Technical Report #1

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This Document is Technical Report #1 for 5th year senior thesis in the Architectural Engineering Departments at The Pennsylvania State University. This Report will include structural concepts and structural existing conditions for the Hospital Patient Tower.

Structural Option

Professor Behr

Hospital Patient Tower

East Coast U.S.A.

10/4/2010

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Executive Summary

Technical Report #1 is a structural systems and existing conditions analysis of the Patient Tower as an expansion of an existing patient tower. This report includes research about the structural system and the process of design. All of the calculations preformed were done in compliance with the most up to date codes for structural design.

This 12 story Patient tower is an expansion to an existing patient tower as an early stage of a large capital expansion plan. This tower utilizes piles and grade beams as a foundation with a concrete structural system. Typical Column size 24" x 24" they have varying rebar placement and design of both vertical and horizontal. Since the Patient Tower needs to line up with the existing structure the floor to floor high was a large consideration in the structural design. Patient Tower has a few design features to make it acceptable to another expansion to be started soon.

The Wind and Seismic loads were both calculated using AISC 7-10. After the calculations of these two loads it was determined that the wind controls in the East-West Direction and Seismic controls in the North-South direction.

For the Stop Checks of the tower three different elements form the towers structure were analyzed and compared to the design that was done by the structural engineer. The gravity load check of an interior column determined that the column was oversized but it was assumed that there were no lateral forces involved. A check of the drop panels was also performed against the punching shear in the slab. A check of the slab system was the third and last spot check of Tech Report #1.

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 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 taking on a more modern look by incorporating aluminum curtain wall and precast concrete panels. The new Tower consists of 12 stories above grade with one level below grade. The Tower is 216,000 Square feet with 174 patient rooms, an operation facilities 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 Hospitals 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 for the Physician, nurses and staff to help make the design as efficient as possible. Medical/surgical patients aging 65years 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 placement for monitoring stations were optimized to ensure patient privacy as well as enhancing the monitoring capabilities.

One of the hospitals goals as well as patient care is as so to lower their impact on the environment. The hospitals 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 1: Sketch by Wilmot Sanz

Structural Systems

Foundations

The geotechnical report was prepared by Schnabel Engineering, LLC, on March 25, 2010. The Foundations of the patient tower is set on piles, with pile caps and grade beams. Each column location has a range from 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 level piles were drilled to reach up to the ground floor level to support irregular building features.

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 3 to the right. The columns are reinforced concrete ranging in size from 30" x 30" to 12" x 18". The main column size is 24" x 24" with vertical reinforcing of #11 bars numbering from 12 to 4 as they move up 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.

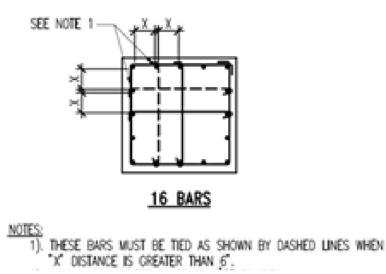


Figure 2: Column reinforcing detail

The structural system of the Patient Tower utilizes column capitals to resist punching shear with in the slab. The typical capital in the tower is 10' x 10' x 6" depth, making the slab thickness at the capitals 15 ½".

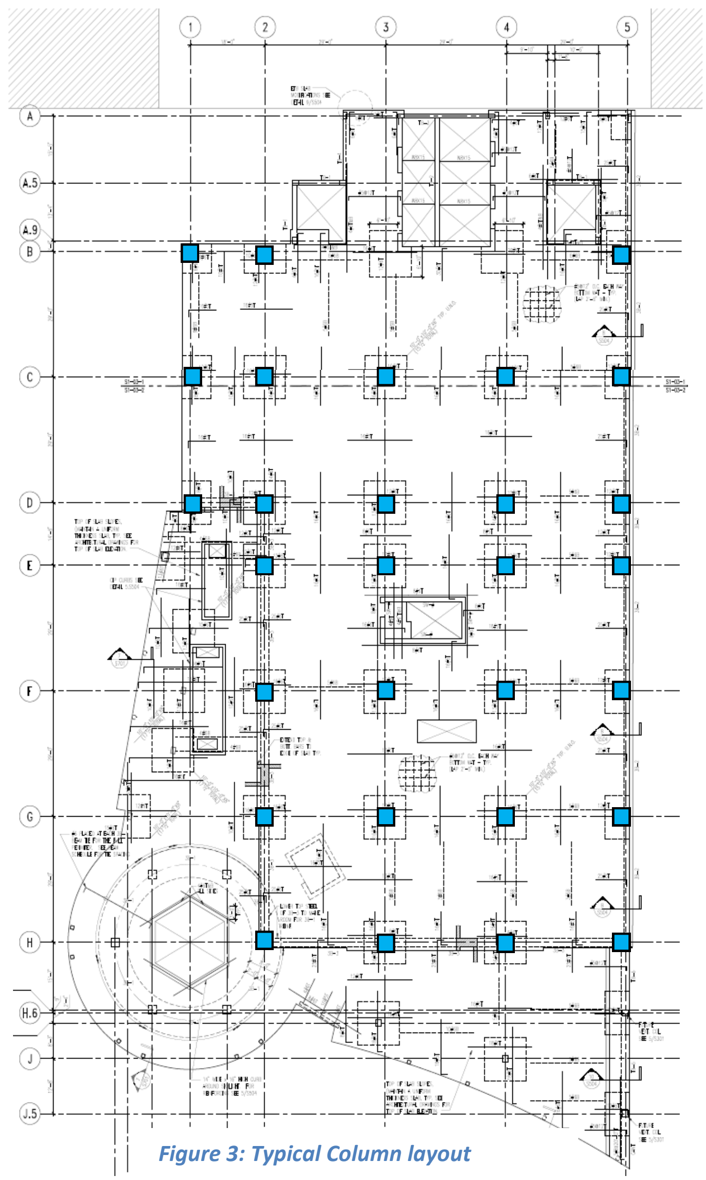


Figure 3: Typical Column layout

Floor System

The Floor system for this patient tower is a 9 ½" 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 larges span for this flat plan is 29' in each direction with square bays. The flat plate system has both top and bottom steel reinforcing. The top steel placed at places 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 is 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 ½" concrete slab. See figure 4 below for details.

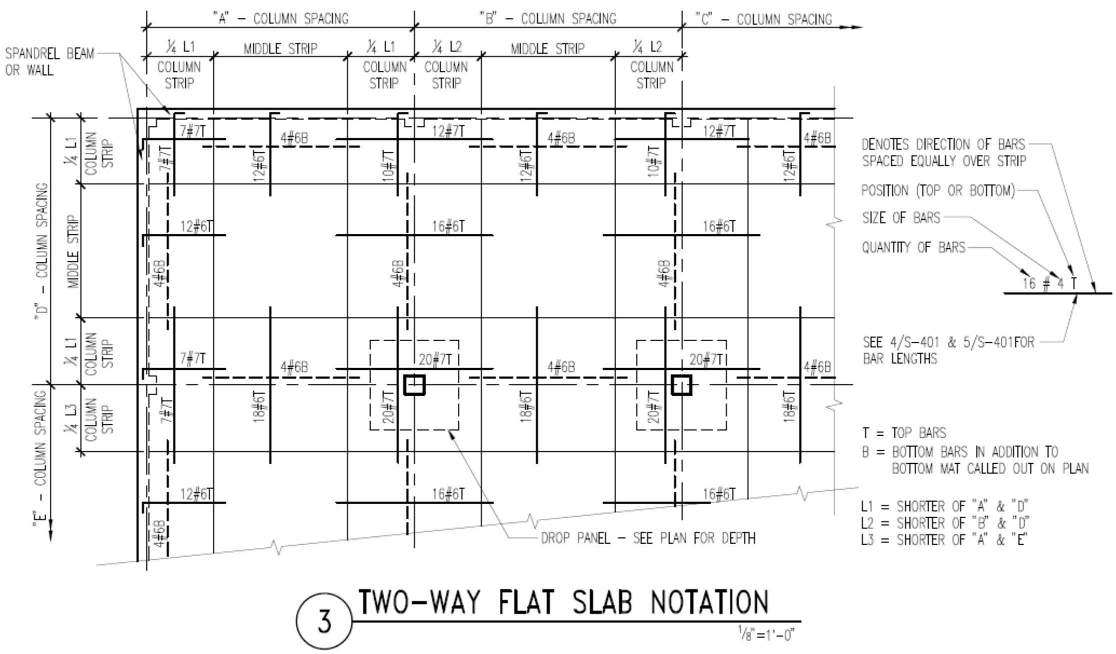


Figure 4: 2-way Slab detail

Roof System

The roof system for the patient tower is designed with the same conditions at a typical floor, a 9 ½" 2-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 bumped up 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 5) for the tower.

