Technical Report #2

Matthew R Peyton

This Document is Technical Report #2 for 5th year senior thesis in the Architectural Engineering Departments at The Pennsylvania State University. This Report is to prepare a study and comparison of at least four different alternation floor framing systems for the structure.

Structural Option

Professor Behr

Hospital Patient Tower

Virginia, U.S.A.

10/4/2010

Table of Contents

Executive Summary

Technical Report #2 is a study and comparison of four different structural floor systems. In this report the existing two-way slab will be compared to 3 different system designed for the Hospital Patient Tower. The three alternative systems studied include:

- Composite deck on Wide flange steel beams
- Non-Composite deck on Open web steel joists
- One-way concrete slab with Beams

The typical bays for the Hospital tower are 29' x29'. All of the bays found in the patient tower are the square 29'x 29' except the center span in the east west direction which is cut in half in one span. With the typical bay sizing for the tower a 3 bay x 3 bay section of a typical 29' x 29' bays was used during the structural analysis.

These alternative structural floor systems will be compared based on a few different criteria. The primary means of these comparisons will be by system weight, architectural impact and serviceability. In addition to those criteria, several other factors will be taken in to account such as; fire protection, constructability and cost.

After the completion of the analysis, it was determined that the best option of the four Systems considered is the existing Two-way flat slab. The major advantage of this system is that it allows the floor elevations of the new tower to line up with those of the existing patient tower while still keeping a low system total thickness.

The composite deck system remains a viable option because it works with the existing column layout and will lower the total building weight affecting the seismic loading and the foundation. The other two alternatives are ruled out from further consideration due to the need for floor layout changes and lack of available mechanical space.

Figure 1: Rendering by Wilmot Sanz

Introduction

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

One of the main design considerations is individual patient rooms. Based on the 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 for the physician, nurses and staff to help make the design as efficient as possible. Medical/surgical patients aging 65 years and older were the focus of this Tower with a special emphasis on their safety and a good healing environment. With the hospital team input the placements for monitoring stations were optimized to ensure patient privacy as well as enhancing the monitoring capabilities.

