Final Report

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This Document is The Final Report for 5th year senior thesis in the Architectural Engineering Department at The Pennsylvania State University.

Structural Option

Professor Behr

Hospital Patient Tower

Virginia, U.S.A.

4/7/2011

Hospital Patient Tower

East Coast U.S.A.

Matthew Peyton

Structural Option

General Building Statistics

Size:	216,000 SF
Number of stories:	12 Above Grade
Cost:	$161 Million
Durations of Construction:	Summer 2010—Fall 2012
Delivery Method:	Design-Bid-Build

Architecture

- 174 private intensive care and medical/surgical rooms.
- 360° patient access for improved care.
- Two story atrium connected to the lobby with a living roof.
- Cantilevered aluminum lovers with glazing as lobby canopy.
- Precast concrete exterior façade with curtain wall sections.

Project Teams

General Contractor	Turner Construction
Architect	Wilmot/Sanz
Structural Engineer	Cagley & Associates
Civil Engineer	Dewberry & Davis
MEP Engineer	RMF Engineering INC.

Mechanical

- 5th Floor mechanical space.
- Five fan cooled AC units.
- Four steam boilers.
- One central and 4 exterior building mechanical risers.
- Stairwell pressurization fan 10,000 CFM.

Structural

- Foundations of piles and grade beams with a 5” S.O.G.
- 9 1/2” Flat plate concrete slab with 2 way steel reinforcing
- Concrete columns with drop panels and edge beams.
- 12” thick concrete shear wall in 7 locations.
- 9 1/2” Flat plate concrete roof slab with Helipad supports .
- 14” Penthouse Slab with steel reinforcing.
- 1 1/2” Metal roof deck on wide flange steel for penthouse connection.

Electrical/Lighting

- Two 2000 KVA transformers provided by DVP.
- 2000 KW Generation feeding a 2000KVA transformer for Emergence back-up.
- 277 V lighting system mostly fluorescent with specialty lighting where needed .

http://www.engr.psu.edu/ae/thesis/portfolios/2011/mrp5082/index.html

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Executive Summary

The Hospital Patient Tower is a 12 story expansion to an existing patient tower. This is one of early steps of a large capital expansion plan. This tower utilizes piles and grade beams as a foundation with a concrete structural system. Typical column size is 24" x 24" with varying rebar placement and design of both vertical and horizontal. The new Patient Tower will connect with an existing patient tower by a bank of elevators and will also await the connection of a women's health facility that is one of the next phases of the Capital Improvement Project. Since the Patient Tower needs to line up with the existing structure the floor to floor high is a major consideration in the structural design.

For this thesis report, the goal was to investigate and discuss the effects of redesigning the structural system for the patient tower from its original cast-in-place concrete system to a steel frame system. While redesigning the structural system it was necessary to maintain the architectural plan as to not affect the functionality of the hospital. The two-way concrete slab with concrete shear wall cores was redesigned to a steel frame with "X" bracing. This new system is known as girder-slab and uses a modified wide flange to create a composite action between the precast plank and the wide flanged steel. Preliminary framing elements were sized using the AISC 13th edition Steel Construction Manual and the Girder-Slab Design Guide. An Etabs model was created to design the lateral force resisting system using calculated wind and seismic loads from ASCE 7 -10.

Two breadth studies were conducted for this report to determine how the structural redesign affects other aspects of the building. The first breath topic is a construction management analysis which was performed to investigate and compare the cost and schedule of the existing concrete structure and the proposed steel frame structure. The schedule was compared using R.S. Means construction cost data, an estimated schedule was generated using time acquired from labor crews and unit amounts. From this study it was concluded that both designs have their pros and cons and both of these structures are feasible options for the Hospital Patient Tower.

The second breadth study was an acoustical study to analyze the Sound Transmission Class (STC) and Impact Insulation Class (IIC) for the two Intensive Care Units (ICU) and there adjacent spaces. Both of the towers ICU units are located either above or below a potential noise source. The regular ICU is located above the towers café which will have a large amount of air borne sound and the Nero ICU is located below the mechanical level on the fifth floor which will have high structural borne noise. These two spaces were check for their specific type of noise so that it does not disturb the occupants. In both cases the existing elements of the design were able to meet the criteria needed for the spaces.

Figure 1: Rendering by Wilmot Sanz

Acknowledgment's

I would like to thank the following individuals and companies for the steadfast support they offered throughout the duration of the thesis process. Without their help, this would not have been possible.

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The Faculty and Staff of the Penn State AE department

Outside Consultants

Frank Malits – Cagley & Associates

Joan Dannemann – Inova Design and Construction Department

I would like to give a special thanks to the professionals at both, Cagley & Associates for sponsoring my thesis project, and, Inova Design and Construction Department for their permission to use The Patient Tower as the subject of my study.

Lastly, I would like to extend my gratitude to my family and friends for their unconditional support and encouragement.

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 on 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 tying in some of the red brick on the exterior walls, while also 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 contract 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 aged 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 teams input, the placements for monitoring stations were optimized to ensure patient privacy as well as enhancing the monitoring capabilities.

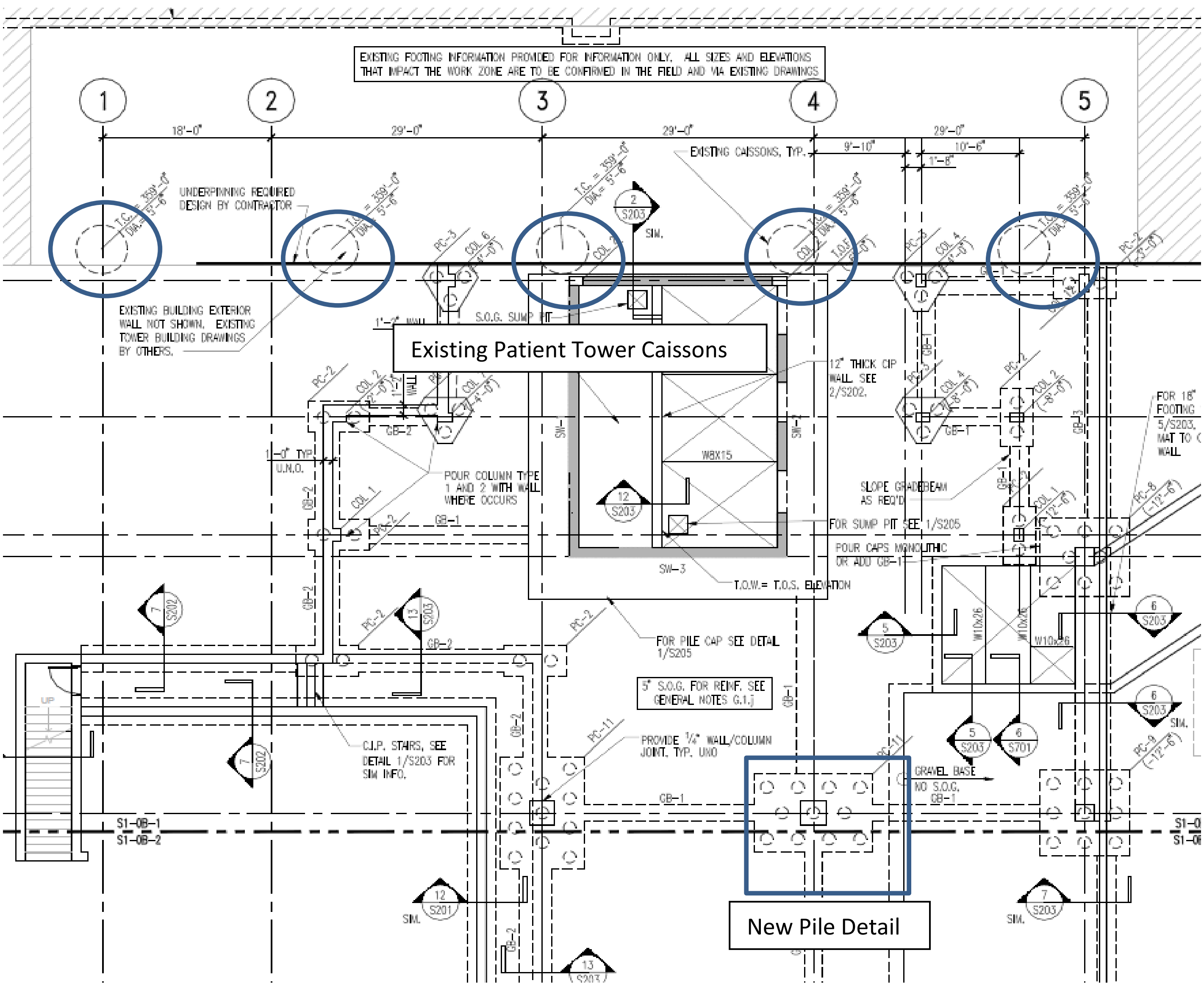
One of the hospital's goals, along with excellent patient care, is also to lower the hospital's impact on the environment. The hospital's plan for this new tower included green features such as living roofs, low flow water fixtures, and rain gardens. The design also calls for no/low VOC building materials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

Existing 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 seen in figure 3. The existing 66" caissons will not support the new tower but some force will be added with the connection of the new tower. In a few locations where no basement exists, piles were placed to reach up to the ground floor level to support irregular building features.

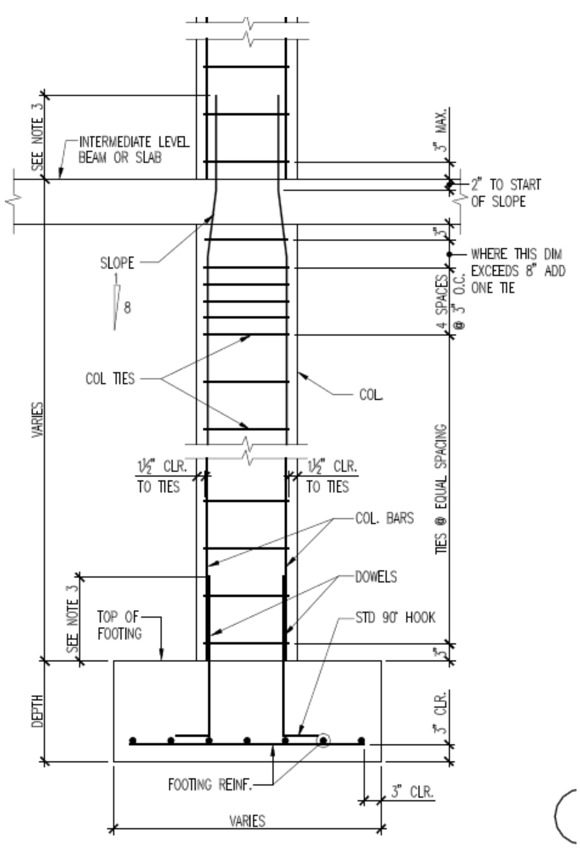


BASEMENT FLOOR FOUNDATION PLAN – NORTH 1/8"=1'-0"

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" as seen in Figure 5. 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".



NOTES: 1) SEE COLUMN SCHEDULE FOR DIMENSIONS AND REINFORCING.
2) CLEARANCE SHOWN IS THE MINIMUM CONCRETE COVER FOR PRIMARY REINFORCEMENT AND TIES.
3) FOR SPLICE LENGTHS OF VERTICAL BARS SEE 1/S-202
4) SEE SCHEDULE FOR TYPICAL COLUMN NOTES.

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

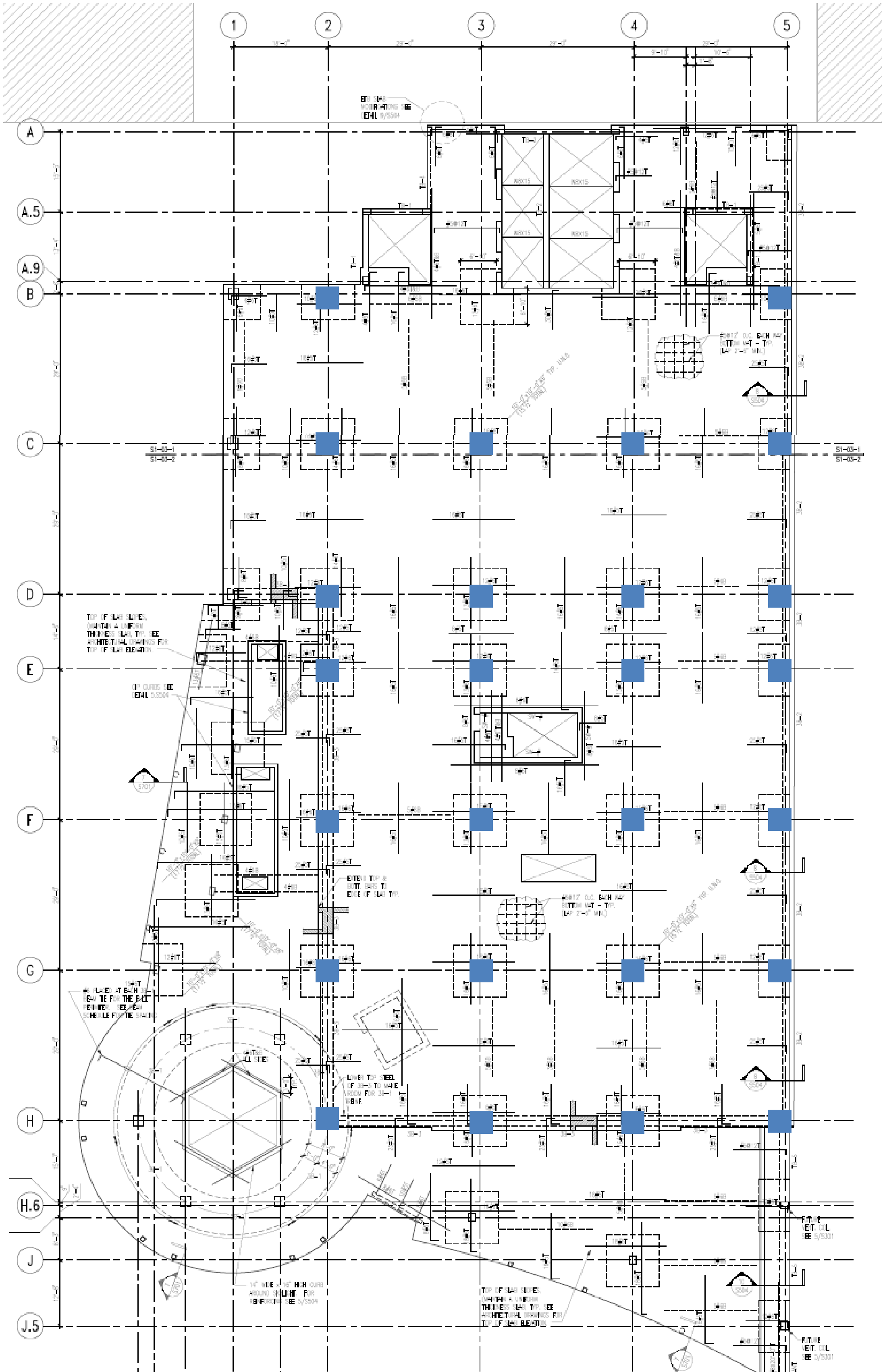


Figure 6: Typical Column layout from Cagley & Associates

Floor System

The floor system for the 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.

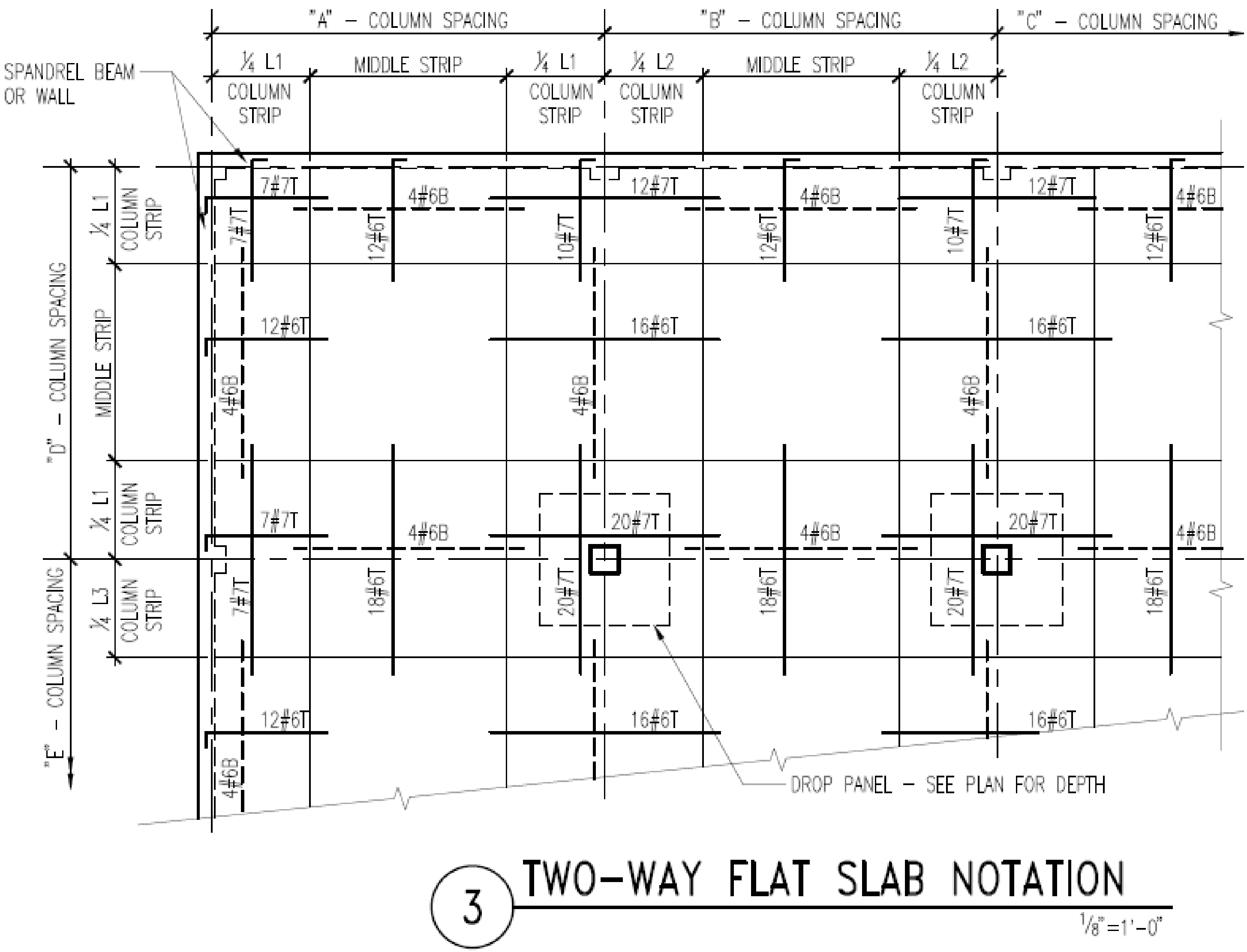


Figure 7: Two-way Flat Slab Detail from Cagley & Associates

Roof System

The roof system for the patient tower is designed with the same conditions of 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.

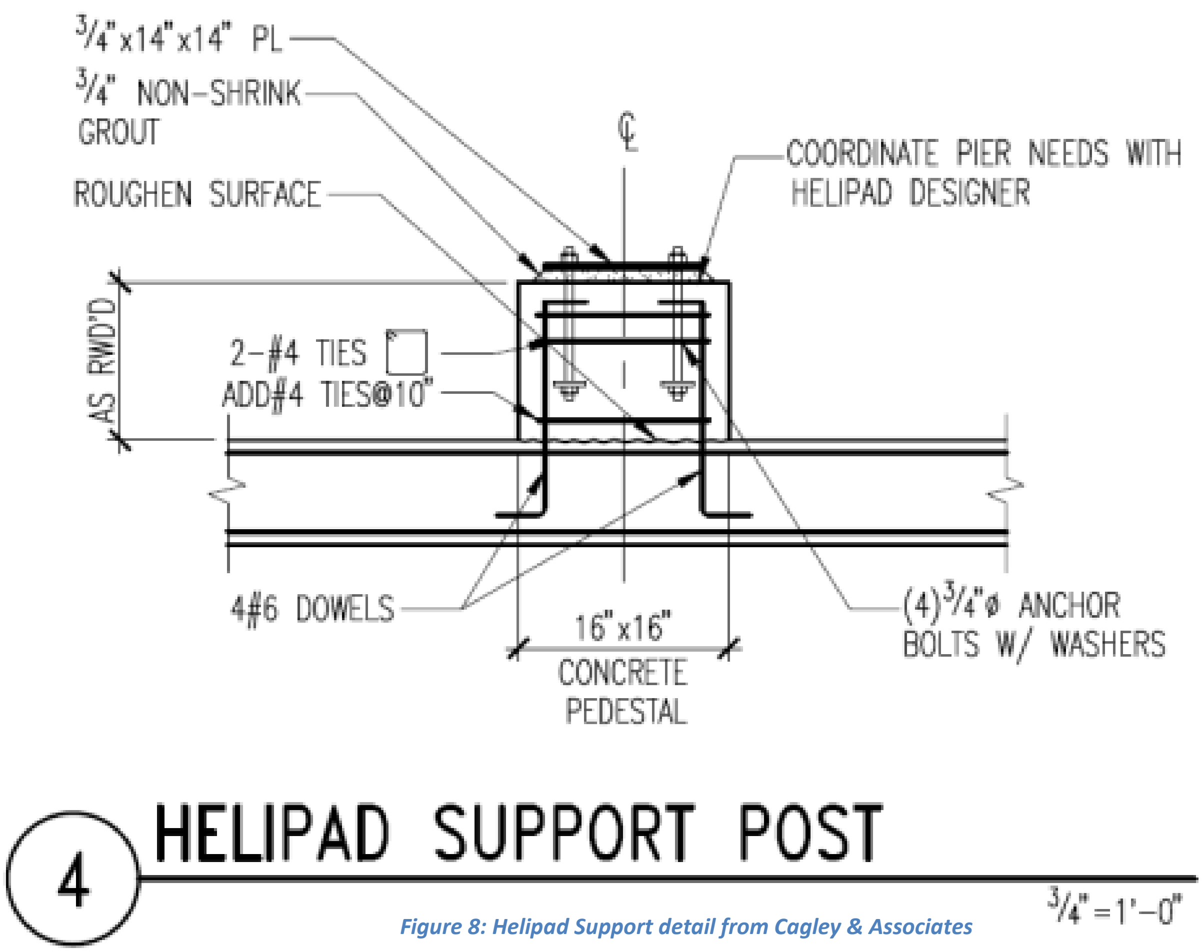


Figure 8: Helipad Support detail from Cagley & Associates

Lateral System

The lateral system for the Hospital Patient Tower consists of seven 12" reinforced concrete shear walls. These walls are located in two shear wall cores, one core is around the elevators and the other is around the main stair case. The shear walls consist of 5000 psi concrete and were run continuously through the tower from the foundations up to the roof. 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 gravity system is a concrete two-way flat plate which will also acts as a concrete moment frame giving it some resistance for the lateral forces. With the combined action of these two systems all of the lateral forces applied to this tower can be resisted. With both of these element types acting in conjunction there is no need for any additional lateral force resisting system. An Etabs model of the lateral system can be seen in figure 9.

