Technical Report #3

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This Document is Technical Report #3 for 5th year senior thesis in the Architectural Engineering Departments at The Pennsylvania State University. In this report the existing reinforced concrete shear wall cores will be analysis and confirmed using the loads obtained in Tech report #1 by using ASCE 7-10.

Structural Option

Professor Behr

Hospital Patient Tower

Virginia, U.S.A.

11/29/2010

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Executive Summary

Technical Report #3 is an analysis and confirmation design study of the lateral system for the patient tower. In this report the existing reinforced concrete shear wall cores will be analysis and confirmed using the loads obtained in Tech report #1 by using ASCE 7-10.

An ETABS model of the patient tower was created for this assignment to compare the analysis results to the hand calculations done for the tower. The ETABS model included the main reinforced concrete system as well as the shear wall cores acting as the main shear resisting system. The calculations done by hand only took into account the shear walls as the lateral resisting system. The lateral loads were applied to the model to determine center of rigidity, torsion, overturning, and story drifts all taken from the ETABS outputs and compared to the hand calculation and allowable limits set forth by the code and industry.

During the comparison of the ETABS results and the hand calculations there were a few differences that were noticed. Since the hand calculations were only taking in to account the shear walls while ETABS was analysis the rigid concrete frame the outputs varied slightly but nothing over what would be expected with including the frame. The outputs for the model and the hand calculations were kept separate during all calculations to maintain consistence between the model and hand calculations without merging the two separate sets of data.

The overturning results show that the dead load of the building will resist any uplift created by the lateral loads since the lateral loads are such a small fraction of the building dead loads. The displacement and story drifts were found to be with in the allowable limits of the code.

Each analysis done on the lateral system of the building can be seen in detail through descriptions and diagrams, as well as, the materials and codes used in the analysis and design. Building layout and detailed calculations for each analysis can be found in the appendix at the end of the report.

Figure 1: Rendering by Wilmot Sanz

Introduction

The Patient Tower is part of the 2015 Capital Improvement Project, of which the Tower Expansion is one of the earlier phases. The new Patient Tower will connect with an existing patient tower by a bank of elevators separated into two sections, one for visitors and the other for patients at every floor. The Tower will also await the connection of a women's health facility that is one of the next phases of the Capital Improvement Project. The Façade of the Patient Tower will blend in with the existing buildings by keeping some of the red brick on the exterior, but also by taking on a more modern look by incorporating an aluminum curtain wall and precast concrete panels. The new tower consists of 12 stories above grade with one level below grade. The patient tower is 216,000 square feet with 174 patient rooms, an operation area and a mechanical level. The contract for this tower was awarded to Turner Construction, the general contractor, in a Design-Bid-Build method with a contact value of $161 million.

One of the main design considerations is individual patient rooms. Based on the hospital's goals for care the individual patient rooms were a large factor in the design of the floor plan. During the design phases the project team requested input from the physicians, nurses and staff to help make the design as efficient as possible. Medical/surgical patients aging 65 years and older were the focus of this tower, with a special emphasis on their safety and a good healing environment. With the hospital team input the placements for monitoring stations were optimized to ensure patient privacy as well as enhancing the monitoring capabilities.

One of the hospital's goals, along with excellent patient care, is also to lower the hospital's impact on the environment. The hospital's plan for this new tower included green features such as living roofs, low flow water fixtures, and rain gardens. The design also calls for no/low VOC building materials to be used in construction of the tower. The tower design has been submitted for a LEED Silver certification.

Figure 2: Sketch by Wilmot Sanz

Structural Systems

Foundations

The geotechnical report was prepared by Schnabel Engineering, LLC, on March 25, 2010. The foundation of the patient tower is set on piles, with pile caps and grade beams. Each column location has a range of 4 to 12 piles. The slab on grade for the tower is 5" with integrated slab pile caps in locations of high stress, such as the elevator shaft and stair well. During the excavation for the new tower the existing basement and caissons supporting the connecting structure were exposed. The existing 66" caissons will support a small portion of the tower connection while the rest will be supported by new piles. In a few locations where there is no basement piles were drilled to reach up to the ground floor level to support irregular building features.

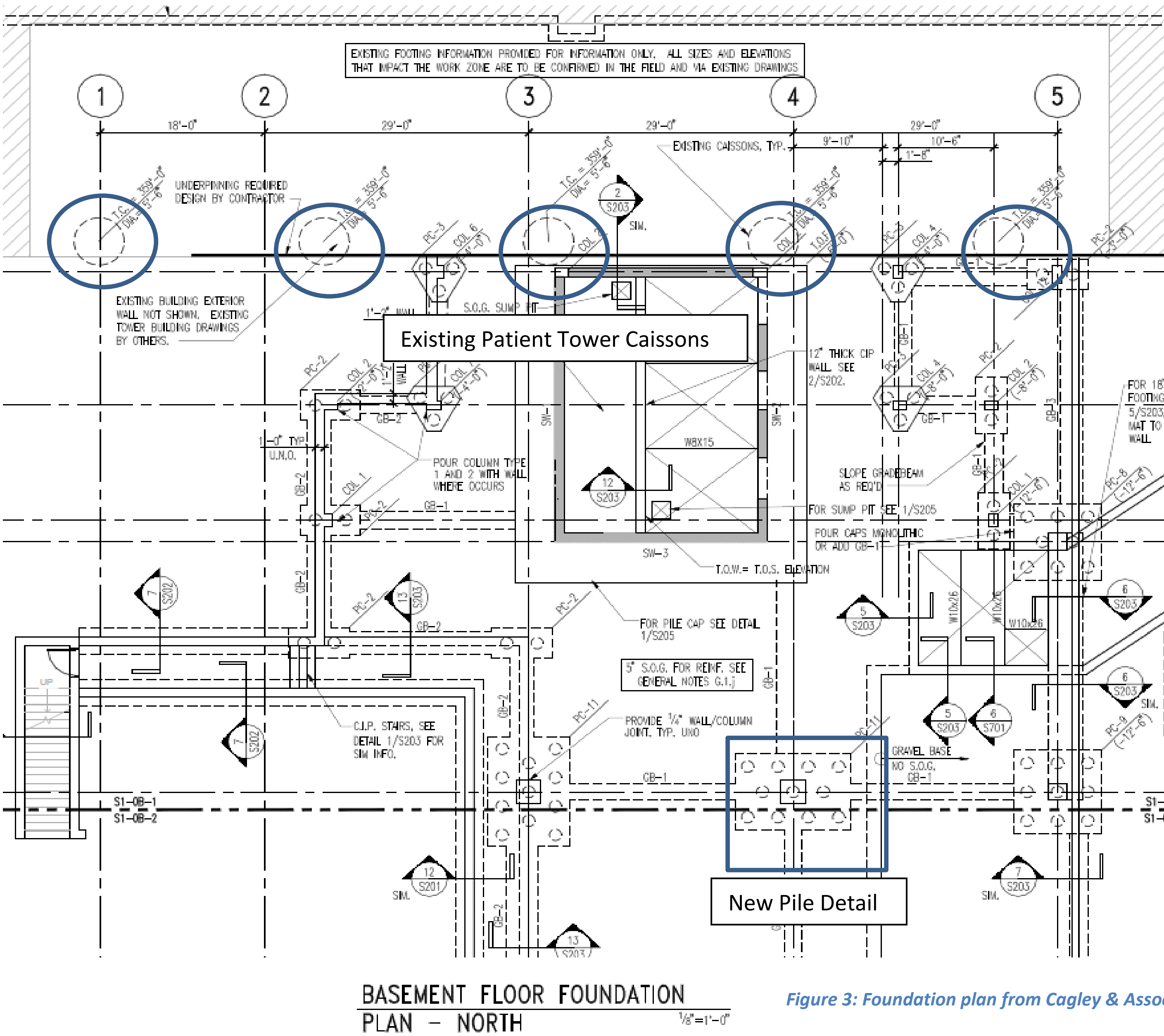


Figure 3: Foundation plan from Cagley & Associates

Columns

The column layout of the patient tower is very regular with a few variations on the 1st through 3rd floors. The bay spacing in the patient tower is mostly square 29' x 29' with a few exceptions as see in Figure 6. The columns are reinforced concrete ranging in size from 30" x 30" to 12" x 18". The typical column size is 24" x 24" with vertical reinforcing of #11 bars numbering from 4 bars to 12 bars as they move through the structure. The vertical reinforcing is tied together with #4 bars placed every 18". The columns on the basement level up through the 4th floor are poured with 7,000 psi concrete and from the 5th floor up they are 5,000 psi concrete. The structural system of the Patient Tower utilizes column capitals to resist punching shear within the slab. The typical capital in the tower is 10' x 10' x 6" depth, making the slab thickness at the capitals 15 ½".

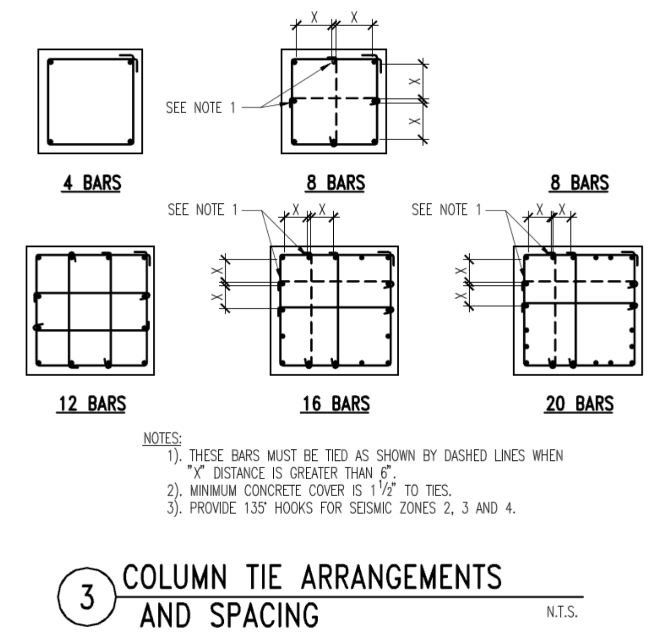


Figure 4: Column Reinforcing Detail from Cagley & Associates

MECH ROOM FLOOR						
MAIN ROOF						
ELEVENTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24 4#11 #4@18'
TENTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24 4#11 #4@18'
NINTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24 4#11 #4@18'
EIGHTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24 4#11 #4@18'
SEVENTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 8#11 #4@18"	24"x24 4#11 #4@18'
SIXTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 8#11 #4@18"	24"x24 4#11 #4@18'
FIFTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 12#11 #4@18"	24"x24 4#11 #4@18'
FOURTH FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 12#11 #4@18"	24"x24 4#11 #4@18'
THIRD FLOOR		24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 12#11 #4@18"	24"x24 8#11 #4@18'
SECOND FLOOR	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 4#11 #4@18"	24"x24" 8#11 #4@18"	26"x26" 16#11 #4@18"	24"x24 8#11 #4@18'
FIRST FLOOR	24"x24" 4#11 #4@18"	24"x24" 8#11 #4@18"	24"x24" 8#11 #4@18"	24"x24" 8#11 #4@18"	26"x26" 16#11 #4@18"	24"x24 12#11 #4@18'
GROUND FLOOR	24"x24" 4#11 #4@18"	24"x24" 8#11 #4@18"	24"x24" 8#11 #4@18"	24"x24" 12#11 #4@18"	28"x28" 20#11 #4@18"	24"x24 12#11 #4@18'
BASEMENT FLOOR TOP OF FOUNDATION			24"x24" 12#11 #4@18"		30"x30" 20#11 #4@18"	26"x26 16#11 #4@18'

