear	Vc=	254.55844 kips				
	ϕ0.5Vc=	95.459415 kips	>Vu	Vs not needed		
	Av=	0.4 in²				
	Vs=	0 kips				
	Vn=Vc+Vs=	254.55844	<	1272.792 kips	ACI section 11.9.3	
	Vn=	254.55844 kips				
	ϕVn=	190.91883 kips				
	Vu<ϕVn		PASS			



DCR= 0.1365 <1 PASS

2/2

Appendix D: Typical Plans

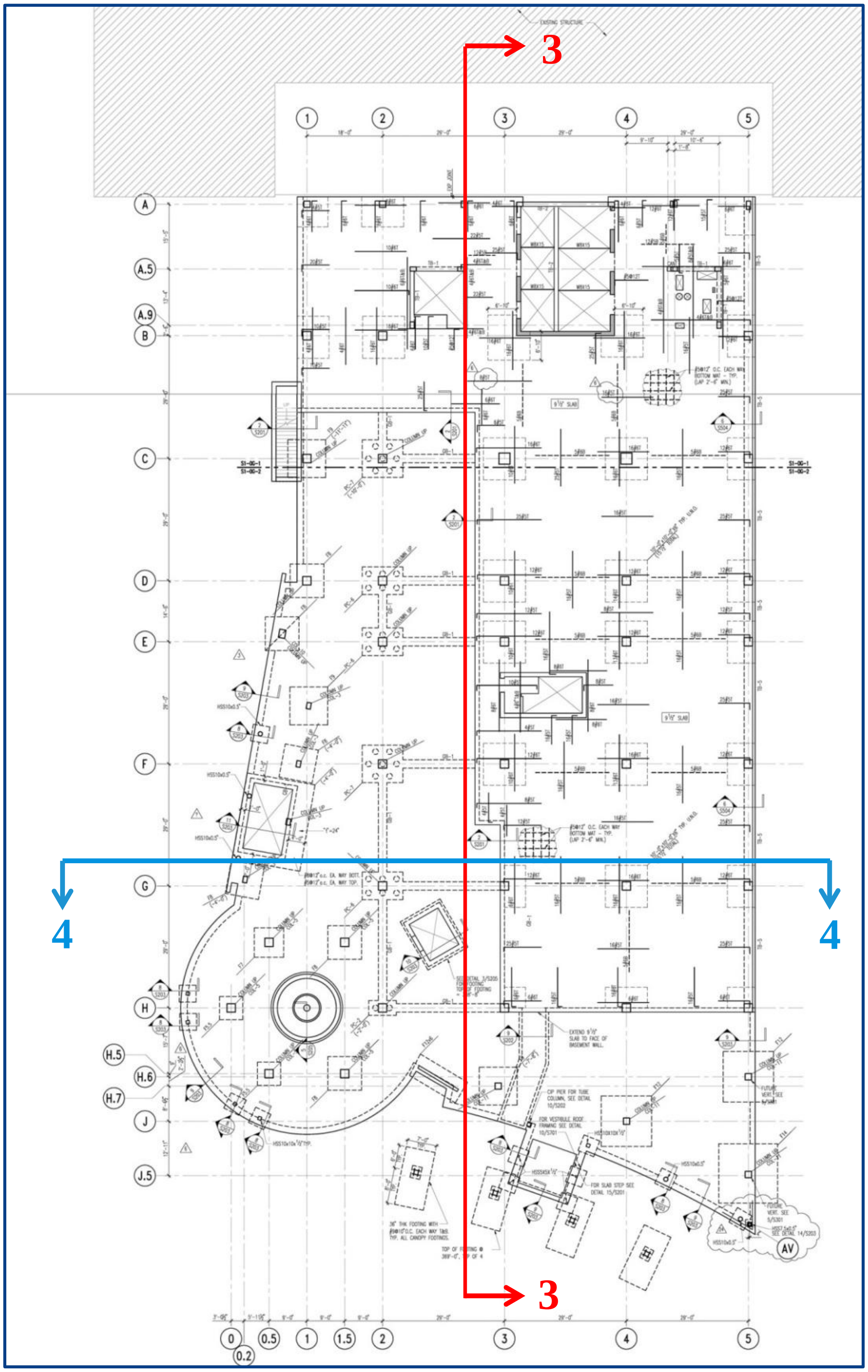


Figure 1:
Ground floor plan (See following figures for sections indicated on the plan)