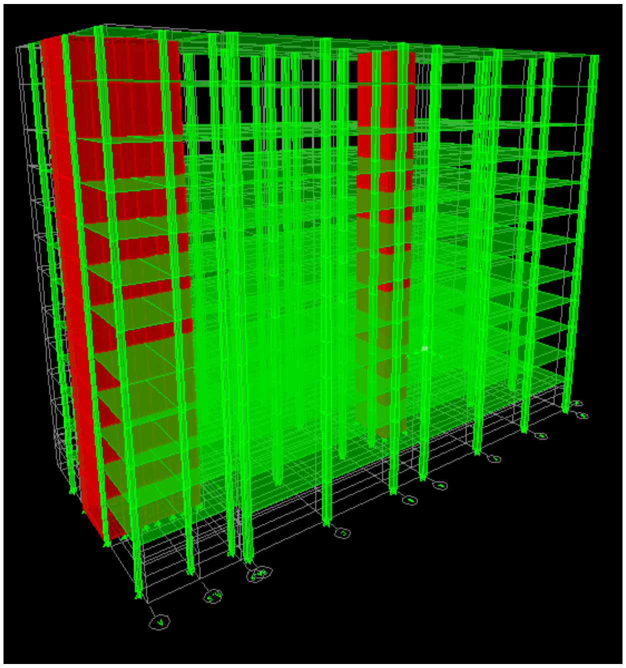


Figure 9: Etabs model of the existing structural system

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

Lateral Drift Criteria

Lateral building drift limited to H/400

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

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 -10. Loads are displayed below.

Table 2 – Dead Loads	
Occupancy	Design Loads
Hollow Core Plank	60 psf
MEP Equipment	15 psf
Superimposed	20 psf
Topping Load	25 psf

Table 3 – 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 4 – Snow Load	
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

$p_f = 0.7C_eC_tI_sp_g$

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 (ASCE7 -10) the design wind speed is 120 MPH for the location of the Patient Tower. For this 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 has been excluded in this analysis. It was also assumed that the building was independent of the connected tower and that the wind was not impeded by any of the structures surrounding the Patient Tower. The four wind load cases in figure 10 below from ASCE 7 were all taken in to account during the analysis of the 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. Tables 7 & 8 show the forces and shear for each wind force direction.

Table 5 - 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

Table 6 - 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

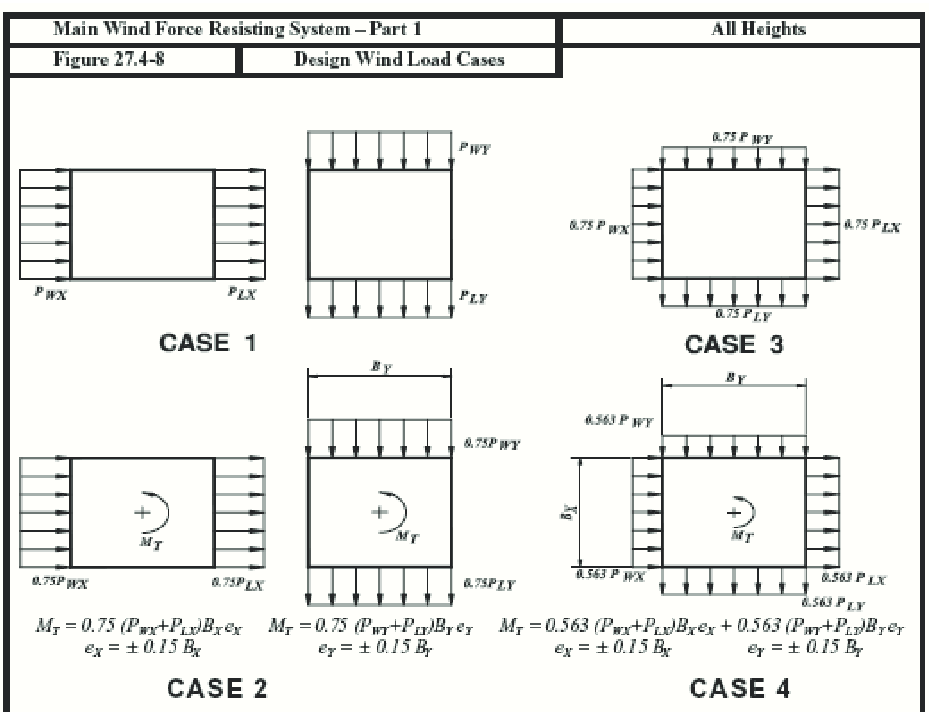


Figure 10: ASCE 7- 10 Wind load cases

Table 7 – North/South Direction										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	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	1.10	34.53	23.5	-14.7	38.2	52	0	0
11	131	11.5	1.07	33.43	22.7	-14.7	37.4	39	52	7520
10	119.5	11.5	1.04	32.52	22.1	-14.7	36.8	38	90	5072
9	108	11.5	1.01	31.64	21.5	-14.7	36.2	37	128	4550
8	96.5	11.5	0.98	30.69	20.9	-14.7	35.5	37	166	4045
7	85	11.5	0.95	29.61	20.1	-14.7	34.8	36	203	3550
6	73.5	11.5	0.90	28.32	19.3	-14.7	33.9	35	239	3062
5	59.5	14	0.85	26.57	18.1	-14.7	32.7	41	274	2581
4	48	11.5	0.80	25.06	17.0	-14.7	31.7	33	315	2454
3	36.5	11.5	0.74	23.15	15.7	-14.7	30.4	31	348	1576
2	25	11.5	0.66	20.68	14.1	-14.7	28.7	30	379	1149
1	13.5	13.5	0.57	17.86	12.1	-14.7	26.8	33	409	744
Ground	0	0	0.00	0.00	0.0	0.0	0.0	0	442	440
									Sum	36742

Table 8 - East/West Direction										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	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	1.10	34.53	23.5	-7.8	31.2	90	0	0
11	131	11.5	1.07	33.43	22.7	-7.8	30.5	67	90	13070
10	119.5	11.5	1.04	32.52	22.1	-7.8	29.9	66	157	8776
9	108	11.5	1.01	31.64	21.5	-7.8	29.3	64	222	7844
8	96.5	11.5	0.98	30.69	20.9	-7.8	28.6	63	286	6947
7	85	11.5	0.95	29.61	20.1	-7.8	27.9	61	349	6070
6	73.5	11.5	0.90	28.32	19.3	-7.8	27.0	59	411	5209
5	59.5	14	0.85	26.57	18.1	-7.8	25.8	69	470	4363
4	48	11.5	0.80	25.06	17.0	-7.8	24.8	54	539	4110
3	36.5	11.5	0.74	23.15	15.7	-7.8	23.5	52	594	2616
2	25	11.5	0.66	20.68	14.1	-7.8	21.8	48	645	1885
1	13.5	13.5	0.57	17.86	12.1	-7.8	19.9	51	693	1199
Ground	0	0	0.00	0.00	0.00	0.00	0.00	0	745	693
									Sum	62782

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 Table 9. Sample seismic calculations along with spreadsheets containing total building calculations are also located in Appendix II. Table 10 includes a summary of the story forces as well as the story shears from the seismic analyses.

Table 9 - 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	3.25
Seismic Response Coefficient	C_S	0.0218
Effective Period	T	0.84

Table 10 - Base Shear and Overturning Moment Distribution									
Floor	Height h_x (ft)	Story Height (ft)	Story Weight w_x (Kip)	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	2126	341	724074	0.16	107	0	0
11	131	11.5	2114	300	634196	0.14	94	107	15626
10	119.5	11.5	2114	269	569556	0.13	84	201	12280
9	108	11.5	2114	239	505967	0.12	75	285	10060
8	96.5	11.5	2130	210	446864	0.10	66	360	8077
7	85	11.5	2130	181	385211	0.09	57	426	6374
6	73.5	11.5	2130	153	324964	0.07	48	483	4840
5	59.5	14	2142	119	255225	0.06	38	531	3530
4	48	11.5	2154	93	199670	0.05	30	568	2245
3	36.5	11.5	2154	67	144925	0.03	21	598	1417
2	25	11.5	3218	43	139036	0.03	21	619	782
1	13.5	13.5	3232	21	67905	0.02	10	640	514
Ground	0	0	0	0	0	0.00	0	650	135
$\sum(w_x h_x^k)$ = 4,397,600			$\sum F_x$ = Base Shear = 650 Kips				Overturning Moment = 65,900 Kips - Ft		

Proposal Problem Statement

The Patient Tower is currently a two – way flat plate reinforced concrete slab supported by reinforced concrete columns. This system is the main gravity load bearing system that transfers each floor load to the foundation of slab on grade and drilled piles. The tower's current lateral system is reinforced concrete shear cores. There are two cores located around the central stair case and the elevator shaft. The strength of concrete used in the shear walls is 5000 psi, with the gravity system using both 5000 and 7000 psi concrete.

The Patient Tower is an addition to an existing hospital campus to provide updated equipment and facilities for care while being integrally connected to the existing patient tower. The goal of this thesis is to decrease the overall cost of the new tower and to decrease the construction time while maintaining the functionality of the tower.

Problem Solution

In order to decrease the overall cost of the tower, decreasing the construction time and the overall building weight are the two main ways that this challenge is confronted. Changing the gravity system of the tower from a two-way concrete slab to a steel frame with hollow concrete plank should help reduce the weight of the structural system and the construction duration.

The new proposed floor system would be hollow core concrete plank ranging from 8” to 12” supported with W-shape steel beams. These planks would be placed in the web of the beam and not placed on top of it as is traditionally done. Since the planks and beams will be placed in conjunction with each other the floor system will be low in depth allowing for the Patient Tower to maintain its floor to floor heights. This will allow for a seamless connection to the existing tower. The current floor system depth is 9.5”, giving the tower enough space in the ceiling cavity for all of the mechanical system. There should not be many issues with ceiling cavity space in the proposed new design. With this change in the floor system the columns for the tower would also need to be redesigned to account for the change in material and loading of the floor system. ASCE7-10 will be used to determine the correct floor loads for the tower as evaluated in Tech Report #2.

The lateral resisting system will remain the same as that in the original design, with the two shear cores surrounding the stairway and the elevator shaft. With the changes to the building weight it may be found that the lateral system is over designed with the new gravity system, but will be maintained during this assignment. If it is found that the shear walls are insufficient in the new design, they will be redesigned to carry the higher loads.

Breadth Topics

The change in the gravity resisting system from a two – way flat plate reinforced concrete slab to a steel frame with precast concrete plank decking will produce a change in the construction management of the project. While steel structural elements are prefabricated and have a longer lead time, we are trying to decrease the weight of the tower. This change would decrease the need for such a bearing ability of the foundations elements. With the faster erection time for steel shortening the length of construction, an overall cost reduction would be realized.

Since we are changing from a concrete gravity system to a steel system the acoustical criteria will need to be checked for areas of importance. In a hospital acoustics will be very important criteria that will need to be kept with in close tolerances to not affect the patients. With a concrete system this criteria is satisfied by the mass and rigidity of the system; whereas with a steel system these criteria will need to be checked. For this study, I would like to check the acoustical performance of the Intensive Care Unit (ICU). The ICU sits above the café that is open to the public so there is a concern that the noise will be carried and disrupting the ICU above. The Patient Tower uses the fifth floor to house the mechanical systems directly below the mechanical floor is the Neuro ICU floor. The acoustics will need to be check for the mechanical floor to make sure that the noise is not transferred to the ICU located below.

Redesigned Structural System

In the structural redesign of the Patient Tower, the structure was changed from a two-way flat plat concrete slab with two shear cores to a structural steel frame with precast composite plank and lateral "X" bracing. In the redesigned structure, the gravity loads were assumed to be the same as in the existing structure with the exception of the dead load which changed with the materials. Both the wind and seismic loads were determined using ASCE 07 – 10.

Columns

The columns for the steel frame were designed in accordance with the LRFD method and the AISC Steel Construction Manual. The columns are designed to resist only the gravity loads on the Patient Tower. The columns for the steel frame redesign are laid out to fit within the existing column layout with modifications to account for the changes in the floor system as well as the lateral system. The original column layout was square 29 foot bays in both the north – south and east – west directions. With the steel frame the column layout maintains the 29 foot bays in the east – west direction, but in the north – south direction the length of the bay was cut from 29 feet to 14.5 feet. The length of the north – south bays needed to be cut down for the redesign because the construction loads could not be supported by the pre-composite steel beams. A column lay out for the steel frame can be seen below in Figure 10. Since the Patient Tower has a very regular footprint the columns were designed in three different categories with included; interior, exterior and corner. Each of these categories has a different tributary area giving different loads at each level. The Columns for the Patient Tower were designed to be all W12 wide flanged steel with splices ever 2 or 4 stories. The wide flanges range from W12 x 120 on the ground floor to W12 x 40 at the roof level the column sizes for each floor can be seen below in Tables 11, 12 & 13. Detailed calculations for all of the columns can be found in Appendix I.

Table 11	
Interior Column Sizing	
Floor	Column Size
Roof	W12 x 50
11	W12 x 50
10	W12 x 50
9	W12 x 50
8	W12 x 79
7	W12 x 79
6	W12 x 79
5	W12 x 79
4	W12 x 120
3	W12 x 120
2	W12 x 120
1	W12 x 120

Table 12	
Corner Column Sizing	
Floor	Column Size
Roof	W12 x 40
11	W12 x 40
10	W12 x 40
9	W12 x 40
8	W12 x 40
7	W12 x 40
6	W12 x 40
5	W12 x 40
4	W12 x 53
3	W12 x 53
2	W12 x 53
1	W12 x 53

Table 13	
Exterior Column Sizing	
Floor	Column Size
Roof	W12 x 40
11	W12 x 40
10	W12 x 40
9	W12 x 40
8	W12 x 53
7	W12 x 53
6	W12 x 53
5	W12 x 53
4	W12 x 72
3	W12 x 72
2	W12 x 72
1	W12 x 72

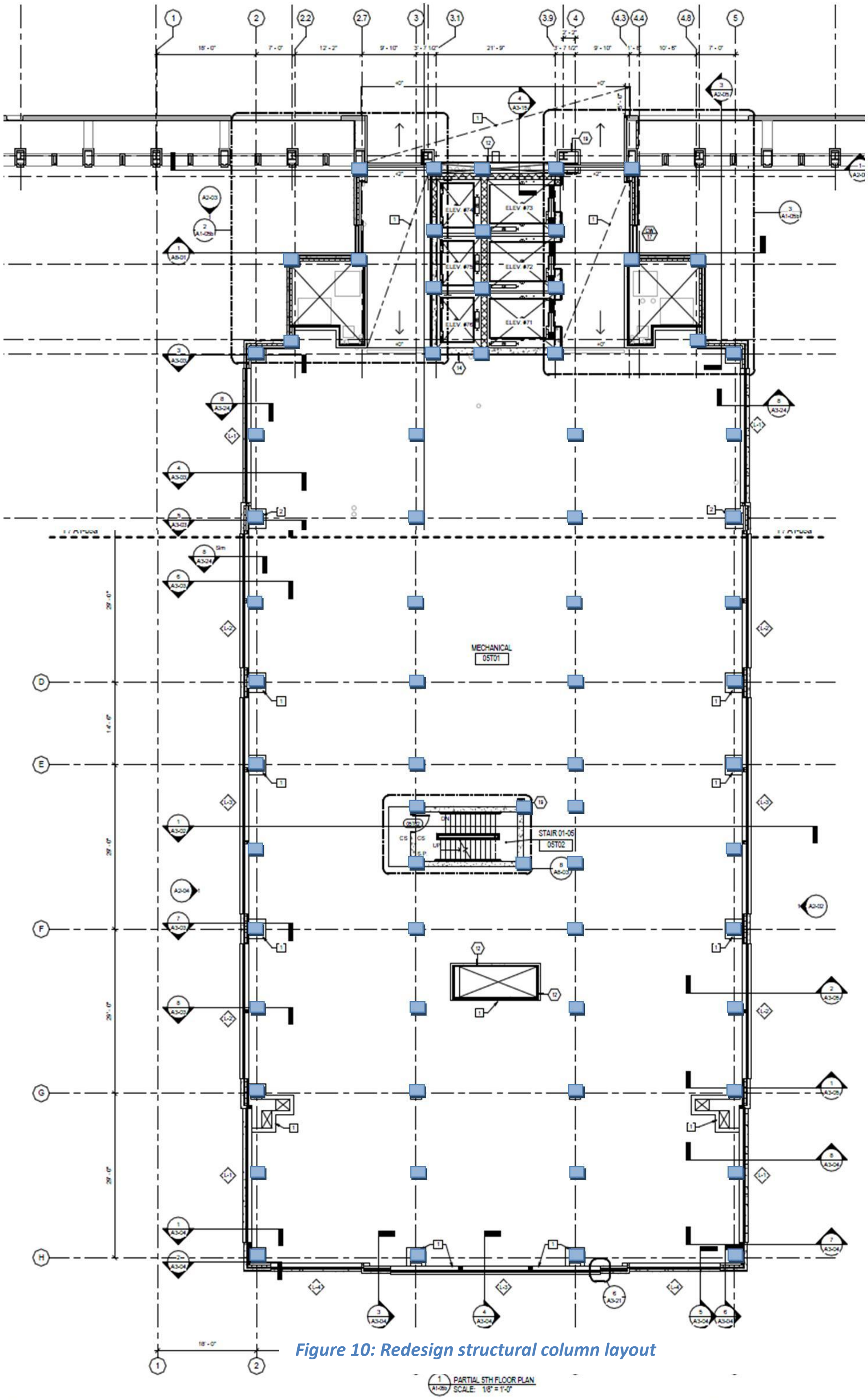


Figure 10: Redesign structural column layout

Floor System

Figure 11: D-beam system from Girder-Slab

The floor system that is being used with this steel frame design is a composite steel and precast system from Girder Slab. This system utilizes modified wide flange steel beams coupled with precast hollow core plank to create a composite action between both of these elements. The modified wide flange is known as a D-Beam in the Girder Slab system (Figure 11), and the concrete plank will bear on the bottom flange of the D-Beam instead of on the top flange of a typical beam section. The modifications to the typical wide flange section include a staggered cut that is placed down the web of a W14 x 61 cutting it in to two equal halves, Once the beam is cut, a Top bar is placed to act as the top flange that was removed in the cutting process. The Top bar is sizes so that it will replace the area of the top flange but will have a lesser width to allow the placement of the plank on the bottom flange. The specified sections that are needed for this design to carry the loads for the Patient Tower include an 8” x 4’ precast concrete plank resting on a DB 9 x 46 with 5000psi grout generation the composite reaction. In this design, a two inch concrete topping was added to this system to allow for a more rigid system and to allow for an ease in the assembly of the floor covering. A detail for the construction of this system can be seen below in Figure 12.

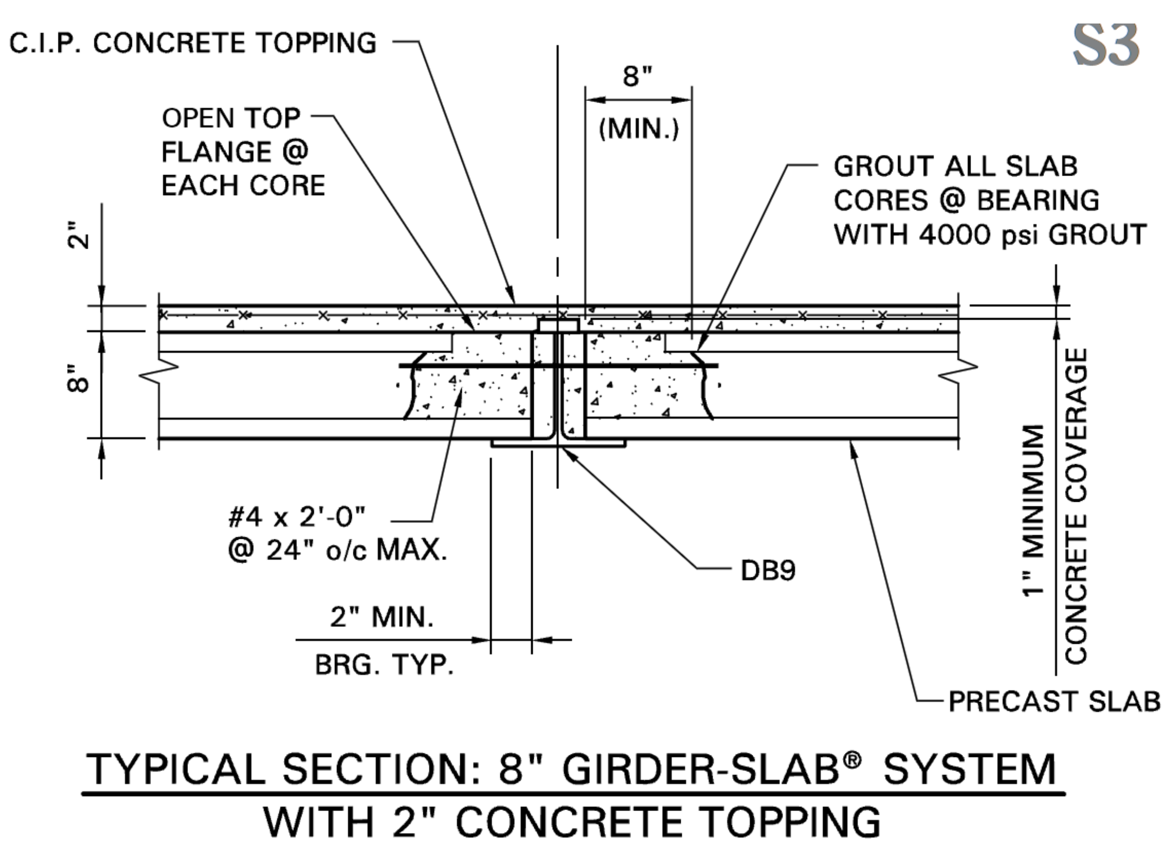
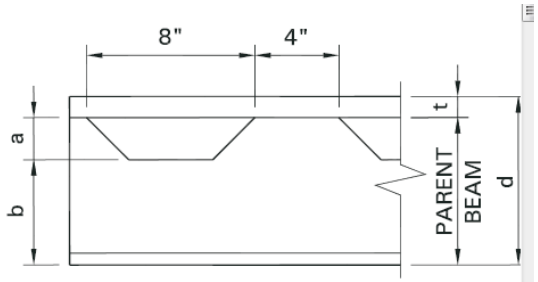


Figure 12: D-beam section system from Girder-Slab

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in²	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5



D-Beam® Reference Calculator is Available on Website. www.girder-slab.com

Figure 13: D-beam properties from Girder-Slab

Strand Pattern Designation
76-S

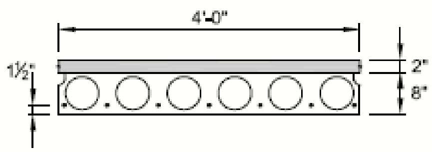
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped		Topped	
A =	215	in.²	311	in.²
I =	1,666	in.⁴	3,071	in.⁴
y_b =	4.00	in.	5.29	in.
y_t =	4.00	in.	4.71	in.
S_b =	417	in.³	581	in.³
S_t =	417	in.³	652	in.³
wt =	224	plf	324	plf
DL =	56	psf	81	psf
V/S =	1.92	in.		