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

Figure 2: Sketch by Wilmot Sanz

Structural Systems

Foundations

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

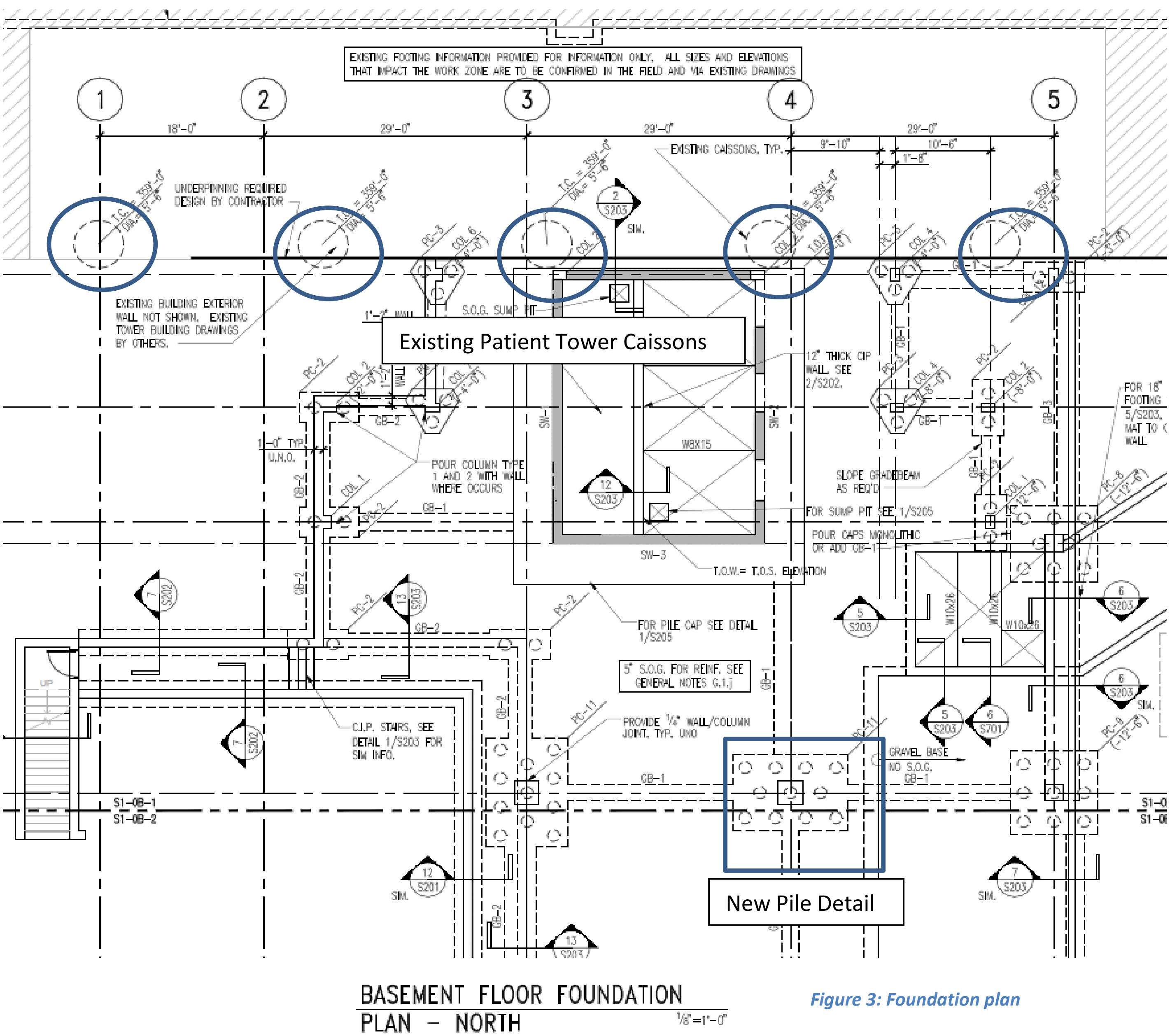


Figure 3: Foundation plan

Columns

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

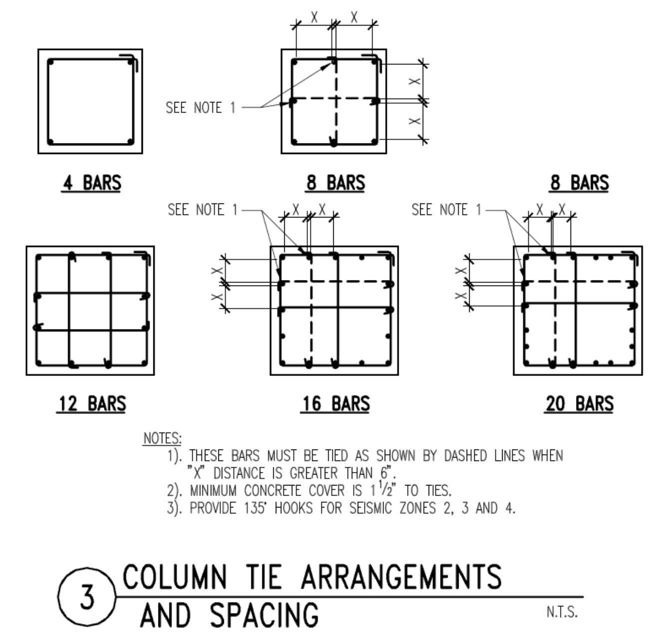


Figure 4: Column Reinforcing Detail

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

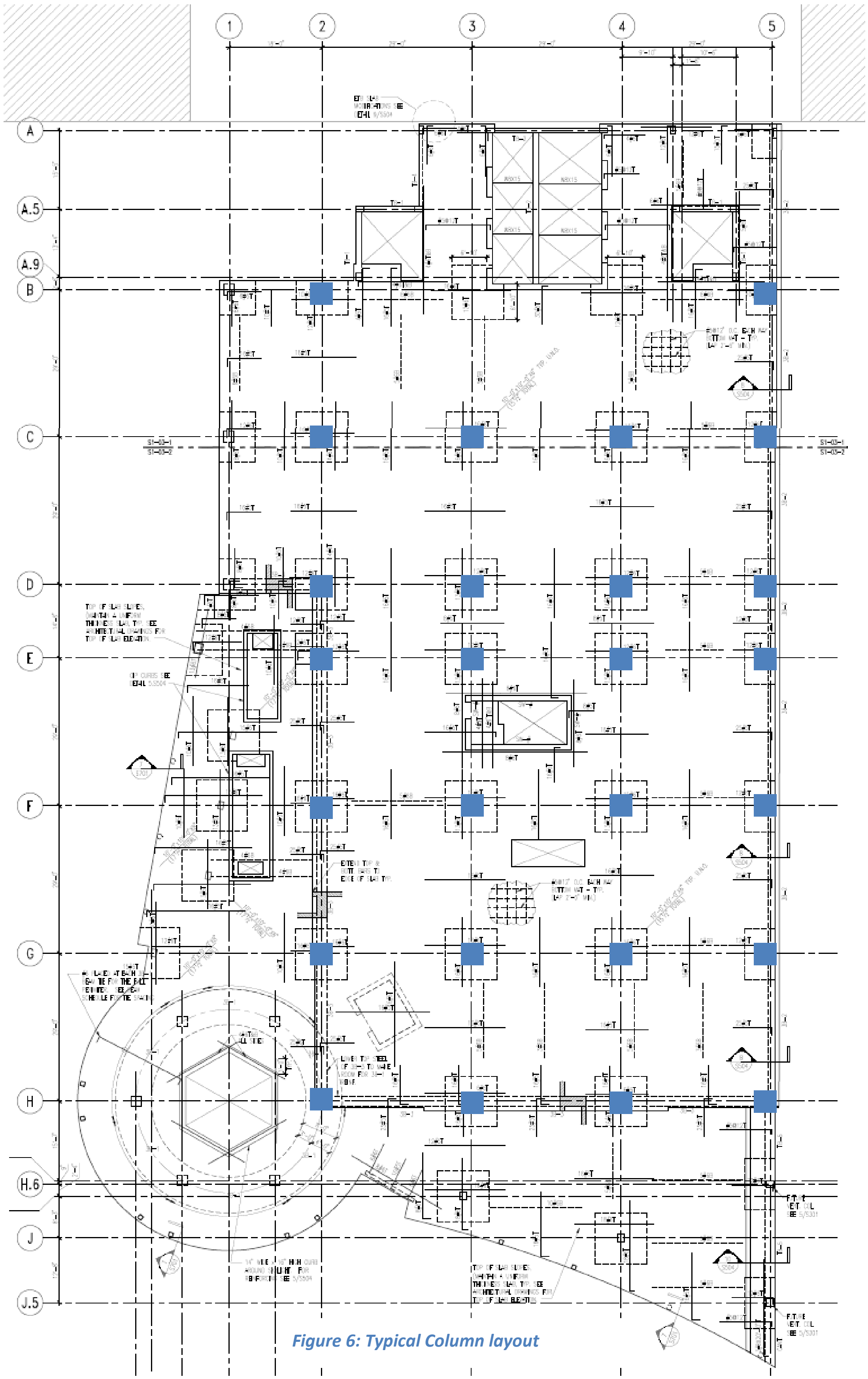


Figure 6: Typical Column layout

Floor System

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

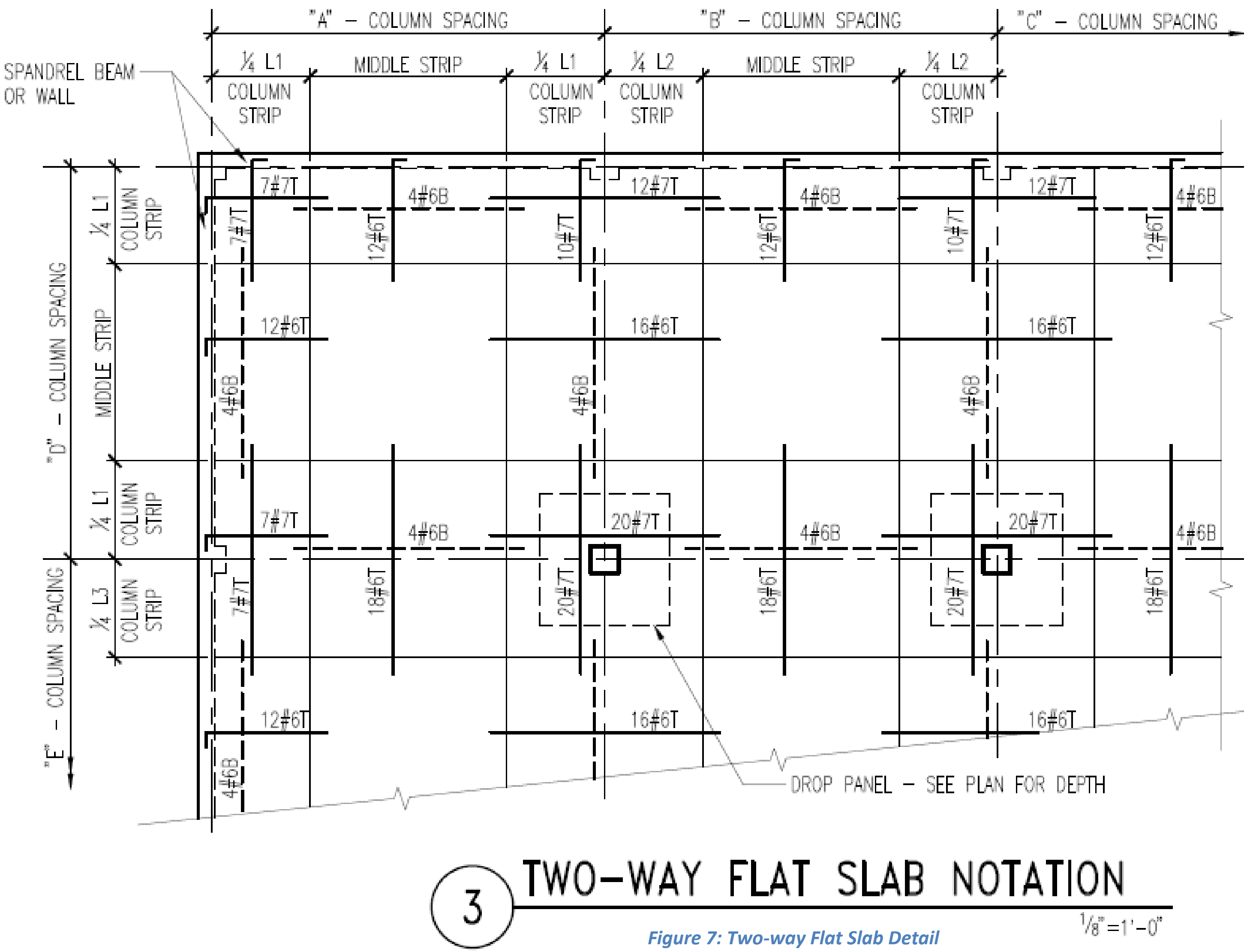


Figure 7: Two-way Flat Slab Detail

Roof System

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

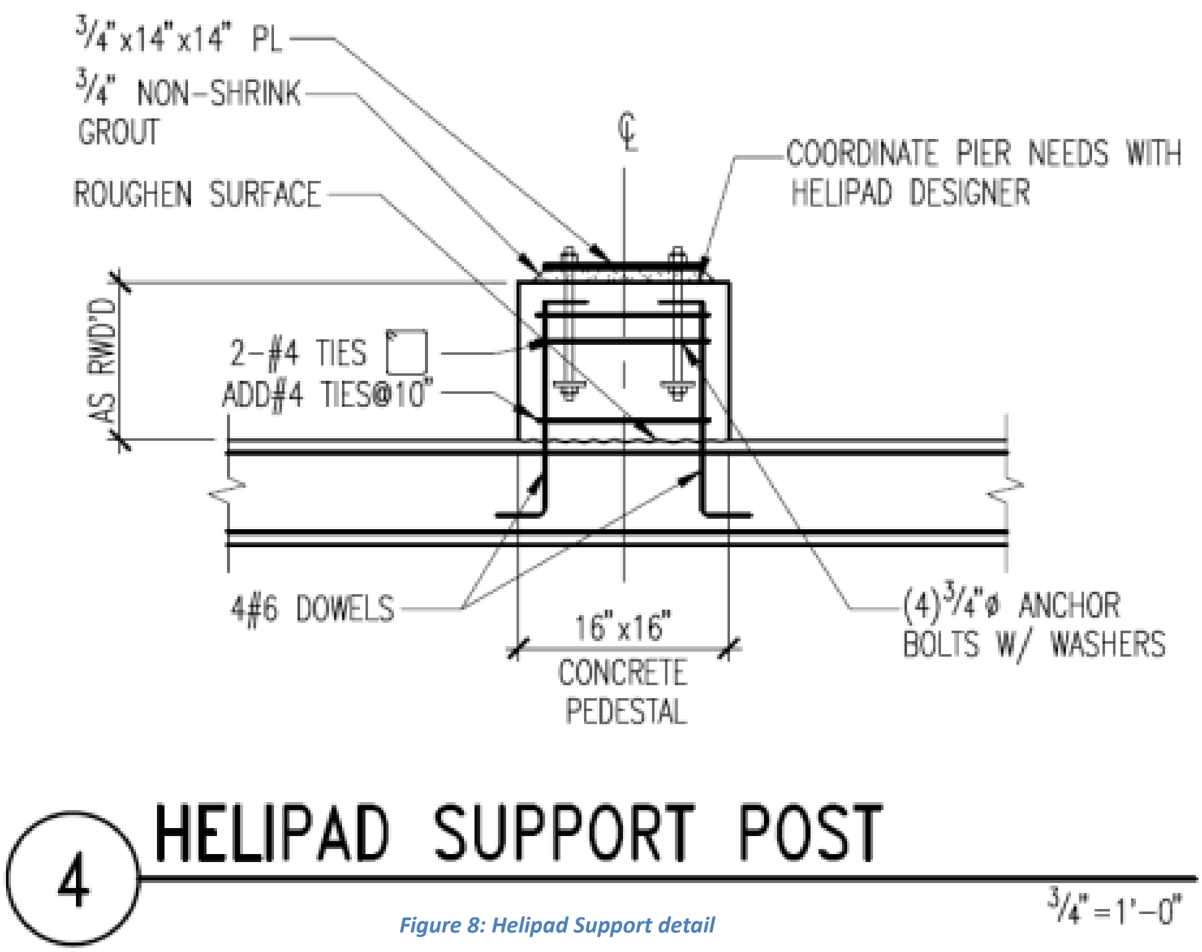


Figure 8: Helipad Support detail

Lateral System

The lateral system in the new patient tower consists of seven 12” reinforced concrete shear walls. These walls are located in different locations throughout the building depicted to the right. The shear walls consisted of 5000 psi concrete and were run continuously through the tower from the foundations up to the roof with the northern core extending through the penthouse. This system of two shear wall cores resists lateral loads in both the north-south and east-west direction based on the orientation of the wall.

Design & Code Review

Design Codes and References

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

Thesis Codes and References

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

Deflection Criteria

Floor Deflection Criteria

Typical Live load Deflection limited to L/360

Typical Total load Deflection limited to L/240

Material Specifications

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

Table 1: Material Specifications

Gravity Loads

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

Dead Loads

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

Table 2: Dead Loads

Live Loads

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

Table 3: Live Loads

Snow Loads

$p_f = 0.7C_eC_tI_sp_g$

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

Table 4: Snow Loads

Design Considerations

In order to have a complete investigation for the floor systems explored in this report, a set of comparison criteria has been established.

Structural	Architectural	Construction	Serviceability
System Weight	System Depth	Constructability	Deflection
Lateral system impact	Floor plan adjustments	Cost	
Foundations impacts			
Fire Protection			

Each of these factors will be discussed in terms of each system design and compared to see if how each system stacks up against the others. Each of these systems was designed using a typical 3 bay x 3 bays section of the towers floor plan.

A large consideration was taken in to account during the design of these floor systems to keep the floors elevations aligned with those of the existing patient tower. During the design of the alternate floor systems a ceiling cavity of 28" from the top of the slab to the bottom of the ceiling was considered to be the maximum allowable spaced used. Within the ceiling cavity there must also be a cavity maintained for the mechanical system. Each system was able to be designed to fit with in the 28" cavity but there were varying amounts of space left for mechanical system.

A wide range of materials and systems were investigated during this assignment. These systems were chosen with a basic idea of their strength and constructability but upon farther investigation strengths and weaknesses for all of the system became more apparent.

Floor Systems

Two-Way Reinforced Flat Plate - Existing

The Existing Two-way reinforced slab was designed to be 9.5"in depth with 6" drop panel around the columns. This system is mildly reinforced with 60k steel rebar. This system was chosen because of the 29'x29' square bays that are used in the floor plan design of the tower which are ideal for a two-way slab. With the two-way slab the total depth is kept to a minimum giving a lot of room for the mechanical systems to be placed in the ceiling cavity. The slab is reinforced with #5 each way bottom mat with spacing @12" O.C. with extra #8 bars in the end spans to support carry over and end span moment s. Top reinforcing is located perpendicular over column lines in both directions and at the end of slab where column are connected. The top bars are #5 and #6 bars with varies spacing depending on the locations and the forces at that location. Detailed layout of a slab section is showed below in Figure #9. Calculations for this system can be found in Appendix II.