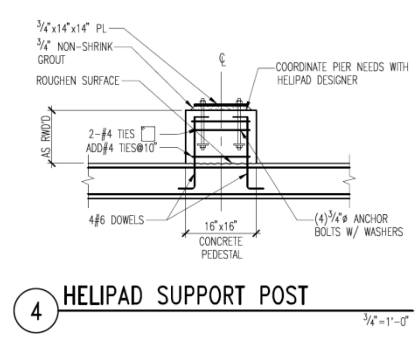


Figure 5: Helipad Support detail

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 thought out the building depicted to the right. The shear walls consisted 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.

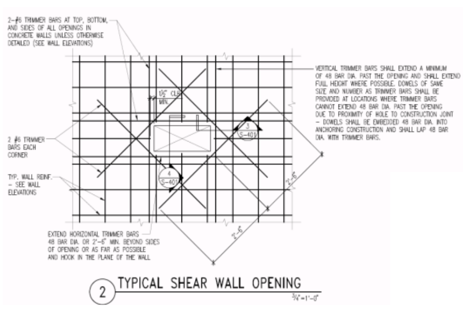


Figure 6: Shear wall reinforcing detail

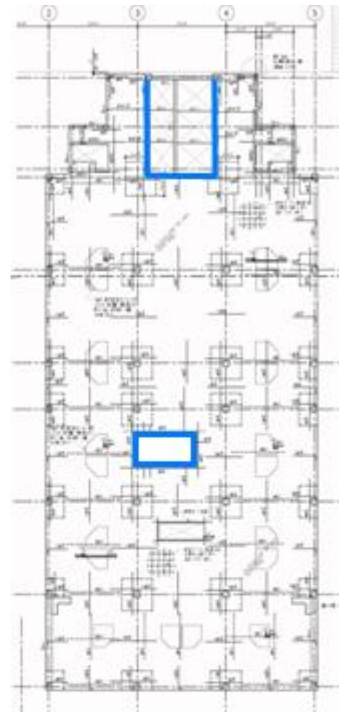


Figure 7: Shear wall layout

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 for 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 1st 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

Lateral 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 I. Appendix I contain sample calculations, spreadsheets including all values used in this analysis and tables including all existing parameters. Figures 8 & 9 show the forces and shear for each wind force direction.

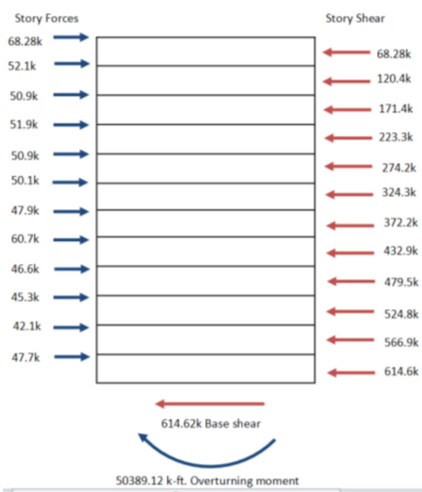


Figure 8: North-South Wind Loads

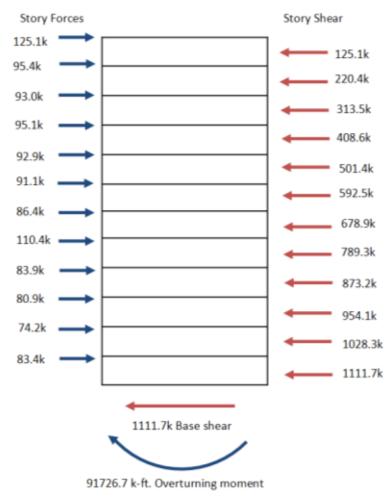


Figure 9: East-West Wind loads

Seismic Loads

In order to calculate the seismic loading of the Patient Tower ASEC 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. All design parameters that were used in this analysis of the seismic loading of the Patient Tower can be found in Appendix II. Sample seismic calculations along with spreadsheets with total building calculations will also be located in Appendix II. Figure 10 is a loading diagram with a summary of the story forces as well as the story shears from the seismic analyses.

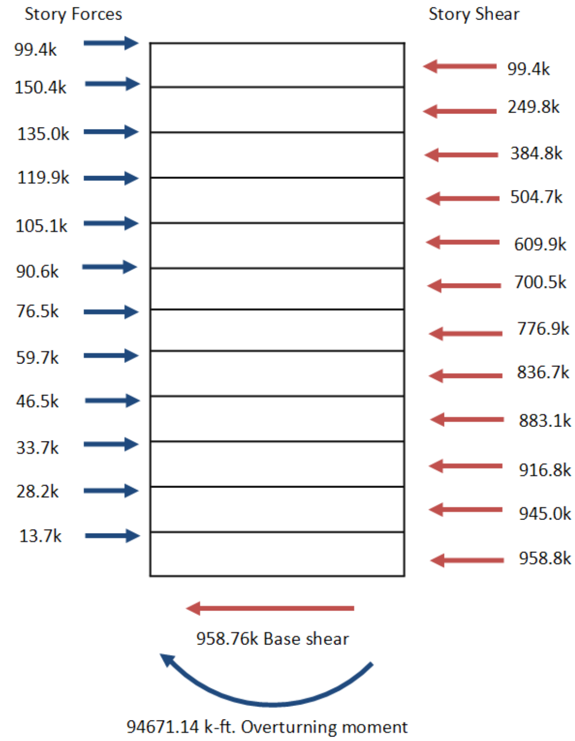


Figure 10: Seismic loads

Spot Checks

Gravity Column Spot Check

For the Column spot check, the axial load of column G4 was check with its current design. The column was checked at the 6th floor for its adequacy to carry the loads accumulated above. Three points on the interaction diagram were calculated; Pure axial, Balanced condition and Pure moment. An assumption was made that the column is not included as part of the lateral system so that the Pure axial load was the constraining factor. After the analysis of the column it is found that the design is oversized for the axial loading that is applied to the column, this could be because the concrete structure it will absorb some of the lateral forces placed on the building. Details of this column analysis can be found in appendix IIII.

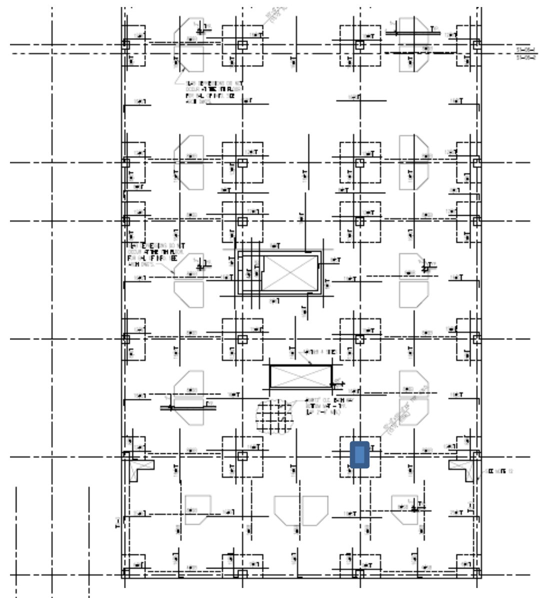


Figure 11: Spot Check Column G4

Drop Panels/Punching Shear Spot Check

The slab and column connection is accompanied by a 10’ x 10’ x 6” drop panel see in Figure 12. This panel is used to prevent the columns from punching thought the slab via shear. The shear resisting capability of the Drop panel is a function of its area and it depth. The capacity of the drop panels to resist the shear created by the drop panels greatly exceeds the needed shear capacity which could be explained by the sizing increments of panels due to ease of construction. Details of this analysis can be found in appendix IIII.