4HC8 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

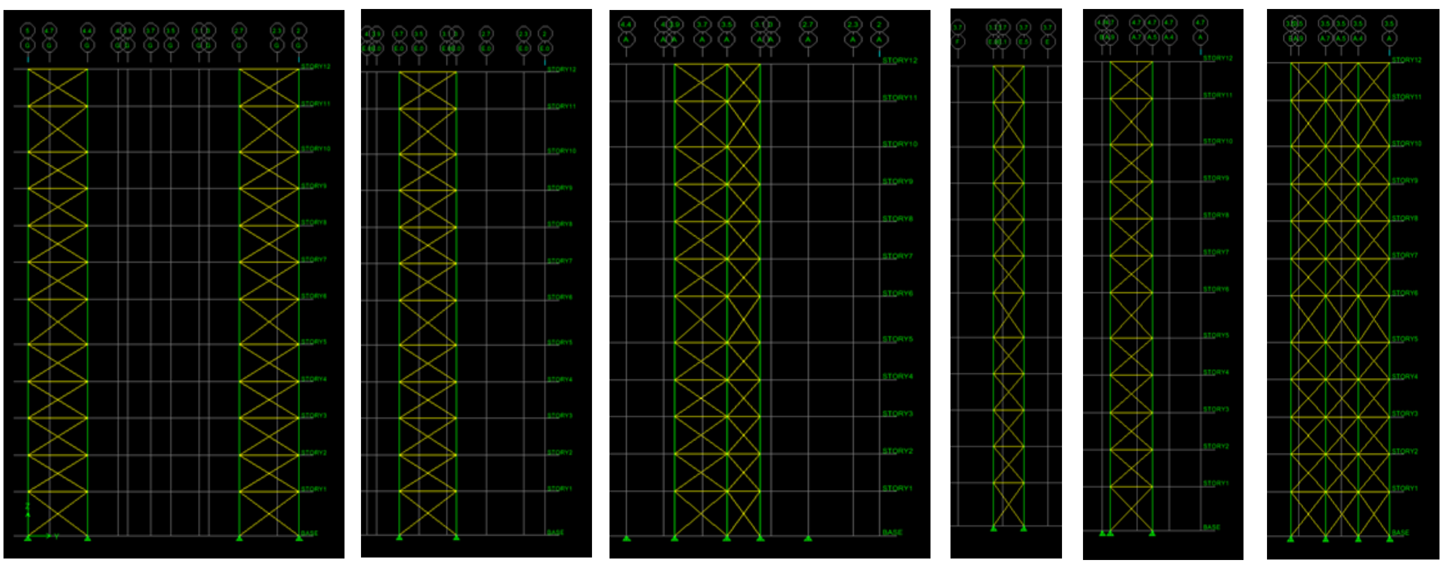
2–32

PCI Design Handbook/Sixth Edition
First Printing/CD-ROM Edition

Figure 14: Hollow core plank tables from PCI

Lateral System

In the design of the lateral force resisting system, a system of “X” braces were used to resist resisting the wind and seismic loads, as well as, the building torsion that would be caused by this loading. These braces were placed along ten different lines of action within the floor plan of the Patient Tower. The layout of these braces can be found in Figure 16. The braces for the steel frame were able to be placed in the space of the existing shear cores with a few additions within existing partitions as to not disrupt the existing floor plan. For the design of these braces, a structural model of the building was constructed in Etabs to be used in the analysis. The loads for the lateral design were found during Technical Report III using ASCE 7-10 and can be found in the lateral loads section of this report. Once all of the loads and load cases were placed in to the Etabs model, an analysis was run with different configurations and locations of the bracing until a suitable combination was found. The “X” braces were designed to be HSS 10” x 12” x 0.5” sections used in tension to avoid issues with buckling. These sections were used in all of the braced frames for ease of construction and since the deflection limit was met without much extra capacity. Section views of each of the braced frames can be found in Figure 15, also addition information and calculations can be found in Appendix IV.



F-1 F-2 F-3 & 4 F-5 & 6 F-7 F-8 & 9 F-10

Figure 15: Braced frame diagrams

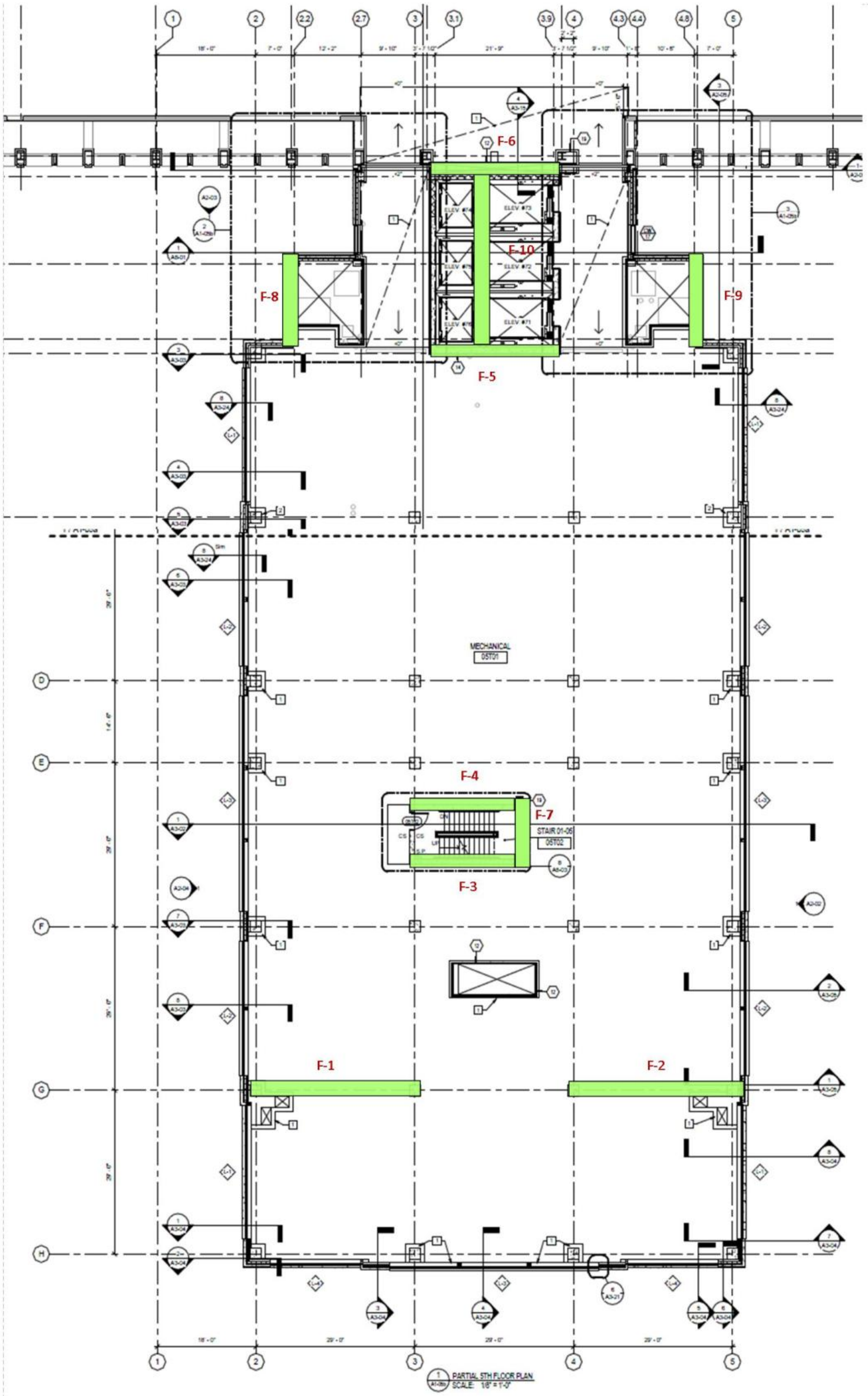


Figure 16: Braced frame locations

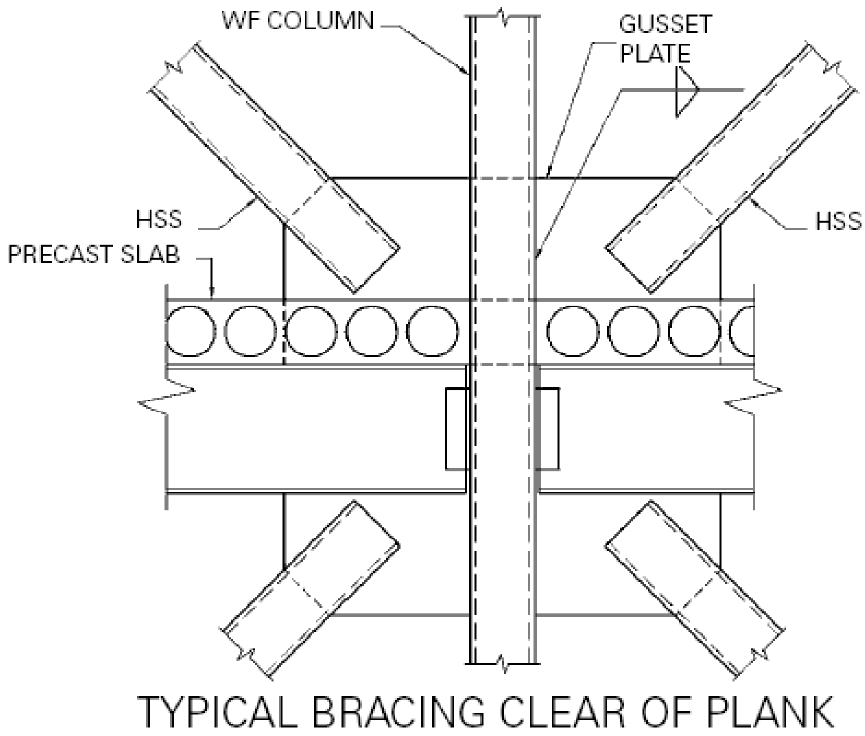


Figure 17: Detail for bracing connection to floor system

Controlling lateral loads

Upon evaluation of the ETABS output, it was determined that Wind Case 1 of Figure 10 above from ASCE 7-10 controls the proposed braced frames LFRS in the y-direction. In the x-direction seismic loads control the design of the braced frame LFRS. See Table 14 summarizing the output from Etabs for the four wind load cases and the seismic loads. Detailed calculations for the lateral load cases can be found in Appendix III.

Table 14 - Lateral Load Cases							
Cases	Location	Load	Base Shear, V_x (k)	Base Shear, V_y (k)	Torsional Moment, M_z (ft-k)	Overturning Moment, M_x (ft-k)	Overturning Moment, M_y (ft-k)
Case 1	Base	WX	-441.6	0	230515.2	0	-437029
Case 1	Base	WY	0	-744.4	-851966	746575.2	0
Case 2	Base	WX	-332	0	119652	0	-329248
Case 2	Base	WY	0	-559	-831764	559398	0
Case 3	Base	WXY	-332	-559	-466472	559398	-329248
Case 4	Base	WXY	-248	-419	-453902	419822	-245568
Seismic	Base	QX	-649.9	0	373398.9	0	-783801
Seismic	Base	QY	0	-649.9	-790728	783800.6	0

Relative Stiffness

In order to calculate the shear and torsion that would be placed on each of the braced frames, the relative stiffness for each frame needed to be found. A unit load method was used for these calculations. A load of 100 kips was placed at the top of each frame separately to measure the deflection. Once a deflection has been found for the frame it is divided by the unit load placed to give the deflection in order to find the Story Stiffness, K_i. To find the Relative story stiffness the sum of K_i is needed for each level as seen in table 15 below. The detailed calculations can be found in Appendix III

$$K_i = P/\Delta p$$

The relative stiffness's for each frame can now be calculated using the formula below. Once the relative stiffness for each frame is found then center of rigidity torsion and shear can be calculated.

$$R_i = K_i/K_{i,total}$$

Table 15 - Relative Story Stiffness, R_{iy}

Level	Total Story Stiffness $K_{iy,total}$	Relative Story Stiffness, R_{iy} $R_{iy} = K_{iy}/K_{iy,total}$						
		Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	∑R
Roof	18.50	0.143	0.143	0.134	0.134	0.222	0.222	1.0
11	20.77	0.143	0.143	0.134	0.134	0.223	0.223	1.0
10	24.37	0.143	0.143	0.134	0.134	0.223	0.223	1.0
9	28.36	0.143	0.143	0.134	0.134	0.223	0.223	1.0
8	33.70	0.143	0.143	0.134	0.134	0.223	0.223	1.0
7	41.16	0.143	0.143	0.134	0.134	0.223	0.223	1.0
6	51.96	0.143	0.143	0.134	0.134	0.224	0.224	1.0
5	73.64	0.143	0.143	0.134	0.134	0.223	0.223	1.0
4	104.09	0.141	0.141	0.135	0.135	0.223	0.223	1.0
3	160.36	0.142	0.142	0.136	0.136	0.223	0.223	1.0
2	288.33	0.145	0.145	0.139	0.139	0.217	0.217	1.0
1	600.00	0.167	0.167	0.167	0.167	0.167	0.167	1.0

Center of Mass and Center of Rigidity

With the addition of more shear walls in the steel redesign, new center of mass and rigidity calculations needed to be performed. Both of these calculations were determined by Etabs and by hand for comparison. The values for the center of mass and center of rigidity can be found in tables 16 & 17 below. In the comparisons there are some discrepancies due to the irregularity of the building shape at the north end that was assumed to regular for the hand calculations.

Table 16 - Center of Mass Etabs Vs. Hand Calculations (inch)

X Center of Mass (Etabs)	X Center of Mass (Hand)	Y Center of Mass (Etabs)	Y Center of Mass (Hand)
522	522	1104	1145

Table 17 - Center of Rigidity Etabs Vs. Hand Calculations (inch)

X Center of Rigidity (Etabs)	X Center of Rigidity (Hand)	Y Center of Rigidity (Etabs)	Y Center of Rigidity (Hand)
542	502	1285	1280

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 distance between the center of mass and the center of rigidity which 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 18 - Overall Building Torsion

North - South Direction						
Story	Lateral Force (k)	Factored Lateral Force (k)	COR-COM (ft)	M_t (Ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Roof	52	82	1.6	132	371	503
11	39	62	1.6	99	279	378
10	38	61	1.6	97	274	372
9	37	60	1.6	96	270	366
8	37	59	1.6	94	265	359
7	36	58	1.6	92	259	352
6	35	56	1.6	90	253	343
5	41	66	1.6	106	297	403
4	33	53	1.6	84	236	320
3	31	50	1.6	81	227	307
2	30	48	1.6	76	214	290
1	33	52	1.6	83	235	318

Table 19 - Overall Building Torsion						
East - West Direction						
Story	Lateral Force (k)	Factored Lateral Force (k)	COR-COM (ft)	M_t (Ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Roof	90	143	15	2148	1368	3516
11	67	107	15	1608	1024	2632
10	66	105	15	1575	1003	2578
9	64	103	15	1544	983	2527
8	63	101	15	1510	961	2471
7	61	98	15	1471	936	2407
6	59	95	15	1425	907	2332
5	69	111	15	1658	1056	2714
4	54	87	15	1308	833	2141
3	52	83	15	1239	789	2029
2	48	77	15	1151	733	1883
1	51	82	15	1232	785	2017

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 20 and 21. Detailed calculations of these values can also be found in Appendix III.

Table 20 - Story Shear, North-South						
Level	Story Force (kips)	Story Shear Per Frame (Kips)				
		Frame 7	Frame 8	Frame 9	Frame 10	Sum
		0.069	0.126	0.126	0.678	1.0
Roof	52	4	6	6	35	52
11	39	3	5	5	26	39
10	38	3	5	5	26	38
9	37	3	5	5	25	37
8	37	3	5	5	25	37
7	36	2	5	5	24	36
6	35	2	4	4	24	35
5	41	3	5	5	28	41
4	33	2	4	4	22	33
3	31	2	4	4	21	31
2	30	2	4	4	20	30
1	33	2	4	4	22	33

Table 21 - Story Shear, East-West								
Level	Story Force (Kips)	Story Shear Per Frame (Kips)						
		Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Sum
		0.145	0.145	0.137	0.137	0.218	0.218	1.0
Roof	90	13	13	12	12	19	19	90
11	67	10	10	9	9	15	15	67
10	66	10	10	9	9	14	14	66
9	64	9	9	9	9	14	14	64
8	63	9	9	9	9	14	14	63
7	61	9	9	8	8	13	13	61
6	59	9	9	8	8	13	13	59
5	69	10	10	9	9	15	15	69
4	54	8	8	7	7	12	12	54
3	52	7	7	7	7	11	11	52
2	48	7	7	7	7	10	10	48
1	51	7	7	7	7	11	11	51

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 can be found in table 22. Detailed calculations of how the torsional shear was calculated can be found in Appendix III.

Table 22 - Torsional Shear

		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance From COM to COR (e) (in)	Distance from $Wall_i$ to Origin (in)	Distance from $Wall_i$ to COR d_i (in)	$(R_i)(d_i)^2$	Torsional Shear (k)
Wall 1	E-W	745	0.145	-181	348	-937	127103	29
Wall 2	E-W	745	0.145	-181	348	-937	127103	29
Wall 3	E-W	745	0.137	-181	834	-451	27955	13
Wall 4	E-W	745	0.137	-181	950	-335	15424	10
Wall 5	E-W	745	0.218	-181	1941	656	93723	-30
Wall 6	E-W	745	0.218	-181	2289	1004	219536	-46
Wall 7	N - S	442	0.069	-20	570	28	54	0
Wall 8	N - S	442	0.126	-20	84	-458	26448	1
Wall 9	N - S	442	0.126	-20	960	418	22030	-1
Wall 10	N - S	442	0.678	-20	492	-50	1696	0
						Sum	637348	

Overturning

Overturning issues in the foundation arise when the forces on the lateral elements are greater than the gravity weight that is applied to lateral element frame. The building foundation can also resist some of these forces with the capacity for soil bearing and pile friction forces. The Patient Towers foundations don't resist any up lift due to overturning because the uplift forces at the base of the braced frames are out weighted by the amount of gravity force applied to the columns. The largest negative force found in the foundation is 716k for an interior column which has a gravity load of 1210k. Since the existing gravity loading is greater than the uplift force created by overturning, the foundation will not need to be designed to resist this force.

Displacement

The displacement of the building should be limited as much as to not disturb the occupants inside the structure. Building displacement falls under the serviceability considerations and is related to the rigidity of each of the buildings braced frames. As a structure gets taller, the more important the displacement of the building becomes and a larger of a factor it will be. The displacement limitation for wind loading is an allowable displacement of $\Delta = L/400$. Seismic drift is limited to $\Delta = 0.015h_{sx}$.

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

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 2.72" and 4.02"in the E/W direction. The drifts in both directions are less than the 4.3" limitation. The ETABS modal analysis does analyze the drift and displacements with all the shear walls working together as a lateral resisting system.

Diaphragm CM Displacements

Edit View

Diaphragm CM Displacements

Story	Diaphragm	Load	UX	UY	UZ
STORY12	D1	COMB1	-0.0026	-0.0145	0.0000
STORY12	D1	COMB2	-0.0022	-0.0124	0.0000
STORY12	D1	COMB3	0.7116	-0.0214	0.0000
STORY12	D1	COMB4	-0.0140	1.8134	0.0000
STORY12	D1	COMB5	1.4254	-0.0304	0.0000
STORY12	D1	COMB6	-0.0258	3.6392	0.0000
STORY12	D1	COMB7	2.7230	0.0153	0.0000
STORY12	D1	COMB8	-0.0170	4.0237	0.0000
STORY12	D1	COMB9	1.4260	-0.0273	0.0000
STORY12	D1	COMB10	-0.0253	3.6423	0.0000
STORY12	D1	COMB11	2.7235	0.0184	0.0000
STORY12	D1	COMB12	-0.0164	4.0268	0.0000
STORY12	D1	COMB13	-0.0022	-0.0124	0.0000
STORY11	D1	DEAD	0.0016	0.0001	0.0000

Figure 18: Diaphragm displacements from Etabs

Schedule Impacts

The existing design of the Patient Tower consists of a reinforced concrete frame with reinforced concrete slab and shear walls. The steps that need to be scheduled for the construction of this concrete design are as follows; form/rebar/pour the columns and walls, Form the deck, Lay the deck rebar, and pour the deck. Each of these tasks in the schedule has a different duration depending on the size and amount of material. In comparison, the steel system tasks are to erect each piece of steel as it is needed, then once enough of the steel sections are in place the concrete plank can be set in place. The schedule for the existing design will take 97 work days for the structure to be completed. In comparison the steel structural design will take just 24 workdays to construct. Using this number it is calculated that the steel design could be constructed 75% faster than the concrete design. With this decrease in length of construction, the tower would be able to be opened for operations about 3 and a half months earlier than the previous design. Detailed schedules for both system designs can be found in Appendix IV.