Figure 5: Partial Column Schedule from Cagley & Associates

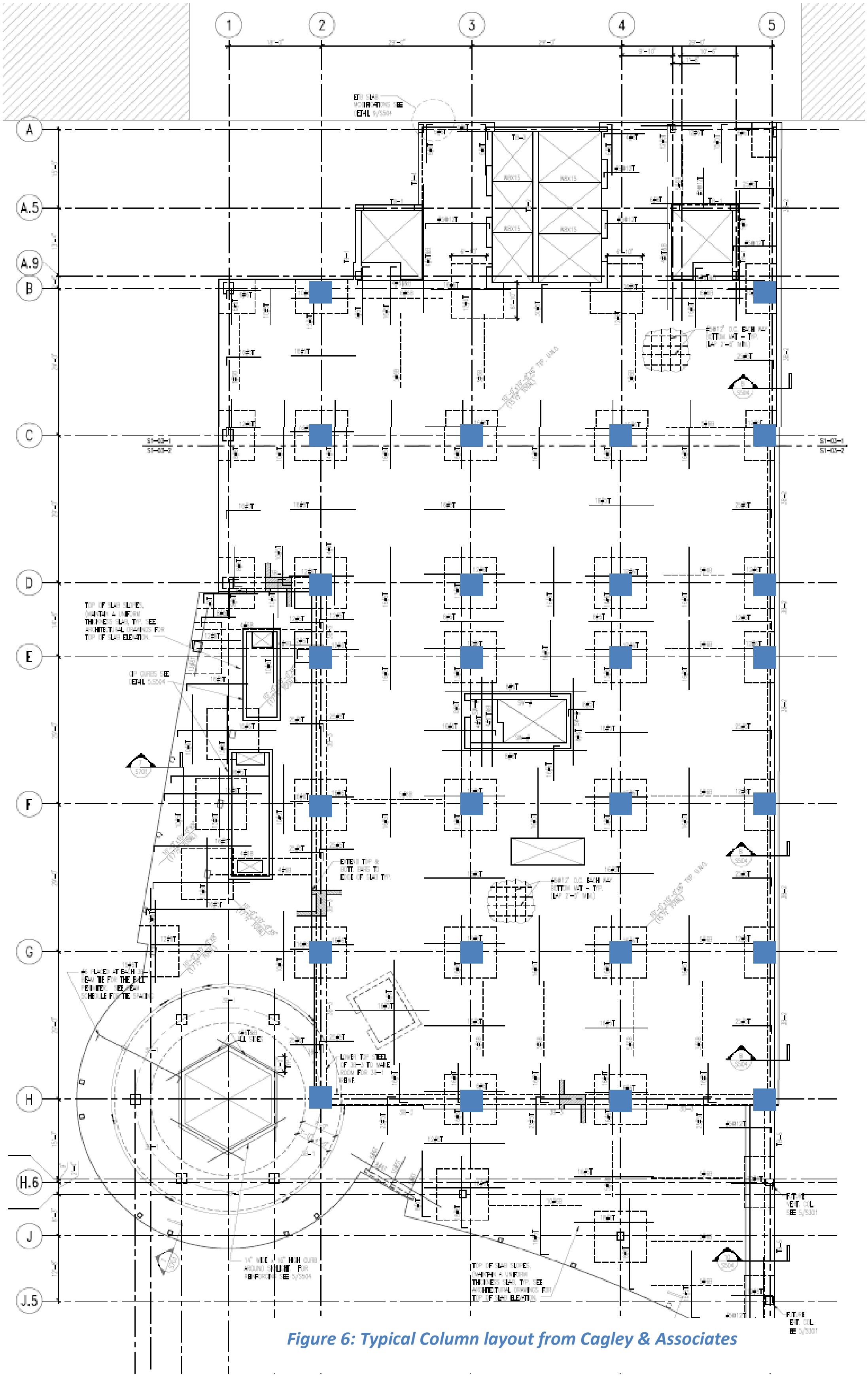


Figure 6: Typical Column layout from Cagley & Associates

Floor System

The floor system for this patient tower is a 9.5" 2-way flat plate. For the ground floor through the 4th floor the slab is 5000 psi concrete with the remaining floors at 4000 psi concrete. The largest span for this flat plate is 29' in each direction with square bays. The flat plate system has both top and bottom steel reinforcing. The top steel placed at regions of negative moment is typical notated with a number of #5 bars. The bottom reinforcing is a 2-way mat of #5 bars at 12" on center. In the end bays of the slab there are extra bottom bars added to handle the carry over moments for the interior span. On the 5th floor of the tower is the mechanical level, which increases the loading on the slab giving it a 10.5" concrete slab. See figure 7 below for details.

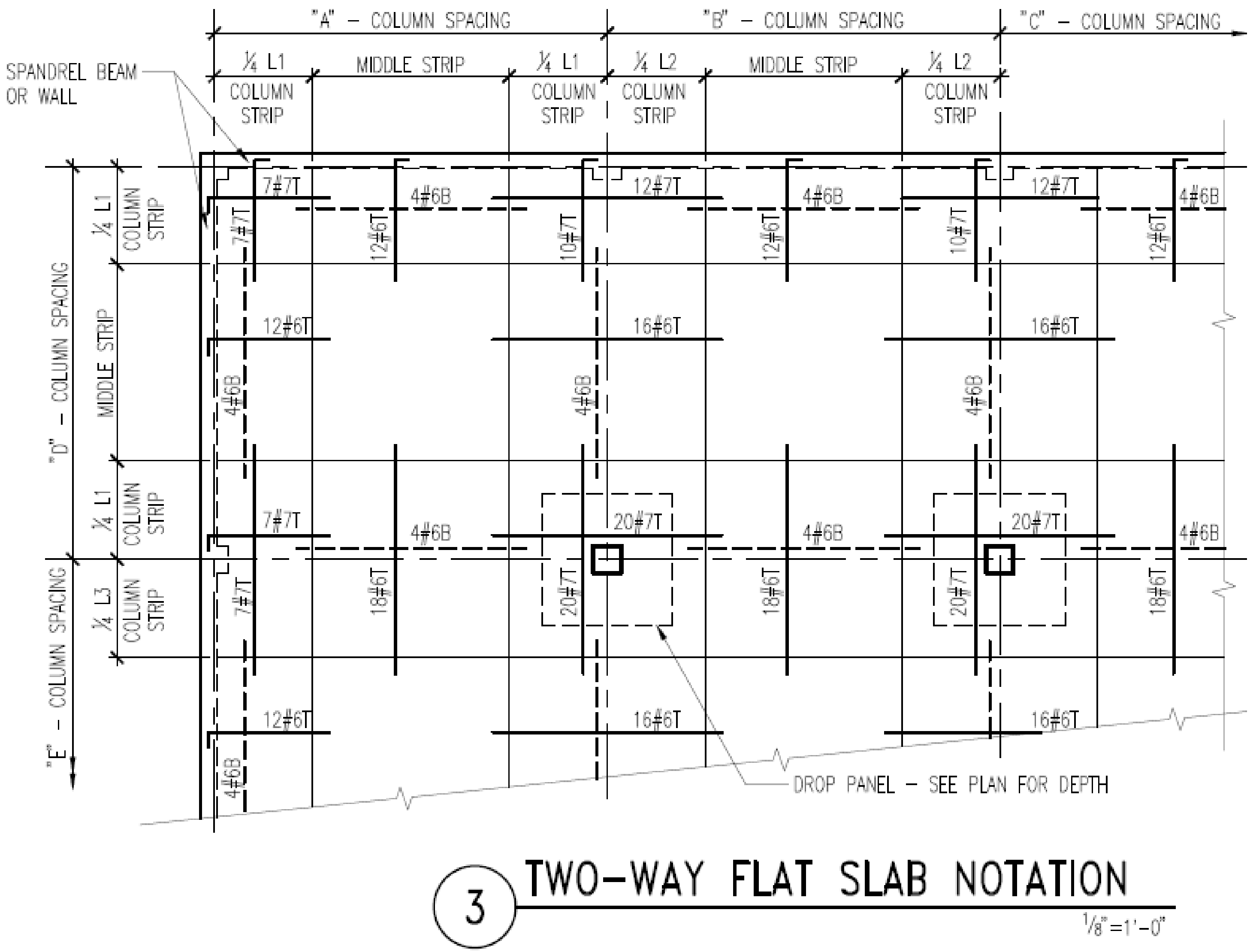


Figure 7: Two-way Flat Slab Detail from Cagley & Associates

Roof System

The roof system for the patient tower is designed with the same conditions at a typical floor, a 9.5" Two-way flat plate with mat and bar reinforcing detailed in the above section. The roof does have a few variations from a typical floor; the roof area that will support the mechanical penthouse has been increased to a 14" slab to support the extra weight of the equipment and there were supports added to the main slab to support the new helipad (Figure 8) for the tower.

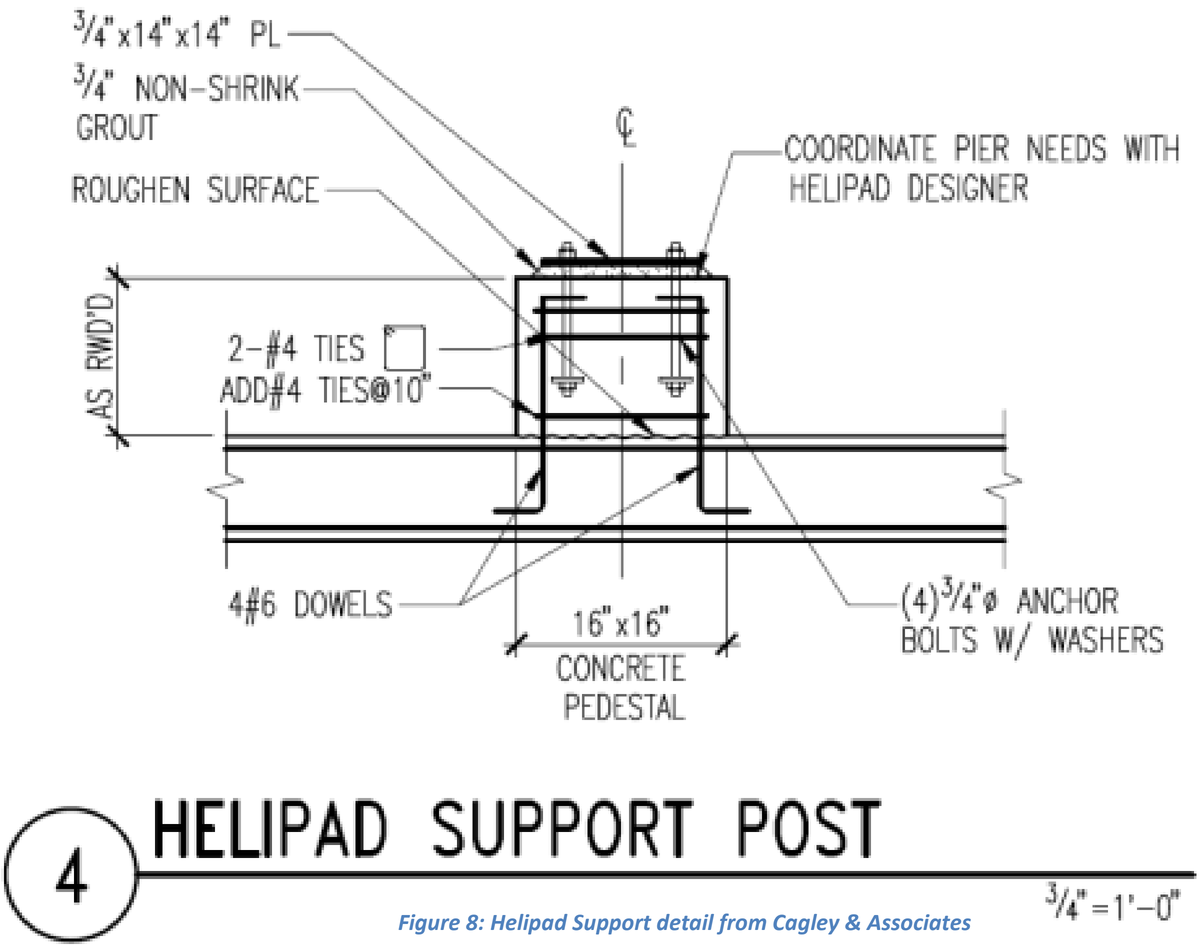


Figure 8: Helipad Support detail from Cagley & Associates

Lateral System

The lateral system in the new patient tower consists of seven 12" reinforced concrete shear walls. These walls are located in different locations throughout the building depicted to the right. The shear walls consists of 5000 psi concrete and were run continuously through the tower from the foundations up to the roof with the northern core extending through the penthouse. This system of two shear wall cores resists lateral loads in both the north-south and east-west direction based on the orientation of the wall. The towers main structural system is a concrete two-way flat plate. This system will also act a concrete moment frame which will also resist lateral forces. Between this two system all of the lateral forces applied to this tower can be resisted.