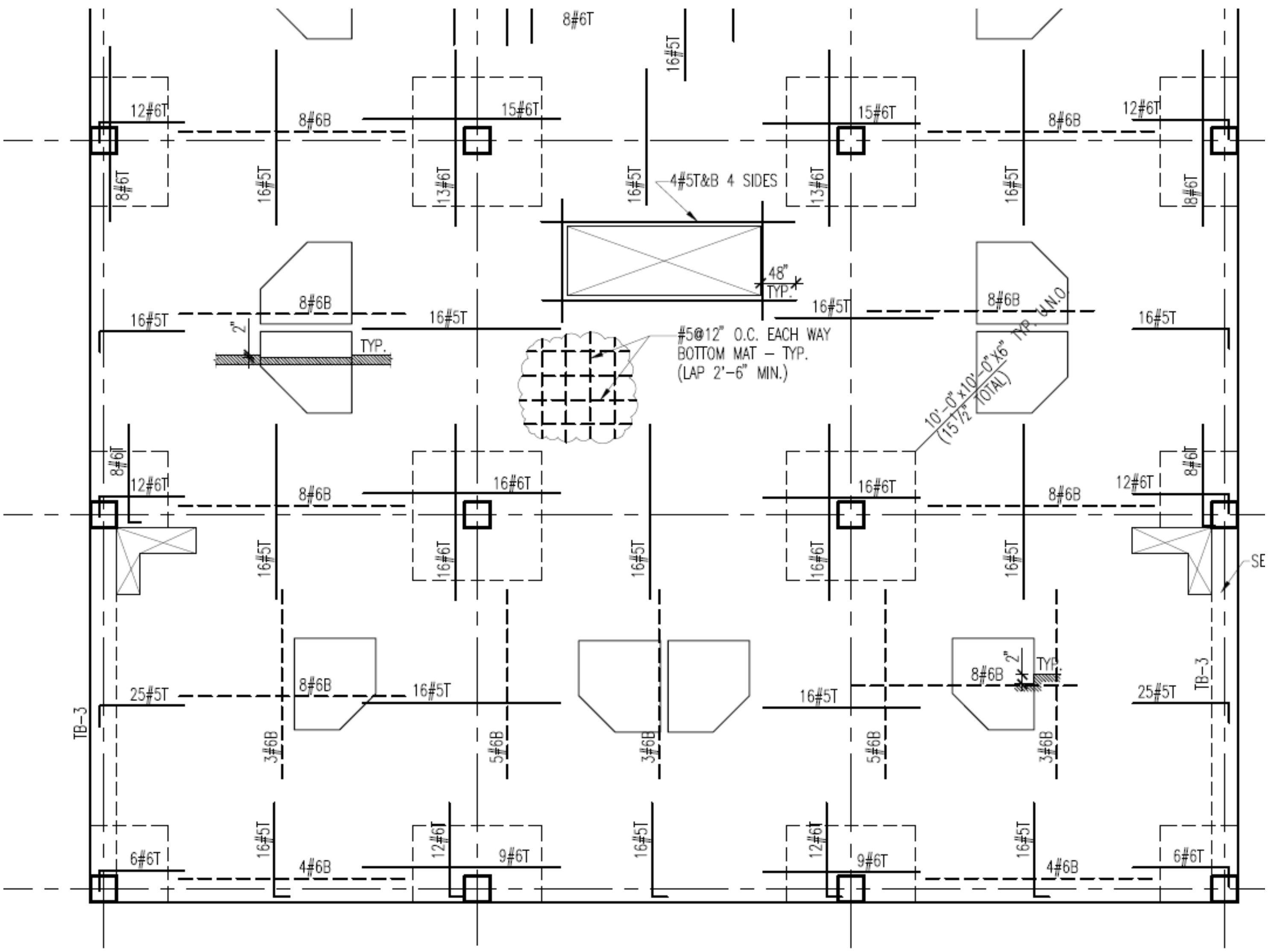


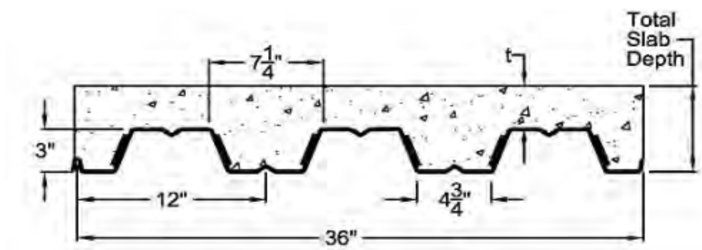
Figure 9: Two-way Slab Layout

W- Flange Beams with Composite Metal Deck

This system was designed using the Vulcraft steel Deck design guide (figure 10) and AISC steel manual. For the typical bay size of 29' x29' a 3" normal weight concrete on 3" on Vulcraft 3VLI Composite deck. Since a 3" normal weight concrete deck does not meet the 2 hour fire rating a fire proofing will need to be applied. The composite deck spans 9'-4" perpendicular to the supporting wide flange beam and parallel to the wide flange girders. The Composite deck 3VLI 18Ga. was found to support 238psf at a span of 9'-6" which is more than adequate to carry the load required. The beams were designed using the AISC Steel manual it was found that a beam size of W10x26 with 30 0.75" shear studs would be adequate to carry the loads needed. A camber of 0.75" is needed in order for the beams to handle the wet concrete deflection in this composite design. For the Girder the same protocols and design criteria were used as in the beam design giving a girder size of W16x45 with 34 shear studs with a diameter of 0.75". In order to keep the floor to floor height at the same level as the existing patient tower there were design decision make accordingly to maintain a proper floor to ceiling height in the new tower.

3 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



Interlocking side lap is not drawn to show actual detail.

STEEL SECTION PROPERTIES

Deck Type	Design Thickness in	Deck Weight psf	Section Properties I_p in^4/ft	S_p in^3/ft	I_n in^4/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
3VLI22	0.0295	1.77	0.730	0.414	0.729	0.426	1528	50
3VLI20	0.0358	2.14	0.920	0.534	0.919	0.551	2698	50
3VLI19	0.0418	2.50	1.104	0.654	1.102	0.676	3678	50
3VLI18	0.0474	2.84	1.254	0.770	1.252	0.797	4729	50
3VLI16	0.0598	3.58	1.580	1.013	1.580	1.013	5309	40

(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span 1 SPAN	2 SPAN	3 SPAN	Superimposed Live Load, PSF Clear Span (ft.-in.) 7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
5.00 (t=2.00) 45 PSF	3VLI22	9'-2	10'-7	11'-8	216	195	176	161	148	109	99	90	83	76	70	64	59	54	50
	3VLI20	10'-8	12'-11	13'-4	241	216	196	178	163	150	139	129	93	85	78	72	66	61	57
	3VLI19	12'-0	14'-4	14'-7	265	237	214	194	178	163	151	140	131	122	115	79	73	67	62
	3VLI18	12'-10	15'-1	15'-1	289	261	238	218	201	186	173	161	151	142	134	127	92	86	80
	3VLI16	13'-5	15'-7	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
5.50 (t=2.50) 51 PSF	3VLI22	8'-9	9'-8	10'-11	247	222	201	184	137	124	113	103	94	87	80	73	67	62	57
	3VLI20	10'-1	12'-4	12'-9	275	247	223	203	186	171	159	116	106	97	89	82	76	70	65
	3VLI19	11'-4	13'-8	14'-2	302	270	244	222	203	186	172	160	149	107	98	90	83	77	71
	3VLI18	12'-5	14'-7	14'-7	330	298	271	248	229	212	197	184	173	162	153	112	105	98	92
	3VLI16	12'-9	14'-11	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102
6.00 (t=3.00) 57 PSF	3VLI22	8'-4	8'-10	10'-1	277	249	226	171	154	140	127	116	106	97	89	82	76	70	65
	3VLI20	9'-8	11'-10	12'-3	309	277	250	228	209	193	143	130	119	109	100	92	85	79	73
	3VLI19	10'-10	13'-2	13'-7	339	304	274	249	227	209	193	179	131	120	110	102	94	87	80
	3VLI18	11'-10	14'-2	14'-2	370	334	304	279	257	238	221	207	194	182	136	126	118	110	103
	3VLI16	12'-2	14'-4	14'-10	400	376	341	311	286	264	245	228	213	200	189	141	132	123	115
	3VLI22	8'-0	8'-3	9'-4	307	277	251	190	171	155	141	129	118	108	99	91	84	78	72

Figure 10: Vulcraft Composite Deck Charts

In figure 11 the beam and girder layout and sizing can be seen. Calculations for this system can be found in Appendix III.

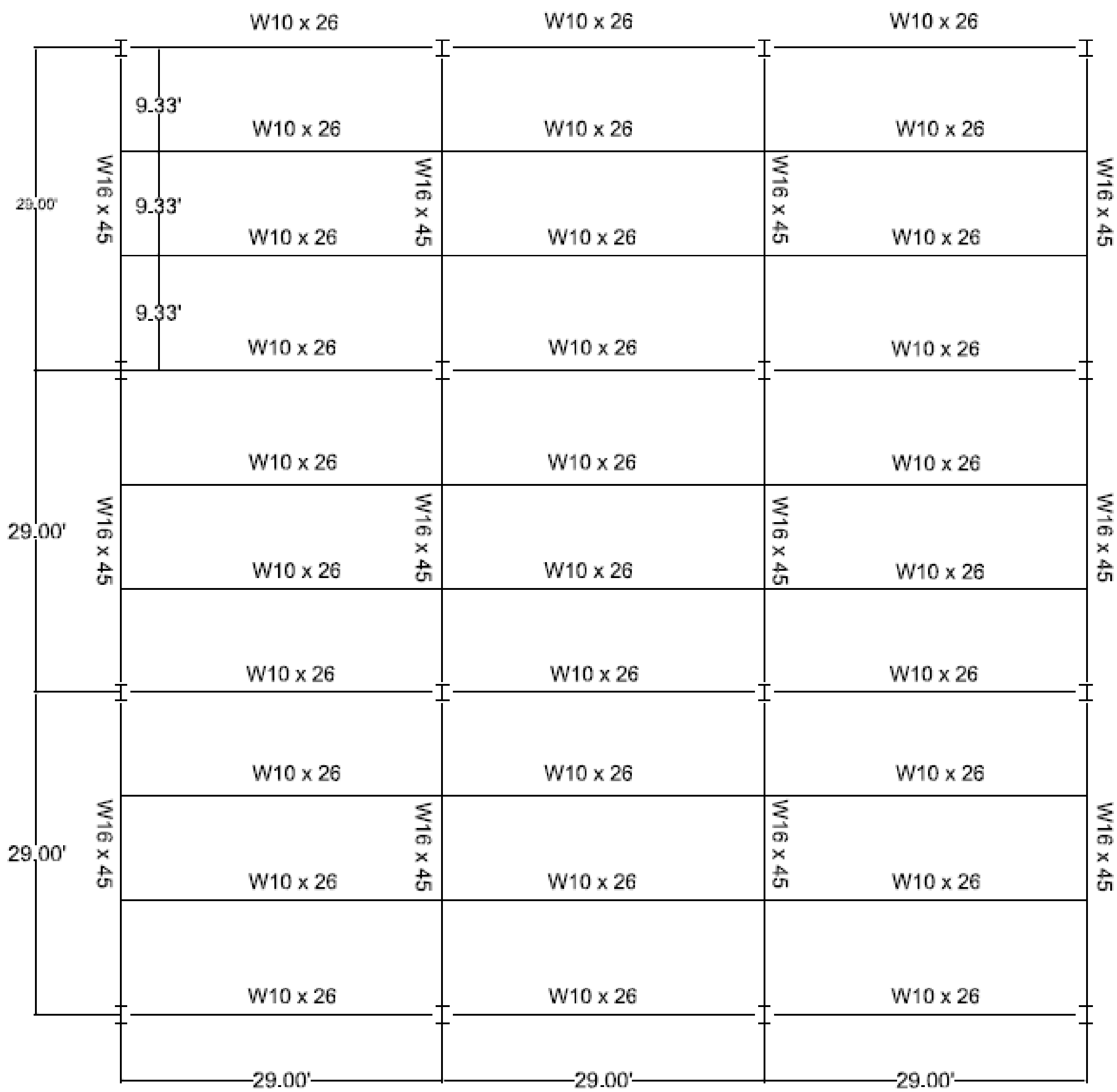


Figure 11: Composite deck system layout

Open Web Steel Joist with Non-composite Metal Deck

This System was designed using the Vulcraft non-composite deck tables (figure 12) and the Long Span steel joists LH-series tables (figure 13). A 3.5" light weight concrete topping was selected to be placed on the 1.3C24 deck which will give the system a 2 hour fire rating. The deck is supported by 20LH06 Vulcraft open web joists spaced at 4'-10" O.C. The Long span joists are 20 inches deep each carrying 1018 lb/ft which is adequate with still keeping the total floor system thickness below the necessary 28" to maintain proper floor alignment with the existing tower. The girders for this system were designed to be wide flange W21x101. This girder size is adequate to carry all of the loads need for the system but since the design needed to stay within the given 28" ceiling to floor cavity there were some economic sacrifices made in this design. Calculations for this system can be found in Appendix IV.