Table 23 - Structural Erection Time comparison (# days)

Concrete Frame with Shear walls	Steel Frame with Hollow core plank
97	24

Cost Impacts

Similar to the differences in construction time, these systems are also very different in cost as well. The original design of the Patient Tower with the reinforced concrete frame has a cheaper cost per square foot then the new design with steel. As can be seen in table 24 below the concrete frame is about nine dollars per square foot cheaper than the new design in steel, which equates to about a two million dollar increase in the base cost of the structure. The cost data was taken for RS means 2011 assemblies tables.

Table 24 - Systems Cost Comparison

Concrete Frame with Shear walls	Steel Frame with Hollow core plank
$ 18/sf	$ 27/sf
$ 4,000,000	$ 6,000,000

With the structure design changing from concrete to steel the total dead weight of the building decreased. The reinforced concrete design of the Patient Tower has a dead weight of approximately 44,000 kips where the weight of the steel design is about 30,000 kips. The difference in weight between the two designs gives a 32% reduction in building dead weight from the concrete structure to the steel structure as can be seen in table 25 below. With this 32% reduction in building weight the current foundation design consisting of drilled piles topped with a slab on grade should also be able to be decreased adding to the saving that have been gained by the steel design.

Table 25 - Total Building Weight Comparison (kips)	
Concrete Frame with Shear walls	Steel Frame with Hollow core plank
44,000	30,000

In comparing steel and concrete frame design of the Patient Tower, it is apparent that there are very distinct advantages and disadvantages for both types of construction. Subsequently comparing the schedule and cost of both of these systems, it seems as though the steel system has a slight advantage. While the steel system is more expensive per square foot by almost 65% it has major advantages in the construction time and dead weight; beating the concrete system by 75% and 32% respectively. With the faster construction time the steel system will allow the building to become operational sooner increasing revenue, and the lower dead weight will allow for a decrease in the bearing capacity of the foundation. The ability to open the hospital as much as 3 and a half months earlier using the steel design will be the controlling factor in the design bases on past average revenue data.

Breath Topic #2

Acoustical Analysis

There are two locations of the Patient Tower that need to be check for their acoustical performance. Both areas are Intensive Care Units within the tower. The general ICU is located above the Café and the other location is the Neuro ICU which is located below the Mechanical floor. Each of these locations would experience a different type of noise; The Café to ICU interaction would have Air-borne noise due to large volume of people and the Neuro ICU to Mechanical interaction would experience structural borne noise through mechanical vibration.

For this study, I would like to check the acoustical performance of the ICU. The ICU is located above the café that is open to the public. There is a concern that the noise will be carried and disrupt the ICU above. The redesigned Hollow core plank system will be complemented by an acoustical ceiling below as the original architectural design called for. With the use of this acoustical ceiling coupled with the Hollow core planks system that is topped with a 2 inch concrete topping the floor system will provide the necessary sound transition class (STC) rating required by the IBC for air-borne sound. For air-borne sound, the IBC requires that applicable walls, partitions and floor/ceiling assemblies have a sound transmission class (STC) of 50 when tested in a laboratory using ASTM E 90. The hollow core system that was used in the structural redesign provides a STC rating of 59 according to the PCI design handbook for the assembly used in the structural redesign.

Table 9.2.6.1 Airborne sound transmission loss (STC) and impact insulation class (IIC) ratings from tests of precast concrete assemblies

	Floor-Ceiling Systems		
8	8 in. hollow-core prestressed units, 57 psf	50	28
9	Assembly 8 with carpet and pad, 58 psf	50	73
10	8 in. hollow-core prestressed units with ½ in. wood block flooring adhered directly, 58 psf	51	47
11	Assembly 10 except ½ in. wood block flooring adhered to ½ in. sound-deadening board underlayment adhered to concrete, 60 psf	52	55
12	Assembly 11 with acoustical ceiling, 62 psf	59	61
13	Assembly 8 with quarry tile, 1¼ in. reinforced mortar bed with 0.4 in. nylon and carbon black spinerette matting, 76 psf	60	54
14	Assembly 13 with suspended 5/8 in. gypsum board ceiling with	61	62

Figure 19: STC and IIC ratings from PCI

The Patient Tower uses the fifth floor to house the mechanical systems directly below the mechanical floor is the Neuro ICU floor. With all of the mechanical equipment sitting on floor/ceiling assemble this will create a structural borne sounds which is classified as an Impact insulations class (IIC). For structure-borne sound, the IBC requires floor/ceiling assemblies to have an impact insulation class (IIC) of 50 when tested in accordance with ASTM E 492. The floor/ceiling assembly used in the redesign for the Patient Tower has an IIC of 61 according to the PIC Handbook. With a higher rating given by the redesign system then is needed per the code the redesign system passes the requirement. All of the Mechanical equipment located on the 5th floor is supported by a four inch concrete housekeeping pad and vibration eliminating mounts. It is specified for the Patient Tower that all of the mechanical connections to the structure must be isolated by a vibration isolator to prevent and structural borne noise.

Design Goals

To evaluate the success of the redesigned structure design goals were set fourth at the beginning of this analysis. The goals are listed below with conclusions and arguments to support whether or not the design goals have been successfully achieved.

1) Design a steel structure that has little impact on the existing architecture of the Hospital Patient Tower.

   This goal was achieved during the redesigning the structural system for the patient tower with a few modifications. The length of the bays had to be shortened in one direction and braced frames have to be added. Both of these elements were placed within existing partitions in order to not change the interior layout of the tower.

2) Maintain a minimal floor to floor height to maintain the proper connection to the existing tower.

   This goal was achieved for the redesign of the Patient Tower with the use of the Girder-slab system; it is a composite steel and precast hollow plank system that will allow this low floor to floor height while still maintaining enough space for the MEP equipment needed for a hospital. The success of this design will allow the connection between the new tower and the existing tower to be kept at every floor for easy transport of hospital personal and patients.

3) Design a steel structure to decreases the cost of the tower.

   The steel redesign for the south patient tower didn't decrease the cost of the structure but it does have an effect on the overall cost of the tower. The steel structural cost was an increase over that of the original concrete design but there is a decrease in foundation and construction time for the steel structure that out weights the structural cost increase.

Conclusion

This thesis report was conducted in order to determine the feasibility for redesigning the Hospital Patient Tower as a steel frame structure. After taking in to account all of the pros and cons of the redesign and the existing systems, it seems that the redesign is just slightly more beneficial do to the decrease in erection time. Through this analysis a better understanding of both types of framing systems was gained as well as how each of these systems affects the rest of the building and its other systems.

For the depth of this thesis report, the structural systems for the hospital Patient Tower was redesigned as a composite precast hollow core plank and steel beam slab with steel columns. This was a redesign from its original cast-in-place two-way slab with concrete columns. The lateral system of the patient Tower was originally design as two cast-in-place concrete shear wall cores. For the redesign, the lateral system was converted to ten frames with “X” bracing. An Etabs model was used in the analysis of the lateral system redesign as well as to check member sizes and layouts. With the criteria that the Patient Tower must connect to an existing tower at every floor it was very critical that the floor to floor height were maintained. This criteria is what lead to the use of a Girder-Slab system for the floor slab. The Girder-Slab allowed for the floor to floor heights to be maintained while will also maintaining the large spans and minimize the effects on the architectural plans.

Two breadth studies were conducted along with the depth analysis to investigate how the structural redesign affects other aspects of the Patient Tower. The first breath topic is a construction management analysis which was performed to investigate and compare the cost and schedule of the existing concrete structure and the proposed steel frame structure. It was determined that the proposed steel frame system would be approximately 2 million dollars more than the existing concrete system. The construction schedule for both the steel and concrete system were also compared, it was found that the steel system could be constructed approximately 3 an a half months faster than the existing concrete system. Due to the decrease in weight of the steel frame compared to the concrete frame there is also the opportunity for a decrease in the bearing capacity of the foundation. Both of these systems have their pros and cons making them both very feasible options for the Patient Tower.

The Second Breadth study was an acoustical study to analyze the Sound Transmission Class and Impact Insulation Class (IIC) for the two Intensive Care Units (ICU) and there adjacent spaces. Both of the towers ICU units are located either above or below a potential noise source. The regular ICU is located above the towers café which will have a large amount of air borne sound and the Neuro ICU is located below the mechanical level on the fifth floor which will have high structural borne noise. These two spaces were check for their specific type of noise so that it does not disturb the occupants. In both cases the existing elements of the design were able to meet the criteria needed for the spaces.

Appendix I

This section of the Final Report is where the supplementary information for the Gravity System Redesign Calculations for the Hospital Patient Tower can be found.

D-Beam® Calculator Reference Tool
1/20/2011

Project Name: Hospital Patient Tower
Job Number: 1.000

Design Information

Dead Load =	60	psf
Partition Load =	35	psf
Live Load =	80	psf
Topping Load =	25	psf
DB Span =	14.5	ft
Plank Span =	29	ft
Grout f'c =	5000	psi
Allowable Δ_{LL} = L /	360	
Allowable Δ_{LL} =	0.48	in

DB Properties

DB Size ------------> DB 9 x 46

Steel Section			**Transformed Section**		
I_s =	195	in^4	I_t =	356	in^4
S_t =	33.7	in^3	S_t =	68.6	in^3
S_b =	50.8	in^3	S_b =	80.6	in^3
M_{scap} =	84.0	ft-k			
t_w =	0.375	in			
b =	5.75	in			

Live Load Reduction (IBC 00/03/06)

Include LLR	☑	(Check for Yes)
% Reduction =	23.28	%
Reduced Load =	61.4	psf

Initial Load - Precomposite

M_{DL} =	45.7	ft-k	<	84.0 ft-k	OK
Δ_{DL} =	0.31	in			
Δ Ratio = L /	569				
Camber D-Beam	☐	(Check for Yes)			
D-Beam Camber	1	in			

Total Load - Composite

M_{sup} =	92.5	ft-k			
M_{TL} =	138.2	ft-k			
S_{REQ} =	55.3	in^3	<	68.6 in^3	OK
Δ_{SUP} =	0.34	in	<	0.48 in	OK
Δ_{TOT} =	0.65	in	= L/ 270		

Superimposed Compressive Stress on Concrete

N value =	7.20				
S_{tc} =	494	in^3			
f_c =	2.25	ksi			
F_c =	2.25	ksi	>	2.25 ksi	OK

Bottom Flange Tension Stress (Total Load)

f_b =	24.6	ksi			
F_b =	45	ksi	>	24.6 ksi	OK

Shear Check

Total Load =	181	psf			
w =	5.26	klf			
R =	38.1	k			
f_v =	17.7	ksi			
Fv =	20	ksi	>	17.7 ksi	OK

D-Beam® Calculator Reference Tool

1/25/2011

Project Name: Hospital Patient Tower Mechanical floor
Job Number: 1.000

Design Information

Dead Load =	60	psf
Partition Load =	15	psf
Live Load =	100	psf
Topping Load =	25	psf
DB Span =	14.5	ft
Plank Span =	29	ft
Grout f'c =	7000	psi
Allowable Δ_{LL} = L /	360	
Allowable Δ_{LL} =	0.48	in

DB Properties

DB Size ------------> DB 9 x 46

Steel Section		Transformed Section	
I_s =	195 in^4	I_t =	356 in^4
S_t =	33.7 in^3	S_t =	68.6 in^3
S_b =	50.8 in^3	S_b =	80.6 in^3
M_{scap} =	84.0 ft-k		
t_w =	0.375 in		
b =	5.75 in		

Live Load Reduction (IBC 00/03/06)

Include LLR ☐ (Check for Yes)
% Reduction = N/A
Reduced Load = N/A

Initial Load - Precomposite

M_{DL} =	45.7 ft-k	<	84.0 ft-k	OK
Δ_{DL} =	0.31 in			
Δ Ratio = L /	569			
Camber D-Beam ☐	(Check for Yes)			
D-Beam Camber	1 in			

Total Load - Composite

M_{sup} =	106.7 ft-k			
M_{TL} =	152.4 ft-k			
S_{REQ} =	61.0 in^3	<	68.6 in^3	OK
Δ_{SUP} =	0.39 in	<	0.48 in	OK
Δ_{TOT} =	0.70 in	= L/ 250		

Superimposed Compressive Stress on Concrete

N value =	6.08			
S_{tc} =	417 in^3			
f_c =	3.07 ksi			
F_c =	3.15 ksi	>	3.07 ksi	OK

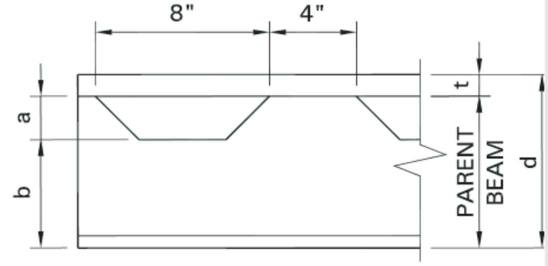
Bottom Flange Tension Stress (Total Load)

f_b =	26.7 ksi			
F_b =	45 ksi	>	26.7 ksi	OK

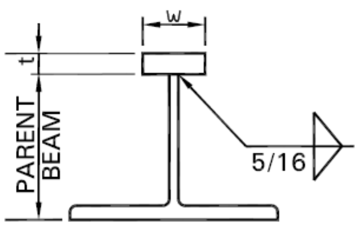
Shear Check

Total Load =	200 psf			
w =	5.80 klf			
R =	42.1 k			
f_v =	19.5 ksi			
Fv =	20 ksi	>	19.5 ksi	OK

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in²	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5



D-Beam® Reference Calculator is Available on Website. www.girder-slab.com



Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Floor System

Loads

Plank DL = 60psf

Superimposed = 35psf (MEP & Partitions)

Live Load = 80psf

Topping Load = 25 psf (installed after grout has cured)

LL_r

Plank Span 29'

D Beam Span 14.5'

Allowable $\Delta_{LL} = L/360 = 14.5(12)/360 = 0.483''$

Initial load pre composite

$M_{DL} = (29')(0.06\ ksf)(14.5')^2/8 = 45.7\ ft \cdot k < 84\ ft \cdot k$ ✓

$$D_{DL} = \frac{5 W l^4}{384\ EI} = \frac{5(29')(0.06\ ksf)(14.5)^4(1728\ in^3/ft^3)}{384(29{,}000\ KSI)(195\ in^4)}$$

$= 0.306\ in < 0.483''$

Total load Composite

$M_{sup} = 29'(0.061 + 0.035 + 0.025\ klf)(14.5)^2/8 = 92.2\ ft \cdot k$

$M_{tot} = M_{sup} + M_{DL} = 92.2 + 45.7\ ft \cdot k = 137.9\ ft \cdot k$ ✓

$S_{req} = (137.9\ ft \cdot k)(12\ in/ft)/(0.6)(50\ ksi) = 55.16\ in^3 < 68.6\ in^3$

$$\Delta_{LL} = \frac{5(29)(0.061 + 0.035 + 0.025\ ksf)(14.5')^4(1728\ in^3/ft^3)}{(384)(356\ in^4)(29000\ k/in^2)}$$

$= 0.338\ in < 0.483\ in$ ✓

Check Compressive Stress on Concrete

$N \text{ value} = \frac{E \text{ steel}}{E \text{ Concrete}} = \frac{29000 \text{ ksi}}{57000(5 \text{ ksi})^{1/2}} = 7.19$

$S_{tc} = 7.19(68.6) = 493.2 \text{ in}^3$

$f_c = M_{sup}(12 \text{ in/ft}) / S_{tc} = 92.2(12 \text{ in/ft}) / 493.2 \text{ in}^3 = 2.243 \text{ ksi}$

$F_c = 0.45(5000)/1000 = 2.25 \text{ ksi}$

$f_c \leq F_c \quad 2.24 \leq 2.25 \text{ ksi}$ ✓

Check bottom flange tension stress (total load)

$f_b = \frac{(45.7 \text{ kft})(12 \text{ in/ft})}{50.8 \text{ in}^3} + \frac{(92.2 \text{ kft})(12 \text{ in/ft})}{80.6 \text{ in}^3} = 24.5 \text{ ksi}$

$F_b = 0.9(50 \text{ ksi}) = 45 \text{ ksi} > 24.5 \text{ ksi}$ ✓

Check Shear

Total load = $(60 + 35 + 61 + 25) = 181 \text{ psf}$

$W_u = (0.181)(29) = 5.25 \text{ k/ft}$

$R = (5.25)(14.5)/2 = 38.1 \text{ k}$

$f_v = 38.1/(0.375)(5.75 \text{ in}) = 17.66 \text{ ksi}$

$F_v = 0.4(50 \text{ ksi}) = 20 \text{ ksi} > 17.66 \text{ ksi}$ ✓

Edge Beams

$1.2D + 1.6L$

Plank DL = 60 psf

Superimposed = 35 psf (MEP + Partions)

Live load = 61 psf

Topping Loads = 25 psf

Curtain Wall = 25 psf × 12' = 300 plf

Span = 14.5'

Area = 14.5' × 15.5' = 224.75 sf

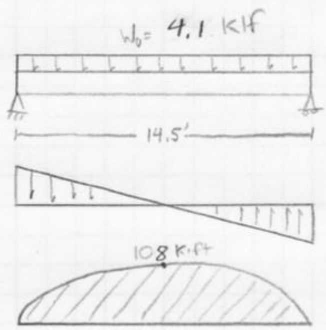
$1.2(120) + 1.6(61)$

$W_u = 242$ psf

$W_u = 242 \times 15.5' = 3751$ PLF

= 3.8 klf + 0.3 klf

= 4.1 klf



$\frac{wL}{2} = \frac{4.1(14.5)}{2} = 30$ k

$\frac{wL^2}{8} = \frac{4.1(14.5^2)}{8} = 108$ k·ft

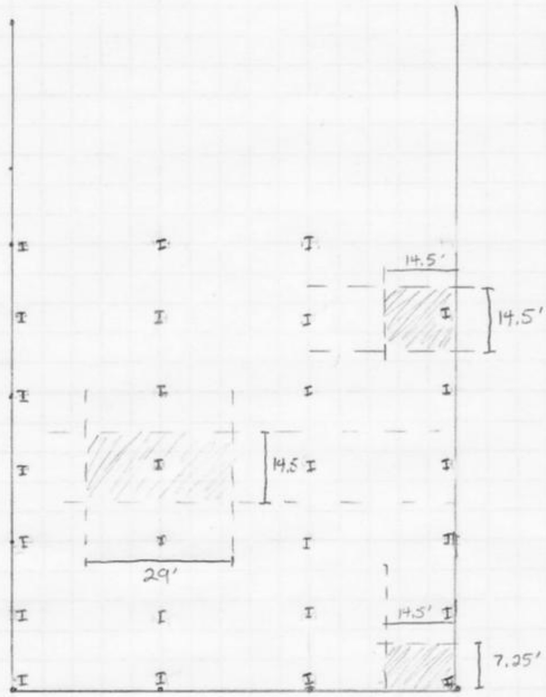
W 12×22 $\phi M_{px} = 110$ k·f $\phi V_n = 96$ k > 30 ✓

Check deflection

$\Delta_{LL} = \frac{5 W_{LL} L^4}{384 E I_x} = \frac{5(0.95)(14.5)^4 \, 1728}{384(29000)(156 \text{ in}^4)} = 0.208 \leq \frac{14.5(12)}{360} = 0.483$ ✓

$\Delta_{TL} = \frac{5(W_{TL}) L^4}{384 E I_x} = \frac{5(2.8)(14.5)^4 \, 1728}{384(29000)(156 \text{ in}^4)} = 0.62 \text{ in} \leq \frac{14.5(12)}{240} = 0.725 \text{ in}$ ✓

Column design



exterion Wall load

assume 25 psf

Corner Column

$L = 21.25'$

$H = 12'$

Area = 33.75 sf

$P = 33.75 \cdot 25$

$= 843.75$ lbs

$= 0.84$ Kips

Loads

all floors except 5

DL = 60 psf

Super = 35 psf

LL = 61 psf

Topping = 25 psf

$1.2(DL) + 1.6(LL)$

$1.2(60 + 35 + 25) + 1.6(61) = 242$ psf

5 floor

DL = 60 psf

Super = 15 psf

LL = 100 psf

Topping = 25 psf

$1.2(60 + 15 + 25) + 1.6(100) = 280$ psf

Roof

DL = 60 psf

Super = 15 psf

LL = 60 psf Non reducable

topping = 25 psf

$1.2(60 + 15 + 25) + 0.5(60) = 150$ psf

Interior Column Loading									
Floor	Area (SF)	Load (PSF)	Column Load (lbs)	Self-Weight (lbs)	Total Load (lbs)	Total Load (Kip)	Column Location	Column Size	ϕP_n (Kips)
Roof	420.5	150	63075	600	63675	64	11	W12 x 50	384
11	420.5	242	101761	600	166036	166	10	W12 x 50	384
10	420.5	242	101761	600	268397	268	9	W12 x 50	384
9	420.5	242	101761	600	370758	371	8	W12 x 50	384
8	420.5	242	101761	948	473467	473	7	W12 x 79	836
7	420.5	242	101761	948	576176	576	6	W12 x 79	836
6	420.5	242	101761	948	678885	679	5	W12 x 79	836
5	420.5	280	117740	948	797573	798	4	W12 x 79	836
4	420.5	242	101761	1440	900774	901	3	W12 x 120	1290
3	420.5	242	101761	1440	1003975	1004	2	W12 x 120	1290
2	420.5	242	101761	1440	1107176	1107	1	W12 x 120	1290
1	420.5	242	101761	1440	1210377	1210	Ground	W12 x 120	1290

Corner Column Loading										
Floor	Area (SF)	Load (PSF)	Column Load (lbs)	Exterior wall Load (lbs)	Self Weight (lbs)	Total Load (lbs)	Total Load (Kip)	Column Location	Column Size	ϕP_n (Kips)
Roof	106	150	15900	6375	480	22755	23	11	W12 x 40	280
11	106	242	25652	6375	480	55262	55	10	W12 x 40	280
10	106	242	25652	6375	480	87769	88	9	W12 x 40	280
9	106	242	25652	6375	480	120276	120	8	W12 x 40	280
8	106	242	25652	6375	480	152783	153	7	W12 x 40	280
7	106	242	25652	6375	480	185290	185	6	W12 x 40	280
6	106	242	25652	6375	480	217797	218	5	W12 x 40	280
5	106	280	29680	6375	480	254332	254	4	W12 x 40	280
4	106	242	25652	6375	636	286995	287	3	W12 x 53	477
3	106	242	25652	6375	636	319658	320	2	W12 x 53	477
2	106	242	25652	6375	636	352321	352	1	W12 x 53	477
1	106	242	25652	6375	636	384984	385	Ground	W12 x 53	477

Exterior Column Loading										
Floor	Area (SF)	Load (PSF)	Column Load (lbs)	Exterior wall Load (lbs)	Self Weight (lbs)	Total Load (lbs)	Total Load (Kip)	Column Location	Column Size	ϕP_n (Kips)
Roof	211	150	31650	4350	480	36480	36	11	W12 x 40	304
11	211	242	51062	4350	480	92372	92	10	W12 x 40	304
10	211	242	51062	4350	480	148264	148	9	W12 x 40	304
9	211	242	51062	4350	480	204156	204	8	W12 x 40	304
8	211	242	51062	4350	636	260204	260	7	W12 x 53	477
7	211	242	51062	4350	636	316252	316	6	W12 x 53	477
6	211	242	51062	4350	636	372300	372	5	W12 x 53	477
5	211	280	59080	4350	636	436366	436	4	W12 x 53	477
4	211	242	51062	4350	876	492654	493	3	W12 x 72	736
3	211	242	51062	4350	876	548942	549	2	W12 x 72	736
2	211	242	51062	4350	876	605230	605	1	W12 x 72	736
1	211	242	51062	4350	876	661518	662	Ground	W12 x 72	736

Appendix II

This section of the Final Report is where the supplementary information for the Wind and Seismic Calculations for the Hospital Patient Tower can be found.