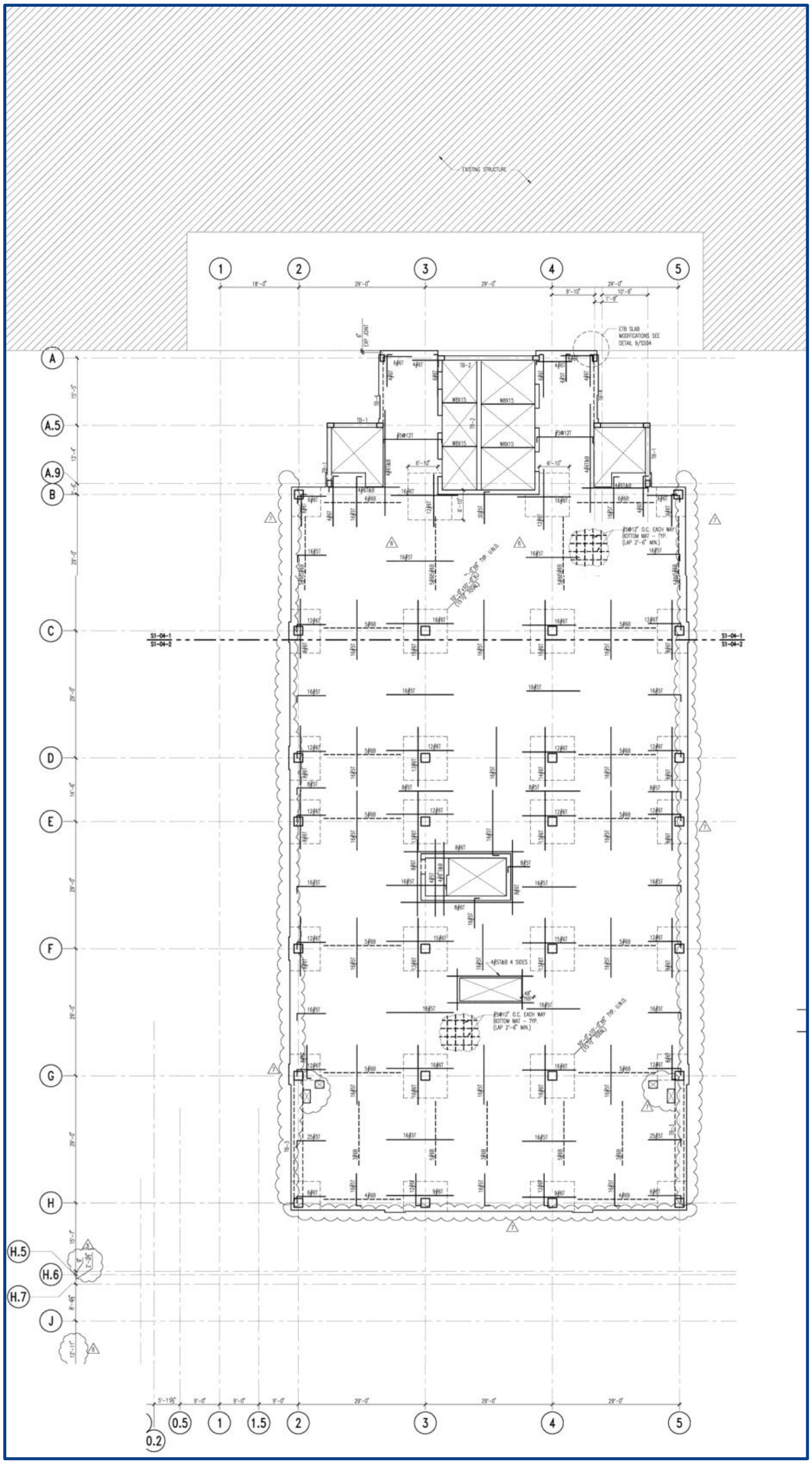


Figure 2:
Typical floor plan (6th – 11th)