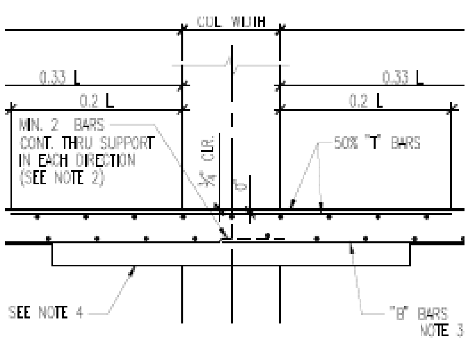


Figure 12: Spot Check Drop Panel

Slab Spot Check

The slab consists of square bays that are 29’ x 29’ with drop panels and no connecting beams. An interior bay was chosen to analyze for this spot check (Figure 13), using the ACI direct design method of analysis of a two way slab. During this analysis a slab thickness of 9.5” was used to check the structural engineer’s design for the slab of the tower. The results of the analysis did not correspond to the slab sizing of the tower because the design moments were greater than those of the slabs capacity. Detailed calculations are provided in appendix IIII.

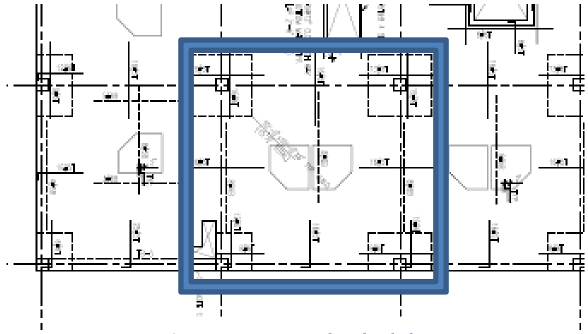


Figure 13: Spot Check Slab

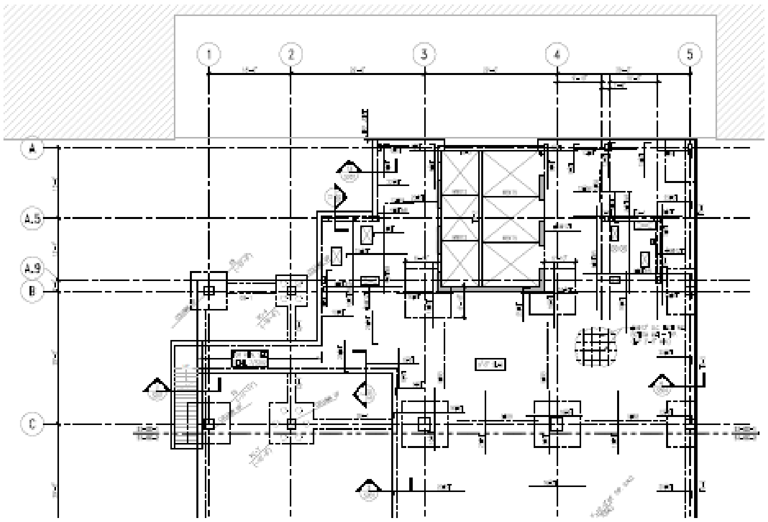
Conclusion

Technical Report #1 is an investigation and analysis of the structure for the Patient Tower in an attempt to better understand the design of the building. This report includes detailed descriptions and calculations of the different aspects of the structural system including the floor system the lateral supporting system and the gravity resisting system.

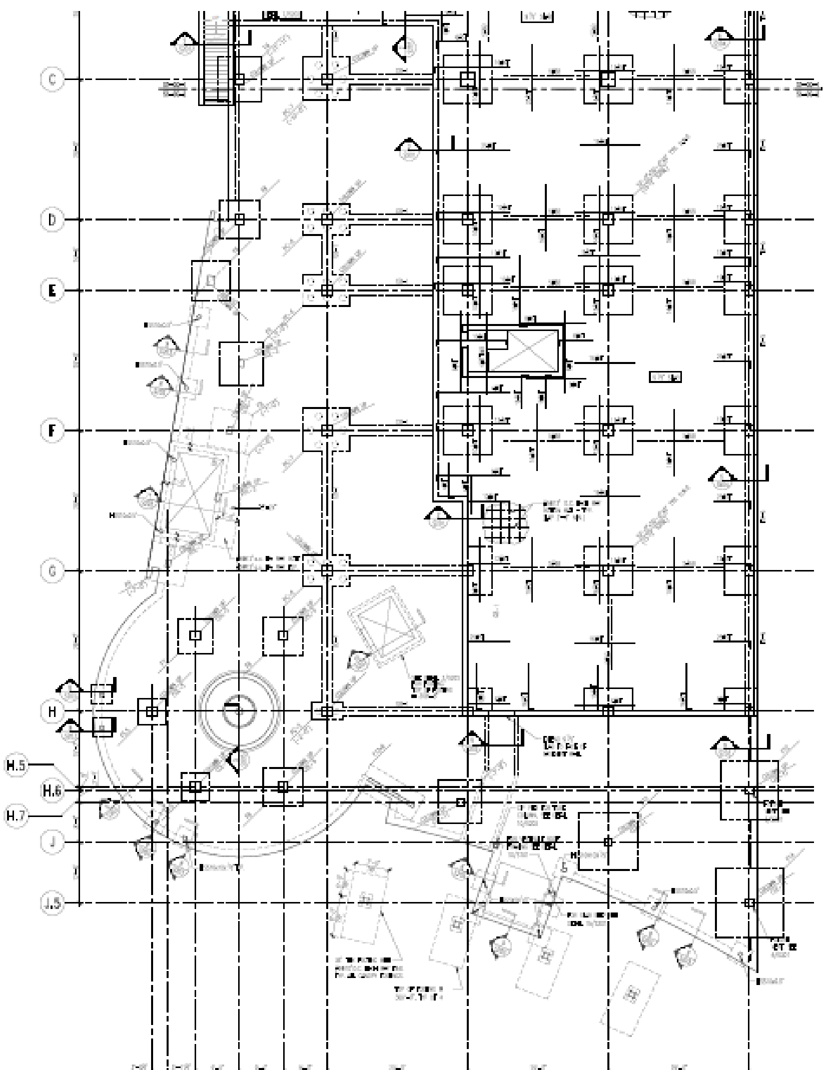
Calculations were run on individual members to verify the structural engineer's calculations. Most of the spot checks that were calculated verified that the structure of the tower was oversized accept for the design of the floor slab. This floor slab will need to be reevaluated in the next report to make sure that it is adequate.

Appendix I

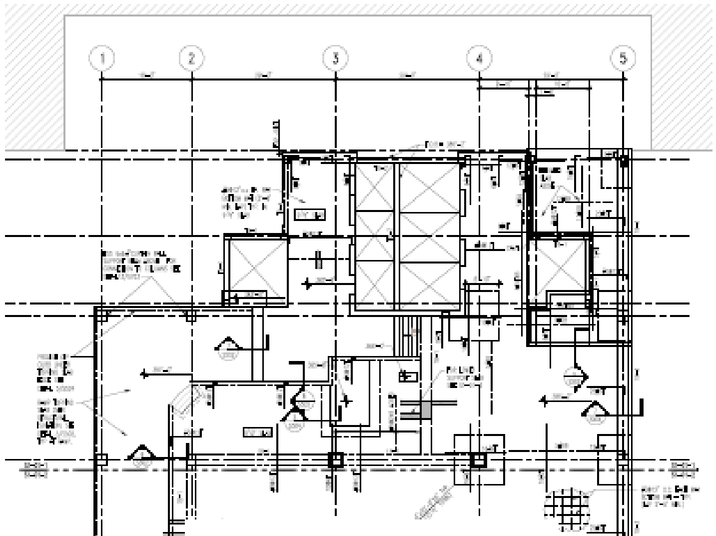
This section of Technical Report #1 is where the supplementary information for the layout and design for the Hospital Patient Tower can be found.