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd^3 / 100 ft^2	Theo. Concrete Volume ft^3 / ft^2	Recommended Welded Wire Fabric
3.3	0.82	0.221	6x6 - W1.4xW1.4
3.8	0.97	0.263	6x6 - W1.4xW1.4
4.3	1.13	0.304	6x6 - W1.4xW1.4
4.55	1.20	0.325	6x6 - W1.4xW1.4
4.8	1.28	0.346	6x6 - W2.1xW2.1
5.3	1.44	0.388	6x6 - W2.1xW2.1
5.55	1.51	0.408	6x6 - W2.1xW2.1
5.8	1.59	0.429	6x6 - W2.1xW2.1

SECTION PROPERTIES

Deck Type	Design Thickness in.	Deck Weight psf	Section Properties I_p in^4/ft	I_n in^4/ft	S_p in^3/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
1.3C26	0.0179	0.99	0.070	0.069	0.097	0.098	1940	60
1.3C24	0.0239	1.33	0.093	0.093	0.132	0.132	3458	60
1.3C22	0.0295	1.62	0.115	0.115	0.163	0.162	4789	60
1.3C20	0.0358	1.97	0.140	0.140	0.197	0.197	5727	60

ALLOWABLE UNIFORM LOAD (PSF)

TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 4- 0	4- 6	5- 0	5- 6	6- 0	6- 6	7- 0	7- 6	8- 0	8- 6	9- 0	9- 6	10- 0
1.3C26	1	Fb = 36,000	145	115	93	77	65	55	47	41	36	32	29	26	23
		Defl. = l/240	72	50	37	28	21	17	13	11	9	7	6	5	5
		Defl. = l/180	96	67	49	37	28	22	18	15	12	10	8	7	6
	2	Fb = 36,000	144	114	93	77	65	55	48	42	37	32	29	26	23
		Defl. = l/240	172	121	88	66	51	40	32	26	21	18	15	13	11
		Defl. = l/180	229	161	117	88	68	53	43	35	29	24	20	17	15
	3	Fb = 36,000	179	142	115	96	81	69	59	52	46	40	36	32	29
		Defl. = l/240	134	94	69	52	40	31	25	20	17	14	12	10	9
		Defl. = l/180	179	126	92	69	53	42	33	27	22	19	16	13	11
1.3C24	1	Fb = 36,000	198	156	126	105	88	75	65	56	49	44	39	35	32
		Defl. = l/240	95	67	49	37	28	22	18	14	12	10	8	7	6
		Defl. = l/180	127	89	65	49	38	30	24	19	16	13	11	9	8
	2	Fb = 36,000	196	155	126	104	87	75	64	56	49	44	39	35	32
		Defl. = l/240	230	161	118	88	68	54	43	35	29	24	20	17	15
		Defl. = l/180	306	215	157	118	91	71	57	46	38	32	27	23	20
	3	Fb = 36,000	243	193	$\boxed{157}$	130	109	93	80	70	62	55	49	44	39
		Defl. = l/240	180	126	$\boxed{92}$	69	53	42	34	27	22	19	16	13	12
		Defl. = l/180	240	168	123	92	71	56	45	36	30	25	21	18	15
		Fb = 36,000	244	193	156	129	108	92	80	69	61	54	48	43	39

Figure 12: Non - Composite deck charts

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft (Joists only)	Depth in inches	SAFE LOAD* in Lbs. Between	CLEAR SPAN IN FEET															
			21-24	25	26	27	28	29	30	31	32	33	34	35	36				
18LH02	10	18	18000	702 313	663 284	627 259	586 234	550 212	517 193	486 175	459 160	433 147	409 135	388 124	367 114				
18LH03	11	18	19950	781 348	739 317	700 289	657 262	613 236	573 213	538 194	505 177	475 161	448 148	424 136	400 124				
18LH04	12	18	23250	906 403	856 367	802 329	750 296	703 266	660 242	619 219	582 200	547 182	516 167	487 153	462 141				
18LH05	15	18	26250	1026 454	972 414	921 378	871 345	814 311	762 282	714 256	672 233	631 212	595 195	562 179	532 164				
18LH06	15	18	31050	1213 526	1123 469	1044 419	972 377	907 340	849 307	796 280	748 254	705 232	664 212	627 195	594 180				
18LH07	17	18	32250	1260 553	1213 513	1170 476	1089 428	1017 386	952 349	892 317	838 288	789 264	744 241	703 222	666 204				
18LH08	19	18	33600	1314 577	1264 534	1218 496	1176 462	1137 427	1075 387	1020 351	961 320	906 292	856 267	810 246	768 226				
18LH09	21	18	36000	1404 616	1351 571	1302 527	1257 491	1215 458	1174 418	1138 380	1069 346	1006 316	949 289	897 266	849 245				
			22-24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
20LH02	10	20	16950	663 306	655 303	646 298	615 274	582 250	547 228	516 208	487 190	460 174	436 160	412 147	393 136	373 126	355 117	337 108	322 101
20LH03	11	20	18000	703 337	694 333	687 317	678 302	651 280	621 258	592 238	558 218	528 200	499 184	474 169	448 156	424 143	403 133	382 123	364 114
20LH04	12	20	22050	861 428	849 406	837 386	792 352	744 320	700 291	660 265	624 243	589 223	558 205	529 189	502 174	477 161	454 149	433 139	412 129
20LH05	14	20	23700	924 459	913 437	903 416	892 395	856 366	816 337	769 308	726 281	687 258	651 238	616 219	585 202	556 187	529 173	504 161	481 150
20LH06	15	20	31650	1233 606	1186 561	1144 521	1084 477	1018 427	952 386	894 351	840 320	790 292	745 267	703 246	666 226	631 209	598 192	568 178	541 165
20LH07	17	20	33750	1317 647	1267 599	1221 556	1179 518	1140 484	1066 438	1000 398	940 362	885 331	834 303	789 278	745 256	706 236	670 218	637 202	606 187
20LH08	19	20	34800	1362 669	1309 619	1263 575	1219 536	1177 500	1140 468	1083 428	1030 395	981 365	931 336	882 309	837 285	795 262	754 242	718 225	685 209
20LH09	21	20	38100	1485 729	1429 675	1377 626	1329 581	1284 542	1242 507	1203 475	1167 437	1132 399	1068 366	1009 336	954 309	904 285	858 264	816 244	775 227
20LH10	23	20	41100	1602 786	1542 724	1486 673	1434 626	1386 585	1341 545	1297 510	1258 479	1221 448	1186 411	1122 377	1060 346	1005 320	954 296	906 274	862 254

Figure 13: Open Web Joists Charts

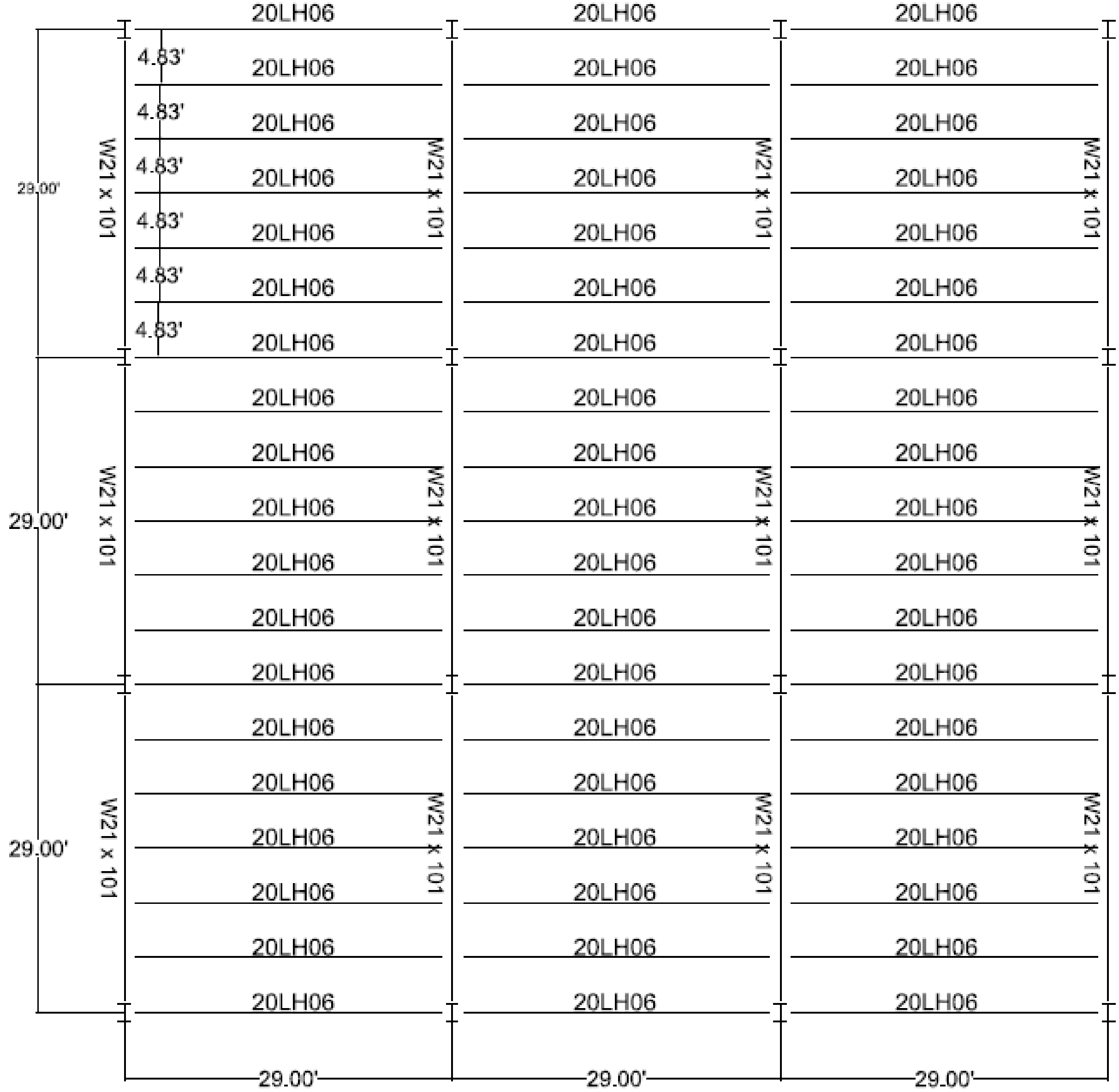


Figure 14: Open web joist system layout

One-Way Slab and Beam

This system consists of a concrete slab and beams, with the tower having square bays both the slab and the beams will span a length of 29'. The beams will be designed in the North-South direction while the slab will span in the east-west direction. The slab thickness was designed using ACI design table 9.5(a) from this table it was determined that a 12.5" slab was needed to support the loads. The slab is reinforced on the top and bottom with a #6 bar @ 12" O.C. From the ACI design table 9.5(a) the beam height was also designed to be 18" and a width of 24". The beams for this system require a minimum of 10 #8 rebar to support the loads and control deflection for this system. With the depth and cover of this system there is no need to provide extra fire protection. Calculations for this system can be found in Appendix V.