Wind Design | Patient tower

Occupancy Catagory 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} ± 0.55

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)

Wind Design | Patient tower

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

$P = (0.85)(0.8) q_z = (0.68) q_z$

E-W

$P = (0.85)(0.8) q_z = (0.68) q_z$

Leeward walls

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

N-S

$P = q_h (0.85)(-0.5) = (-0.425) q_h$

E-W

$P = q_h (0.85)(-0.3) = (-0.281) q_h$

Wind Design | Patient Tower

Natural frequency <u>N-S</u>

building height 146' < 300' ✓

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

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

$A_B = 17190 \text{ ft}^2$

$h = 146'$

$h_i = 146'$

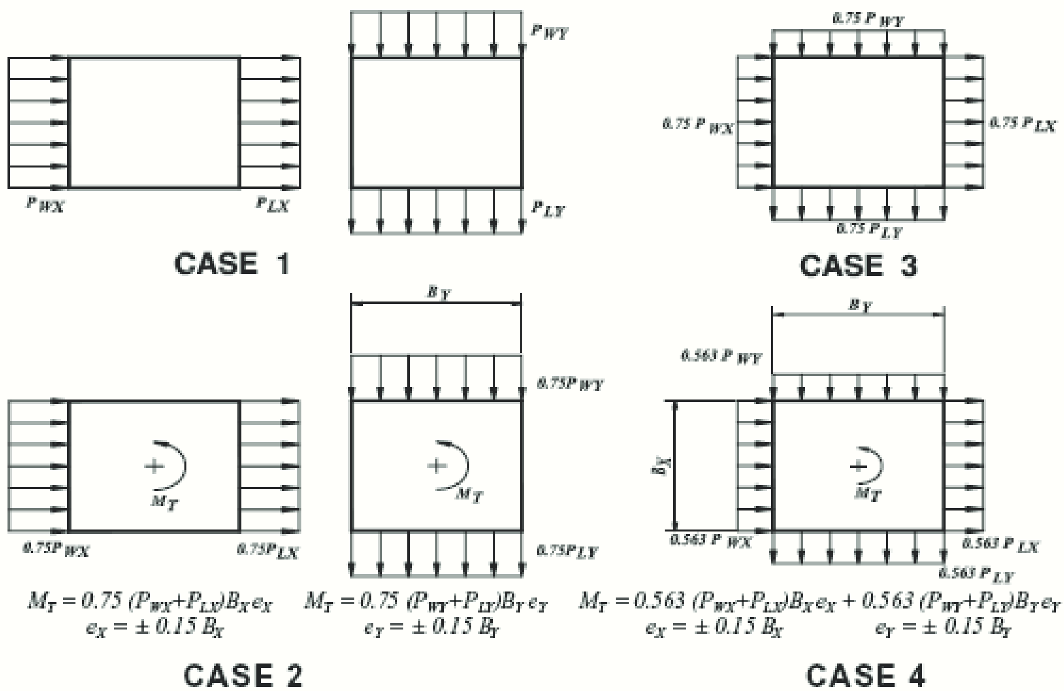
$n_a = 385 (C_w)^{0.5} h = \boxed{3.45 > 1 \text{ Rigid Structure}}$

<u>E-W</u>

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

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

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY} : Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY} : Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Case 1 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	1.10	34.53	23.5	-14.7	38.2	52	0	0
11	131	11.5	1.07	33.43	22.7	-14.7	37.4	39	52	7520
10	119.5	11.5	1.04	32.52	22.1	-14.7	36.8	38	90	5072
9	108	11.5	1.01	31.64	21.5	-14.7	36.2	37	128	4550
8	96.5	11.5	0.98	30.69	20.9	-14.7	35.5	37	166	4045
7	85	11.5	0.95	29.61	20.1	-14.7	34.8	36	203	3550
6	73.5	11.5	0.90	28.32	19.3	-14.7	33.9	35	239	3062
5	59.5	14	0.85	26.57	18.1	-14.7	32.7	41	274	2581
4	48	11.5	0.80	25.06	17.0	-14.7	31.7	33	315	2454
3	36.5	11.5	0.74	23.15	15.7	-14.7	30.4	31	348	1576
2	25	11.5	0.66	20.68	14.1	-14.7	28.7	30	379	1149
1	13.5	13.5	0.57	17.86	12.1	-14.7	26.8	33	409	744
Ground	0	0	0.00	0.00	0.0	0.0	0.0	0	442	440
									Sum	36742

Case 1 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	1.10	34.53	23.5	-7.8	31.2	90	0	0
11	131	11.5	1.07	33.43	22.7	-7.8	30.5	67	90	13070
10	119.5	11.5	1.04	32.52	22.1	-7.8	29.9	66	157	8776
9	108	11.5	1.01	31.64	21.5	-7.8	29.3	64	222	7844
8	96.5	11.5	0.98	30.69	20.9	-7.8	28.6	63	286	6947
7	85	11.5	0.95	29.61	20.1	-7.8	27.9	61	349	6070
6	73.5	11.5	0.90	28.32	19.3	-7.8	27.0	59	411	5209
5	59.5	14	0.85	26.57	18.1	-7.8	25.8	69	470	4363
4	48	11.5	0.80	25.06	17.0	-7.8	24.8	54	539	4110
3	36.5	11.5	0.74	23.15	15.7	-7.8	23.5	52	594	2616
2	25	11.5	0.66	20.68	14.1	-7.8	21.8	48	645	1885
1	13.5	13.5	0.57	17.86	12.1	-7.8	19.9	51	693	1199
Ground	0	0	0.00	0.00	0.00	0.00	0.00	0	745	693
									Sum	62782

Case 2 ($M_t = 0.75((P_{wx}+P_{lx})B_x(e_x))$ North - South Direction					
Floor	0.75	$(P_{wx}+P_{lx})B_x$	$0.75(P_{wx}+P_{lx})B_x$	e_x	M_t
Roof	0.75	52	39	13.5	521
11	0.75	39	29	13.5	392
10	0.75	38	29	13.5	386
9	0.75	37	28	13.5	379
8	0.75	37	28	13.5	372
7	0.75	36	27	13.5	365
6	0.75	35	26	13.5	356
5	0.75	41	31	13.5	418
4	0.75	33	25	13.5	332
3	0.75	31	24	13.5	319
2	0.75	30	22	13.5	301
1	0.75	33	24	13.5	330

Case 2 ($M_t = 0.75((P_{wy}+P_{ly})B_y(e_y))$ East - West Direction					
Floor	0.75	$(P_{wy}+P_{ly})B_y$	$0.75(P_{wy}+P_{ly})B_y$	e_y	M_t
Roof	0.75	90	67	28.7	1924
11	0.75	67	50	28.7	1440
10	0.75	66	49	28.7	1410
9	0.75	64	48	28.7	1382
8	0.75	63	47	28.7	1352
7	0.75	61	46	28.7	1317
6	0.75	59	45	28.7	1276
5	0.75	69	52	28.7	1484
4	0.75	54	41	28.7	1171
3	0.75	52	39	28.7	1110
2	0.75	48	36	28.7	1030
1	0.75	51	39	28.7	1103

Case 3 $(0.75(P_{wx}+P_{lx}))+(0.75(P_{wy}+P_{ly}))$					
Floor	0.75	$(P_{wx}+P_{lx})$	$0.75*(P_{wx}+P_{lx})$	$(P_{wy}+P_{ly})$	$0.75(P_{wy}+P_{ly})$
Roof	0.75	52	39	90	67
11	0.75	39	29	67	50
10	0.75	38	29	66	49
9	0.75	37	28	64	48
8	0.75	37	28	63	47
7	0.75	36	27	61	46
6	0.75	35	26	59	45
5	0.75	41	31	69	52
4	0.75	33	25	54	41
3	0.75	31	24	52	39
2	0.75	30	22	48	36
1	0.75	33	24	51	39

Case 4 $M_t = 0.563(P_{wx}+P_{lx})B_x(e_x) + 0.563(P_{wy}+P_{ly})B_y(e_y)$								
Floor	0.563	$(P_{wx}+P_{lx})B_x$	$0.563(P_{wx}+P_{lx})B_x$	e_x	$(P_{wy}+P_{ly})B_y$	$0.563(P_{wy}+P_{ly})B_y$	e_y	M_t
Roof	0.563	52	29	13.5	90	50	28.7	1033
11	0.563	39	22	13.5	67	38	28.7	774
10	0.563	38	21	13.5	66	37	28.7	759
9	0.563	37	21	13.5	64	36	28.7	744
8	0.563	37	21	13.5	63	35	28.7	729
7	0.563	36	20	13.5	61	35	28.7	711
6	0.563	35	20	13.5	59	33	28.7	689
5	0.563	41	23	13.5	69	39	28.7	804
4	0.563	33	18	13.5	54	31	28.7	635
3	0.563	31	18	13.5	52	29	28.7	604
2	0.563	30	17	13.5	48	27	28.7	563
1	0.563	33	18	13.5	51	29	28.7	606

Seismic Design | Patient Tower

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)

Steel ordinary Concentrically braced frames
$R = 3.25$ (12.2-1)
$\Omega = 2$
$C_d = 3.25$
Section 14.1

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

$S_{M1} = F_v S_1 = 2.4(0.055) = \boxed{0.132}$

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

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

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

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

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

Seismic Design | Patient Tower

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 Catagory B (12.6-1.2)

$$C_{s\,min} \begin{cases} \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.088}{0.84\left(\frac{3.25}{1.25}\right)} = \boxed{0.040} \quad \text{Controles} \\[2ex] \dfrac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} = \dfrac{0.088}{(0.84)^2\left(\frac{3.25}{1.25}\right)} = 0.048 \\[2ex] \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.144}{\left(\frac{3.25}{1.25}\right)} = 0.55 \end{cases}$$

$$C_s = 0.040$$

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

Building Dead load Weight

$$W_t = 29805 \text{ K}$$

Equivalent lateral force procedure (12.8)

$$V = C_s W = 0.040\,(29805) = 1192 \text{ K}$$

base shear

Seismic Design | Patient Tower

Vertical distribution (12.8.3)

$F_x = C_{vx} V$

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

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

See excel sheet

$\sum w_i h_i^k = 724074$

Seismic Story Forces + Shear

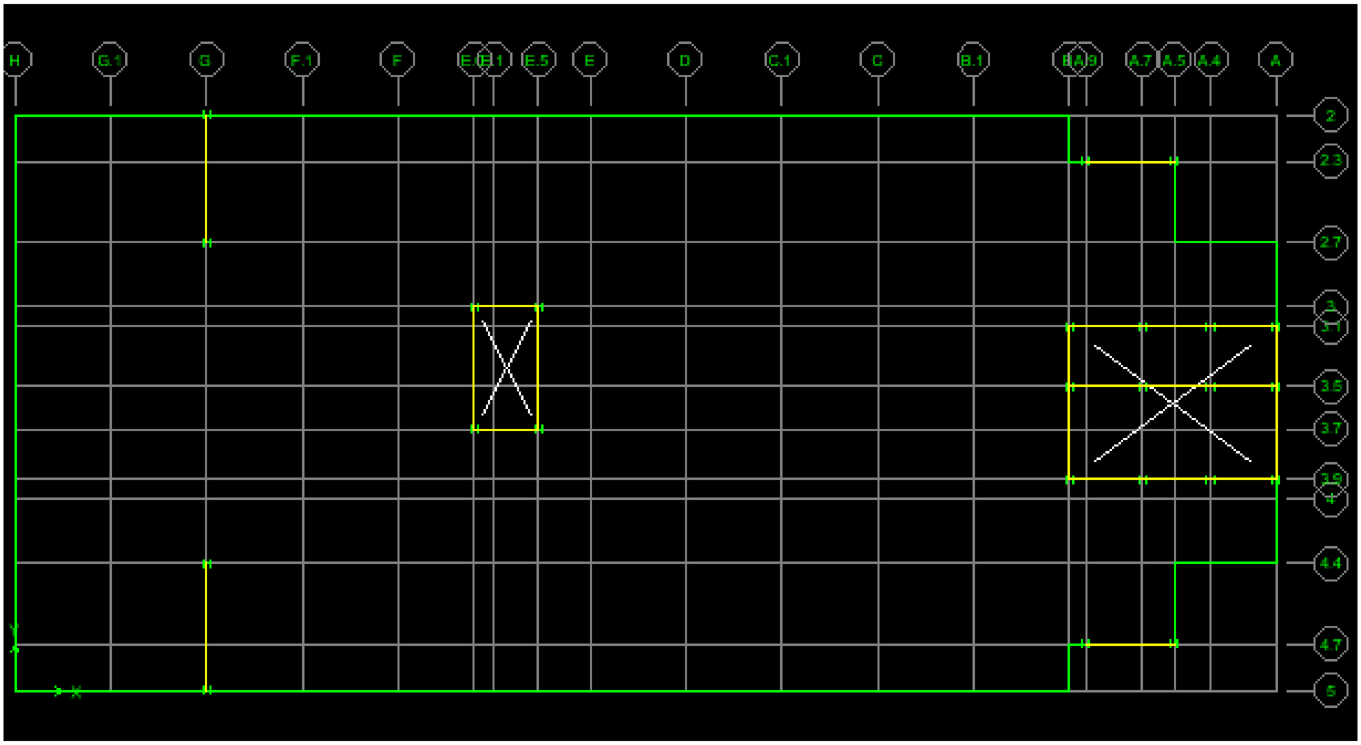
Story Forces	Level	Story Shear
107^k	Roof (146')	
94^k	Floor 11 (131')	107^k
84^k	Floor 10 (119.5')	201^k
75^k	Floor 9 (108')	285^k
66^k	Floor 8 (96.5')	360^k
57^k	Floor 7 (85')	426^k
48^k	Floor 6 (73.5')	483^k
38^k	Floor 5 (59.5')	531^k
30^k	Floor 4 (48')	568^k
21^k	Floor 3 (36.5')	598^k
21^k	Floor 2 (25')	619^k
10^k	Floor 1 (13.5')	640^k
	Ground (0')	650^k

650^k

65,900 k-ft

Appendix III

This section of the Final Report is where the supplementary information for the Lateral bracing Redesign Calculations for the Hospital Patient Tower can be found.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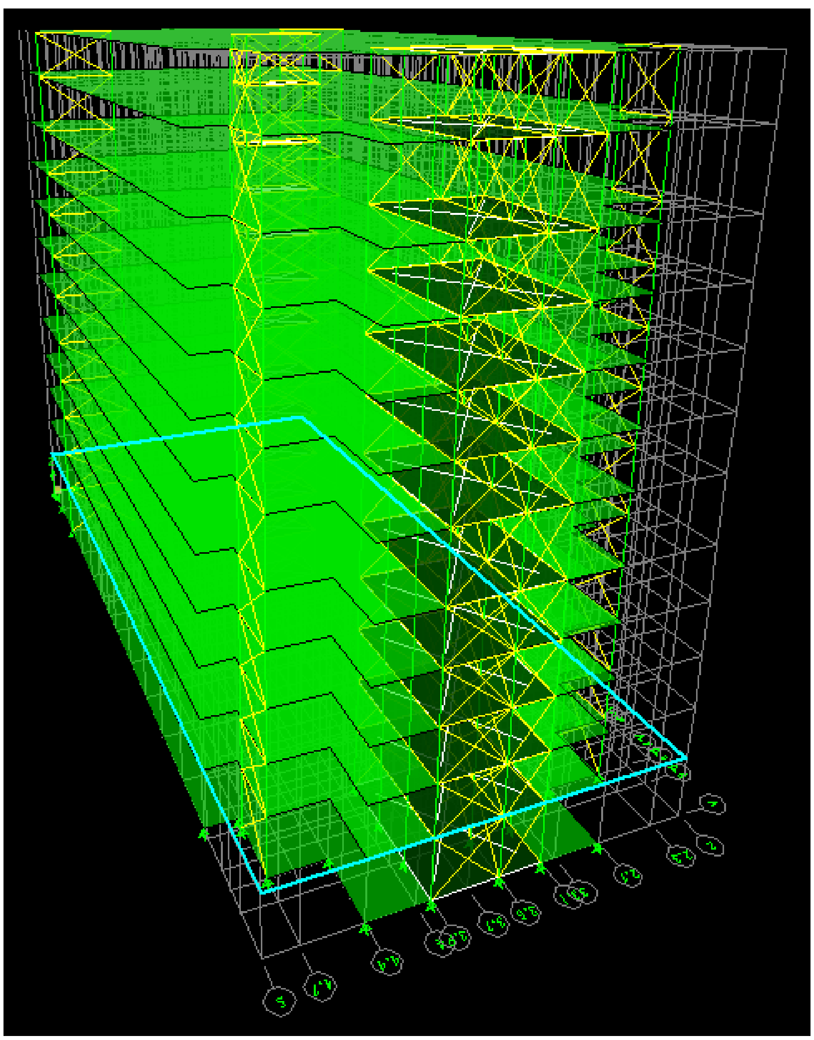

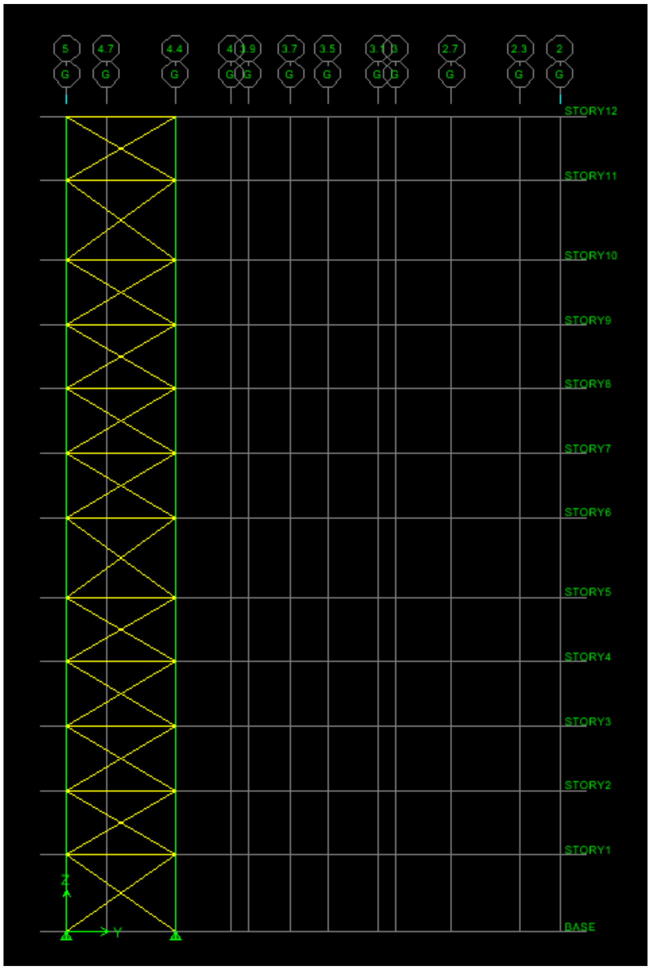
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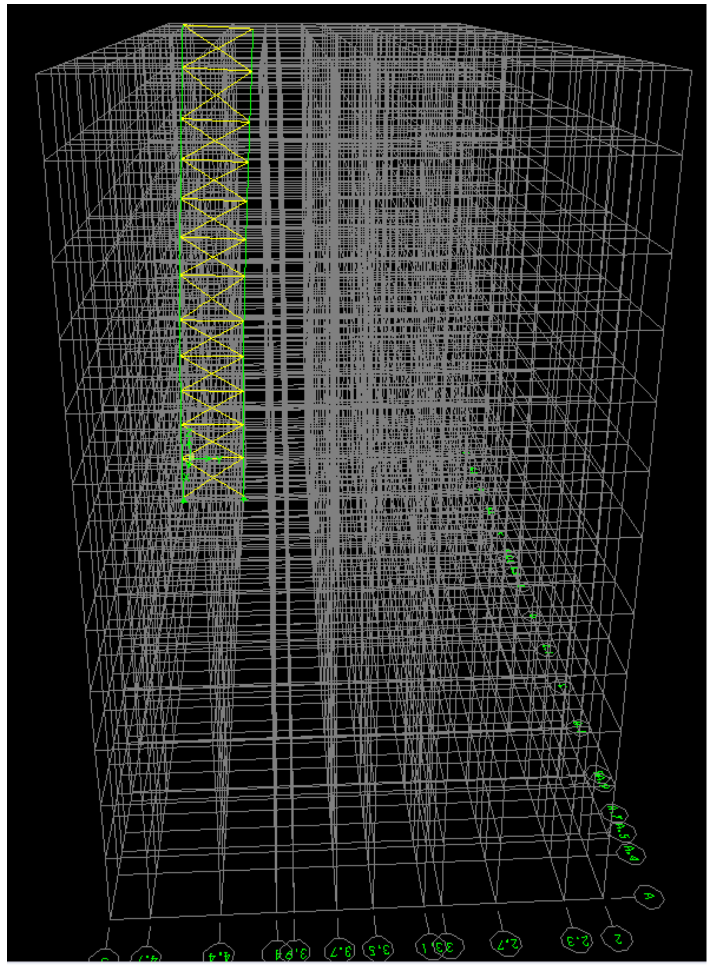