ETABS model

An ETABS model was constructed of the buildings structural system to be used in the analysis of the lateral reinforcing system. This model includes the concrete gravity reinforcing system as well as the two shear reinforcing systems. Both the wind and seismic loads that were found in Tech Report were input at the center of rigidity. The results for the model were compared to hand calculations fo the center of mass, center of rigidity and story displacements.

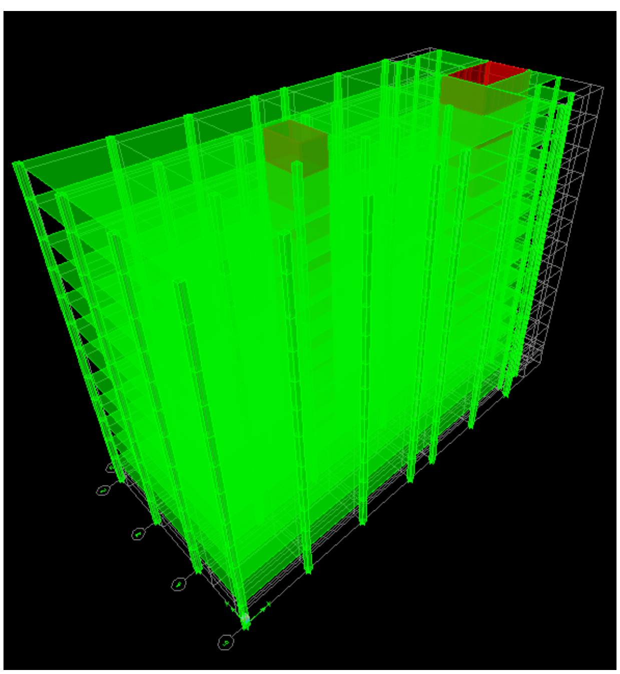


Figure 9: ETABS structural model

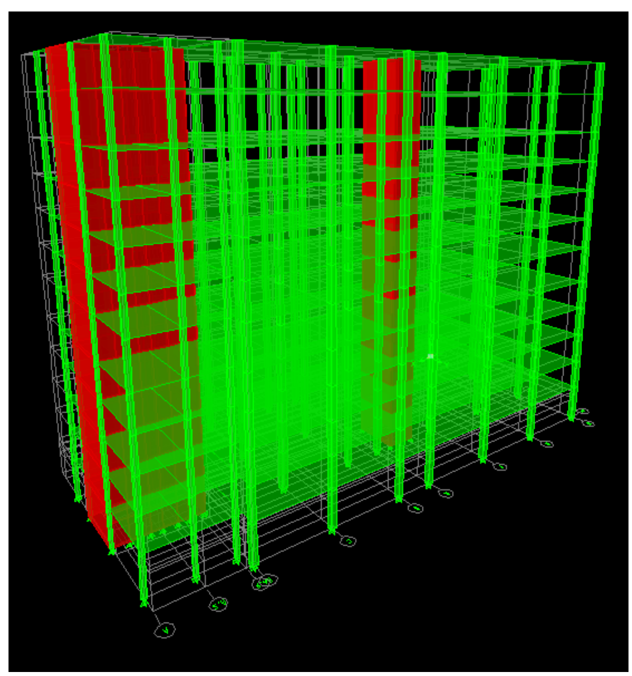


Figure 10: ETABS structural model

Design & Code Review

Design Codes and References

- International Building Code – 2006 "International Code Council".
- ASCE 7 – 05 "Minimum Design loads for Buildings and Other Structures" American Society of Civil Engineers.
- ACI 318-05 "Building Code Requirements for Structural Concrete" American Concrete Institute.
- ACI Manual of Concrete Practice.
- AISC "Manual of Steel Construction – Allowable Stress Design".

Thesis Codes and References

- International Building Code – 2006 "International Code Council".
- ASCE 7 – 10 "Minimum Design loads for Buildings and Other Structures" American Society of Civil Engineers.
- ACI 318-08 "Building Code Requirements for Structural Concrete" American Concrete Institute.

Deflection Criteria

Floor Deflection Criteria

Typical Live load Deflection limited to L/360

Typical Total load Deflection limited to L/240

Material Specifications

Materials	Grade	Strength
Concrete		
• Piles	-	f'_c = 4,000 psi
• Foundations	-	f'_c = 3,000 psi
• Slab-on-grade	-	f'_c = 3,500 psi
• Shear Walls	-	f'_c = 5,000 psi
• Columns	-	f'_c = 5,000/7,000 psi
• Floor Slabs	-	f'_c = 4,000/5,000 psi
W Flange Shapes	ASTM A992	F_y = 65,000 psi
HSS Round	ASTM A53 grade B	F_y = 35,000 psi
HSS Rectangular	ASTM A500 grade B	F_y = 46,000 psi
Reinforcing bars	ASTM 615 grade 60	F_y = 60,000 psi
Steel Decking	ASRM A653 SS Grade 33	F_y = 33,000 psi

Table 1: Material Specifications

Gravity Loads

Loads for the Patient Tower were calculated from IBC 2006 in Reference with ASCE 7 -05. Loads are displayed below.

Dead Loads

Occupancy	Design Loads
Normal Weight Concrete	150 psf
MEP Equipment	15 psf
Superimposed	20 psf

Table 2: Dead Loads

Live Loads

Occupancy	ASCE 7 – 10 Loads
Corridors First floor	100 psf
Hospitals	
• Operating Rooms, Laboratories	60 psf
• Patient Rooms	40 psf
• Corridors above 1^{st} floor	80 psf
Helipads	60 psf
Lobby	100 psf
Roof with Garden	100 psf

Table 3: Live Loads

Snow Loads

$p_f = 0.7C_eC_tI_sp_g$

Factor	Value
Exposure Factor C_e	0.9
Thermal Factor C_t	1.0
Importance Factor I_s	1.10
Ground Snow Loads p_g	25 psf
Flat Roof Snow Load p_f	17.3 psf ≈ 20 psf

Table 4: Snow Loads

Wind Loads

According the IBC 2006 the wind analyses procedures to be used are in ASCE 7-10 chapter 27. To examine the lateral wind loads in both the North-south and East-west wind direction, the MWFRS Directional Procedure (Table 27.2-1). According to Figure 26.5-1B the design wind speed is 120 MPH for the location of the Patient Tower. For this Tech Report, a few assumptions were made during the wind analyses procedures. One of the assumptions was that the building was completely regular from the ground to the roof elevation. On the first through third floors there is a glass atrium that extends passed the regular structure that was excluded in this analysis. It was also assumed that the building was independent of the connected tower and also that the wind was not impeded by any of the structures surrounding the new Patient Tower. The Details of these calculations can be found in Appendix II. Appendix II contains sample calculations, spreadsheets including all values used in this analysis and tables including all existing parameters. Figures 11 & 11 show the forces and shear for each wind force direction.