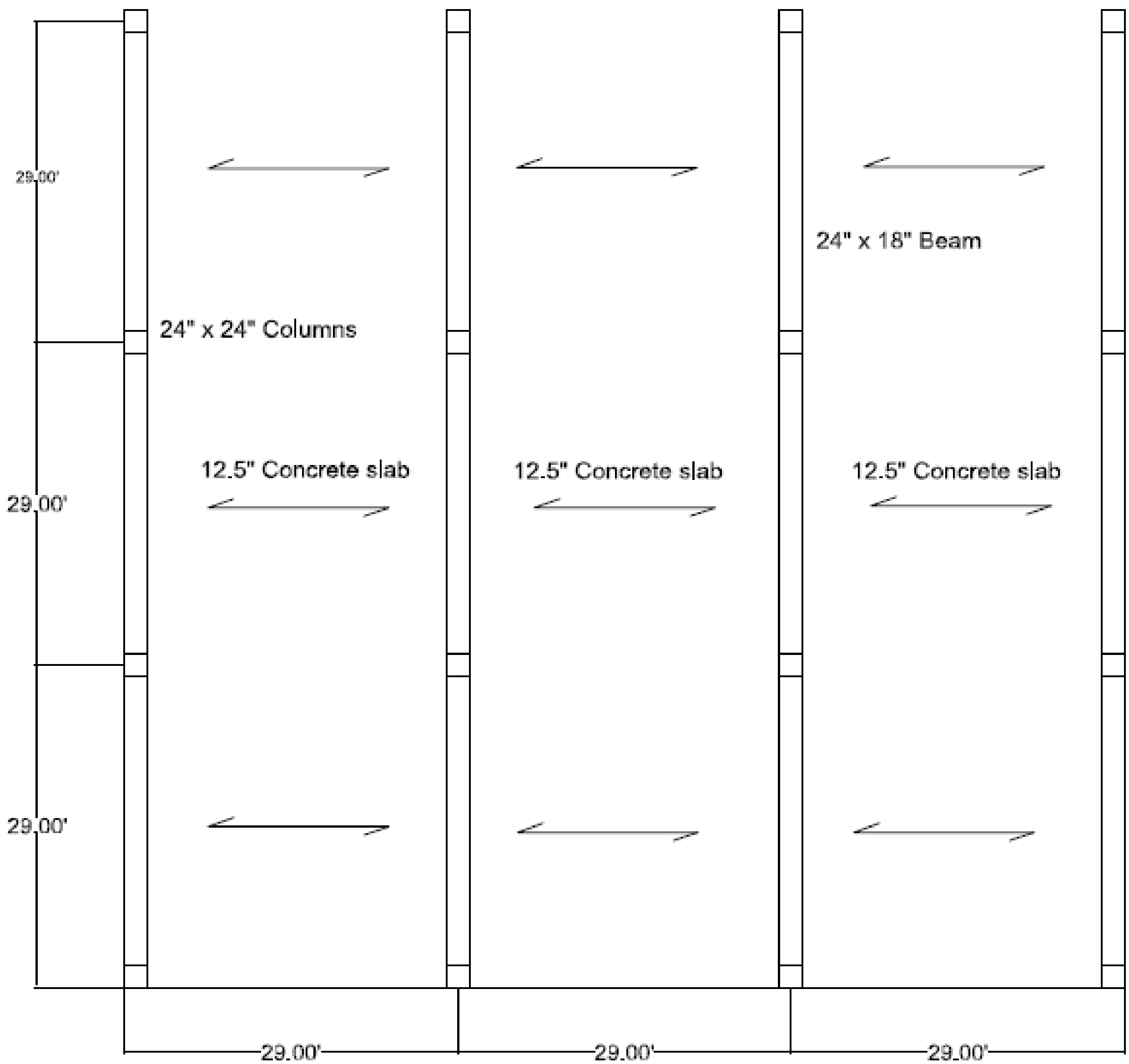


Figure 15: Open web joist system layout

Floor System Comparison

Structural

The floor system has a significant impact on the lateral force resisting system of a building. Since the existing system is a cast in place concrete system the lateral system shear walls are also cast in place concrete allowing the use of the same lateral force resisting for another concrete system. With the one-way concrete slab and beam system the design of the existing lateral system would be suitable to resist the load. With the beams in the one way system there will be an internal moment connection in the North –South direction so there maybe the ability to remove some of the shear walls from the existing design for the direction.

The composite deck and the open web joist systems will require significant changes to the lateral system. Since the frame type is changing from concrete to steel the lateral system would also need to be redesigned for these systems. With the use of a steel gravity system it would likely to use steel bracing to act as the lateral force resisting system.

The current concrete floor system as well as the alternate concrete system is heavier system then the other two alternate composite steel systems making the building stiffer and able to resist wind forces more easily.

With the composite steel alternate systems the building will decrease in weight of the structure and allow the foundations to intern be decreases. The number of piles per column would be able to be decreases as well as the size of the grade beams.

An advantage of the concrete systems is that they will not need to have fireproofing applied. Both of the composite systems will need to have an applied fireproofing in order for them to reach the required two hour rating that a concrete system has naturally.

Seismic considerations must also be considered. From Tech Report #1 it was determined that seismic loads controlled in the North - South lateral design. If the concrete system is switched out for a composite system the seismic load will be decreased and might lead to the wind load controlling in both directions.

Architectural

Each of the floor systems was designed for a 29' x29' bay but a square bay is more efficient for some systems then other. The existing two-way flat slab is ideal for square bays with the higher load that we are working with. The alternative composite deck and beam is also a good design when spanning a square with normal column layout. For both the open web joist and the one way concrete system the design is capable of spanning the square bays but the systems could be made more efficient by shorting the bays in one direction to make the more rectangular. The girder supporting the open web joists has to support a large deflection force; with a shorter span you would be able to decrease that member size.

The column size of 24" x 24" for the existing system as well as the one-way alternative is a considerable size in comparison to the estimated size of a steel column to support the other two alternative systems. To support alterative systems two and three the column would range from a W14 to a W18 which would give up to 10 inches of space for expansion of walls.

Another significant architectural consideration is the depth of the floor system. Special considerations for this were taken in to account during the design of these alternate systems. Since the New patient tower will be meeting with an existing tower the floor to floor heights need to remain at the current design level. The existing system has a total depth of 15.5 inches under the drop panels with a slab depth of only 9.5". The Composite beam and the one way slab and beam have a total depth of 22" and 18" respectively. Both of these systems leave space for mechanical equipment without increasing the total height of the building but these cavities would be a challenging space for the designers of those systems. The open web joist system has a total depth of 25.8" which occupies all but 2.2" of the ceiling cavity. The advantage to the open web joist system is that the mechanical equipment will be able to run though the web of these members. If special consideration during the design was not taken for the systems the elevations of the new patient tower would not match the levels of the existing tower.

Construction

The easiest of the floor systems to build is the composite metal deck because the deck acts at the formwork that there is no shoring needed unlike conventional concrete systems. The Open web joist has similar conditions to the composite beam and deck with easy construction due no formwork or shoring. The existing concrete two-way slab and the alternative one-way slab with beams will both need to be formed and shored which is a very labor intensive and slow process. Concrete does have a shorter lead time compared to steel and concrete can also be delivered as needed allowing less area for shake out.

Constructability doesn't necessarily equate to lower system cost. The least expensive floor system between the alternates and the existing system is the two-way flat slab at $17/SF. The most easily constructible of the floor systems the composite deck and beam system is at a cost of $19/SF. These calculations can be seen in Appendix VI.

Serviceability

With the need of each floor system to stay with in the 28” ceiling cavity and maintain space for the mechanical system the deflection criteria was something that need to be monitored. With the design of the alternate system as member were designed to support the deflection member tended to grow much larger in weight to compensate for the inability to make the system deeper. The Open web system girders were designed to be W21-101 to support the live load deflection the weight on this girder was need due to the inability for the girder to be a deeper member that would have supported the load more efficiently. The Composite deck system also had to use a specialty design to support the total load on the system. The beams of the composite deck system were designed with .75” of camber to support the total load deflection.

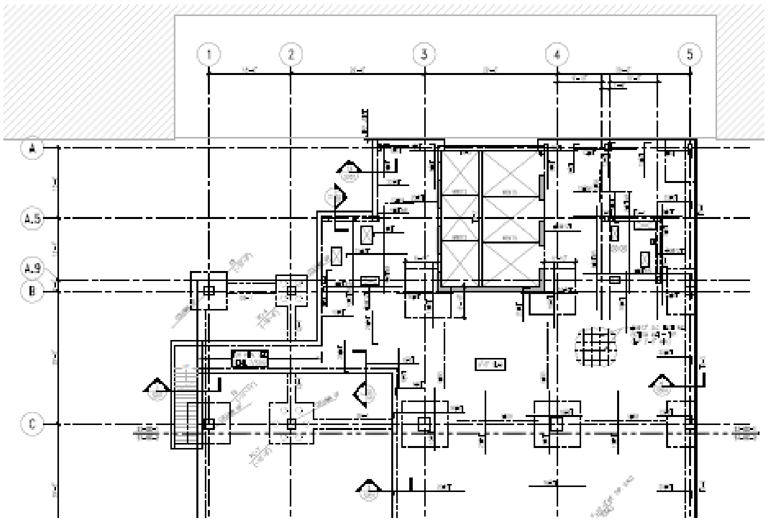
Design Comparison

Floor Systems	Two –way flat slab	Composite Beam	Open web Joist	One-way slab with beams
Lateral Impacts	None	Braced frame in building core with moment connections if needed	Braced frame in building core with moment connections if needed	Existing lateral system design should be sufficient.
Weight	120 psf	62 psf	47 psf	173 psf
Foundation Impacts	None	Lower total building weight	Lower total building weight	Existing piles and footing design is sufficient
Fire Protection	No Fireproofing	Fireproof girders, beams and slab to achieve 2 hour rating	Fireproof joists and girder to achieve 2 hour rating	No Fireproofing
Depth	9.5"	6"	4.8"	12.5"
Total Depth	9.5"-15.5"	22"	25.8"	18"
Floor Plan Impact	None	None	Would a more suitable system for a rectangular bay	Needs short bays to decrease slab thickness
Constructability	Labor intensive formwork with longer time to strength	Faster construction with quick to strength	Faster construction with quick to strength	Labor intensive formwork with longer time to strength
Cost	$17/sf	$19/sf	$20/sf	$18/sf
Deflection	No issues	Beam camper was needed to resist total load deflection	High girder weight to resist total load deflection	No issues
Viable Alternative	Existing	Yes, no need to rearrange the floor plan and will decrease total building weight.	No, System does not yield enough ceiling cavity for the mechanical system	No, there is an increase in concrete and no other benefits compared to the existing system.