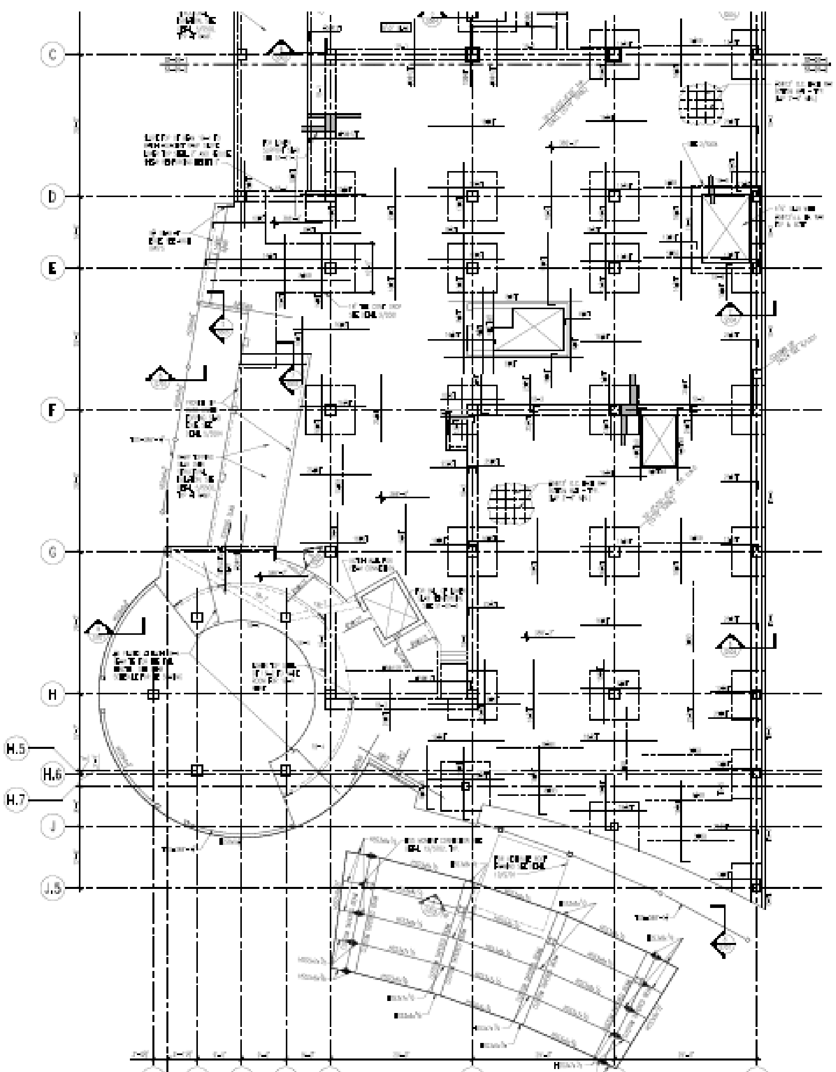
North Ground Floor Plan



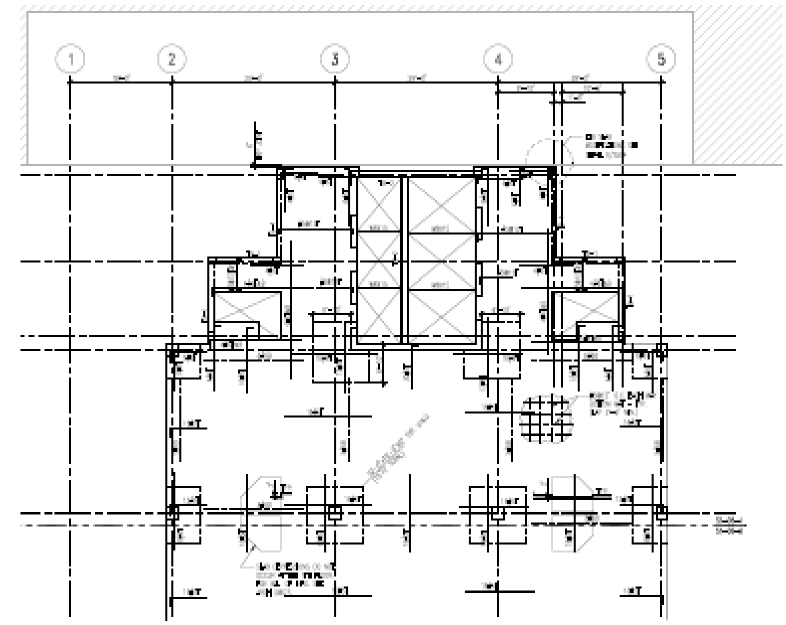
South Ground Floor Plan



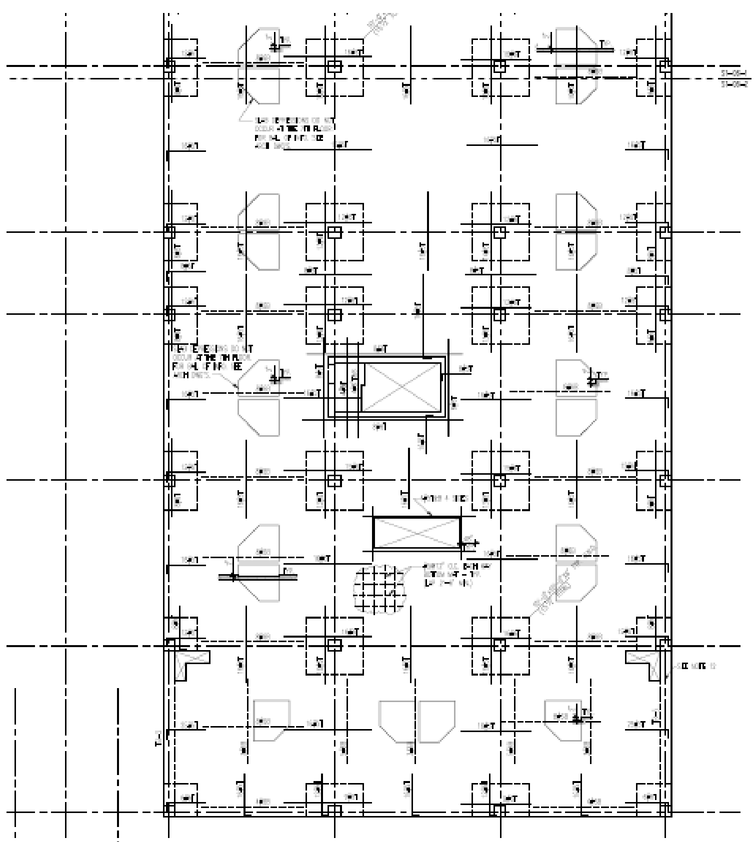
North 1st Floor Plan



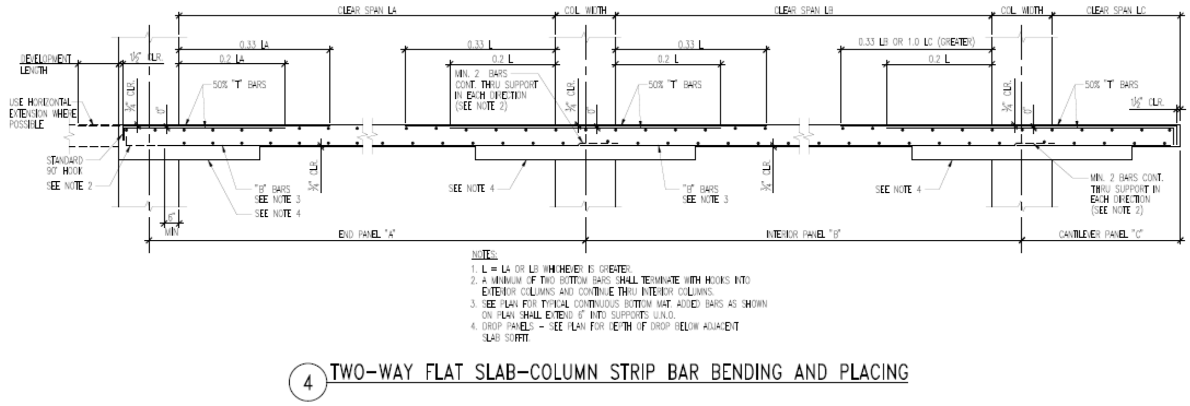
South 1st Floor Plan



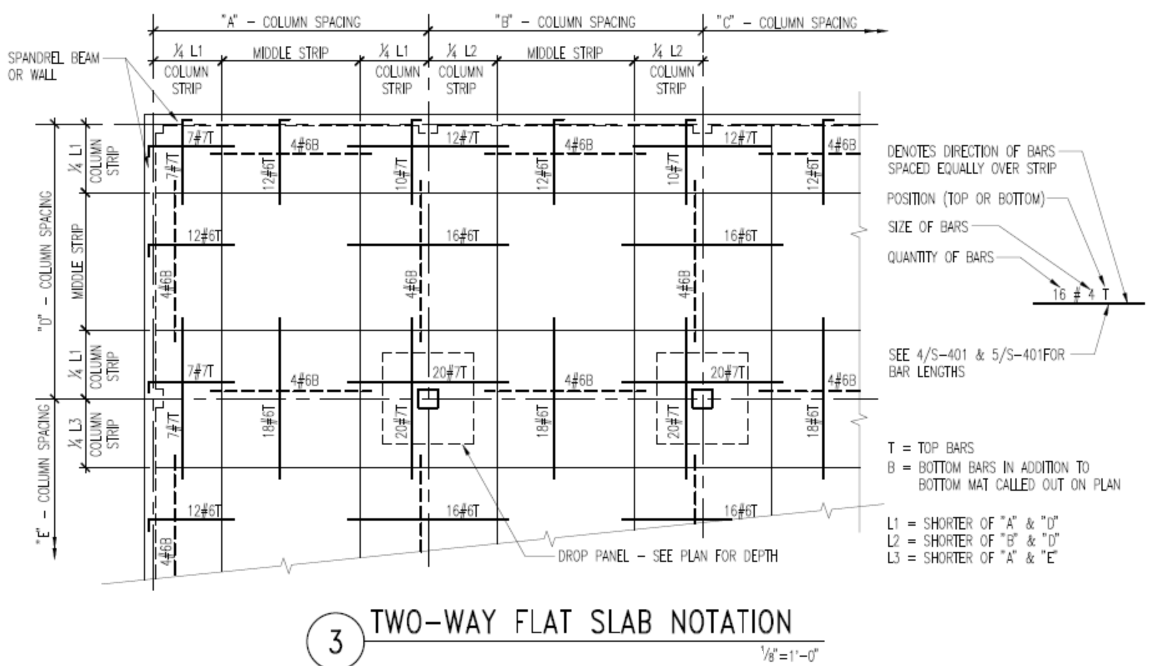
North Typical Floor Plan



South Typical Floor Plan



Two-way flat slab column strip rebar



Two-way flat slab notation

Appendix II

This section of Technical Report #1 is where the supplementary information for the Wind load analysis for the Hospital Patient Tower 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	Enclosed
Internal pressure coefficient (GC_{pi})	± 0.18

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

North - South Direction					
Height	K_z	q_z	Wind Pressures		
			Wind N-S	Lee N-S	Total N-S
146	1.102	34.53	29.69	-20.89	50.58
140	1.09	34.15	29.44	-20.89	50.32
120	1.04	32.58	28.37	-20.89	49.26
100	0.99	31.02	27.31	-20.89	48.19
90	0.96	30.08	26.67	-20.89	47.55
80	0.93	29.14	26.03	-20.89	46.92
70	0.89	27.88	25.18	-20.89	46.06
60	0.85	26.63	24.32	-20.89	45.21
50	0.81	25.38	23.47	-20.89	44.36
40	0.76	23.81	22.41	-20.89	43.29
30	0.7	21.93	21.13	-20.89	42.02
25	0.66	20.68	20.28	-20.89	41.16
20	0.62	19.42	19.42	-20.89	40.31
0-15	0.57	17.86	18.36	-20.89	39.25

East - West Direction					
Height	K_z	q_z	Wind Pressures		
			Wind E-W	Lee E-W	Total E-W
146	1.102	34.53	29.69	-13.98	43.67
140	1.09	34.15	29.44	-13.98	43.42
120	1.04	32.58	28.37	-13.98	42.35
100	0.99	31.02	27.31	-13.98	41.29
90	0.96	30.08	26.67	-13.98	40.65
80	0.93	29.14	26.03	-13.98	40.01
70	0.89	27.88	25.18	-13.98	39.16
60	0.85	26.63	24.32	-13.98	38.31
50	0.81	25.38	23.47	-13.98	37.45
40	0.76	23.81	22.41	-13.98	36.39
30	0.7	21.93	21.13	-13.98	35.11
25	0.66	20.68	20.28	-13.98	34.26
20	0.62	19.42	19.42	-13.98	33.41
0-15	0.57	17.86	18.36	-13.98	32.34

North - South Direction								
Floor	Height (ft.)	Story Height (ft.)	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	29.69	-20.89	50.58	68.28	0.00	0.00
11	131	11.5	29.44	-20.89	50.33	52.09	68.28	9969.32