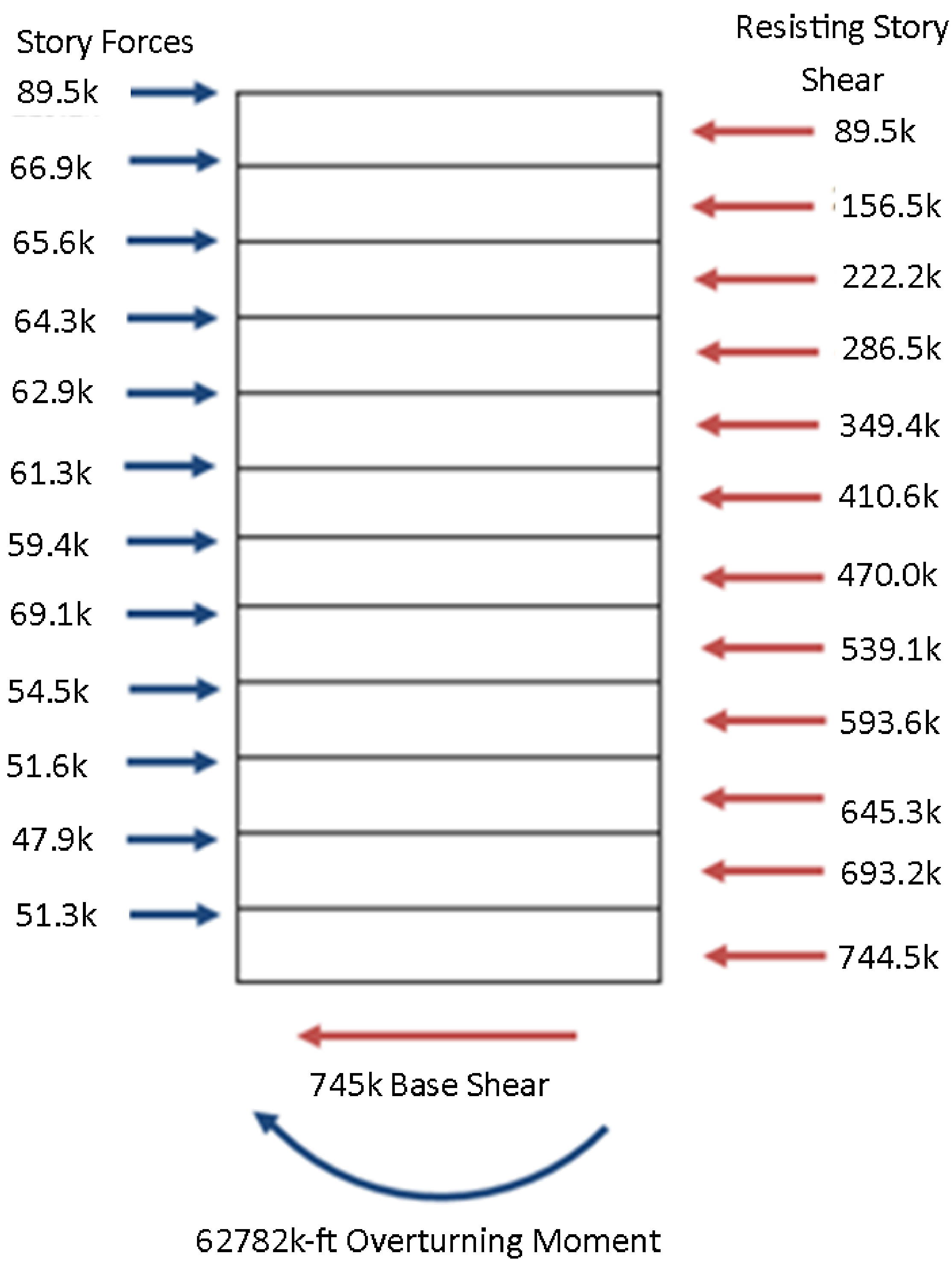


Figure 11: East-West Wind loads

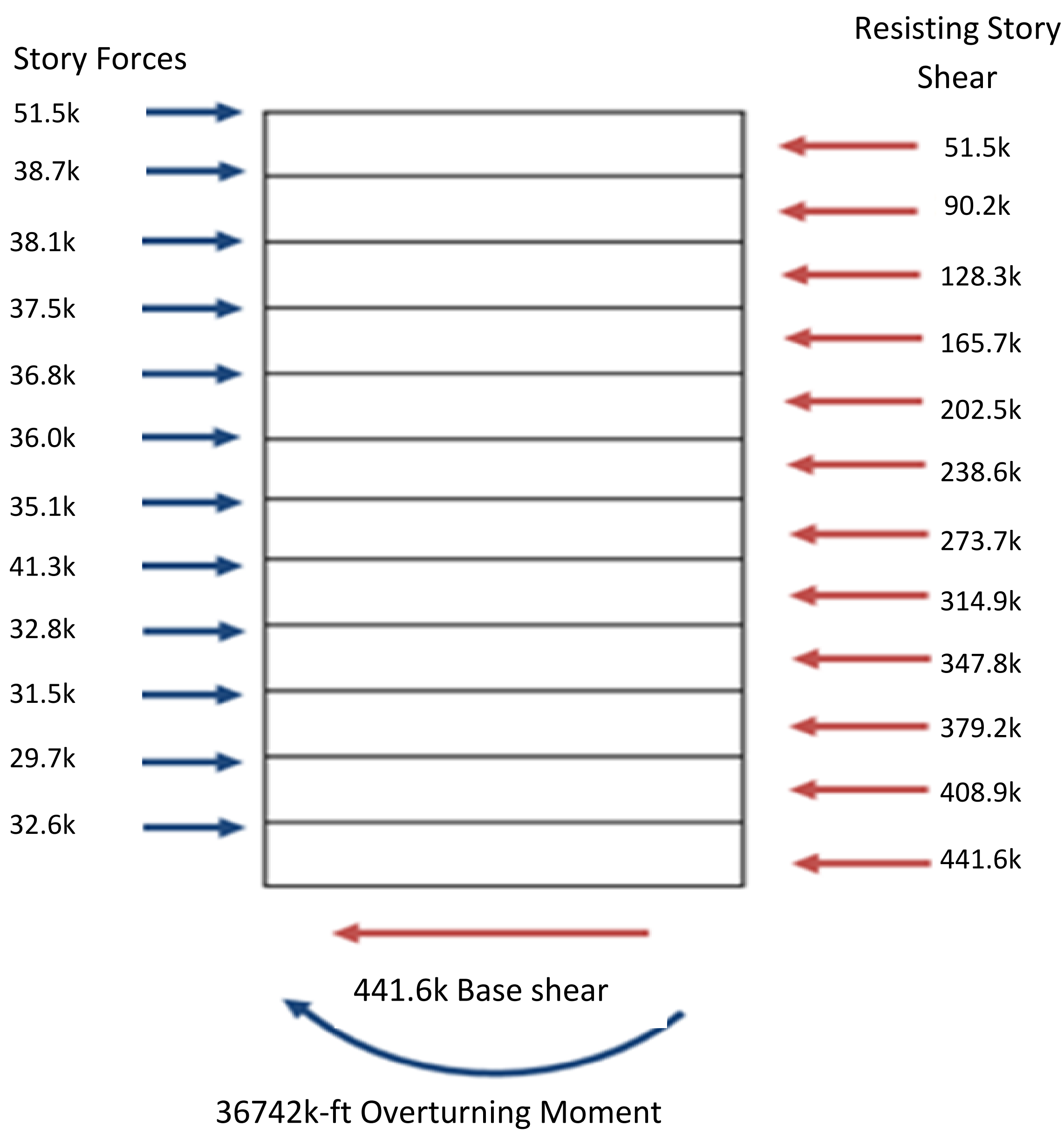


Figure 12: North – South Wind

(Table 5) North - South Direction										
Floor	Height (ft)	Story Height (ft)	Kz	qz	Wind Pressures (psf)			Story Force (Kips)	Story Shear (Kips)	Overturning moment (kips - Ft)
					Wind N-S	Lee N-S	Total N-S			
Roof	146	15	1.102	34.53	23.48	-14.67	38.15	51.50	0.00	0.00
11	131	11.5	1.067	33.43	22.73	-14.67	37.41	38.71	51.50	7519.53
10	119.5	11.5	1.038	32.52	22.11	-14.67	36.79	38.07	90.22	5071.58
9	108	11.5	1.01	31.64	21.52	-14.67	36.19	37.46	128.29	4549.95
8	96.5	11.5	0.9795	30.69	20.87	-14.67	35.54	36.79	165.75	4045.41
7	85	11.5	0.945	29.61	20.13	-14.67	34.81	36.02	202.54	3549.75
6	73.5	11.5	0.904	28.32	19.26	-14.67	33.93	35.12	238.56	3062.06
5	59.5	14	0.848	26.57	18.07	-14.67	32.74	41.25	273.68	2581.34
4	48	11.5	0.8	25.06	17.04	-14.67	31.72	32.83	314.93	2454.48
3	36.5	11.5	0.739	23.15	15.74	-14.67	30.42	31.48	347.76	1575.70
2	25	11.5	0.66	20.68	14.06	-14.67	28.73	29.74	379.24	1149.09
1	13.5	13.5	0.57	17.86	12.14	-14.67	26.82	32.58	408.98	743.50
Ground	0	0	0	0.00	0.00	0.00	0.00	0.00	441.56	439.86
									Sum	36742.27

(Table 6) East - West Direction										
Floor	Height (ft)	Story Height (ft)	Kz	qz	Wind Pressures (psf)			Story Force (Kips)	Story Shear (Kips)	Overturning moment (kips - Ft)
					Wind E-W	Lee E-W	Total E-W			
Roof	146	15	1.102	34.53	23.48	-7.77	31.25	89.52	0.00	0.00
11	131	11.5	1.067	33.43	22.73	-7.77	30.50	66.99	89.52	13069.77
10	119.5	11.5	1.038	32.52	22.11	-7.77	29.88	65.64	156.51	8776.14
9	108	11.5	1.01	31.64	21.52	-7.77	29.29	64.33	222.15	7843.54
8	96.5	11.5	0.9795	30.69	20.87	-7.77	28.64	62.90	286.47	6947.22
7	85	11.5	0.945	29.61	20.13	-7.77	27.90	61.28	349.37	6069.74
6	73.5	11.5	0.904	28.32	19.26	-7.77	27.03	59.37	410.66	5209.17
5	59.5	14	0.848	26.57	18.07	-7.77	25.83	69.08	470.02	4363.39
4	48	11.5	0.8	25.06	17.04	-7.77	24.81	54.50	539.11	4110.33
3	36.5	11.5	0.739	23.15	15.74	-7.77	23.51	51.64	593.60	2615.96
2	25	11.5	0.66	20.68	14.06	-7.77	21.83	47.95	645.25	1885.03
1	13.5	13.5	0.57	17.86	12.14	-7.77	19.91	51.34	693.20	1198.69
Ground	0	0	0	0.00	0.00	0.00	0.00	0.00	744.54	693.12
									Sum	62782.10

Seismic Loads

In order to calculate the seismic loading of the Patient Tower ASCE 7-10 was referenced. Chapters 11, 12, 20-22 were all used to find parameters, procedures and references to complete the analyses of the seismic loading. Located in the geotechnical report the site classification was determined to be Class D for the Patient Tower in Virginia. All design parameters that were used in this analysis of the seismic loading of the Patient Tower can be found in Appendix III. Sample seismic calculations along with spreadsheets with total building calculations will also be located in Appendix III. Table 8 includes a summary of the story forces as well as the story shears from the seismic analyses.

(Table 7) General Seismic Information		
Occupancy		III
Site Class		D
Seismic Design Category		B
Short Period Spectral Response	S_s	13.5 % g
Spectral Response (1 Sec.)	S_1	5.5% g
Maximum Short Period Spectral Response	S_{MS}	0.216
Maximum Spectral Response (1 Sec.)	S_{M1}	0.132
Design Short Spectral Response	S_{DS}	0.144
Design Spectral Response (1 Sec.)	S_{D1}	0.088
Response Modification Coefficient	R	6
Seismic Response Coefficient	C_S	0.0218
Effective Period	T	0.84

(Table 8) Base Shear and Overturning Moment Distribution									
Floor	Height h_x (ft)	Story Height (ft)	Story Weight w_x (lbs)	h_x^k	$w_x*h_x^k$	C_{vx}	Lateral Force F_x (Kips)	Shear Force V_x (Kips)	Moment M_x (Kips - ft)
Roof	146	15	2022	340.64	688769.63	0.10	99.40	0.00	0.00
11	131	11.5	3472	300.06	1041806.67	0.16	150.35	99.40	14512.25
10	119.5	11.5	3472	269.48	935621.44	0.14	135.02	249.75	19695.47
9	108	11.5	3472	239.39	831161.68	0.13	119.95	384.77	16135.26
8	96.5	11.5	3472	209.84	728579.04	0.11	105.14	504.72	12954.40
7	85	11.5	3472	180.89	628058.16	0.09	90.64	609.86	10146.40
6	73.5	11.5	3472	152.60	529829.22	0.08	76.46	700.50	7704.18
5	59.5	14	3472	119.17	413775.30	0.06	59.71	776.96	5619.93
4	48	11.5	3472	92.69	321834.03	0.05	46.45	836.67	3552.95
3	36.5	11.5	3472	67.28	233594.37	0.04	33.71	883.12	2229.36
2	25	11.5	4524	43.21	195484.54	0.03	28.21	916.83	1230.45
1	13.5	13.5	4524	21.01	95063.35	0.01	13.72	945.04	705.28
Ground	0	0	1450	0	0.00	0.00	0.00	958.76	185.21
$\sum(w_xh_x^k)$ = 6643577.42			$\sum F_x$ = Base Shear = 959 Kips			Overturning Moment = 94671 Kips - Ft			

Load Combinations

The load combinations used for the analysis are listed below. These combinations must be considered during design per ASCE7-10

1. 1.4D

2. 1.2D + 1.6L + 0.5(Lr or S or R)

3. 1.2D + 1.6(Lr or S or R) + (L or 0.5W)

4. 1.2D + 1.0W + L + 0.5(Lr or S or R)

5. 1.2D + 1.0E + L + 0.2S

6. 0.9D + 1.0W

7. 0.9D + 1.0E

Load Distribution

Center of Rigidity

The center of rigidity is calculated using the stiffness of each of the 8 shear walls that are found in the patient tower. These shear walls are located surrounding the staircase and the elevator shaft of the tower; both of these cores are comprised of 12 inch thick reinforced concrete walls. These walls vary in length and are located different distances for the center of rigidity of the building. The thickness, height and distance from the center of rigidity all affect the center of rigidity and altering the relative stiffness of each wall.

The center of the rigidity was calculated by the computer model in ETABS as well as by hand, both the ETABS and the hand calculations are compared below in table # and more detailed hand calculations can be found in appendix IV.