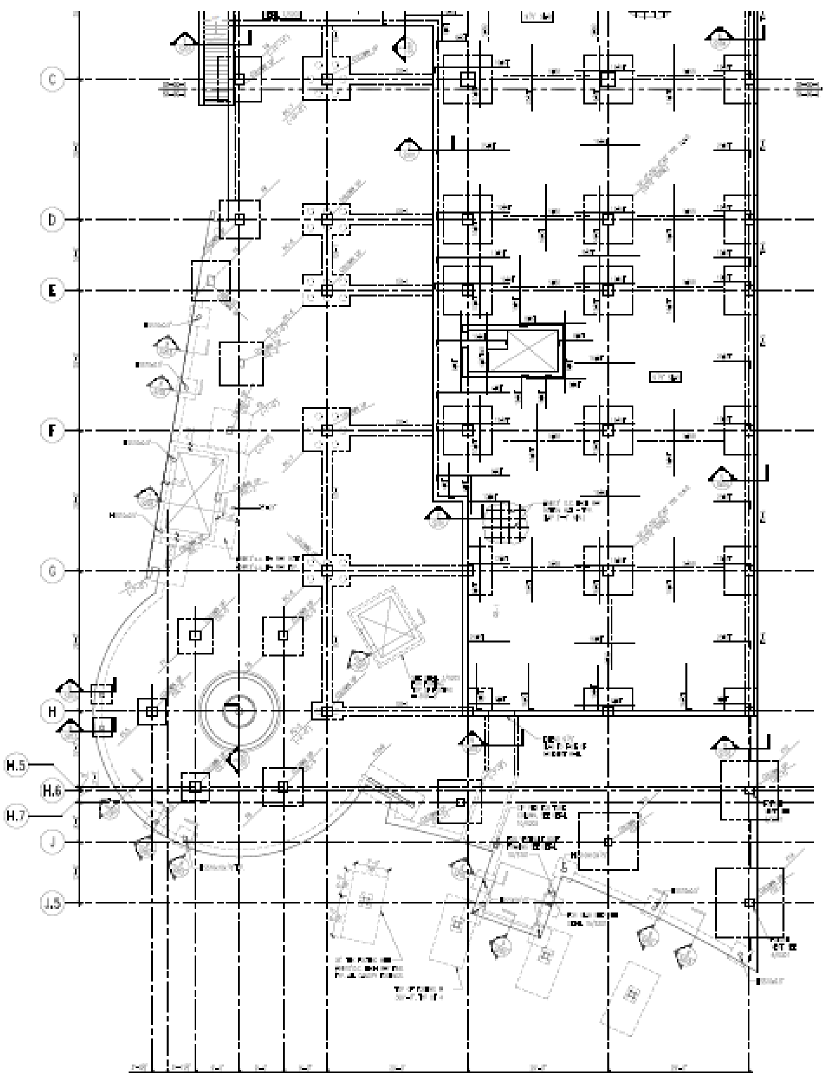
Table 5: System Comparison

Appendix I

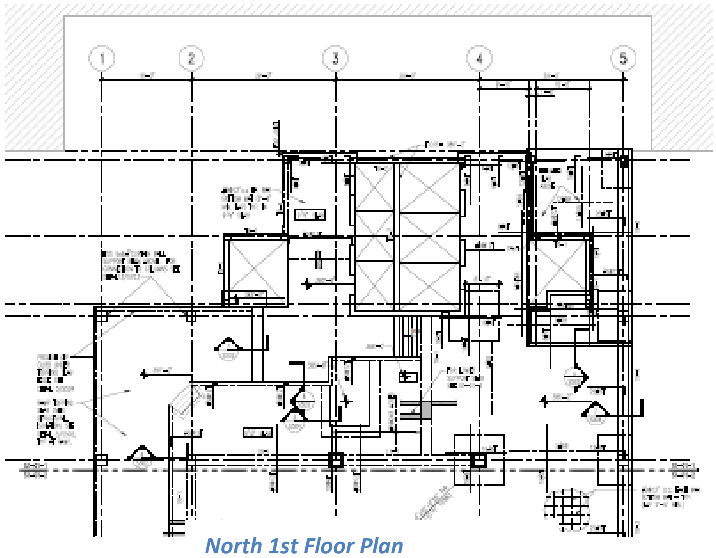
This section of Technical Report #1 is where the supplementary information for the layout and design for the Hospital Patient Tower can be found.