10	119.5	11.5	28.37	-20.89	49.26	50.98	120.37	6823.99
9	108	11.5	29.3	-20.89	50.19	51.95	171.36	6092.60
8	96.5	11.5	28.3	-20.89	49.19	50.91	223.31	5610.24
7	85	11.5	27.5	-20.89	48.39	50.08	274.22	4912.97
6	73.5	11.5	25.36	-20.89	46.25	47.87	324.30	4257.11
5	59.5	14	27.3	-20.89	48.19	60.72	372.17	3518.35
4	48	11.5	24.2	-20.89	45.09	46.67	432.89	3612.80
3	36.5	11.5	22.84	-20.89	43.73	45.26	479.56	2240.07
2	25	11.5	19.8	-20.89	40.69	42.11	524.82	1652.01
1	13.5	13.5	18.36	-20.89	39.25	47.69	566.93	1052.85
Ground	0	0	0	0	0	0.00	614.62	643.80
							Sum	50386.12

East - West Direction									
Floor	Height (ft.)	Story Height (ft.)	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	29.69	-13.98	43.67	125.11	0.00	0.00
11	131	11.5	29.44	-13.98	43.42	95.37	125.11	18266.72
10	119.5	11.5	28.37	-13.98	42.35	93.02	220.49	12493.74
9	108	11.5	29.3	-13.98	43.28	95.06	313.51	11116.10
8	96.5	11.5	28.3	-13.98	42.28	92.87	408.57	10266.97
7	85	11.5	27.5	-13.98	41.48	91.11	501.44	8961.76
6	73.5	11.5	25.36	-13.98	39.34	86.41	592.55	7744.42
5	59.5	14	27.3	-13.98	41.28	110.38	678.96	6351.16
4	48	11.5	24.2	-13.98	38.18	83.86	789.34	6567.77
3	36.5	11.5	22.84	-13.98	36.82	80.88	873.21	4025.39
2	25	11.5	19.8	-13.98	33.78	74.20	954.08	2951.94
1	13.5	13.5	18.36	-13.98	32.34	83.39	1028.28	1854.94
Ground	0	0	0	0	0	0.00	1111.67	1125.75
							Sum	91726.67

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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. 26.9.1)

— Enclosed Buildings (table 26.11-1)

GC_{pi} ± 0.18

Velocity Pressures exposure coefficient (K_z) Building Height 174'-4" (including penthouse)

Heigth (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

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

N-S

$$P = (0.85)(0.8) q_z - (\pm 0.18) q_h = (0.68) q_z + 6.214 \text{ psf}$$

E-W

$$P = (0.85)(0.8) q_z - (\pm 0.18) q_h = (0.68) q_z + 6.214 \text{ psf}$$

Leeward walls

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

N-S

$$P = q_h (0.85)(-0.5) - q_h (\pm 0.18) = (-0.425) q_h - (0.18) q_h \text{ psf}$$