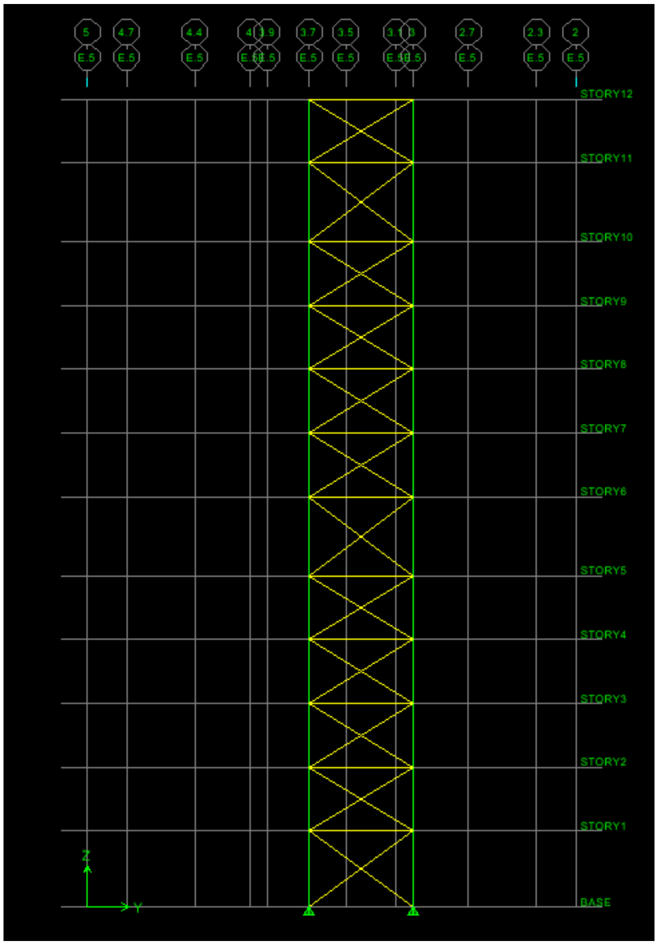
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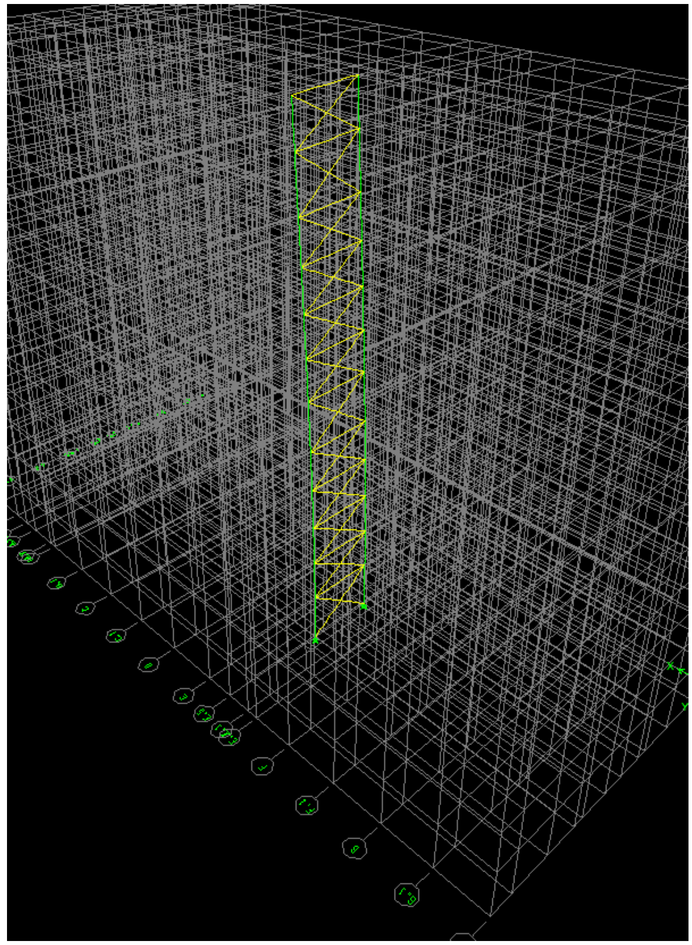

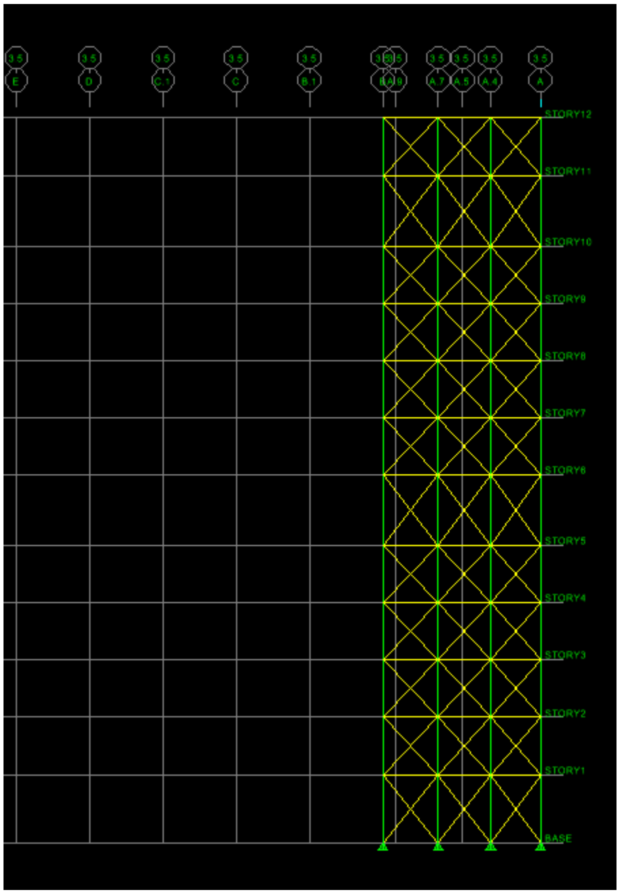
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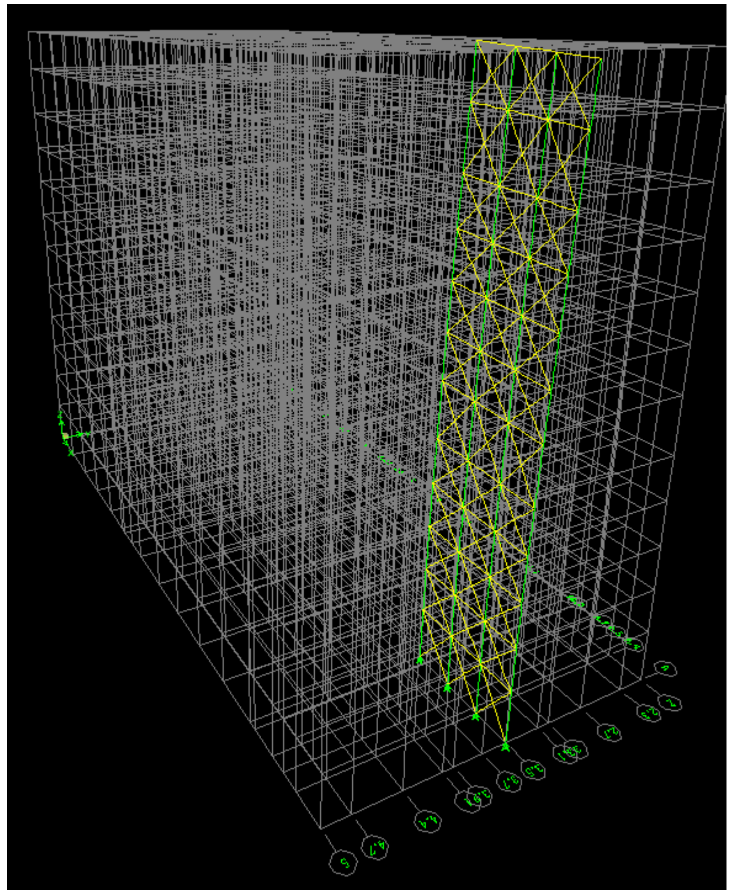
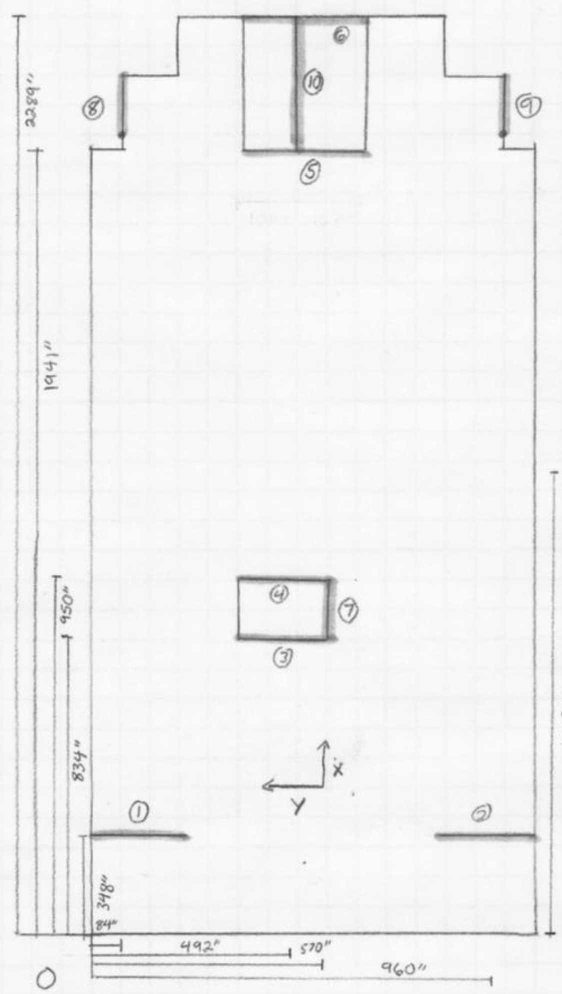




↑N

Center of Rigity

Y- Direction

$\Sigma R_{iy} = 1.0$

$$\frac{\Sigma R_{iy} \cdot d}{\Sigma R_{iy}}$$

$$= \frac{(R_{F_1}(348) \cdot 2) + R_{F_3}(834) + \ldots}{1.0}$$

$\approx 1280''$

X - Direction

$\Sigma R_{ix} = 1.0$

$$\frac{\Sigma R_{ix} \cdot d}{\Sigma R_{ix}}$$

$$= \frac{(R_{F7}(570'') + R_{F8}(84) \ldots)}{1.0}$$

$\approx 502''$

Hand

(502, 1280)

Etabs

(542, 1285)

Center of Mass

Y- Direction

$2289/2 = 1144.5''$

X-Direction

$1044/2 = 522''$

Hand

(522, 1145)

Etabs

(522, 1104)

See table for detailed Calculations

North – South Direction

Unit Load displacement							
Level	North-South Frames (Y-Direction) Displacement, Δ_p (in.)						Arbitrary Unit Load, P (kips)
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	
Roof	37.7	37.7	40.3	40.3	24.3	24.3	100
11	33.7	33.7	35.9	35.9	21.6	21.6	100
10	28.7	28.7	30.6	30.6	18.4	18.4	100
9	24.7	24.7	26.3	26.3	15.8	15.8	100
8	20.8	20.8	22.1	22.1	13.3	13.3	100
7	17.0	17.0	18.1	18.1	10.9	10.9	100
6	13.5	13.5	14.4	14.4	8.6	8.6	100
5	9.5	9.5	10.1	10.1	6.1	6.1	100
4	6.8	6.8	7.1	7.1	4.3	4.3	100
3	4.4	4.4	4.6	4.6	2.8	2.8	100
2	2.4	2.4	2.5	2.5	1.6	1.6	100
1	1.0	1.0	1.0	1.0	1.0	1.0	100

Story Stiffness, K_{iy}							
Level	Arbitrary Unit Load, P (kips)	Story Stiffness, K_i $K_{iy}= P/\Delta_p$					
		Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6
Roof	100	2.65	2.65	2.48	2.48	4.12	4.12
11	100	2.97	2.97	2.79	2.79	4.63	4.63
10	100	3.48	3.48	3.27	3.27	5.43	5.43
9	100	4.05	4.05	3.80	3.80	6.33	6.33
8	100	4.81	4.81	4.52	4.52	7.52	7.52
7	100	5.88	5.88	5.52	5.52	9.17	9.17
6	100	7.41	7.41	6.94	6.94	11.63	11.63
5	100	10.53	10.53	9.90	9.90	16.39	16.39
4	100	14.71	14.71	14.08	14.08	23.26	23.26
3	100	22.73	22.73	21.74	21.74	35.71	35.71
2	100	41.67	41.67	40.00	40.00	62.50	62.50
1	100	100.00	100.00	100.00	100.00	100.00	100.00

Relative Story Stiffness, R_{iy}								
Level	Total Story Stiffness $K_{iy,total}$	Relative Story Stiffness, R_i $R_{iy} = K_{iy}/K_{iy,total}$						
		Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	∑R
Roof	18.50	0.143	0.143	0.134	0.134	0.222	0.222	1.0
11	20.77	0.143	0.143	0.134	0.134	0.223	0.223	1.0
10	24.37	0.143	0.143	0.134	0.134	0.223	0.223	1.0
9	28.36	0.143	0.143	0.134	0.134	0.223	0.223	1.0
8	33.70	0.143	0.143	0.134	0.134	0.223	0.223	1.0
7	41.16	0.143	0.143	0.134	0.134	0.223	0.223	1.0
6	51.96	0.143	0.143	0.134	0.134	0.224	0.224	1.0
5	73.64	0.143	0.143	0.134	0.134	0.223	0.223	1.0
4	104.09	0.141	0.141	0.135	0.135	0.223	0.223	1.0
3	160.36	0.142	0.142	0.136	0.136	0.223	0.223	1.0
2	288.33	0.145	0.145	0.139	0.139	0.217	0.217	1.0
1	600.00	0.167	0.167	0.167	0.167	0.167	0.167	1.0

Distance from Origin to Frame Y-Direction						
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6
Distance	348	348	834	950	1941	2289

Center of Rigidity Y-direction							
Level	Relative Story Stiffness * Distance from origin, ∑R *d						∑R*d/∑R
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Total
Roof	49.9	49.9	111.9	127.4	431.8	509.2	1280
11	49.7	49.7	111.9	127.4	432.8	510.3	1282
10	49.7	49.7	111.8	127.4	432.8	510.4	1282
9	49.7	49.7	111.8	127.4	433.2	510.8	1283
8	49.6	49.6	112.0	127.5	433.0	510.7	1282
7	49.7	49.7	111.9	127.5	432.6	510.2	1282
6	49.6	49.6	111.5	127.0	434.4	512.3	1284
5	49.7	49.7	112.1	127.7	432.1	509.6	1281
4	49.2	49.2	112.8	128.5	433.6	511.4	1285
3	49.3	49.3	113.1	128.8	432.3	509.8	1283
2	50.3	50.3	115.7	131.8	420.7	496.2	1265
1	58.0	58.0	139.0	158.3	323.5	381.5	1118

East – West Direction

Unit Load displacement					
Level	East - West Frames (X-Direction) Displacement, Δ_p (in.)				Arbitrary Unit Load, P (kips)
	Frame 7	Frame 8	Frame 9	Frame 10	
Roof	139.4	74.9	74.9	11.9	100
11	123.8	66.6	66.6	10.7	100
10	104.6	56.4	56.4	9.1	100
9	89.3	48.2	48.2	7.9	100
8	74.6	40.4	40.4	6.7	100
7	60.6	32.9	32.9	5.5	100
6	47.4	25.8	25.8	4.4	100
5	32.7	17.9	17.9	3.1	100
4	22.6	12.5	12.5	2.3	100
3	14.1	7.9	7.9	1.5	100
2	7.3	4.1	4.1	1.0	100
1	2.6	1.5	1.5	0.5	100

Story Stiffness, K_{ix}					
Level	Arbitrary Unit Load, P (kips)	Story Stiffness, K_i $K_{ix} = P/\Delta_p$			
		Frame 7	Frame 8	Frame 9	Frame 10
Roof	100	0.72	1.34	1.34	8.40
11	100	0.81	1.50	1.50	9.35
10	100	0.96	1.77	1.77	10.99
9	100	1.12	2.07	2.07	12.66
8	100	1.34	2.48	2.48	14.93
7	100	1.65	3.04	3.04	18.18
6	100	2.11	3.88	3.88	22.73
5	100	3.06	5.59	5.59	32.26
4	100	4.42	8.00	8.00	43.48
3	100	7.09	12.66	12.66	66.67
2	100	13.70	24.39	24.39	100.00
1	100	38.46	66.67	66.67	200.00

Relative Story Stiffness, R_{ix}						
Level	Total Story Stiffness $K_{iy,total}$	Relative Story Stiffness, R_i $R_{ix} = K_{ix}/K_{ix,total}$				
		Frame 7	Frame 8	Frame 9	Frame 10	∑R
Roof	11.79	0.0608	0.1132	0.1132	0.7127	1.0
11	13.16	0.0614	0.1141	0.1141	0.7104	1.0
10	15.49	0.0617	0.1145	0.1145	0.7094	1.0
9	17.93	0.0625	0.1157	0.1157	0.7061	1.0
8	21.22	0.0632	0.1167	0.1167	0.7035	1.0
7	25.91	0.0637	0.1173	0.1173	0.7017	1.0
6	32.59	0.0647	0.1189	0.1189	0.6974	1.0
5	46.49	0.0658	0.1202	0.1202	0.6939	1.0
4	63.90	0.0692	0.1252	0.1252	0.6804	1.0
3	99.08	0.0716	0.1278	0.1278	0.6729	1.0
2	162.48	0.0843	0.1501	0.1501	0.6155	1.0
1	371.79	0.1034	0.1793	0.1793	0.5379	1.0

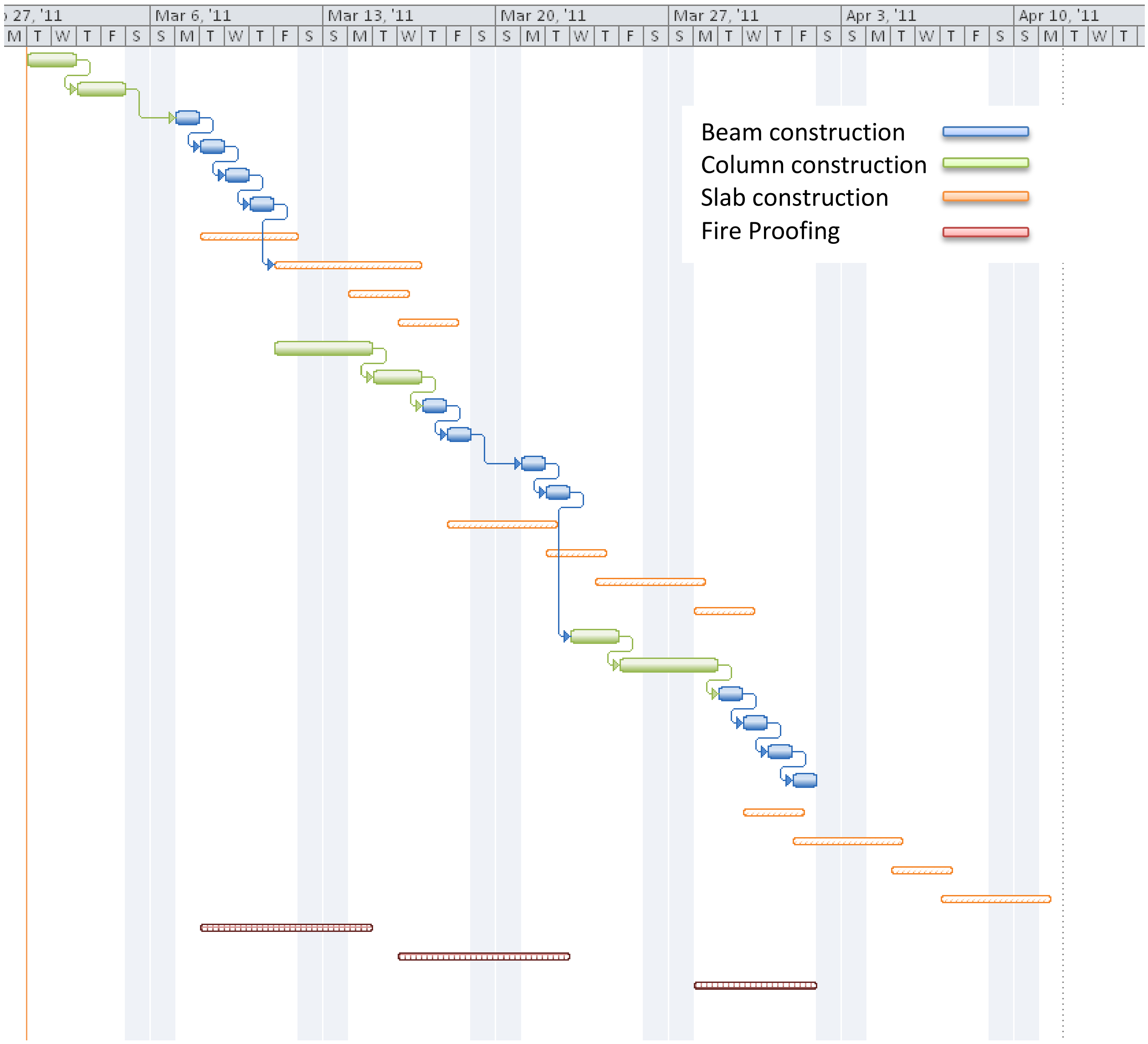
Distance from Origin to Frame X-Direction				
	Frame 7	Frame 8	Frame 9	Frame 10
Distance	570	84	940	492

Center of Rigidity x-direction					
Level	Relative Story Stiffness * Distance from origin, Ri ∑R *d				∑R*d/∑R
	Frame 7	Frame 8	Frame 9	Frame 10	Total
Roof	34.7	9.5	106.4	350.6	501
11	35.0	9.6	107.3	349.5	501
10	35.2	9.6	107.6	349.0	501
9	35.6	9.7	108.8	347.4	502
8	36.0	9.8	109.7	346.1	502
7	36.3	9.9	110.3	345.2	502
6	36.9	10.0	111.8	343.1	502
5	37.5	10.1	113.0	341.4	502
4	39.5	10.5	117.7	334.7	502
3	40.8	10.7	120.1	331.1	503
2	48.1	12.6	141.1	302.8	505
1	59.0	15.1	168.6	264.7	507

Appendix IV

This section of the Final Report is where the supplementary information for the Lateral bracing Redesign Calculations for the Hospital Patient Tower can be found.