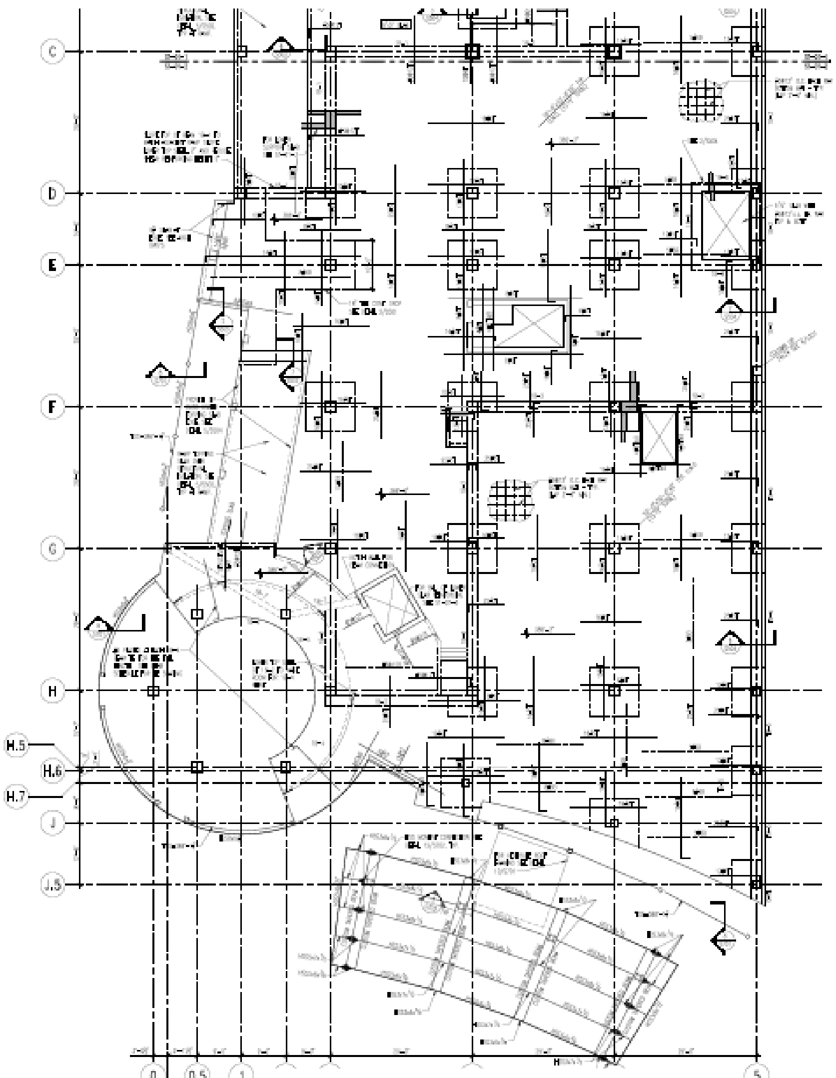
North Ground Floor Plan



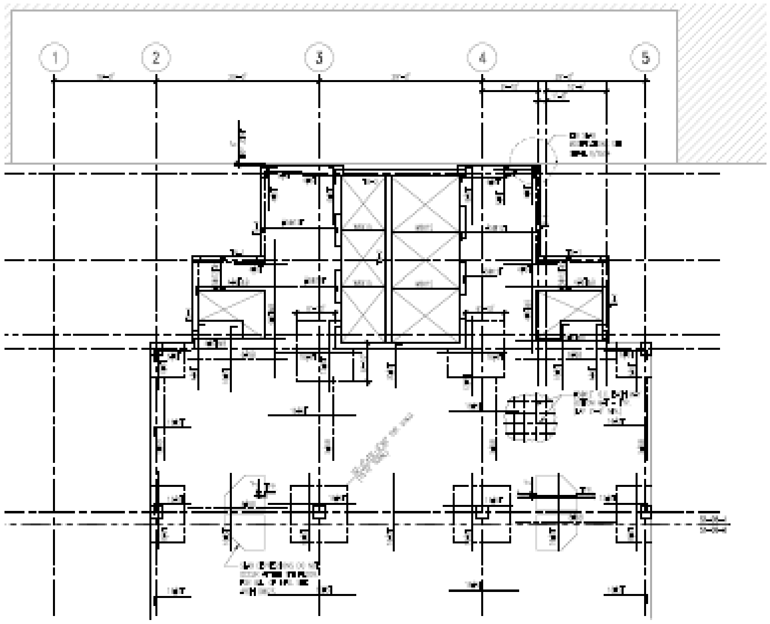
South Ground Floor Plan



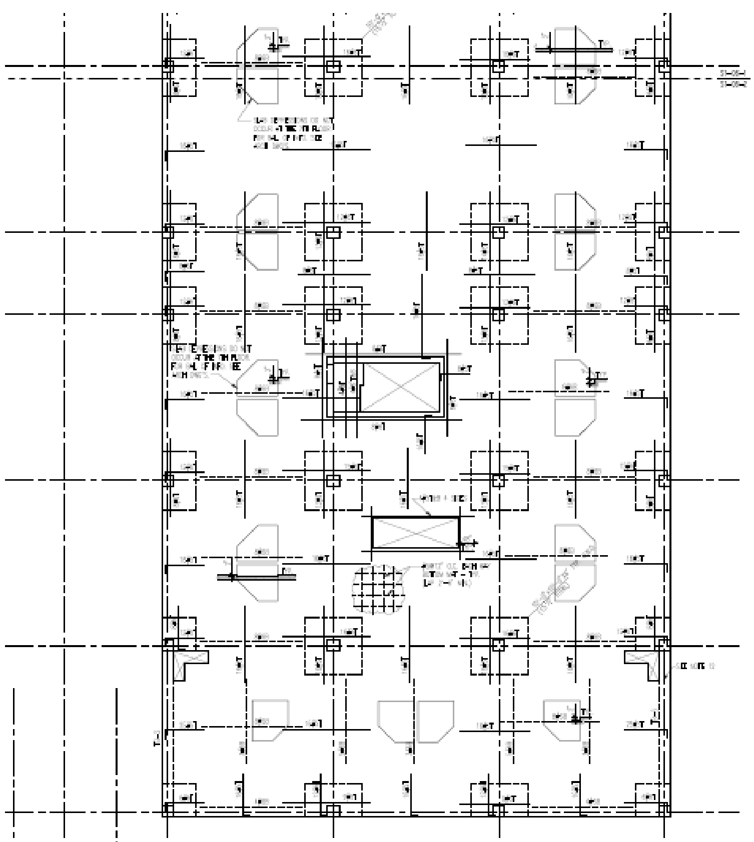
North 1st Floor Plan



South 1st Floor Plan



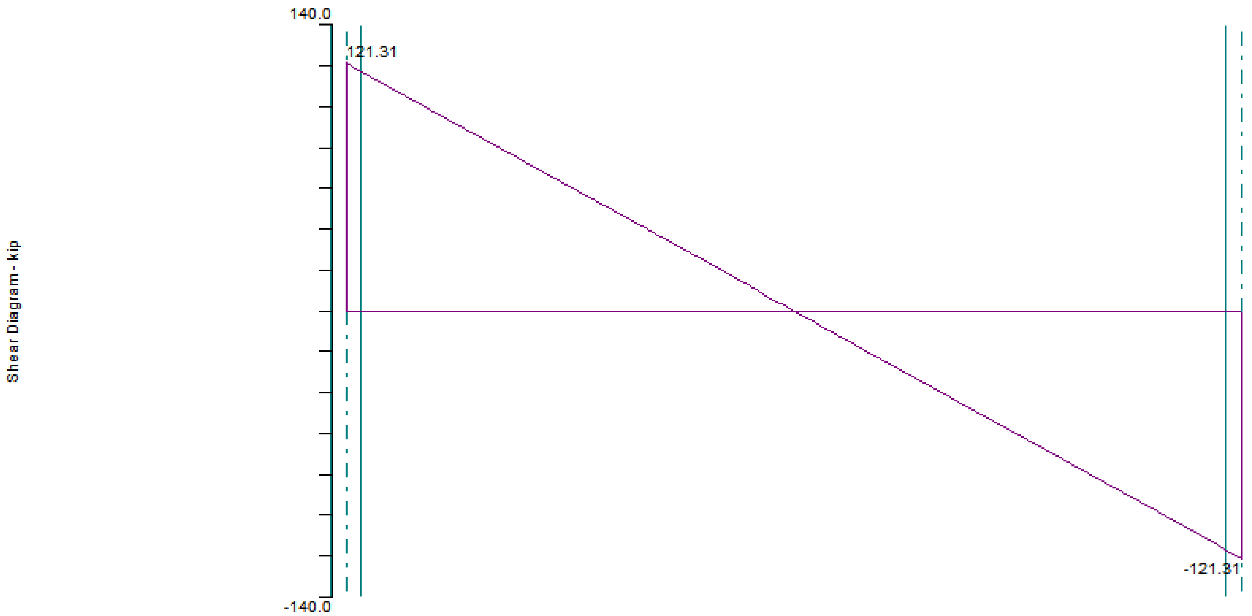
North Typical Floor Plan

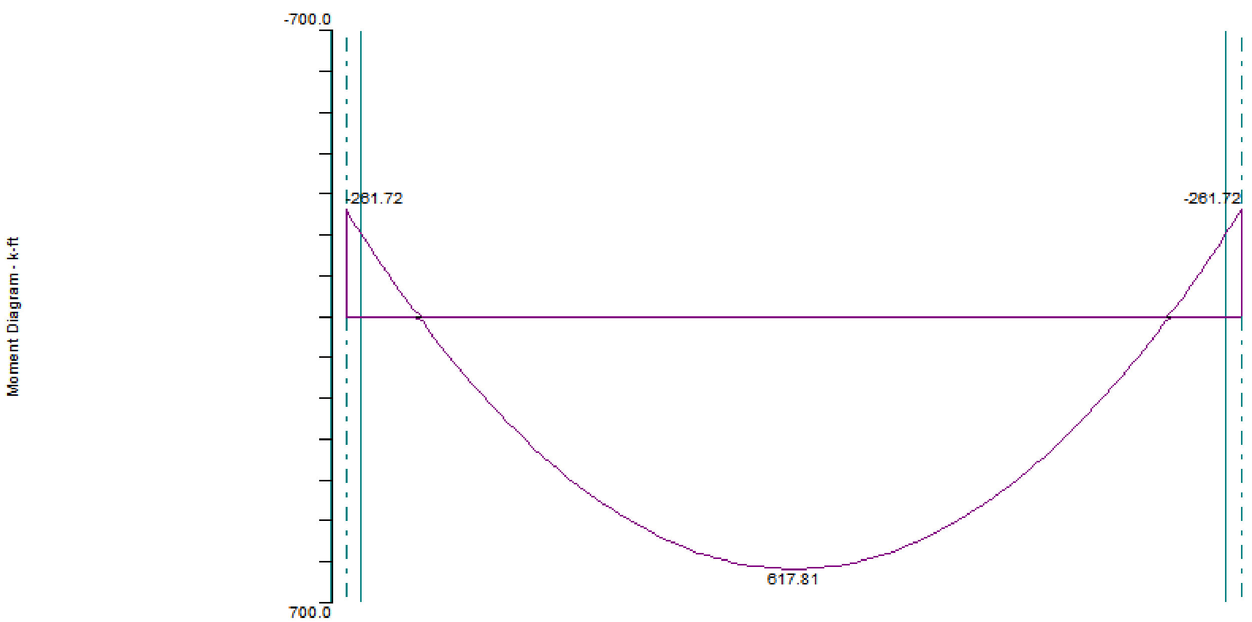


South Typical Floor Plan

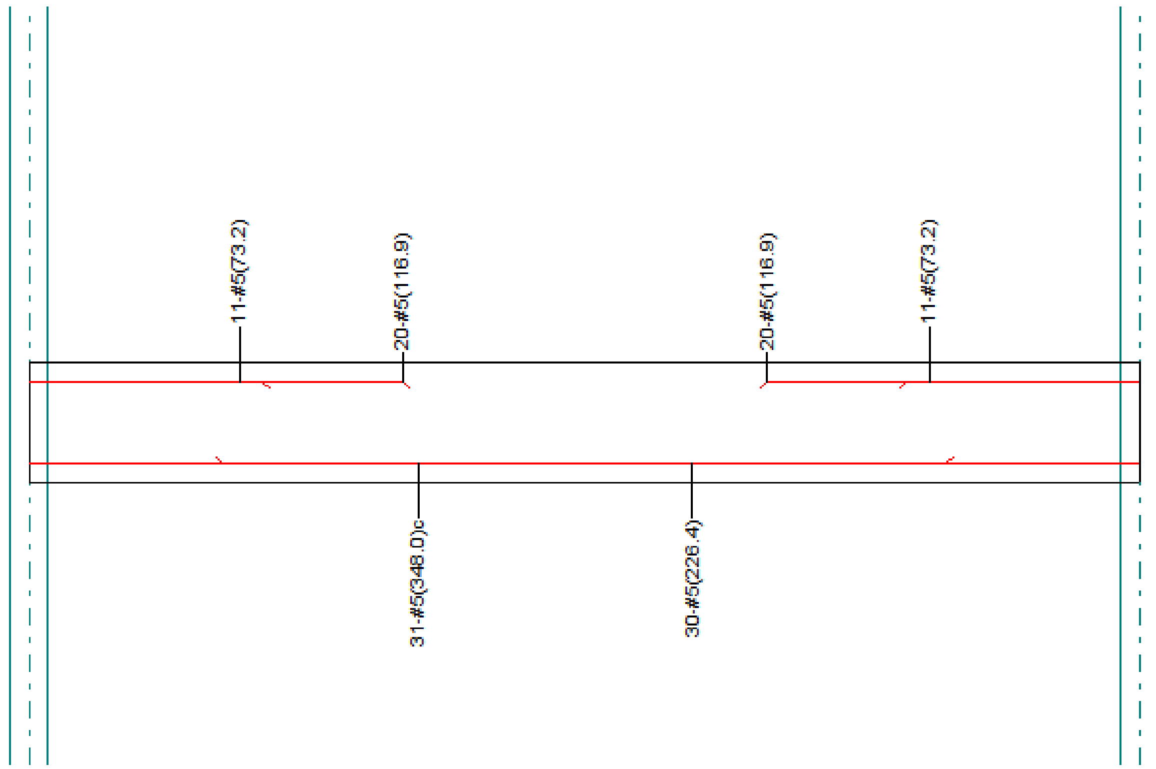
Appendix II

This section of Technical Report #2 is where the supplementary information for analysis of the existing Two-way flat slab designed using SPbeam


Shear Diagram - kip
140.0
121.31
-121.31
-140.0


Moment Diagram - k-ft
-700.0
-261.72
-261.72
617.81
700.0

.


11-#5(73.2)
20-#5(116.9)
20-#5(116.9)
11-#5(73.2)
31-#5(348.0)c
30-#5(226.4)

Appendix III

This section of Technical Report #2 is where the supplementary information for Alternate system #1, Composite Deck with Beam for the Hospital Patient Tower can be found.

Tech Report #2 | System # 1 | Patient Tower | ½

Alternative Floor # 1

W-Flange Beams, with Composite Metal Deck Slab and Normal weight concrete

Typical Floor Slab

- 3" NW Concrete on 3" composite Steel deck

Vulcraft catalog

- 3VLI with 3" topping
- Max Unshored Span > 9'-4"

$A_t = 280.3'$

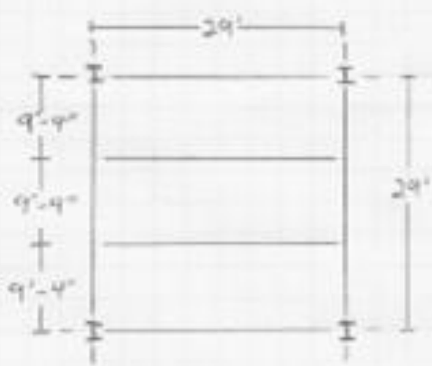
Dead Load = 57 psf (from Vulcraft)

15 psf MEP/SIDL

Total Dead = 72 psf

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

$LL_{red} = 80 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{2 \times 280.3}} = 0.884 \end{array} \right.$ $LL = 71$ psf



$w_u = 1.2\,DL + 1.6\,LL$

$= 1.2(72) + 1.6(71)$

$= 200$ psf

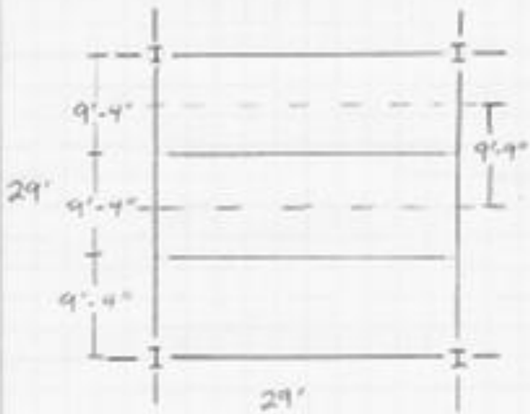
Use 3LI 17 @ 9'-6" span

251 psf > 200 psf ✓ OK

14'-2" > 9'-4" ✓ OK

Tech Report # 2 | System # 1 | Patient Tower | 2/4

Beam Design



Floor DL = 72 psf

Live Load(red) = 71 psf

$w_u = 1.2(72) + 1.6(71) = 200$ psf

$w_u = 200(9.33) = 1.86$ klf

$M_u = \frac{w_u L^2}{8} = \frac{1.86(29)^2}{8}$

$= 195.5$ k·ft

$V_u = \frac{w_u L}{2} = \frac{1.86(29)}{2} = 26.97$

$b_{eff} = \min \begin{cases} \text{Spacing} = 112" \\ \frac{\text{Span}}{4} = 87" \leftarrow \text{controls} \end{cases}$

assume $a = 1$ in

$Y_2 = t_{slab} - \frac{a}{2} = 6 - \frac{1}{2} = 5.5$ in

$Q_n = 17.2$ for 4 ksi NWC w/ Perpendicular Deck

Try W12×26 $\quad \Sigma Q_n = \frac{197^k}{17.2} = 11.5 \Rightarrow 24$ Studs

$\phi M_n = 255$ k·ft

→ W10×26 $\quad \Sigma Q_n = \frac{254^k}{17.2} = 14.8 \Rightarrow 30$ Studs

$\phi M_n = 251$ k·ft

$a = \frac{254}{0.85(4)(87)} = 0.86 \quad Y_2 = 6 - \frac{0.86}{2} = 5.57" > 5.5"$

$\phi M_n = 251$ k-ft $> M_u$ 195.5 k·ft

Tech Report #2	System #1	Patient Tower	3/4

Check Unshored Strength

W 10 × 26 $\phi M_p = 117^{k \cdot ft}$

$W_u = 1.4(57(9.33)+26)) = 0.781^k$

$= 1.2(57(9.33)+26)) + 1.6(20(9.33)) = 0.970^k$

$M_u = \frac{0.970(29)^2}{8} = 102^k < 117^k \therefore$ OK ✓

Check deflection

$W_{LL} = 71(9.33) = 0.662^k$

$I_{LB} = 491 in^4$ (Table 3-20 AISC)

$\Delta_{LL} = \frac{5(0.662)(29)^4\ 1728}{384(29000)(491)} = 0.74'' < 0.466 \therefore$ OK ✓

Max $\Delta_{LL} = \frac{29(12)}{360} = 0.466''$

Check Wet Concrete deflection

$W_{wc} = 57(9.33)+26 = 0.557^{k \cdot ft}$

$\Delta_{wc} = \frac{5(0.557)(29)^4(1728)}{384(29000)(144)} = 2.12 in > 1.45''$ No Good

$\Delta_w = \frac{L}{240} = \frac{29 \times 12}{240} = 1.45 in$ $2.12 - 0.75 = 1.37'' < 1.45'' \therefore$ OK ✓

W10×26 with ¾" Camber is adequate to carry the loads with 30 ¾" ø studs

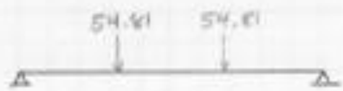
Tech Report # 2	System # 1	Patient Tower	4/4

Girder Design

$W_u = 1.86 + 0.026 = 1.89$ klf

$P_u = 1.89(29) = 54.81$

$M_u = PL/3 = \frac{54.8(29)}{3} = 520$ k-ft



b_{eff} | $\frac{29 \times 12}{4} = 87"$ ⇐ controles
29×12

Assume $a = 1$ $Y_2 = 6 - ½ = 5.5$ in

$Q_n = 21.5$ for 4 ksi NWC w/ deck parellel (table 3-21 AISC)

Try W16×45 $\Sigma Q_n = \frac{365}{21.5} = 16.9 = 34$ studs

$\phi M_n = 547$ (PNA = 4)

Check unshored strength

W16×45 $\phi M_p = 309^{k\text{-}ft}$

$P_u = 1.2(57(29) + 26) + 1.6(20(29)) \times 9.33 = 27.4^k$

$w_u = 1.2(0.045) = 0.054$ kIF

$M_u = w_u l^2/8 + P_u l/3 = 0.054(29)^2/8 + 27.4(29)/3 = 270.54^{k\text{-}ft}$

$M_u < \phi M_p$ ∴ OK ✓

Check Deflection (2 point loads)

$P_{LL} = 80(29)(9.33)/1000 = 21.65^k$

$I_{LB} = 1450\ in^4$ (table 3-20 AISC)

$\Delta_{LL} = \frac{21.65(29)^3(1728)}{28(29000)(1450)} = 0.77" < Max\ \Delta_{LL} = 0.966"$

∴ OK ✓

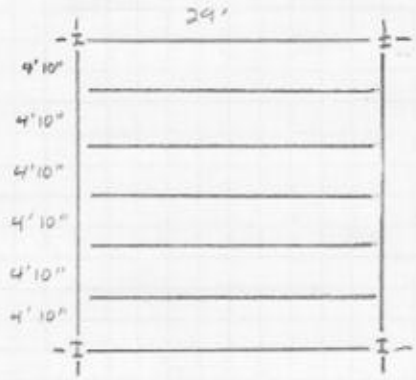
Appendix IV

This section of Technical Report #2 is where the supplementary information for Alternate system #2, Composite Deck with Beam for the Hospital Patient Tower can be found.