E-W

$$P = q_h (0.85)(-0.3) - q_h (\pm 0.18) = (-0.281) q_h - (0.18) q_h \text{ psf}$$

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Natural frequency N-S

building height 146' < 300' ✓

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

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

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

$n_a = 385 (C_w)^{0.5} h$ = 3.45 > 1 Rigid Structure

E-W

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

$n_a = 2.168 > 1$ Rigid Structure

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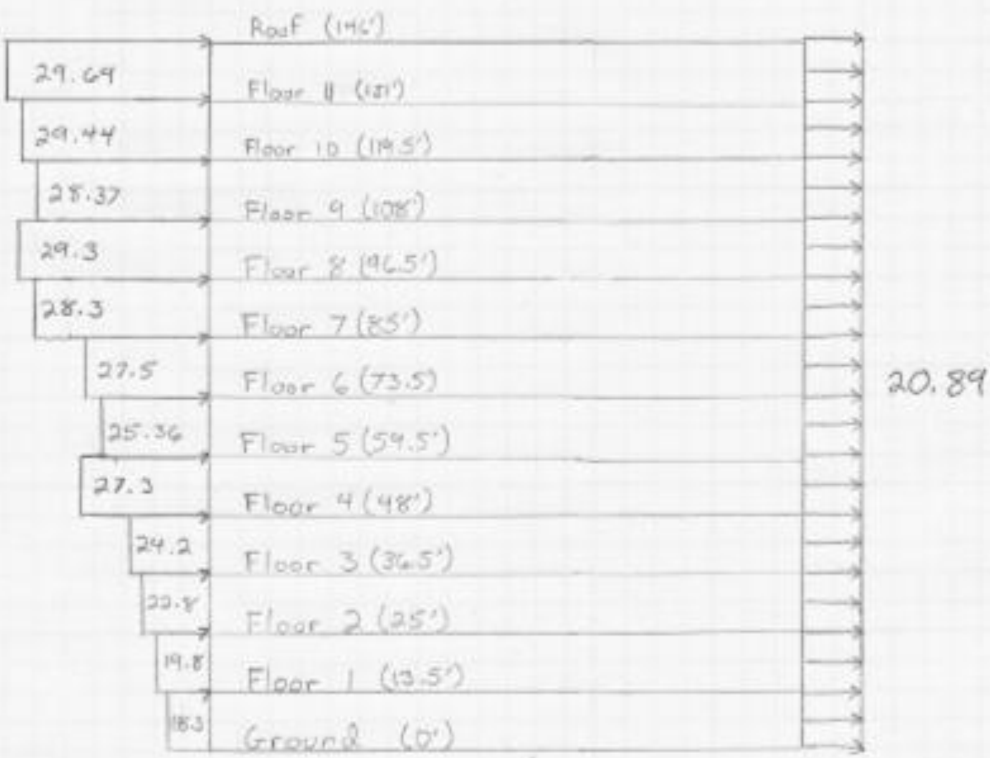
Wind Pressure Diagram



Pressure Distribution N-S Direction
(all values in pounds per sq ft)

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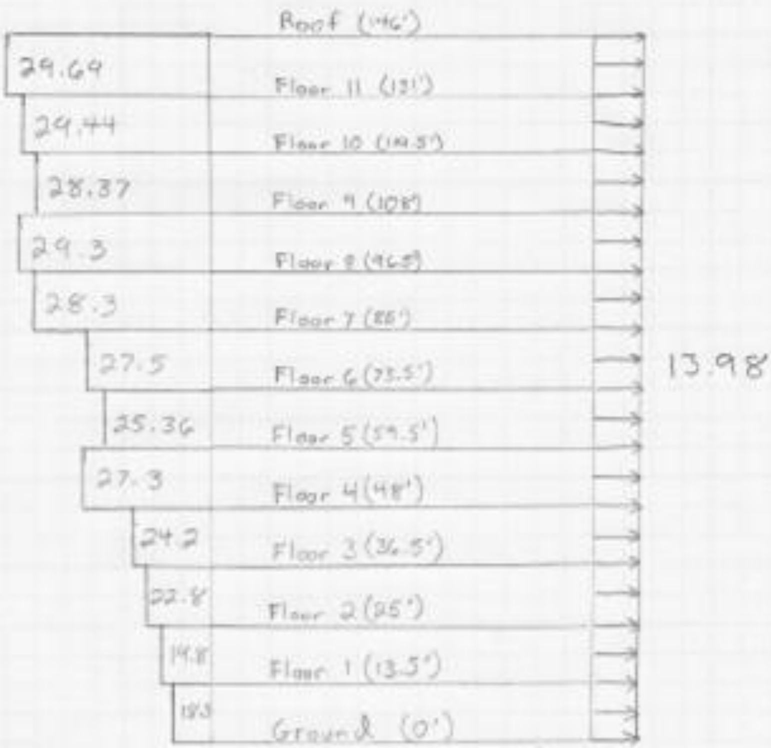
Wind Pressure Diagrams



Pressure Distribution E-W Direction

(all values in pounds per sq ft.)

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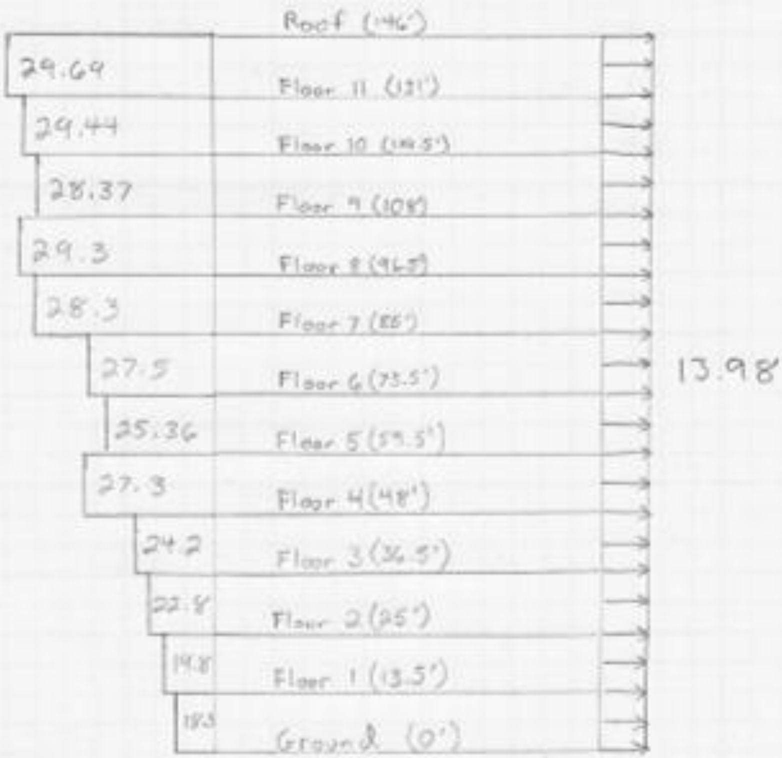
Wind Pressure Diagrams



Pressure Distribution E-W Direction

(all values in pounds per sq ft.)