Task Mode	Task Name	Duration	Start	Finish
	Column lines H-E L1-4	2 days	Tue 3/1/11	Wed 3/2/11
	Column lines D-A L1-4	2 days	Thu 3/3/11	Fri 3/4/11
	Beam Level 1	1 day	Mon 3/7/11	Mon 3/7/11
	Beam Level 2	1 day	Tue 3/8/11	Tue 3/8/11
	Beam Level 3	1 day	Wed 3/9/11	Wed 3/9/11
	Beam Level 4	1 day	Thu 3/10/11	Thu 3/10/11
	Plank Level 1	4 days	Tue 3/8/11	Fri 3/11/11
	Plank Level 2	4 days	Fri 3/11/11	Wed 3/16/11
	Plank Level 3	2.5 days	Mon 3/14/11	Wed 3/16/11
	Plank Level 4	2.5 days	Wed 3/16/11	Fri 3/18/11
	Column lines H-E L5-8	2 days	Fri 3/11/11	Mon 3/14/11
	Column lines D-A L5-8	2 days	Tue 3/15/11	Wed 3/16/11
	Beam Level 5	1 day	Thu 3/17/11	Thu 3/17/11
	Beam Level 6	1 day	Fri 3/18/11	Fri 3/18/11
	Beam Level 7	1 day	Mon 3/21/11	Mon 3/21/11
	Beam Level 8	1 day	Tue 3/22/11	Tue 3/22/11
	Plank Level 5	2.5 days	Fri 3/18/11	Tue 3/22/11
	Plank Level 6	2.5 days	Tue 3/22/11	Thu 3/24/11
	Plank Level 7	2.5 days	Thu 3/24/11	Mon 3/28/11
	Plank Level 8	2.5 days	Mon 3/28/11	Wed 3/30/11
	Column lines H-E L9-12	2 days	Wed 3/23/11	Thu 3/24/11
	Column lines D-A L9-12	2 days	Fri 3/25/11	Mon 3/28/11
	Beam Level 9	1 day	Tue 3/29/11	Tue 3/29/11
	Beam Level 10	1 day	Wed 3/30/11	Wed 3/30/11
	Beam Level 11	1 day	Thu 3/31/11	Thu 3/31/11
	Beam Level 12	1 day	Fri 4/1/11	Fri 4/1/11
	Plank Level 9	2.5 days	Wed 3/30/11	Fri 4/1/11
	Plank Level 10	2.5 days	Fri 4/1/11	Tue 4/5/11
	Plank Level 11	2.5 days	Tue 4/5/11	Thu 4/7/11
	Plank Level 12	2.5 days	Thu 4/7/11	Mon 4/11/11
	Fire Proofing 1	5 days	Tue 3/8/11	Mon 3/14/11
	Fire Proofing 2	5 days	Wed 3/16/11	Tue 3/22/11
	Fire Proofing 3	5 days	Mon 3/28/11	Fri 4/1/11

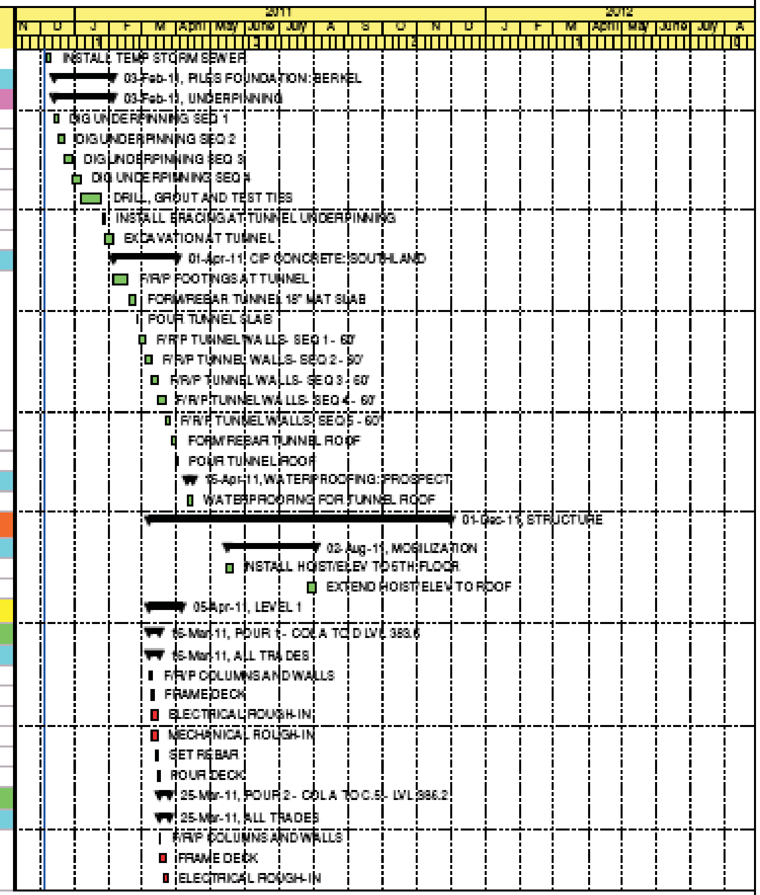

27, '11
Mar 6, '11
Mar 13, '11
Mar 20, '11
Mar 27, '11
Apr 3, '11
Apr 10, '11
M T W T F S S M T W T F S S M T W T F S S M T W T F S S M T W T F S S M T W T F S S M T W T
Beam construction
Column construction
Slab construction
Fire Proofing

TURNER CONSTRUCTION COMPANY

INOVA SOUTH PATIENT TOWER

Run Date: 09-Dec-10 01:3

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
SPT-P01120	INSTALL TEMP STORM SEWER	5	5	06-Dec-10	10-Dec-10
PILES FOUNDATION: BERKEL		37	37	13-Dec-10	03-Feb-11
UNDERPINNING		37	37	13-Dec-10	03-Feb-11
SPT-P01450	DIG UNDERPINNING SEQ 1	4	4	13-Dec-10	16-Dec-10
SPT-P01460	DIG UNDERPINNING SEQ 2	4	4	17-Dec-10	22-Dec-10
SPT-P01470	DIG UNDERPINNING SEQ 3	4	4	23-Dec-10	29-Dec-10
SPT-P01480	DIG UNDERPINNING SEQ 4	4	4	30-Dec-10	05-Jan-11
SPT-P01500	DRILL, GROUT AND TEST TIES	13	13	06-Jan-11	24-Jan-11
SPT-P01510	INSTALL BRACING AT TUNNEL UNDERPINNING	3	3	25-Jan-11	27-Jan-11
SPT-P01520	EXCAVATION AT TUNNEL	5	5	28-Jan-11	03-Feb-11
CIP CONCRETE: SOUTHLAND		41	41	04-Feb-11	01-Apr-11
SPT-P01530	F/R/P FOOTINGS AT TUNNEL	10	10	04-Feb-11	17-Feb-11
SPT-P01540	FORM/REBAR TUNNEL 18" MAT SLAB	5	5	18-Feb-11	24-Feb-11
SPT-P01550	POUR TUNNEL SLAB	1	1	25-Feb-11	25-Feb-11
SPT-P01560	F/R/P TUNNEL WALLS- SEQ 1 - 60'	4	4	28-Feb-11	03-Mar-11
SPT-P01570	F/R/P TUNNEL WALLS- SEQ 2 - 60'	4	4	04-Mar-11	09-Mar-11
SPT-P01580	F/R/P TUNNEL WALLS- SEQ 3 - 60'	4	4	10-Mar-11	15-Mar-11
SPT-P01590	F/R/P TUNNEL WALLS- SEQ 4 - 60'	4	4	16-Mar-11	21-Mar-11
SPT-P01600	F/R/P TUNNEL WALLS- SEQ 5 - 60'	4	4	22-Mar-11	25-Mar-11
SPT-P01610	FORM/REBAR TUNNEL ROOF	4	4	28-Mar-11	31-Mar-11
SPT-P01620	POUR TUNNEL ROOF	1	1	01-Apr-11	01-Apr-11
WATERPROOFING: PROSPECT		5	5	11-Apr-11	15-Apr-11
SPT-P01820	WATERPROOFING FOR TUNNEL ROOF	5	5	11-Apr-11	15-Apr-11
STRUCTURE		192	192	08-Mar-11	01-Dec-11
MOBILIZATION		56	56	16-May-11	02-Aug-11
SPT-3280	INSTALL HOIST/ELEV TO 5TH FLOOR	5	5	16-May-11	20-May-11
SPT-3300	EXTEND HOIST/ELEV TO ROOF	5	5	27-Jul-11	02-Aug-11
LEVEL 1		21	21	08-Mar-11	05-Apr-11
POUR 1 - COL A TO D LVL 383.6		7	7	08-Mar-11	16-Mar-11
ALL TRADES		7	7	08-Mar-11	16-Mar-11
SPT-P02440	F/R/P COLUMNS AND WALLS	3	3	08-Mar-11	10-Mar-11
SPT-P02450	FRAME DECK	2	2	10-Mar-11	11-Mar-11
SPT-P02460	ELECTRICAL ROUGH-IN	2	2	11-Mar-11	14-Mar-11
SPT-P02470	MECHANICAL ROUGH-IN	2	2	11-Mar-11	14-Mar-11
SPT-P02480	SET REBAR	2	2	14-Mar-11	15-Mar-11
SPT-P02490	POUR DECK	1	1	16-Mar-11	16-Mar-11
POUR 2 - COL A TO C.5 - LVL 396.2		7	7	17-Mar-11	25-Mar-11
ALL TRADES		7	7	17-Mar-11	25-Mar-11
SPT-P02380	F/R/P COLUMNS AND WALLS	1	1	17-Mar-11	17-Mar-11
SPT-P02390	FRAME DECK	3	3	18-Mar-11	22-Mar-11
SPT-P02400	ELECTRICAL ROUGH-IN	3	3	21-Mar-11	23-Mar-11



Remaining Level of Effort · Actual Level of Effort · Actual Work · Remaining Work · Critical Remaining Work · Milestone · Summary

Master Schedule Update

Through 05-Dec-10

Data Date: 05-Dec-10

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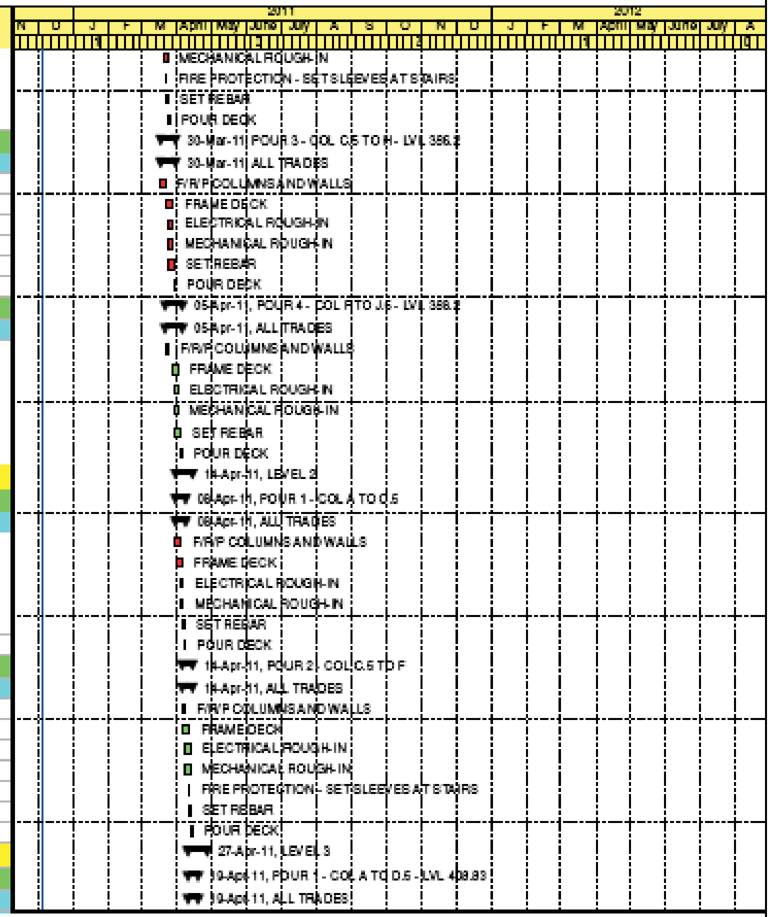
Project ID: SPT-M-1

TURNER CONSTRUCTION COMPANY

INOVA SOUTH PATIENT TOWER

Run Date: 09-Dec-10 01:3

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
SPT-P02410	MECHANICAL ROUGH-IN	3	3	21-Mar-11	23-Mar-11
SPT-P02520	FIRE PROTECTION - SET SLEEVES AT STAIRS	1	1	22-Mar-11	22-Mar-11
SPT-P02420	SET REBAR	2	2	23-Mar-11	24-Mar-11
SPT-P02430	POUR DECK	1	1	25-Mar-11	25-Mar-11
POUR 3 - COL C.5 TO H - LVL 386.2		**9**	**9**	**18-Mar-11**	**30-Mar-11**
ALL TRADES		**9**	**9**	**18-Mar-11**	**30-Mar-11**
SPT-P02320	F/R/P COLUMNS AND WALLS	2	2	18-Mar-11	21-Mar-11
SPT-P02330	FRAME DECK	4	4	23-Mar-11	28-Mar-11
SPT-P02340	ELECTRICAL ROUGH-IN	3	3	24-Mar-11	28-Mar-11
SPT-P02350	MECHANICAL ROUGH-IN	3	3	24-Mar-11	28-Mar-11
SPT-P02360	SET REBAR	3	3	25-Mar-11	29-Mar-11
SPT-P02370	POUR DECK	1	1	30-Mar-11	30-Mar-11
POUR 4 - COL F TO J.5 - LVL 388.2		**11**	**11**	**22-Mar-11**	**05-Apr-11**
ALL TRADES		**11**	**11**	**22-Mar-11**	**05-Apr-11**
SPT-P02260	F/R/P COLUMNS AND WALLS	3	3	22-Mar-11	24-Mar-11
SPT-P02270	FRAME DECK	4	4	29-Mar-11	01-Apr-11
SPT-P02280	ELECTRICAL ROUGH-IN	3	3	30-Mar-11	01-Apr-11
SPT-P02290	MECHANICAL ROUGH-IN	3	3	30-Mar-11	01-Apr-11
SPT-P02300	SET REBAR	3	3	31-Mar-11	04-Apr-11
SPT-P02310	POUR DECK	1	1	05-Apr-11	05-Apr-11
LEVEL 2		**11**	**11**	**31-Mar-11**	**14-Apr-11**
POUR 1 - COL A TO C.5		**7**	**7**	**31-Mar-11**	**08-Apr-11**
ALL TRADES		**7**	**7**	**31-Mar-11**	**08-Apr-11**
SPT-P02200	F/R/P COLUMNS AND WALLS	3	3	31-Mar-11	04-Apr-11
SPT-P02210	FRAME DECK	3	3	01-Apr-11	05-Apr-11
SPT-P02220	ELECTRICAL ROUGH-IN	3	3	04-Apr-11	06-Apr-11
SPT-P02230	MECHANICAL ROUGH-IN	3	3	04-Apr-11	06-Apr-11
SPT-P02240	SET REBAR	2	2	06-Apr-11	07-Apr-11
SPT-P02250	POUR DECK	1	1	08-Apr-11	08-Apr-11
POUR 2 - COL C.5 TO F		**7**	**7**	**06-Apr-11**	**14-Apr-11**
ALL TRADES		**7**	**7**	**06-Apr-11**	**14-Apr-11**
SPT-P02140	F/R/P COLUMNS AND WALLS	3	3	06-Apr-11	08-Apr-11
SPT-P02150	FRAME DECK	3	3	07-Apr-11	11-Apr-11
SPT-P02160	ELECTRICAL ROUGH-IN	3	3	08-Apr-11	12-Apr-11
SPT-P02170	MECHANICAL ROUGH-IN	3	3	08-Apr-11	12-Apr-11
SPT-P02510	FIRE PROTECTION - SET SLEEVES AT STAIRS	1	1	11-Apr-11	11-Apr-11
SPT-P02180	SET REBAR	2	2	12-Apr-11	13-Apr-11
SPT-P02190	POUR DECK	1	1	14-Apr-11	14-Apr-11
LEVEL 3		**13**	**13**	**11-Apr-11**	**27-Apr-11**
POUR 1 - COL A TO D.5 - LVL 408.83		**7**	**7**	**11-Apr-11**	**19-Apr-11**
ALL TRADES		**7**	**7**	**11-Apr-11**	**19-Apr-11**



Remaining Level of Effort · Actual Level of Effort · Actual Work · Remaining Work · Critical Remaining Work · Milestone · Summary

Master Schedule Update

Through 05-Dec-10

Data Date: 05-Dec-10

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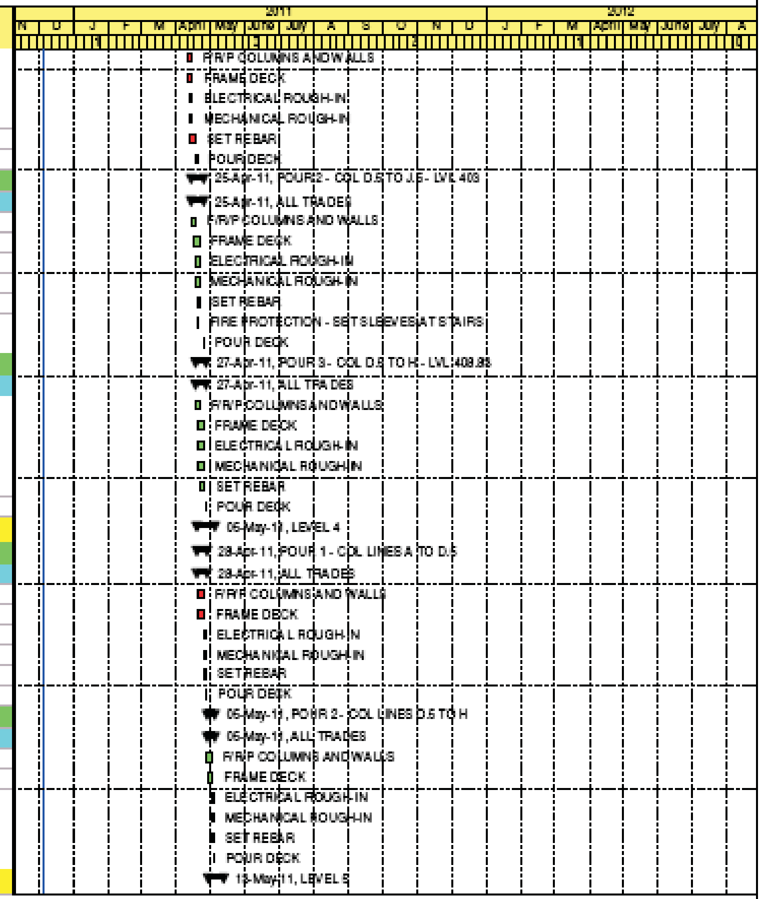
Project ID: SPT-M-1

TURNER CONSTRUCTION COMPANY

INOVA SOUTH PATIENT TOWER

Run Date: 09-Dec-10 01:3

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
SPT-F02080	F/R/P COLUMNS AND WALLS	4	4	11-Apr-11	14-Apr-11
SPT-F02090	FRAME DECK	4	4	12-Apr-11	15-Apr-11
SPT-F02100	ELECTRICAL ROUGH-IN	3	3	13-Apr-11	15-Apr-11
SPT-F02110	MECHANICAL ROUGH-IN	3	3	13-Apr-11	15-Apr-11
SPT-F02120	SET REBAR	3	3	14-Apr-11	18-Apr-11
SPT-F02130	POUR DECK	1	1	19-Apr-11	19-Apr-11
POUR 2 - COL D.5 TO J.5 - LVL 403		7	7	15-Apr-11	25-Apr-11
ALL TRADES		7	7	15-Apr-11	25-Apr-11
SPT-F02020	F/R/P COLUMNS AND WALLS	2	2	15-Apr-11	18-Apr-11
SPT-F02030	FRAME DECK	4	4	18-Apr-11	21-Apr-11
SPT-F02040	ELECTRICAL ROUGH-IN	3	3	19-Apr-11	21-Apr-11
SPT-F02050	MECHANICAL ROUGH-IN	3	3	19-Apr-11	21-Apr-11
SPT-F02060	SET REBAR	3	3	20-Apr-11	22-Apr-11
SPT-F02500	FIRE PROTECTION - SET SLEEVES AT STAIRS	1	1	20-Apr-11	20-Apr-11
SPT-F02070	POUR DECK	1	1	25-Apr-11	25-Apr-11
POUR 3 - COL D.5 TO H - LVL 408.83		7	7	19-Apr-11	27-Apr-11
ALL TRADES		7	7	19-Apr-11	27-Apr-11
SPT-F01990	F/R/P COLUMNS AND WALLS	3	3	19-Apr-11	21-Apr-11
SPT-F01980	FRAME DECK	4	4	20-Apr-11	25-Apr-11
SPT-F02000	ELECTRICAL ROUGH-IN	3	3	21-Apr-11	25-Apr-11
SPT-F02010	MECHANICAL ROUGH-IN	3	3	21-Apr-11	25-Apr-11
SPT-F01970	SET REBAR	3	3	22-Apr-11	26-Apr-11
SPT-F01960	POUR DECK	1	1	27-Apr-11	27-Apr-11
LEVEL 4		12	12	20-Apr-11	05-May-11
POUR 1 - COL LINES A TO D.5		7	7	20-Apr-11	28-Apr-11
ALL TRADES		7	7	20-Apr-11	28-Apr-11
SPT-F03520	F/R/P COLUMNS AND WALLS	4	4	20-Apr-11	25-Apr-11
SPT-F03510	FRAME DECK	4	4	21-Apr-11	26-Apr-11
SPT-F03530	ELECTRICAL ROUGH-IN	3	3	25-Apr-11	27-Apr-11
SPT-F03540	MECHANICAL ROUGH-IN	3	3	25-Apr-11	27-Apr-11
SPT-F03500	SET REBAR	2	2	26-Apr-11	27-Apr-11
SPT-F03490	POUR DECK	1	1	28-Apr-11	28-Apr-11
POUR 2 - COL LINES D.5 TO H		6	6	28-Apr-11	05-May-11
ALL TRADES		6	6	28-Apr-11	05-May-11
SPT-F03580	F/R/P COLUMNS AND WALLS	3	3	28-Apr-11	02-May-11
SPT-F03570	FRAME DECK	3	3	29-Apr-11	03-May-11
SPT-F03590	ELECTRICAL ROUGH-IN	3	3	02-May-11	04-May-11
SPT-F03600	MECHANICAL ROUGH-IN	3	3	02-May-11	04-May-11
SPT-F03560	SET REBAR	2	2	03-May-11	04-May-11
SPT-F03550	POUR DECK	1	1	05-May-11	05-May-11
LEVEL 5		11	11	29-Apr-11	13-May-11