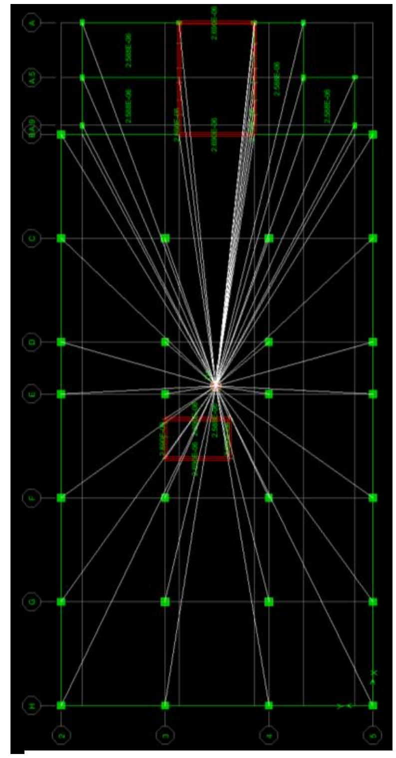


Figure 13: Center of Rigidity ETABS

(Table 9) Center of Mass and Center of Rigidity (Etabs Vs. Hand Calculation)

Story	Etabs		By Hand		Etabs		By Hand	
	COM y-direction (in)	COM x-direction (In)	COM y-direction (in)	COM x-direction (In)	COR y - Direction (In)	COR x - Direction (In)	COR y – Direction (In)	COR x - Direction (In)
Roof	1070.022	526.445	1146	540	1571.126	673.014	1550	567
STORY11	1070.795	526.391	1146	540	1570.733	672.138	1550	567
STORY10	1071.042	526.373	1146	540	1569.95	669.911	1550	567
STORY9	1071.342	526.364	1146	540	1569.202	667.521	1550	567
STORY8	1071.521	526.358	1146	540	1568.202	664.431	1550	567
STORY7	1071.641	526.354	1146	540	1566.727	660.399	1550	567
STORY6	1071.624	526.352	1146	540	1564.439	654.943	1550	567
STORY5	1071.611	526.351	1146	540	1559.584	645.234	1550	567
STORY4	1071.68	526.349	1146	540	1552.568	633.407	1550	567
STORY3	1071.687	526.347	1146	540	1540.499	615.138	1550	567
STORY2	1071.648	526.345	1146	540	1520.063	586.031	1550	567
STORY1	1072.417	526.237	1146	540	1472.727	543.097	1550	567

Torsion

When the center of rigidity and the center of mass are not at the same location, torsion is present in the structure. Eccentricity is the difference between the center of mass and the center of rigidity. The eccentricity of the structure allows that development of moments and torsional shear is then introduced as an additional force on the building.

For rigid diaphragms, two separate moments need to be taken into account when determining torsion in a building. Torsion in a rigid diaphragm is the sum of the inherent moment and the accidental moment. The accidental moment, M_{ta}, is due to the rigidity of the slab. The accidental moment takes into account an assumed displacement of the center of mass. The displacement is a distance equal to 5% of the center of mass dimension each way from the actual location perpendicular to the direction of the applied force. The inherent moment, M_t, is caused by the eccentricity between the center of rigidity and the center of mass. The lateral force exerted on the building at that level; times the eccentricity of the floor gives the inherent moment.

(Table 10) Overall Building Torsion					
North - South Direction					
Story	Factored Lateral Force (k)	COR-COM (ft)	M_t (Ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Roof	82.4	-2.19	-180.46	370.80	190.34
11	61.936	-2.19	-135.64	278.71	143.07
10	60.96	-2.19	-133.50	274.32	140.82
9	59.936	-2.19	-131.26	269.71	138.45
8	58.864	-2.19	-128.91	264.89	135.98
7	57.6	-2.19	-126.14	259.20	133.06
6	56.16	-2.19	-122.99	252.72	129.73
5	66.08	-2.19	-144.72	297.36	152.64
4	52.48	-2.19	-114.93	236.16	121.23
3	50.4	-2.19	-110.38	226.80	116.42
2	47.52	-2.19	-104.07	213.84	109.77
1	52.16	-2.19	-114.23	234.72	120.49

(Table 11) Overall Building Torsion					
East - West Direction					
Story	Factored Lateral Force (k)	COR-COM (ft)	M_t (Ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Roof	143.232	-33.7	-4826.92	1367.87	-3459.05
11	107.184	-33.7	-3612.10	1023.61	-2588.49
10	105.024	-33.7	-3539.31	1002.98	-2536.33
9	102.928	-33.7	-3468.67	982.96	-2485.71
8	100.64	-33.7	-3391.57	961.11	-2430.46
7	98.048	-33.7	-3304.22	936.36	-2367.86
6	95.04	-33.7	-3202.85	907.63	-2295.22
5	110.56	-33.7	-3725.87	1055.85	-2670.02
4	87.2	-33.7	-2938.64	832.76	-2105.88
3	82.624	-33.7	-2784.43	789.06	-1995.37
2	76.72	-33.7	-2585.46	732.68	-1852.79
1	82.08	-33.7	-2766.10	783.86	-1982.23

Shear

In order to calculate the shear forces at each level of the patient tower, direct and torsional forces need to be accounted for. The combination of the two forces is the total shear that the building will be experiencing. Direct shear is related to the stiffness of each of the shear walls and there relative stiffness as compared to each of the walls. The torsional shear is caused by the variations in location of each wall from the center of mass.

Direct Shear

The lateral forces that are acting on the building must be distributed to each of the frame elements so that they can be transferred down the load paths. The story shear that is applied at each story of the building is then distributed to the shear elements found at each floor. Depending on the relative stiffness of each of the shear elements depends then on how much of the force at that story is distributed to the wall. The greater the stiffness of the shear element the greater the load the wall can receive. The direct shear that is applied to each wall can be seen below in table's 12 and 13. Detailed calculations of these values can also be found in Appendix V.

(Table 12) North - South Direction						
Load Combinations 1.2D + 1.0L + 1.0W + 1.0E	Force (k)	Factored Force (k)	Distributed Force (k)			
			Wall 2-1	Wall 2-2	Wall 3-1	Wall 3-2
Roof	51.50	51.50	3.61	3.61	22.20	22.20
11	38.71	38.71	2.71	2.71	16.69	16.69
10	38.07	38.07	2.67	2.67	16.41	16.41
9	37.46	37.46	2.62	2.62	16.14	16.14
8	36.79	36.79	2.57	2.57	15.85	15.85
7	36.02	36.02	2.52	2.52	15.53	15.53
6	35.12	35.12	2.46	2.46	15.14	15.14
5	41.25	41.25	2.89	2.89	17.78	17.78
4	32.83	32.83	2.30	2.30	14.15	14.15
3	31.48	31.48	2.20	2.20	13.57	13.57
2	29.74	29.74	2.08	2.08	12.82	12.82
1	32.58	32.58	2.28	2.28	14.04	14.04

(Table 13) East - West Direction						
Load Combinations 1.2D + 1.0L + 1.0W + 1.0E	Force (k)	Factored Force (k)	Distributed Force (k)			
			Wall 1-1	Wall 1-2	Wall 4-1	Wall 4-2
Roof	89.52	89.52	20.41	20.41	24.26	24.26
11	66.99	66.99	15.27	15.27	18.16	18.16
10	65.64	65.64	14.97	14.97	17.79	17.79
9	64.33	64.33	14.67	14.67	17.43	17.43
8	62.90	62.90	14.34	14.34	17.05	17.05
7	61.28	61.28	13.97	13.97	16.61	16.61
6	59.37	59.37	13.54	13.54	16.09	16.09
5	69.08	69.08	15.75	15.75	18.72	18.72
4	54.50	54.50	12.43	12.43	14.77	14.77
3	51.64	51.64	11.77	11.77	14.00	14.00
2	47.95	47.95	10.93	10.93	12.99	12.99
1	51.34	51.34	11.71	11.71	13.91	13.91

Torsional Shear

Torsion Shear is created by distance of the wall element from the center of rigidity where the lateral force is acting. The shear walls within the building will have to resist a torsional shear force that will be distributed to them in the same way as the direct shear, where the greater the relative stiffness the greater the shear force on that wall. The torsional shear forces were determined for the shear walls supporting story 6 and can be found in table 14. Detailed calculations of how the torsional shear was calculated can be found in Appendix V.

(Table 14) Torsional Shear at Story 6 N-S Direction							
		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance From COM to COR e (in)	Distance from $Wall_i$ to COR d_i (in)	$(R_i)(d_i)^2$	Torsional Shear (k)
Wall 1-1	E-W	657.1	0.228	-404.3	707.3	114062.3	-107.875
Wall 1-2	E-W	657.1	0.228	-404.3	603.3	82985.36	-92.013
Wall 4-1	E-W	657.1	0.271	-404.3	-366.7	36441.07	66.47543
Wall 4-2	E-W	657.1	0.271	-404.3	-735.7	146680	133.3678
Wall 2-1	N - S	381.7	0.07	-26.1	-165.7	1921.954	0.290957
Wall 2-2	N - S	381.7	0.07	-26.1	68.3	326.5423	-0.11993
Wall 3-1	N - S	381.7	0.431	-26.1	-122.7	6488.83	1.32657
Wall 3-2	N - S	381.7	0.431	-26.1	138.3	8243.69	-1.49523
					Sum	397149.7	

Strength Check

Shear Strength Check

With the direct shear forces and the torsional forces acting on each shear wall, a check needs to be done on each wall to determine if the reinforcement is sufficient to support the loads. Shear strength calculations done on the shear walls supporting Floor 6 were conducted and detailed calculations can be found in Appendix VI. Each shear wall was within the capacity determined by the shear strength. The reinforcement for each wall proved to be adequately designed. The shear wall checks and verifications can be found in Table 15.

(Table 15) Shear Wall Strength Check (supporting Floor 6)												
Wall	Direct Shear (k)	Torsional Shear (k)	V_u (k)	Vertical Reinf.	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in2)	α_c	ρ_t	ϕV_c (k)	
Wall 1-1	13.5354	-107.875	-94.3393	(2) #5	24	234	12	2808	2	0.002153	569.8134	Adequate
Wall 1-2	13.5354	-92.013	-78.4776	(2) #5	24	234	12	2808	2	0.002153	569.8134	Adequate
Wall 4-1	16.08813	66.47543	82.56356	(2) #5	24	261	12	3132	2	0.002153	635.5611	Adequate
Wall 4-2	16.08813	133.3678	149.4559	(2) #5	24	261	12	3132	2	0.002153	635.5611	Adequate
Wall 2-1	2.458416	0.290957	2.749373	(2) #5	24	128	12	1536	2	0.002153	311.6928	Adequate
Wall 2-2	2.458416	-0.11993	2.338486	(2) #5	24	128	12	1536	2	0.002153	311.6928	Adequate
Wall 3-1	15.13682	1.32657	16.46339	(2) #5	24	375	12	4500	2	0.002153	913.1625	Adequate
Wall 3-2	15.13682	-1.49523	13.64159	(2) #5	24	375	12	4500	2	0.002153	913.1625	Adequate

Displacement and Drift

The overall drift of the building should be limited as much as possible due to comfortability inside the structure. Building drift falls under the serviceability considerations and is related to the rigidity of each of the buildings shear walls. As a structure gets taller the more important the overall drift of the building becomes and a larger of a factor it will be. The drift limitation for wind loading is an allowable drift of Δ = L/400. The seismic drift is limited to an allowable drift of $\Delta = 0.015h_{sx}$.

$$\Delta_{Limit} = 1722''/400 = 4.305''$$

One wall was analyzed at each floor to determine an approximate story displacement and story drift, adding up to overall building drift. A hand calculation was done to determine the displacements on each floor. The hand calculations done were determined using the following equation:

$$\Delta_{Cantilever} = \Delta_{Flextural} + \Delta_{Shear}$$

The ETABS model also analyzed the story drift of the building. The drifts for the patient tower were taken both the North – South and East – West directions. The drift in the N/S direction is 0.3382" and 1.23"in the E/W direction. The drifts in both directions are less than the 4.3" limitation. In order to computer the story drift and displacements of all the shear walls working together by hand would be beyond the scope of this assignment. The ETABS modal analysis does analyze the drift and displacements with all the shear walls working together as a lateral resisting system. The values computed by hand can't be directly compared with the ETAB results due to this difference in analysis parameters.

The hand calculations used to determine the drift for wall 3-1 can be found in Appendix VI

Overturning Analysis

The lateral forces against the building result in overturning moments. These moments cause a rotational force that acts against the foundations in an reaction to overturn the structure. The dead load of the tower would serve as the system to resist the overturning. Since the earthquake loading is providing the largest lateral force it would control in the overturning analysis. This lateral force applied to the building would be resisted by the dead weight of the building acting on the foundations. A rough estimate was done to check if the overturning would be an issue to the patient tower. The stresses from the lateral loads were compared with the stresses due to the self-weight of the building. The stresses for the lateral loads are a small fraction of the stresses for the dead loads which will provide minimal overturning effects on the foundation. Detailed calculations of the overturning check can be found in Appendix VI.