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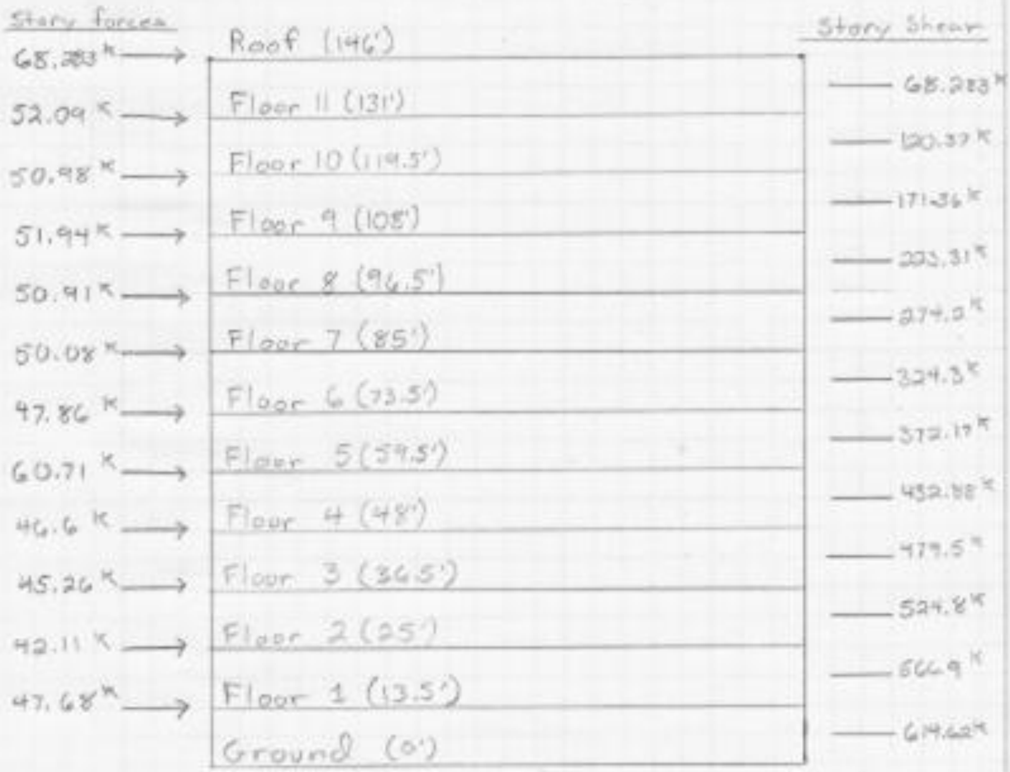
Wind Story loads



614.62k Base Shear

46237.4 k-ft Over turning moments

North - South Wind Direction

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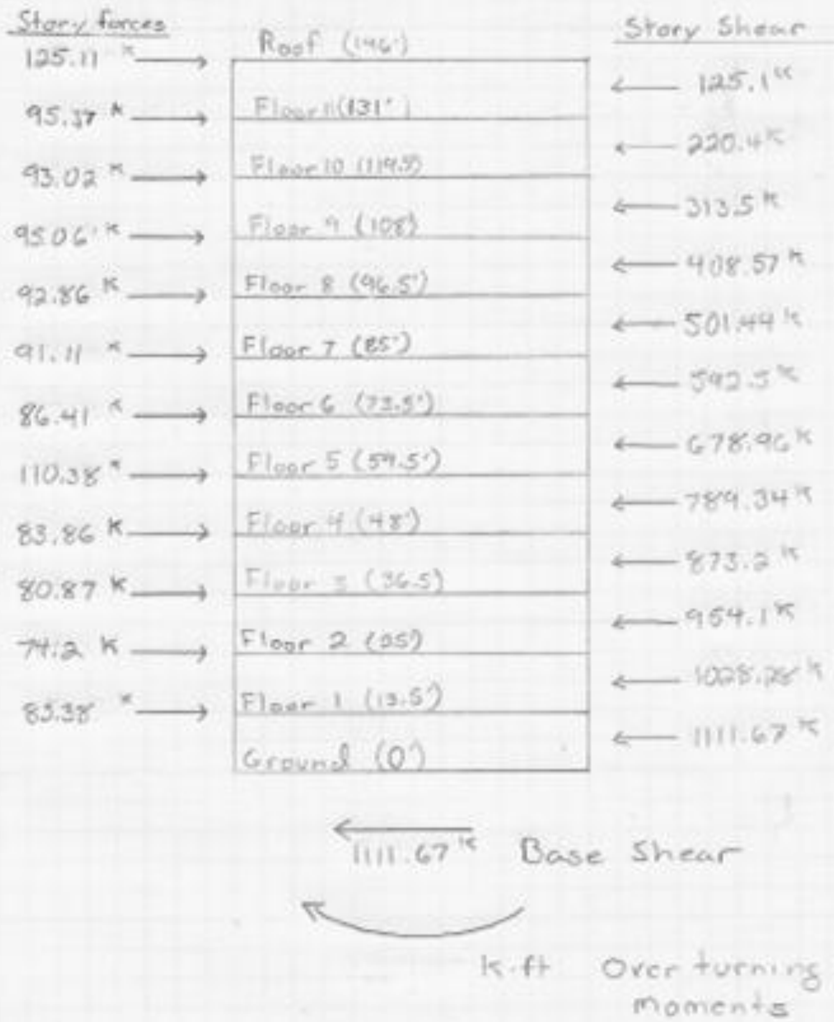
Wind Story loads



East - West Wind Direction

Appendix III

This section of Technical Report #1 is where the supplementary information for the seismic analysis for the Hospital Patient Tower can be found.

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
Base Shear (k)		958.76
Overturning Moment (k-ft.)		94671.14

Slab Weights			
Floor	Floor (sq. ft.)	Concrete slab (cubic ft.)	Weight (lbs.)
Roof	17026	13478.92	2021837.5
11	17026	13478.92	2021837.5
10	17026	13478.92	2021837.5
9	17026	13478.92	2021837.5
8	17026	13478.92	2021837.5
7	17026	13478.92	2021837.5
6	17026	13478.92	2021837.5
5	17026	14897.75	2234662.5
4	17026	13478.92	2021837.5
3	17026	13478.92	2021837.5
2	25889	20495.46	3074318.75
1	25889	20495.46	3074318.75
Ground	25889		
Sum	222038	177198.9167	26579837.5

Column and Misalanius Weights					
Column Size	Area (sq. ft.)	Number	Total Sq. ft.	Cubic ft.	Weight (lbs.)
24" x 24"	4	110	440	5280	792000
12" x 18"	1.5	97	145.5	1746	261900
12" x 24"	2	3	6	72	10800
18" x 18"	2.25	3	6.75	81	12150
26" x 26"	4.69	2	9.38	112.56	16884
28" x 28"	5.44	4	21.76	261.12	39168
Sum		219		7552.68	1132902
Shear walls			148	21608	3241200
Drop Panels	20	219	4380	24090	3613500
Curtain walls			82052		1641040
Superimposed					3330570
MEP					4440760

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
$\sum(w_x h_x^k)$ = 6643577.42			*$\sum F_x$ = Base Shear = 958.76 Kips*				*Overturning Moment = 94671.14 Kips - Ft*		

Tech Report #1 | Seismic Design | Patient Tower | 1/3

Seismic Loading

Spectral response acceleration

$S_s = 13.5\% g$ $S_1 = 5.5\% g$ (Figure 22.1)

Site Class - D $F_a = 1.6$ (11.4-1)

$F_v = 2.4$ (11.4-2)

ordinary reinforced concrete shear walls

$R = 6$ (12.2-1)

$\Omega = 2\frac{1}{2}$

$C_d = 5$

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

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

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

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 \frac{0.088}{0.144} = \boxed{0.122}$

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

$T_L = 8$

For Periods

$< T_0$ $S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) = \boxed{0.65}$

$> T_0, < T_S$ $S_a = S_{DS} = \boxed{0.144}$

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

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

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Seismic design Category

Occupancy category III $I_e = 1.25$

$S_{DS} = 0.144 \rightarrow$ Category A
$S_{D1} = 0.088 \rightarrow$ Category B
} used Category B (12.6-1,2)

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

$$C_s = 0.0218$$

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

Building Dead load Weight

$$W_t = 43,980 \text{ K}$$

Equivalent lateral force procedure (12.8)

$$V = C_s W = 0.0218(43,980) = 958.76^{K}$$

base shear

Tech Report #1 | Seismic Design | Patient Tower 3/3

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		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

99671.14 k·ft

Appendix IIII

This section of Technical Report #1 is where the supplementary information for the spot check analysis for the Hospital Patient Tower can be found.