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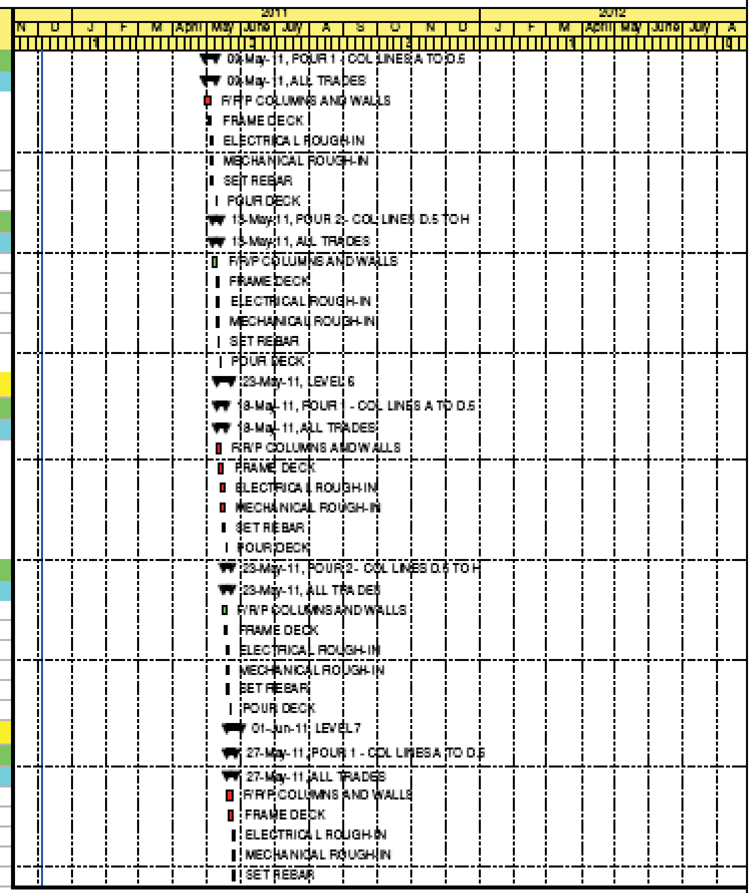
	POUR 1 - COL LINES A TO D.5	7	7	29-Apr-11	09-May-11
	ALL TRADES	7	7	29-Apr-11	09-May-11
SPT-F03400	F/R/P COLUMNS AND WALLS	4	4	29-Apr-11	04-May-11
SPT-F03390	FRAME DECK	4	4	02-May-11	05-May-11
SPT-F03410	ELECTRICAL ROUGH-IN	3	3	04-May-11	06-May-11
SPT-F03420	MECHANICAL ROUGH-IN	3	3	04-May-11	06-May-11
SPT-F03380	SET REBAR	2	2	05-May-11	06-May-11
SPT-F03370	POUR DECK	1	1	09-May-11	09-May-11
	POUR 2 - COL LINES D.5 TO H	6	6	06-May-11	13-May-11
	ALL TRADES	6	6	06-May-11	13-May-11
SPT-F03460	F/R/P COLUMNS AND WALLS	3	3	06-May-11	10-May-11
SPT-F03450	FRAME DECK	3	3	09-May-11	11-May-11
SPT-F03470	ELECTRICAL ROUGH-IN	3	3	10-May-11	12-May-11
SPT-F03480	MECHANICAL ROUGH-IN	3	3	10-May-11	12-May-11
SPT-F03440	SET REBAR	2	2	11-May-11	12-May-11
SPT-F03430	POUR DECK	1	1	13-May-11	13-May-11
	LEVEL 6	10	10	10-May-11	23-May-11
	POUR 1 - COL LINES A TO D.5	7	7	10-May-11	18-May-11
	ALL TRADES	7	7	10-May-11	18-May-11
SPT-F03280	F/R/P COLUMNS AND WALLS	4	4	10-May-11	13-May-11
SPT-F03270	FRAME DECK	4	4	11-May-11	16-May-11
SPT-F03290	ELECTRICAL ROUGH-IN	3	3	13-May-11	17-May-11
SPT-F03300	MECHANICAL ROUGH-IN	3	3	13-May-11	17-May-11
SPT-F03260	SET REBAR	2	2	16-May-11	17-May-11
SPT-F03250	POUR DECK	1	1	18-May-11	18-May-11
	POUR 2 - COL LINES D.5 TO H	6	6	16-May-11	23-May-11
	ALL TRADES	6	6	16-May-11	23-May-11
SPT-F03340	F/R/P COLUMNS AND WALLS	3	3	16-May-11	18-May-11
SPT-F03330	FRAME DECK	3	3	17-May-11	19-May-11
SPT-F03350	ELECTRICAL ROUGH-IN	3	3	18-May-11	20-May-11
SPT-F03360	MECHANICAL ROUGH-IN	3	3	18-May-11	20-May-11
SPT-F03320	SET REBAR	2	2	19-May-11	20-May-11
SPT-F03310	POUR DECK	1	1	23-May-11	23-May-11
	LEVEL 7	10	10	19-May-11	01-Jun-11
	POUR 1 - COL LINES A TO D.5	7	7	19-May-11	27-May-11
	ALL TRADES	7	7	19-May-11	27-May-11
SPT-F03160	F/R/P COLUMNS AND WALLS	4	4	19-May-11	24-May-11
SPT-F03150	FRAME DECK	4	4	20-May-11	25-May-11
SPT-F03170	ELECTRICAL ROUGH-IN	3	3	24-May-11	26-May-11
SPT-F03180	MECHANICAL ROUGH-IN	3	3	24-May-11	26-May-11
SPT-F03140	SET REBAR	2	2	25-May-11	26-May-11



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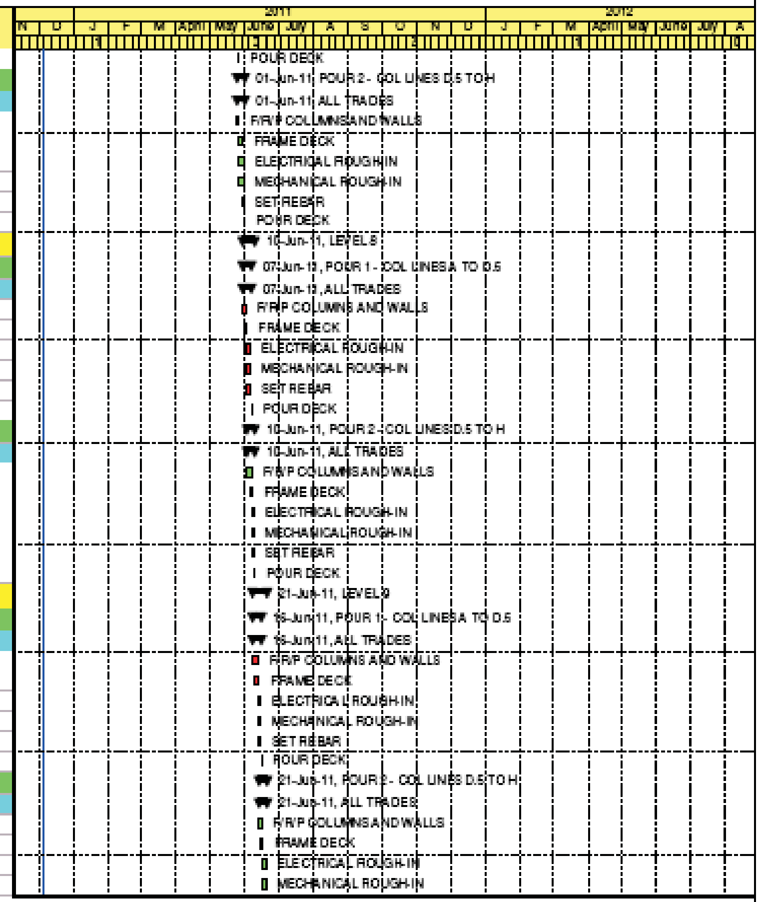
SPT-F03130	POUR DECK	1	1	27-May-11	27-May-11
	POUR 2 - COL LINES D.5 TO H	6	6	25-May-11	01-Jun-11
	ALL TRADES	6	6	25-May-11	01-Jun-11
SPT-F03220	F/R/P COLUMNS AND WALLS	3	3	25-May-11	27-May-11
SPT-F03210	FRAME DECK	3	3	26-May-11	30-May-11
SPT-F03230	ELECTRICAL ROUGH-IN	3	3	27-May-11	31-May-11
SPT-F03240	MECHANICAL ROUGH-IN	3	3	27-May-11	31-May-11
SPT-F03200	SET REBAR	2	2	30-May-11	31-May-11
SPT-F03190	POUR DECK	1	1	01-Jun-11	01-Jun-11
	LEVEL 8	10	10	30-May-11	10-Jun-11
	POUR 1 - COL LINES A TO D.5	7	7	30-May-11	07-Jun-11
	ALL TRADES	7	7	30-May-11	07-Jun-11
SPT-F03040	F/R/P COLUMNS AND WALLS	4	4	30-May-11	02-Jun-11
SPT-F03030	FRAME DECK	4	4	31-May-11	03-Jun-11
SPT-F03050	ELECTRICAL ROUGH-IN	3	3	02-Jun-11	06-Jun-11
SPT-F03060	MECHANICAL ROUGH-IN	3	3	02-Jun-11	06-Jun-11
SPT-F03020	SET REBAR	2	2	03-Jun-11	06-Jun-11
SPT-F03010	POUR DECK	1	1	07-Jun-11	07-Jun-11
	POUR 2 - COL LINES D.5 TO H	6	6	03-Jun-11	10-Jun-11
	ALL TRADES	6	6	03-Jun-11	10-Jun-11
SPT-F03100	F/R/P COLUMNS AND WALLS	3	3	03-Jun-11	07-Jun-11
SPT-F03090	FRAME DECK	3	3	06-Jun-11	08-Jun-11
SPT-F03110	ELECTRICAL ROUGH-IN	3	3	07-Jun-11	09-Jun-11
SPT-F03120	MECHANICAL ROUGH-IN	3	3	07-Jun-11	09-Jun-11
SPT-F03080	SET REBAR	2	2	08-Jun-11	09-Jun-11
SPT-F03070	POUR DECK	1	1	10-Jun-11	10-Jun-11
	LEVEL 9	10	10	08-Jun-11	21-Jun-11
	POUR 1 - COL LINES A TO D.5	7	7	08-Jun-11	16-Jun-11
	ALL TRADES	7	7	08-Jun-11	16-Jun-11
SPT-F02920	F/R/P COLUMNS AND WALLS	4	4	08-Jun-11	13-Jun-11
SPT-F02910	FRAME DECK	4	4	09-Jun-11	14-Jun-11
SPT-F02930	ELECTRICAL ROUGH-IN	3	3	13-Jun-11	15-Jun-11
SPT-F02940	MECHANICAL ROUGH-IN	3	3	13-Jun-11	15-Jun-11
SPT-F02900	SET REBAR	2	2	14-Jun-11	15-Jun-11
SPT-F02890	POUR DECK	1	1	16-Jun-11	16-Jun-11
	POUR 2 - COL LINES D.5 TO H	6	6	14-Jun-11	21-Jun-11
	ALL TRADES	6	6	14-Jun-11	21-Jun-11
SPT-F02980	F/R/P COLUMNS AND WALLS	3	3	14-Jun-11	16-Jun-11
SPT-F02970	FRAME DECK	3	3	15-Jun-11	17-Jun-11
SPT-F02990	ELECTRICAL ROUGH-IN	3	3	16-Jun-11	20-Jun-11
SPT-F03000	MECHANICAL ROUGH-IN	3	3	16-Jun-11	20-Jun-11



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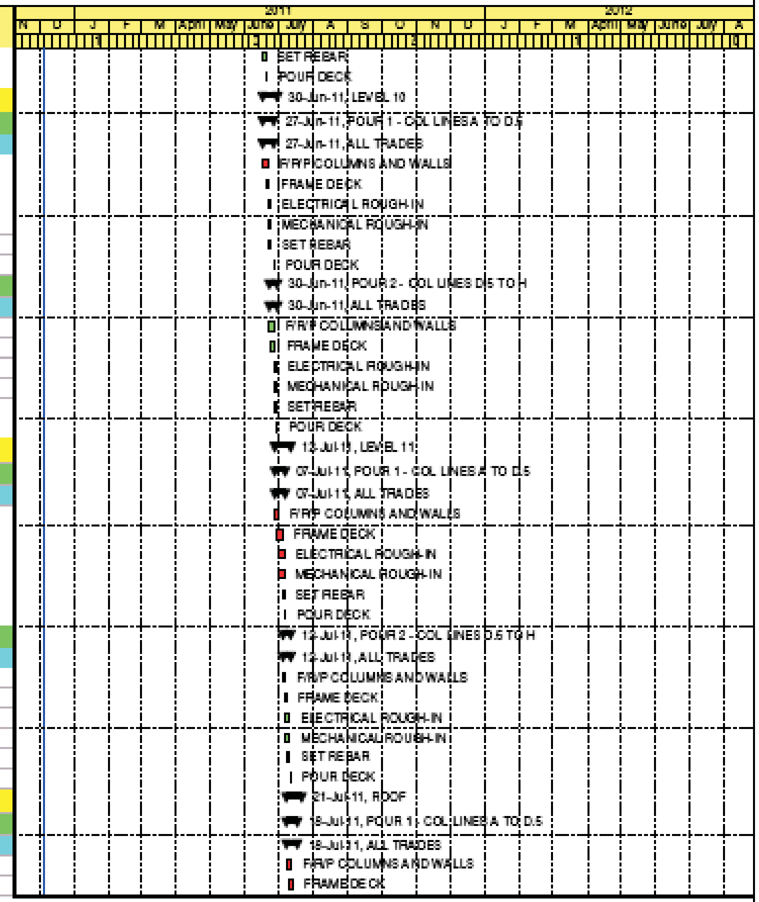
SPT-P02950	SET REBAR	2	2	17-Jun-11	20-Jun-11
SPT-P02960	POUR DECK	1	1	21-Jun-11	21-Jun-11
LEVEL 10		10	10	17-Jun-11	30-Jun-11
POUR 1 - COL LINES A TO D.5		7	7	17-Jun-11	27-Jun-11
ALL TRADES		7	7	17-Jun-11	27-Jun-11
SPT-P02800	F/R/P COLUMNS AND WALLS	4	4	17-Jun-11	22-Jun-11
SPT-P02790	FRAME DECK	4	4	20-Jun-11	23-Jun-11
SPT-P02810	ELECTRICAL ROUGH-IN	3	3	22-Jun-11	24-Jun-11
SPT-P02820	MECHANICAL ROUGH-IN	3	3	22-Jun-11	24-Jun-11
SPT-P02780	SET REBAR	2	2	23-Jun-11	24-Jun-11
SPT-P02770	POUR DECK	1	1	27-Jun-11	27-Jun-11
POUR 2 - COL LINES D.5 TO H		6	6	23-Jun-11	30-Jun-11
ALL TRADES		6	6	23-Jun-11	30-Jun-11
SPT-P02860	F/R/P COLUMNS AND WALLS	3	3	23-Jun-11	27-Jun-11
SPT-P02850	FRAME DECK	3	3	24-Jun-11	28-Jun-11
SPT-P02870	ELECTRICAL ROUGH-IN	3	3	27-Jun-11	29-Jun-11
SPT-P02880	MECHANICAL ROUGH-IN	3	3	27-Jun-11	29-Jun-11
SPT-P02840	SET REBAR	2	2	28-Jun-11	29-Jun-11
SPT-P02830	POUR DECK	1	1	30-Jun-11	30-Jun-11
LEVEL 11		10	10	28-Jun-11	12-Jul-11
POUR 1 - COL LINES A TO D.5		7	7	28-Jun-11	07-Jul-11
ALL TRADES		7	7	28-Jun-11	07-Jul-11
SPT-P02680	F/R/P COLUMNS AND WALLS	4	4	28-Jun-11	01-Jul-11
SPT-P02670	FRAME DECK	4	4	29-Jun-11	05-Jul-11
SPT-P02690	ELECTRICAL ROUGH-IN	3	3	01-Jul-11	06-Jul-11
SPT-P02700	MECHANICAL ROUGH-IN	3	3	01-Jul-11	06-Jul-11
SPT-P02660	SET REBAR	2	2	05-Jul-11	06-Jul-11
SPT-P02650	POUR DECK	1	1	07-Jul-11	07-Jul-11
POUR 2 - COL LINES D.5 TO H		6	6	05-Jul-11	12-Jul-11
ALL TRADES		6	6	05-Jul-11	12-Jul-11
SPT-P02740	F/R/P COLUMNS AND WALLS	3	3	05-Jul-11	07-Jul-11
SPT-P02730	FRAME DECK	3	3	06-Jul-11	08-Jul-11
SPT-P02750	ELECTRICAL ROUGH-IN	3	3	07-Jul-11	11-Jul-11
SPT-P02760	MECHANICAL ROUGH-IN	3	3	07-Jul-11	11-Jul-11
SPT-P02720	SET REBAR	2	2	08-Jul-11	11-Jul-11
SPT-P02710	POUR DECK	1	1	12-Jul-11	12-Jul-11
ROOF		10	10	08-Jul-11	21-Jul-11
POUR 1 - COL LINES A TO D.5		7	7	08-Jul-11	18-Jul-11
ALL TRADES		7	7	08-Jul-11	18-Jul-11
SPT-P02530	F/R/P COLUMNS AND WALLS	4	4	08-Jul-11	13-Jul-11
SPT-P02540	FRAME DECK	4	4	11-Jul-11	14-Jul-11



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SPT-F02550	ELECTRICAL ROUGH-IN	3	3	13-Jul-11	15-Jul-11
SPT-F02560	MECHANICAL ROUGH-IN	3	3	13-Jul-11	15-Jul-11
SPT-F02570	SET REBAR	2	2	14-Jul-11	15-Jul-11
SPT-F02580	POUR DECK	1	1	18-Jul-11	18-Jul-11
POUR 2 - COL LINES D.5 TO H		6	6	14-Jul-11	21-Jul-11
ALL TRADES		6	6	14-Jul-11	21-Jul-11
SPT-F02590	F/R/P COLUMNS AND WALLS	3	3	14-Jul-11	18-Jul-11
SPT-F02600	FRAME DECK	3	3	15-Jul-11	19-Jul-11
SPT-F02610	ELECTRICAL ROUGH-IN	3	3	18-Jul-11	20-Jul-11
SPT-F02620	MECHANICAL ROUGH-IN	3	3	18-Jul-11	20-Jul-11
SPT-F02630	SET REBAR	2	2	19-Jul-11	20-Jul-11
SPT-F02640	POUR DECK	1	1	21-Jul-11	21-Jul-11
PENTHOUSE		51	51	19-Jul-11	27-Sep-11
HIGH PENTHOUSE - COL LINES A TO B.2		6	6	19-Jul-11	26-Jul-11
ALL TRADES		6	6	19-Jul-11	26-Jul-11
SPT-F03640	F/R/P COLUMNS AND WALLS	2	2	19-Jul-11	20-Jul-11
SPT-F03630	FRAME DECK	2	2	20-Jul-11	21-Jul-11
SPT-F03650	ELECTRICAL ROUGH-IN	1	1	21-Jul-11	21-Jul-11
SPT-F03660	MECHANICAL ROUGH-IN	1	1	21-Jul-11	21-Jul-11
SPT-F03620	SET REBAR	2	2	22-Jul-11	25-Jul-11
SPT-F03610	POUR DECK	1	1	26-Jul-11	26-Jul-11
HIGH PENTHOUSE ROOF		45	45	27-Jul-11	27-Sep-11
ALL TRADES		45	45	27-Jul-11	27-Sep-11
SPT-F03700	F/R/P COLUMNS AND WALLS	1	1	27-Jul-11	27-Jul-11
SPT-F03690	FRAME DECK	2	2	28-Jul-11	29-Jul-11
SPT-F03710	ELECTRICAL ROUGH-IN	1	1	29-Jul-11	29-Jul-11
SPT-F03720	MECHANICAL ROUGH-IN	1	1	29-Jul-11	29-Jul-11
SPT-F03680	SET REBAR	1	1	01-Aug-11	01-Aug-11
SPT-F03670	POUR DECK	1	1	02-Aug-11	02-Aug-11
SPT-F03750	CONCRETE STRUCTURE SUBSTANTIALLY COMPLETE	0	0		02-Aug-11
SPT-F03760	ERECT STEEL FRAMING	5	5	31-Aug-11	06-Sep-11
SPT-F03770	DETAIL STRUCTURE STEEL	5	5	07-Sep-11	13-Sep-11
SPT-F03780	INSTALL DECK	5	5	14-Sep-11	20-Sep-11
SPT-F03790	POUR SLAB ON METAL DECK	5	5	21-Sep-11	27-Sep-11
HELIPAD		60	60	09-Sep-11	01-Dec-11
ALL TRADES		60	60	09-Sep-11	01-Dec-11
SPT-F03800	INSTALL POSTS FOR HELIPAD	5	5	09-Sep-11	15-Sep-11
SPT-F03810	INSTALL HELIPAD FRAMING	15	15	16-Sep-11	06-Oct-11
SPT-F03820	INSTALL HELIPAD DECK AND WALKWAY	10	10	07-Oct-11	20-Oct-11
SPT-F03830	INSTALL ICE MELT SYSTEM/DELUGE SYSTEMS	15	15	21-Oct-11	10-Nov-11
SPT-F03840	HELIPAD TRIM AND TEST	15	15	11-Nov-11	01-Dec-11

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