Conclusion

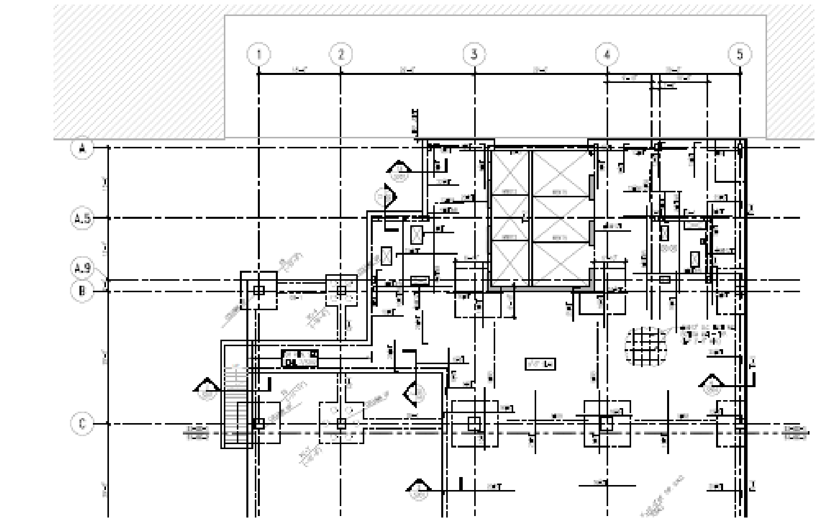
In analyzing the existing lateral system of the patient tower, the loads determined in Tech Report #1 were applied to the later system of the building. ASCE 07-10 was used in determining the load combinations that would be used in this analysis. The controlling load combinations were determined using the ETABS model output, which gave 1.2D + 1.0L + 1.0W + 1.0E as the controlling load case in both the North-south and East-west directions.

The ETABS model was used as a reference and in comparison to verify that the model and hand calculations were providing similar outputs that were also reasonable. During the hand calculations only previous information that was calculated by hand was used in order to maintain consistence and to not move back and forth between ETABS output and hand calculations. Also, with this being the first attempt at using ETABS to model the building, there was some uncertainty as to whether everything was input under the proper assumptions that the hand calculations made.

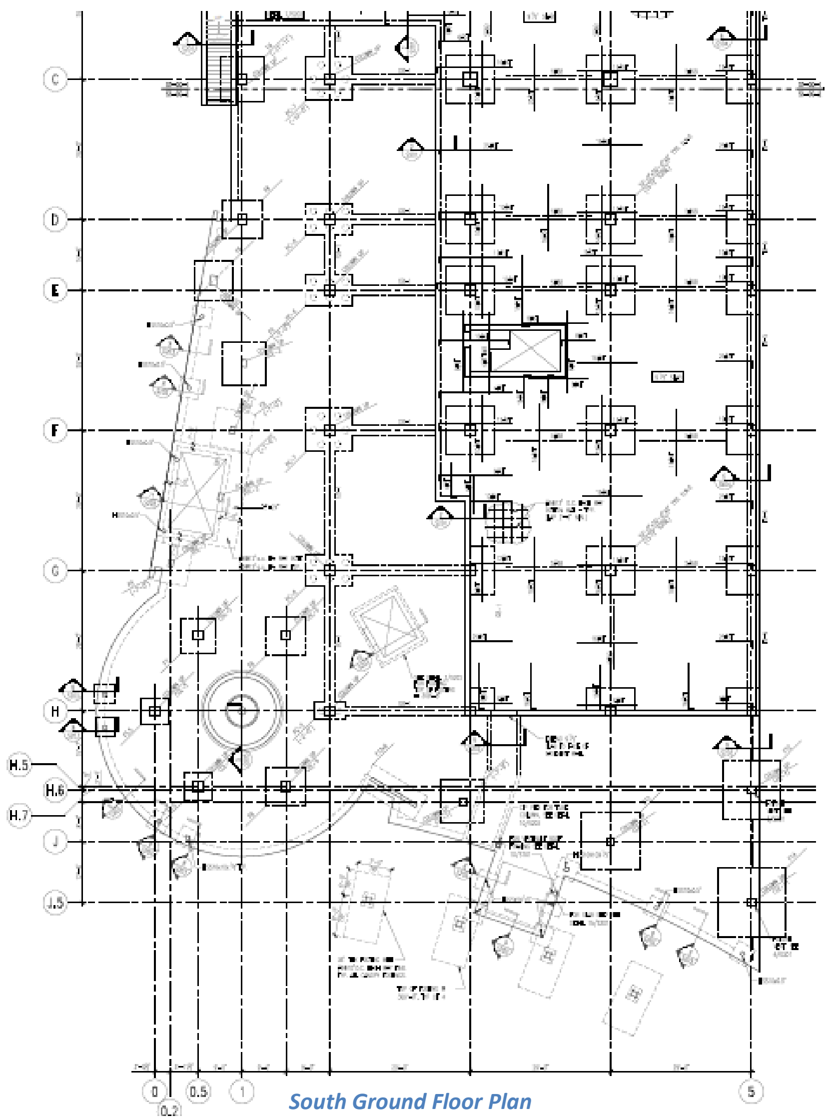
Through this analysis, it confirmed that the lateral resisting system in the patient tower is sufficient to support the loads generated in that area. The original design of this building was done using ASCE 07 -05 but I used a new version of the code ASCE 07-10 which in turn increased the loads that were applied to the structure. Even with the increase in loads by using the new version of ASCE the lateral system was still adequate in resisting the applied loads. The center of rigidity and center of mass in the tower were calculated to be in different locations producing a torsional effect in the lateral system. With the addition of the torsional shear to the direct shear the existing wall were found to adequately support that shear affects. The overall building drift was determined using the ETABS model to be within the allowable limits of the building determined by the code. A check for overturning was completed to find that overturning was present in the building due to the lateral loads on the tower. It was found after a stress check that the self-weight of the building resisting these loads makes the issue irrelevant due to over powering dead load. The overall analysis of the patient tower has determined that the shear wall cores in the building are satisfactory to resist the various loads that are present on the building.

Appendix I

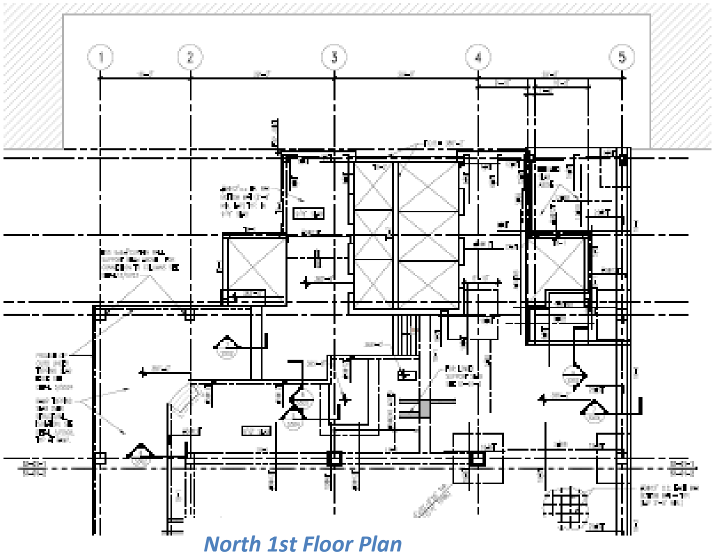
This section of Technical Report #3 is where the supplementary information for the layout and design for the Hospital Patient Tower can be found.