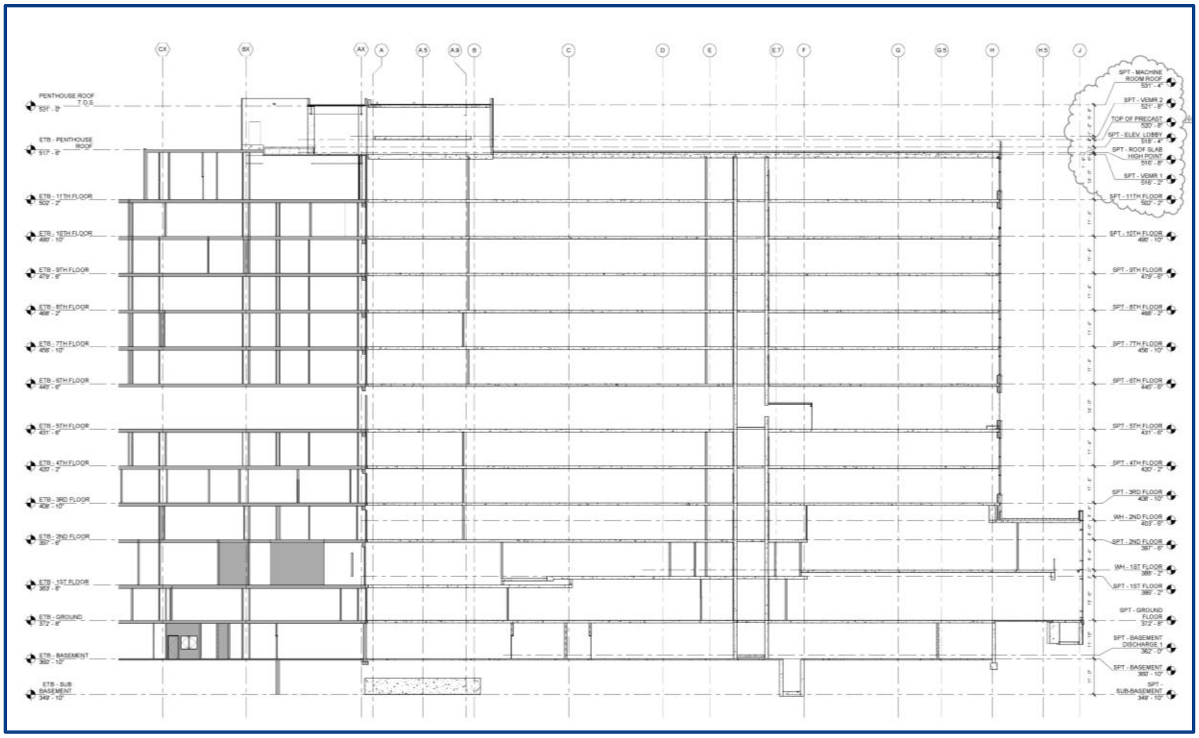


Figure 3:
North – South section cut

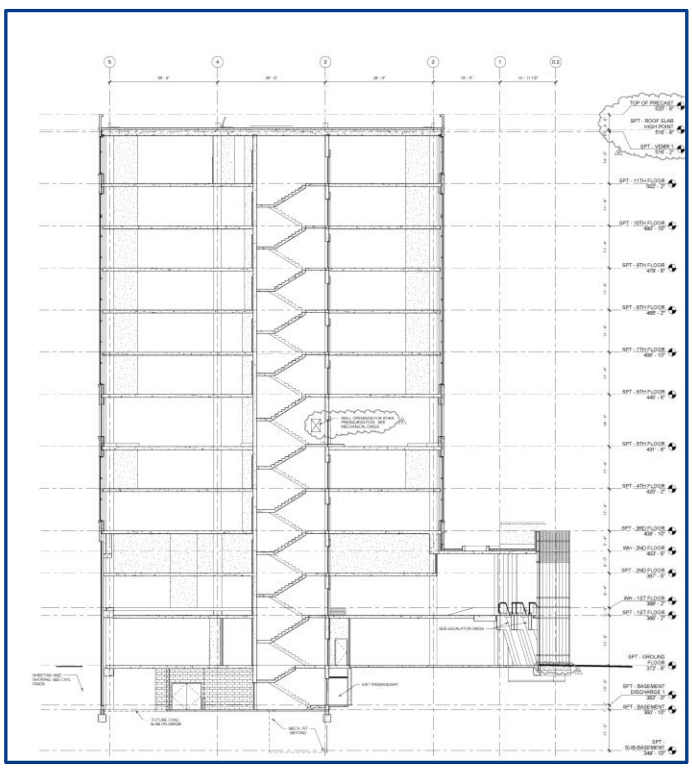


Figure 4:
East – West section cut