Tech Report #2	System #2	Patient Tower	1/3

Alternative floor # 2

Open Web Steel Joist with Non-composite Metal deck Slab + Light weight Concrete



3½" LWC topping for 2 hour fire

Vulcraft deck

1.3C24 4.8" total with 3½" topping

light weigh concrete 110 pcf

Max span 7'-1"

Dead loads = 40 psf (Vulcraft)
= 15 psf SIDL

total = 55 psf

Live load = 60 + 20 psf partions = 80 psf Unreducable $A_t < 400 ft^2$

total load = 135 psf

F_b = 36,000 = 157 psf > 135 ∴ ok ✓

DEFL = 1/240 = 92 psf > 80 psf ∴ OK ✓

Construction Span max = 7'-1" > 4'-10" ∴ OK ✓

Tech Report | System #2 | Patient Tower | 7/3

Open Web Steel Joist

$w_{u,ll} = 1.2(55) + 1.6(80) = 194_{psf}\,(4.833') = 938\ ^{lb}/_{ft}$ + Self Weight

From Vulcraft

Standard LRFD Longspan Steel Joists LH-Series

20LH06

Depth 20" Weigh 15 $^{lbs}/_{ft}$

$938\ ^{lb}/_{ft} + 1.6(15) = 962\ ^{lb}/_{ft} < 1018\ ^{lb}/_{ft}\ \therefore$ OK ✓

Check Deflection

$w_{LL} = 80_{psf}\,(4.833') = 387\ ^{lb}/_{ft} < 427\ ^{lb}/_{ft}$ (Vulcraft table)

$\therefore$ OK ✓

Tech Report #2 | System #2 | Patient Tower

Girder Design

$A_t = 29' \times 29' = 841$

assume distributed

$W_{LL} = 1.2(55) + 1.2(16) + 1.6(80)$
$= 212 \text{ psf } (29')$
$= 6.1 \text{ KIF}$

$b_{eff} = 87"$ (from previous)

29' Span

$M_u = W_u L^2 / 8$

$= \frac{6.1(29)^2}{8}$

$= 641.3 \text{ k}\cdot\text{ft}$

From table 3-2 AISC

Try W18x 86 for moment capacity + Maintaining floor to floor height

M_u w/ Girder weight $= 650.3 \text{ k}\cdot\text{ft}$

$\phi M_p = 698 > 650.3 \text{ k}\cdot\text{ft} \therefore$ ok ✓

$L_b = 4.83' < 9.29' \therefore$ ok ✓

Check Live Load deflection

$W_{LL} = 1.6(80)(29') = 3.7 \text{ klf}$

$\Delta_{LL} = \frac{5 w_{LL} L^4 (1728)}{384 EI} \leq \frac{L}{360}$

$I = \frac{675 W_{LL} L^3}{29000} = 2100.4 \text{ in}^4$

W18x119 needed to support Δ_{LL} $I = 2190 \text{ in}^4$

Check Total Load deflection

$\Delta_{TL} \leq L/240$

$W_{TL} = 6.1 \text{ klf}$

$I_{req} = \frac{450 W_{TL} L^3}{E} = 2308.5$

W21x101 needed to support Δ_{TL} $I = 2420 \text{ in}^4$

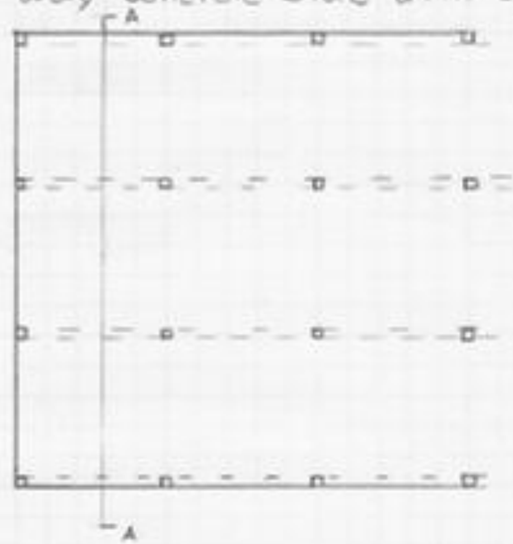
Appendix V

This section of Technical Report #2 is where the supplementary information for Alternate system #3, One-way concrete slab and beam system for the Hospital Patient Tower can be found.

Tech Report #2 | System #3 | Patient Tower

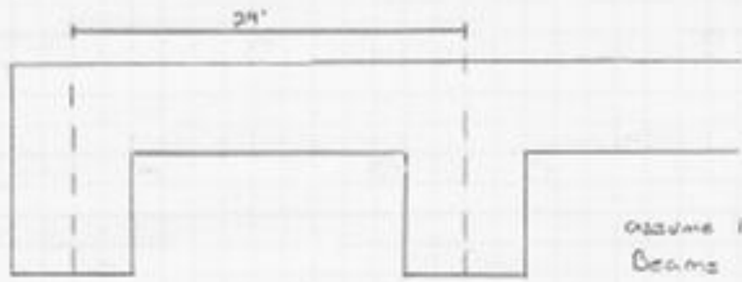
Alternative floor #

One Way Concrete Slab with Beams



$f'_c = 5000$ psi

$f_y = 60{,}000$ psi



assume 12" wide Beams

Minimum thickness (h) of nonprestressed one-way slab continuous

$l/28$ from (ACI 9.5(a))

$l = 29'$ $\quad 29 \times 12/28 = 12.4" \approx 12.5"$

Loads

Dead = 150 pcf $(12.5/12)$ = 156.25 psf

Live = 80 psf

SIDL = 15 psf

$w_u = 1.2(172) + 1.6(80) = 334.4$ psf

Tech Report #2 | System #3 | Patient Tower

Factored Moment

$w_u L^2 / 11$ $\quad L_n = 29' - 2' = 27' \quad w_u = 0.334\text{ksf}(1') = 0.334\text{klf}$

$\frac{0.334(27)^2}{11} = 22.1 \text{ k·ft}$

$A_s \geq \frac{M_u}{\phi f_y (jd)} \quad jd = 0.95d \quad d = 12.5" - \underset{\text{cover}}{1"} = 11.5"$

$\frac{22.1(12)}{0.9(60)(10.93)} = 0.449 \text{ in}^2/\text{ft}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.45(60)}{0.85(5)(12)} = 0.529 \text{ in}$

$A_s \geq \frac{M_u}{\phi f_y (d - a/2)} = \frac{22.1(12)}{0.9 \times 60(12.5 - 0.265)} = 0.40 \text{ in}^2/\text{ft}$

$\rho = \frac{A_s/\text{ft}}{bd} \quad \frac{0.40}{(12)(12.5)} \quad 0.0026$

Check whether thickness is adequate for Shear

$V_u = \frac{1.15 w_u L}{2} = \frac{1.15(0.334)(27)}{2} = 5.18 \text{ k per 1' width}$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2 \cdot 1 \cdot \sqrt{5000}\,(12)(12.5) = 21.2\text{k per 1' width}$

$\lambda = 1$ for NWC

$\phi V_c > V_u = 0.75(21.2) > 5.18$ so 12.5" slab is OK for Shear

$A_s = 0.4 \text{ in}^2/\text{ft} \qquad$ #6@12"

Tech Report #2 | System #3 | Patient Tower

Check Reinforcing for Crack Control

$$S = 15\left(\frac{40{,}000}{f_s}\right) - 2.5\,C_c \leq 12\left(\frac{40000}{f_s}\right)$$

$f_s = \frac{2}{3} f_y$

$= 40000$

$C_c = 1''$

$= 12.5 \leq 12$

$S = 12'' \therefore$ OK

Determine the Shrinkage + temp reinforcement

$A_s(S+T) = 0.0018(b)(h) = 0.0018 \times (12)(12.5) = 0.27\ \text{in}^2/\text{ft}$

Max Spacing $\begin{cases} \leq 5 \times h = 62.5'' \\ \leq 18'' \end{cases}$ ← controls

$A_s = \#6 @ 12'' \therefore$ OK ✓

Tech Report #2 | System #3 | Patient Tower

Beam design

$$h = \ell/21 = \frac{29\times 12}{21} = 16.5'' \approx 18'' \quad (\text{ACI } 9.5a)$$

$$b_w = 0.75\,h = 13.5''$$

$$w_{web} = \frac{(18-12.5)\times 13.5}{144} \times 150\ \text{lb/ft}^3 = 77.3\ \text{plf}$$

$$w_{slab} + \text{SDL} = \left(\tfrac{12.5}{12}(150) + 15\right) = \frac{171.25 \times 29'}{1000} = 4.97\ \text{klf}$$

$$w_u = 1.2(4.97) + 1.6(80(29)/1000) = 8.65\ \text{k/ft}$$

$$b_{eff} = \begin{cases} b_w + 16h_f = 13.5 + 16(12.5) = 213.5'' \\ b_w + 2(\tfrac{1}{2}\text{ clear span}) = 13.5 + (29\times 12) = 361.5'' \\ \tfrac{1}{4}\text{ span length} = \tfrac{1}{4}(29\times 12) = 87'' \end{cases}$$

$$b_{eff} = 87''$$

$$M_u = w_u L^2/11 = \frac{8.65\,(27)^2}{11} = 573.3\ \text{k-ft}$$

$$A_s = M_u / \phi f_y (jd) \qquad jd = 0.9(18) = 16.2$$

$$d = 18 - \underset{\text{cover}}{1''} - \underset{\text{bar}}{0.5''} = 16.5''$$

$$= \frac{573.3\times 12}{0.9(60)(16.2)} = 7.9\ \text{in}^2$$

Check $C = T$ assumption

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{7.9(60)}{0.85(5)(24)} = 4.64 \qquad c = \frac{4.64}{0.8} = 5.8$$

$$\epsilon_t = \frac{0.003}{5.8}(16.2 - 5.8) = 0.0057 \geq 0.005$$

Appendix VI

This section of Technical Report #2 is where the supplementary information for the floor system cost analysis for the Hospital Patient Tower can be found.

These table values were taken from RSMeans Assemblies Cost Data manual 2011. The location factor for Virginia is 0.92 to be multiplied against the average data given.

System	Locations Factor	Material Cost ($/SF)	Installations Cost ($/SF)	Total Cost ($/SF)
Two-way Slab	0.92	7.60	10.15	16.33
Composite Beam	0.92	13.70	6.65	18.72
Open Web Joists	0.92	13.90	7.40*	19.6
One-way Slab & Beam	0.92	6.95	12.70	18.1

*Material cost for Open web joist does not take in to account the increase weight of the girder to support the total load deflection as discussed above. This would only add to the cost of the system making it the most expensive by a larger margin