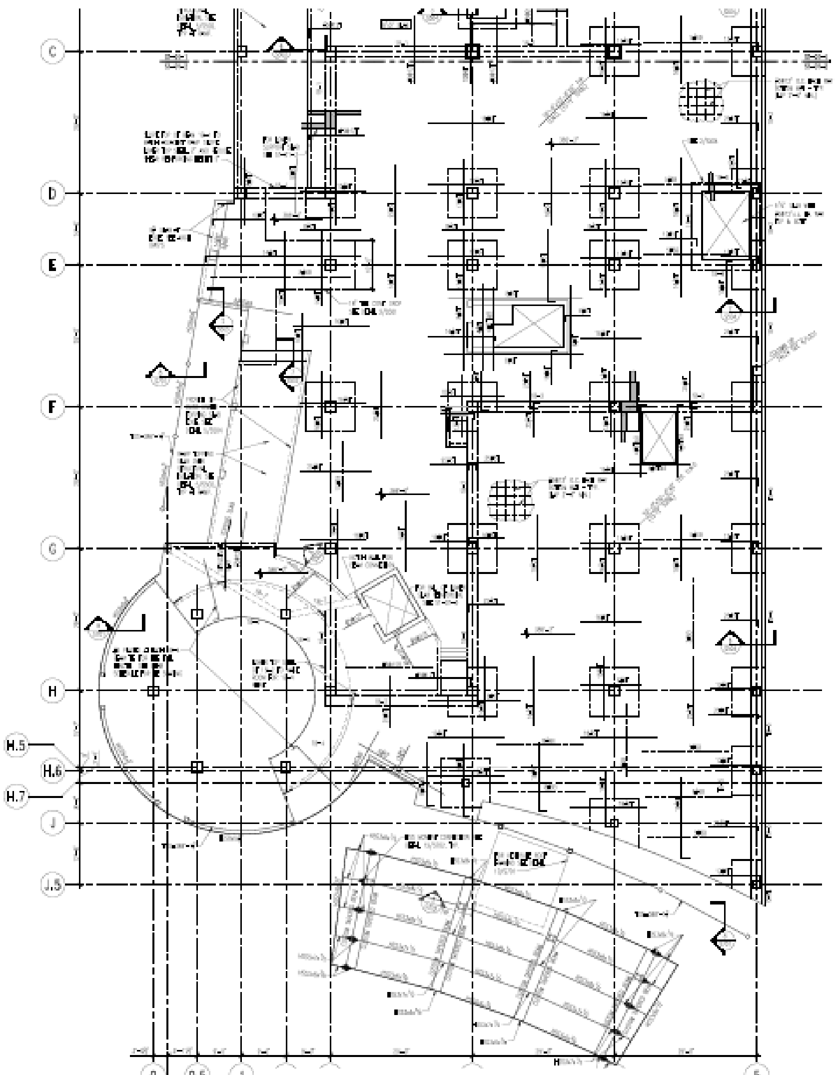
North Ground Floor Plan



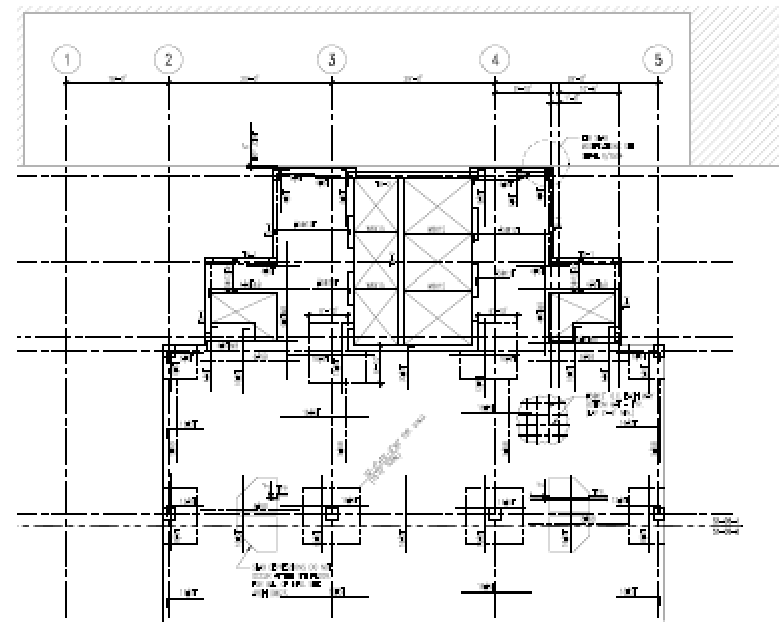
South Ground Floor Plan



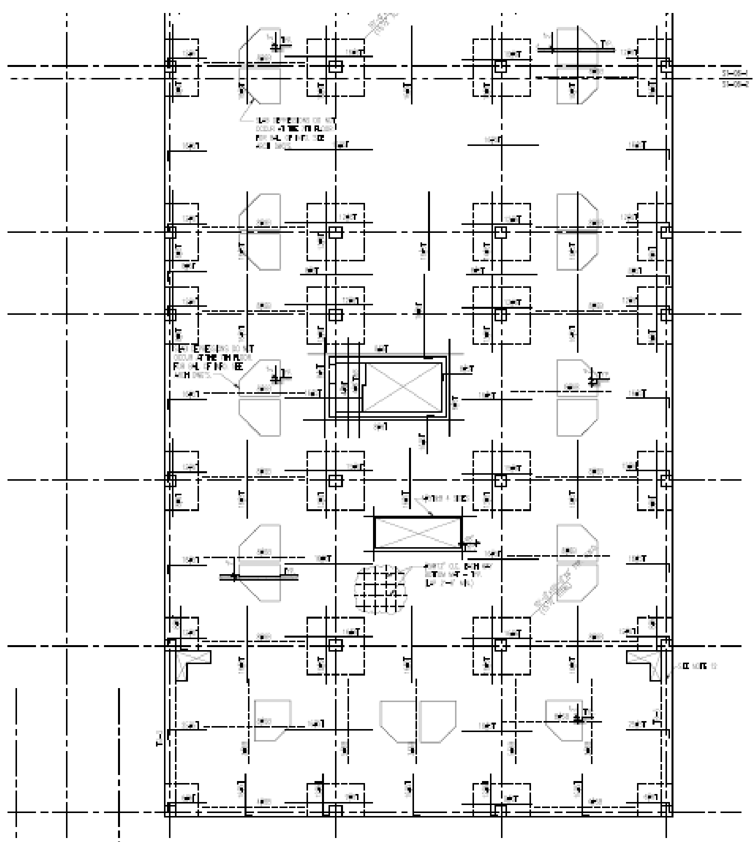
North 1st Floor Plan



South 1st Floor Plan



North Typical Floor Plan



South Typical Floor Plan

Appendix II

This section of Technical Report #3 is where the supplementary information for analysis of the wind forces acting on the building can be found.

Wind Load Parameters	
Wind directionality factor (k_d)	0.85
Exposure Category	B
Topographic Factor (K_{zt})	1.0
Gust Effect Factor (G)	0.85
Enclosure classification	Partially Enclosed
Internal pressure coefficient (GC_{pi})	± 0.55

Building Information	
Number of Stories	12
Building Height (feet)	146
N-S Building Length (feet)	191
E-W Building Length (feet)	90
L/B in N-S Direction	2.12
L/B in E-W Direction	0.47

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Occupancy Category III

Wind Speed (V) = 120 mph (from Figure 26.5-1B ASCE 7-10)

Wind Load Parameters

$K_d = 0.85$ (table 26.6-1)

Exposure Category B (Section 26.7.3)

$K_{zt} = 1.0$

$G = 0.85$ (Sect. 29.1.1)

Partially enclosed Buildings (table 26.11-1)

GC_{pi} ± 0.18

Velocity Pressures exposure coefficient (K_z) Building Height 174'-9" (Including penthouse)

Height (ft)	Exposure
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17

Vertical Pressure (q_z) (eq 27.3-1)

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

$q_z = 0.00256\, K_z (1.0)(0.85)(120)^2$

$= 31.33\, K_z$

*See chart for calculated values

External pressure coefficient (C_p)
(Figure 27.4-1)

Wind in North - South Direction

Windward wall: $C_p = 0.8$ (use w/ q_z)

Leeward wall: ($L/B = 0.47$) $C_p = -0.5$ (use w/ q_h)

Side wall: $C_p = -0.7$ (use w/ q_h)

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Wind in East-West direction

Wind ward Wall: $C_p = 0.8$

Lee ward Wall: $(L/B = 2.1) = -0.3$

Side walls : $C_p = -0.7$

Design Wind Pressures (Eq. 27.4-1)

Windward walls

$$P = q_z G C_p - q_i (G C_{pi})$$

N-S

$$P = (0.85)(0.8)\, q_z \cancel{\pm (0.18)\, q_h} = (0.68)\, q_z$$

E-W

$$P = (0.85)(0.8)\, q_z \cancel{\pm (0.18)\, q_h} = (0.68)\, q_z$$

Leeward walls

$$P = q_h G C_p - q_h (G C_{pi})$$

N-S

$$P = q_h (0.85)(-0.5) - \cancel{q_h (\pm 0.18)} = (-0.425)\, q_h$$

E-W

$$P = q_h (0.85)(-0.3) - \cancel{q_h (\pm 0.18)} = (-0.281)\, q_h$$

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Natural frequency N-S

building height 146' < 300' ✓

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} \quad \frac{27886}{146} = 191 \times 4 = 764 > 146 \checkmark$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1+0.83\left(\frac{h_i}{D_i}\right)^2}$$

$A_B = 17190\ ft^2$
$h = 146'$
$h_i = 146'$

$$n_a = 385\,(C_w)^{0.5}\,h = \boxed{3.45 > 1 \quad \text{Rigid Structure}}$$

E-W

$$L_{eff} = \frac{13140}{146} = 90 \times 4 = 360 > 146 \checkmark$$

$$\boxed{n_a = 2.168 > 1 \quad \text{Rigid Structure}}$$

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Moment Calculations

N-S Direction

$$M = 68.3^k(139.25') + 52.1^k(124.75') + 49.26^k(112.75') + 51.9^k(101.25') + 50.9^k(89.75') + 50.1^k(78.25') + 47.8^k(66.75') + 60.7^k(52.75') + 46.7^k(41.25') + 45.3^k(29.75') + 42.1^k(18.75') + 47.7^k(6.75')$$

$$\boxed{= 46237.44 \text{ k-ft}}$$

E-S Direction

$$M = 58.95(139.25') + 44.9(124.75') + 43.83(112.75) + 44.79(101.25') + 43.75(89.75) + 42.9(78.25') + 40.7(66.75') + 52.0(52.75') + 39.5(41.25') + 38.1(29.75') + 34.96(18.75') + 39.3(6.75')$$

$$\boxed{= 39713.72 \text{ k-ft}}$$

Appendix III

This section of Technical Report #3 is where the supplementary information for the seismic force acting on the Hospital Patient Tower can be found.

Tech Report #3 | Seismic Design | Patient Tower

Seismic Loading

Spectral response acceleration

$S_s = 13.5\% g \quad S_1 = 5.5\% g$ (Figure 22-1)

Site Class - D $\quad F_a = 1.6$ (11.4-1)
$F_v = 2.4$ (11.4-2)

ordinary reinforced concrete Shear walls
$R = 6$
$\Omega = 2\frac{1}{2}$ (12.2-1)
$C_d = 5$

$S_{ms} = F_a S_s = 1.6(0.135) = \boxed{0.216}$

$S_{m1} = F_v S_1 = 2.4(0.055) = \boxed{0.132}$

$S_{DS} = \frac{2}{3} S_{ms} = \boxed{0.144}$ (11.4-3)

$S_{D1} = \frac{2}{3} S_{m1} = \boxed{0.088}$ (11.4-4)

$T = C_t h_n^x = 0.02(146')^{0.75} = \boxed{0.84}$ (12.8-2)

$T_o = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 \frac{0.088}{0.144} = \boxed{0.122}$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.088}{0.144} = \boxed{0.611}$

$T_L = 8$

For Periods

$< T_o \quad S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_o}\right) = \boxed{0.65}$

$> T_o, < T_s \quad S_a = S_{DS} = \boxed{0.144}$

$> T_s, < T_L \quad S_a = \frac{S_{D1}}{T} = \boxed{0.105}$

$> T_L \quad S_a = \frac{S_{D1} T_L}{T^2} = \boxed{0.997}$

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Seismic design Category

Occupancy category III $\quad I_e = 1.25$

$S_{DS} = 0.144 \rightarrow$ Category A
$S_{D1} = 0.088 \rightarrow$ Category B

} used category B (12.6-1,2)

$$C_{s\,min} \begin{cases} \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.088}{0.84\left(\frac{6}{1.25}\right)} = \boxed{0.0218} \quad \text{Controles} \\ \dfrac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} = \dfrac{0.088}{(0.84)^2\left(\frac{6}{1.25}\right)} = 0.0259 \\ \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.144}{\left(\frac{6}{1.25}\right)} = 0.3 \end{cases}$$

$$C_s = 0.0218$$

$$f = \frac{1}{T} = \frac{1}{0.84} = 1.19 > 1 \quad \text{Rigid Diaphragm}$$

Building Dead load Weight

$$W_t = 43{,}980\ k$$

Equivalent lateral force procedure (12.8)

$$V = C_s W \quad = 0.0218(43{,}980) = 958.76^k$$

base shear

Tech Report #3 | Seismic Design | Patient Tower

Vertical distribution (12.8.3)

$F_x = C_{vx} V$

$k = 0.75 + 0.5(T) = 1.17$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

See excel sheet

$\sum W_i h_i^k = 6643577.42$

Seismic Story Forces + Shear

Story Forces	Level	Story Shear
99.4^k	Roof (146')	
		99.4^k
150.35^k	Floor 11 (131')	
		249.75^k
135.02^k	Floor 10 (119.5')	
		384.77^k
119.95^k	Floor 9 (108')	
		504.72^k
105.14^k	Floor 8 (96.5')	
		609.86^k
90.64^k	Floor 7 (85')	
		700.5^k
76.46^k	Floor 6 (73.5')	
		776.96^k
59.71^k	Floor 5 (59.5')	
		836.67^k
46.45^k	Floor 4 (48')	
		883.12^k
33.71^k	Floor 3 (36.5')	
		916.83^k
28.21^k	Floor 2 (25')	
		945.04^k
13.72^k	Floor 1 (13.5')	
		958.76^k
	Ground (0')	

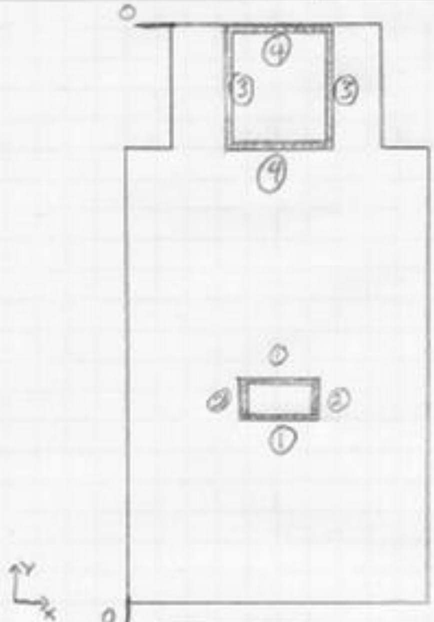
958.76^k

94671.14 k-ft

Appendix IV

This section of Technical Report #3 is where the supplementary information for the Center of Rigidity and Center of Mass calculations for the Hospital Patient Tower can be found.