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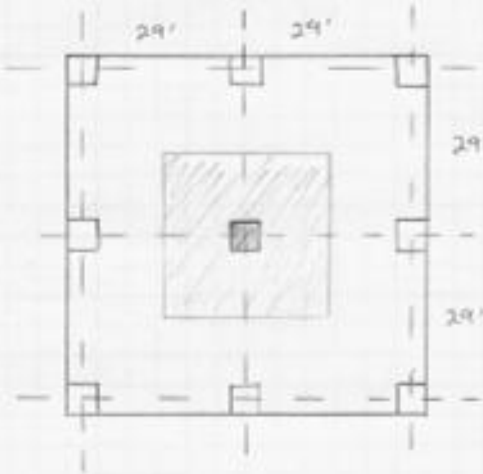
Typical Interior Column G4

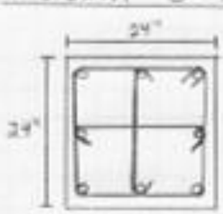


Tributary area
29'x29' = 841 sq ft

$f'_c = 5000$ psi
$f_y = 60{,}000$ psi

Minimum Cover 1.5"

Column G-4 @ Floor 6



8 #11 (Bars)
#4 @ 18" (ties)

$A_g = 24" \times 24" = 576 \text{ in}^2$
$A_s = (8 \times 1.56 \text{ in}^2) = 12.48 \text{ in}^2$

Pure axial Strength (P_o)

$P_o = 0.85 f'_c (A_g - A_s) + f_y A_s$
$= 0.85(5000)(576 - 12.48) + 60{,}000(12.48)$
$= 3143760$ psi
$\boxed{= 3143.8^k}$

$\phi \alpha P_o = 0.65(0.8)(3143.8^k)$ $\boxed{= 1634.78^k}$

Tech Report #1 | Spot Checks | Patient Tower

Balanced Condition

$\varepsilon_y = f_y / E_y = \frac{60}{29{,}000} = 0.00207$

$c_b = \frac{0.003}{0.003 + 0.00207}(24 - 2.28) = 12.85''$

$\varepsilon_{s1} = \frac{0.003}{12.85''}(12.85 - 2.28) = 0.0025 > 0.00207 \therefore f_{s1} = 60 \text{ ksi}$

$\varepsilon_{s2} = \frac{0.003}{12.85}(12.85 - 12) = 0.000198 < 0.00207 \therefore f_{s2} = \varepsilon_{s2}(E) = 5.75 \text{ ksi}$

$\varepsilon_{s3} = \frac{0.003}{12.85}(12.85 - 21.72) = -0.00207 = \varepsilon_y \therefore f_{s3} = -60 \text{ ksi}$

$P_b = 0.85 f'_c b \beta c + \Sigma A_{si} f_{si}$

$= 0.85(5)(24)(0.85)(12.85) + 3(1.56)(60) + 2(1.56)(5.75) + 3(1.56)(-60)$

$= 1132^k$

$M_b = 0.85 f'_c b \beta_1 c \left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + \Sigma A_{si} f'_c \left(\frac{h}{2} - d_i\right)$

$= 0.85(5)(24)(0.85)(12.85)\left(\frac{24}{2} - \frac{0.85(12.85)}{2}\right) + 3(1.56)(60)\left(\frac{24}{2} - 2.28\right)$

$+ 2(1.56)(5.75)(12 - 12) + 3(1.56)(60)(12 - 21.72)$

$= 12743.5 \text{ in}\cdot\text{k} = 1061.96 \text{ ft}\cdot\text{k}$

Pure bending M_o

$f_{s1} = \frac{0.003}{c}(c - 2.5)(29000)$ $\underbrace{f_{s2} = -60^k \; f_{s3} = -60^k}_{\text{assumptions}}$

$\Sigma F = 0 = 0.85(5)(24)(0.85)c + 3\left(\frac{0.003}{c}(c - 2.28)(29000)\right)(1.56)$

$+ 2(-60)(1.56) + 3(-60)(1.56)$

$86.7c + \frac{0.015}{c}(4.677c - 10.66)(135720) - 187.2 - 280.8$

Tech Report #1 | Spot Checks | Patient Tower | 3/6

$$0 = 86.7c + 407.2 - 928.32/c - 468$$

$$0 = 86.7c^2 - 60.8c - 928.32$$

$$c = 3.64''$$

$$f_{s1} = 32.5^k < 60^k$$

$$\varepsilon_{s2} = \frac{0.003}{3.64}(3.64 - 12) = -0.0068$$

$$\varepsilon_{s3} = \frac{0.003}{3.64}(3.64 - 21.72) = -0.0149$$

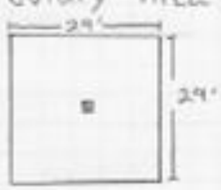
$< .00207$

$$M_n = 0.85(5)(24)(0.85)(3.64)\left(12 - \frac{0.85(3.64)}{2}\right)$$

$$+ 3(1.56)(32.5)(12 - 2.28) + 3(1.56)(12 - 21.72)(-60)$$

$$= 7640.75^{\text{in.k}} \quad \boxed{= 636.7^{\text{ft.k}}}$$

Tributary Area



Weights

Slab $9.5/12\,(150) = 118.75$ psf 5 floors

Roof $9.5/12\,(150) = 118.75$ psf

Misc $= 15$ psf

Live Load $= 60$ psf $+ 20$ partitions $= 80$ psf

DL $= 5(118.75) + 118.75 + 15 = 725.5$ psf

LL $= 5(80) = 400$ psf

$$P_u = 1.2(725.5) + 1.6(400)$$

$$= \frac{1510.6(24^2)}{1000} = \boxed{1270.4^k}$$

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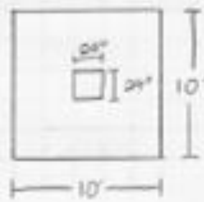
Punching Shear



15.5" thick

LL = 80 psf

DL =

$b_o = (24 + 15.5)4 = 158''$

$d = 15.5''$

$\phi V_c = 0.75\,(4) \times \sqrt{4000}\,(158)(15.5) = 464.6\text{ k}$

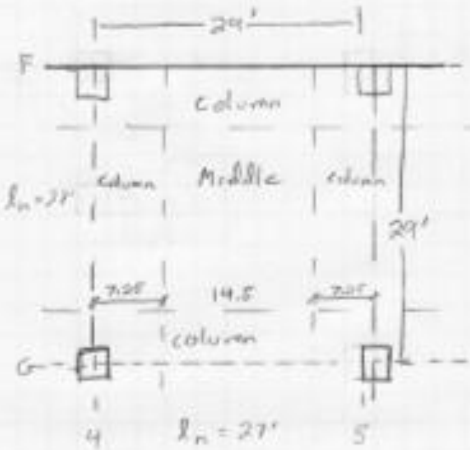
$W_u = 1.2((9.5/12)(150) + 20) + 1.6(80) = 294.5\text{ psf} = 0.294.5\text{ ksf}$

$V_u = 0.294.5(29^2) = \boxed{247.67 < 464.6\text{ k}}$

Drop panel resists slab punching shear

Tech Report #1 | Spot Checks | Patient Tower

Slab



Two way Flat Slab
No beams

$DL = \left(\frac{4.5}{12}\right)(150) + 20$

$= 138.75$

$LL = 80\ psf$

$w_u = 1.2(138.75) + 1.6(80)$

$= 0.295$

$l_n = 324\ in$

$h = \frac{l_n}{36} = \frac{324}{36} = 9"$

$$M_o = \frac{w_u L_2 L_n^2}{8} = \frac{0.295(29)(27)^2}{8} = 778.25$$

Interior $0.70 M_o = 544.78$

Mid $0.52 M_o = 404.69$

Interior column

Interior

Column - 75% $M = 0.75(544.78) = 408.59$ ft·k

Mid - 65% $M = 0.65(544.78) = 354.1$ ft·k

Mid

Column - 75% $M = 0.75(404.69) = 303.5$ ft·k

Mid - 65% $M = 0.65(404.69) = 263$ ft·k

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Positive Moments

$w_u l_n^2/16$ $\qquad w_u = 8.55$ plf

$8.55(27)^2/16 = 389.78\ \text{f}\cdot\text{k} > 303.5\ \text{f}\cdot\text{k}$ ∴ No Good

negative moments

$w_u l_n^2/11$

$8.55(27)^2/11 = 566.96\ \text{f}\cdot\text{k} > 408.59\ \text{f}\cdot\text{k}$ ∴ No Good