Tech Report # 3 | Center of Rigidity | Patient Tower



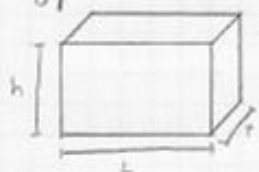
Rigidity

Typical floor Shear wall layout

$$R = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$$

$$E = 57000\sqrt{f'_c}$$

$$= 57000\sqrt{5000} = 403 \times 10^6 \text{ psi}$$

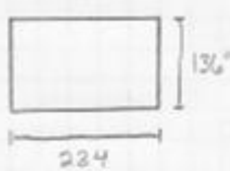


h = height at each Story

L = Length of wall

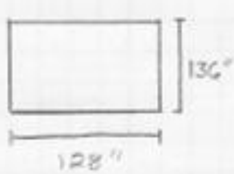
t = 12" thick

Wall #1



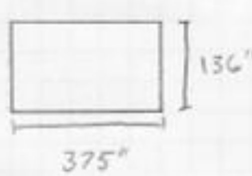
$$R = \frac{(4.03\times10^3 \text{ ksi})(12")}{4\left(\frac{136}{224}\right)^3 + 3\left(\frac{136}{224}\right)} = 19114.6$$

Wall #2



$$R = \frac{(4.03\times10^3 \text{ ksi})(12)}{4\left(\frac{136}{128}\right)^3 + 3\left(\frac{136}{128}\right)} = 6056.4$$

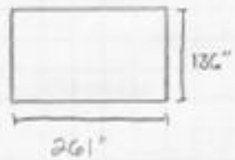
Wall #3



$$R = \frac{(4.03\times10^3 \text{ ksi})(12)}{4\left(\frac{136}{375}\right)^3 + 3\left(\frac{136}{375}\right)} = 37810.79$$

Tech Report # 3 | Center of Rigidity | Patient Tower

Wall # 4



$$R = \frac{(4.03 \times 10^{3} ksi)(12")}{4\left(\frac{136}{261}\right)^3 + 3\left(\frac{136}{261}\right)} = 22704.2$$

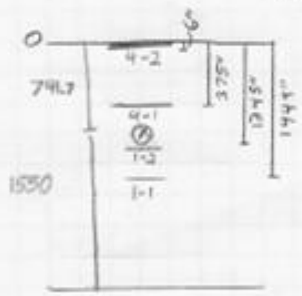
Y - Coordinate

$\Sigma R = 19114.6(2) + 22704.2(2) = 83637.6$

Relative stiffness

$\% \frac{R}{\Sigma R}$ Wall # 1 $\frac{19114.6}{83637.6} = 22.8\%$

Wall # 4 $\frac{22704.2}{83637.6} = 27.1\%$



$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{22704.2(5") + 22704.2(375) + 19114.6(1345) + 19114.6(1449)}{83637.6}$$

$= 741.7" = $ Y-coordinate

$(191 \times 12) - 741.7" = 1550$ to compare to Etabs

X - Coordinate

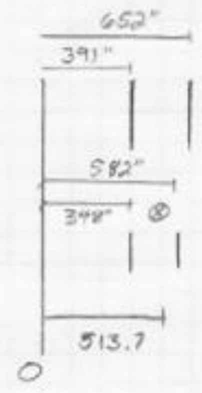
$\Sigma R = 2(6056.4) + 2(37810.79) = 87734.4$

Relative Stiffness

$\% \frac{R}{\Sigma R}$ Wall # 2 $\frac{6056.4}{87734.4} = 7\%$

Wall # 3 $\frac{37810.8}{87734.4} = 43.1\%$

Tech Report # 3 | Center of Rigidity | Patient Tower



$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{(348)(6056.4) + 582(6056.4) + (391)(37810.8) + 652(37810.8)}{87734.4}$$

$= 513.7" =$ X-Coordinate $\quad (90 \times 12) - 513.7 = 567"$ to compare to Etabs

Center of Rigity

(513.7", 741.7")

(567", 1550") to compare to etabs

Center of mass

Since building foot print is rectangular

Center of Mass coordinates: (540", 1146")

Appendix V

This section of Technical Report #3 is where the supplementary information for the shear force calculations for the Hospital Patient Tower can be found.

Tech Report #3 | Shear | Patient Tower

Shear

- Controlling loads

North/South: $1.2D + 1.0L + 1.0W + 1.0E$

East/West: $1.2D + 1.0L + 1.0W + 1.0E$

- Direct shear

$= (\text{factored Story Force}) \times \dfrac{\text{relative Stiffness \%}}{100}$

example: Floor 6 in N/S direction @ Wall 2-1

$35.12^k \times 0.07 = 2.46^k$

- direct shear values for each floor can be found in tables

- Torsional Shear

$T = \dfrac{V_{tot}\, e\, d_i R_i}{J}$

V_{tot} = Story shear
e = distance from COM to COR
d_i = distance from COR to element
R_i = relative stiffness
J = Torsional moment of Inertia

example Wall 3-1 supporting floor 6

factored Story Shear $= (1.6 \times 238.56) = 381.69^K$

center of Rigidity (x-coord) = 513.7"

Center of mass (x-coord) = 540"

$e = 513.7 - 540 = -26.1$

$R_i = 0.431$

Tech Report #3 | Shear | Patient Tower

Torsion

- Overall Building torsion

$M_{t,tot} = M_t + M_{ta}$

Factored lateral force = 1.6 W

W = (force of wind pressure @ story)

→ M_t = (factored lateral force) × (eccentricity)

e = COR − COM

example @ floor 6 N/S Direction

$e = 513.7'' - 540'' = -26.3'' = -2.19'$

Factored lateral force = 1.6 (41.25) = 66.1 K

$M_t = 66.1 \times -2.19 = -144.7$ K·ft

M_{ta} = (factored lateral force) (5% assumed displacement each way of center of Mass)

example @ floor 6 N/S Direction

center of mass = 540"

5% in each direction = 54" = 4.5'

factored lateral force = 66.1 K

$M_{ta} = 66.1\ K \times 4.5' = 297.4$ K·ft

$M_{t,tot} = M_t + M_{ta} = -144.7^{K\cdot ft} + 297.4^{K\cdot ft} = 152.6^{K\cdot ft}$

Tech Report #3	Shear	Patient Tower

location of wall 3-1 = 391" (x-coord)

$d_i = \text{Wall}_i - COR_y = 391 - 513.7 = -122.7"$

$R_i \times d_i^2 = 0.431 \times (-122.7)^2 = 6488.8$

$J = 397149.7$

$$T = \frac{(381.7)(-26.1)(-122.7)(0.431)}{397149.7} = 1.3^k$$

Values for all walls supporting level 6 can be found in table

Appendix VI

This section of Technical Report #3 is where the supplementary information for Strength Check, Displacement and the Overturning analysis for the Hospital Patient Tower can be found.

Tech Report #3 | Strength Check | Patient Tower

Shear Strength

- Structural Walls shall not exceed V_n

$$\phi V_n = \phi A_c (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$$

$\phi = 0.75$

A_c = Gross area of Concrete

$\alpha_c = 2.0$ if $h_w/l_w > 2.0$

$\rho_t = \dfrac{A_v}{S \cdot h}$ S = Shear reinforcement Spacing
h = Wall thickness = 12"

example of Wall 3-1 supporting floor 6

direct Shear @ Wall 3-1 above floor 6

= 118 from direct shear table

torsional Shear = 1.32 from torsional table

$V_u = 118 + 1.32 = 119.3$

Vertical reinforcement (2) #5 @ 24" o.c.

$$\rho_t = \frac{2(0.31)}{24(12)} = 0.00215$$

$$A_{cv} = 375(12) = 4500$$

$$\phi V_n = 0.75(4500)\left[2\frac{\sqrt{5000}}{1000} + 0.0022(60)\right] = 913.16^k > 16.46^k$$

adequate ✓

Tech Report #3	Displacement	Patient Tower

Story Displacement

- an approximate method used to determine story shear

- Story Drift

$\Delta = 0.015h_{sx}$

$\Delta_{cont} = \Delta_{flexural} + \Delta_{shear}$

$\Delta_{flex} = \dfrac{Ph^3}{3E_c I}$ $\quad\quad$ $\Delta_{shear} = \dfrac{1.2\,ph}{E_r A}$

$E_c = 57000\sqrt{5000} = 4.03\times10^3$ ksi

E_r = Modulus of rigidity $= 0.4\,E_c$

$= 1.61\times10^3$ ksi

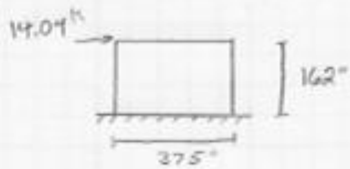
thickness 12"

$A = (\text{length})\times(\text{thickness})$ $\quad\quad$ $I = \dfrac{(\text{thickness})(\text{length})^3}{3}$

example for wall 3-1 in the N-S direction

$1.2D + 1.0L + 1.0W + 1.0E$

Floor 1 supported



$A = 12" \times 375" = 4500\,in^2$

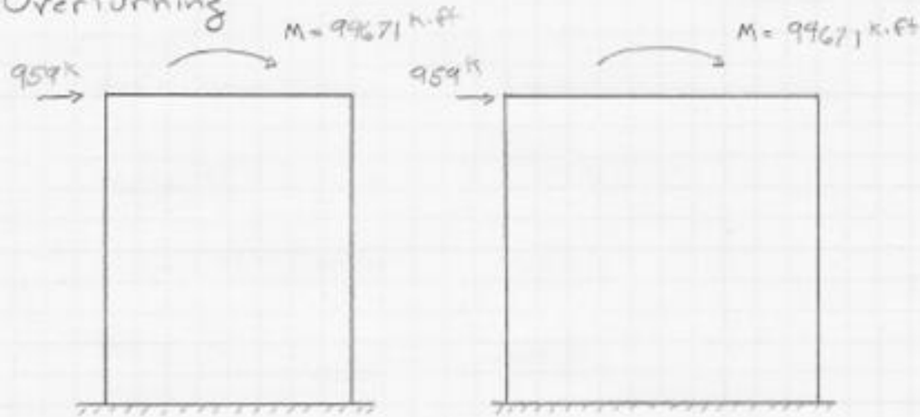
$I = \dfrac{12" \times 375"^3}{3} = 210937500$

$\Delta_1 = \dfrac{14.04^k (162)^3}{3(4.03\times10^3)(I)} + \dfrac{1.2(12.82)(162)}{1.61\times10^3(4500)} = 0.0004002\,in$

Wall 3-1 Story Displacements									
Floor Supported	Lateral Force (k)	E_c (ksi)	E_f (ksi)	Thickness (in)	Length (in)	Height (in)	Δ_{flex}	Δ_{Shear}	Story Displacement (in)
Roof	22.20	4030	1610	12	136	1722	0.0000219	0.0005	0.0005219
11	16.69	4030	1610	12	168	1586	0.0000310	0.000464	0.0004953
10	16.41	4030	1610	12	136	1418	0.0000162	0.00037	0.0003858
9	16.14	4030	1610	12	136	1282	0.0000159	0.000364	0.0003796
8	15.85	4030	1610	12	136	1146	0.0000156	0.000357	0.0003728
7	15.53	4030	1610	12	136	1010	0.0000153	0.00035	0.0003651
6	15.14	4030	1610	12	168	874	0.0000281	0.000421	0.0004493
5	17.78	4030	1610	12	136	706	0.0000175	0.0004	0.0004180
4	14.15	4030	1610	12	136	570	0.0000140	0.000319	0.0003327
3	13.57	4030	1610	12	136	434	0.0000134	0.000306	0.0003190
2	12.82	4030	1610	12	136	298	0.0000126	0.000289	0.0003014
1	14.04	4030	1610	12	162	162	0.0000234	0.000377	0.0004002
Inertia I (in^4) =		210937500		Area (in^2) =		4500	Total Displacement (in)		0.0047412

Tech Report # 3	Overturning	Patient Tower

Overturning



E-W Direction   N-S Direction

- Seismic loads control in both E-W + N-S Directions
- Overturning moments caused by the lateral forces will be counteracted by the Dead loads

Foundation area = 25,889 sf

Building Weight = 43979.8 K

Stress due to dead load

$$= \frac{\text{Building Weight}}{\text{foundation area}} = \frac{43979.8}{25889} = 1.6 \times 1000\text{ lb} = 1600\text{ psf}$$

Stress due to Seismic

$$= \frac{959^{k}(1000^{lb})}{25889} = 37.04\text{ psf} \qquad = \frac{37.04}{1600} = 2.3\% \text{ of Dead load}$$