Franklin Square Hospital Center Patient Tower

Baltimore, MD

Technical Report 2

Pro-Con Structural Study of Alternate Floor Systems

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Structural Option

AE 481W Senior Thesis

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Table of Contents:

Executive Summary:

The intent of this report is to study the existing floor system and three alternative floor systems for the Franklin Square Hospital Center Patient Tower in Baltimore, MD. The existing floor system is a 10" flat plate system. See Figure 1, "Typical Structural Floor Plan."

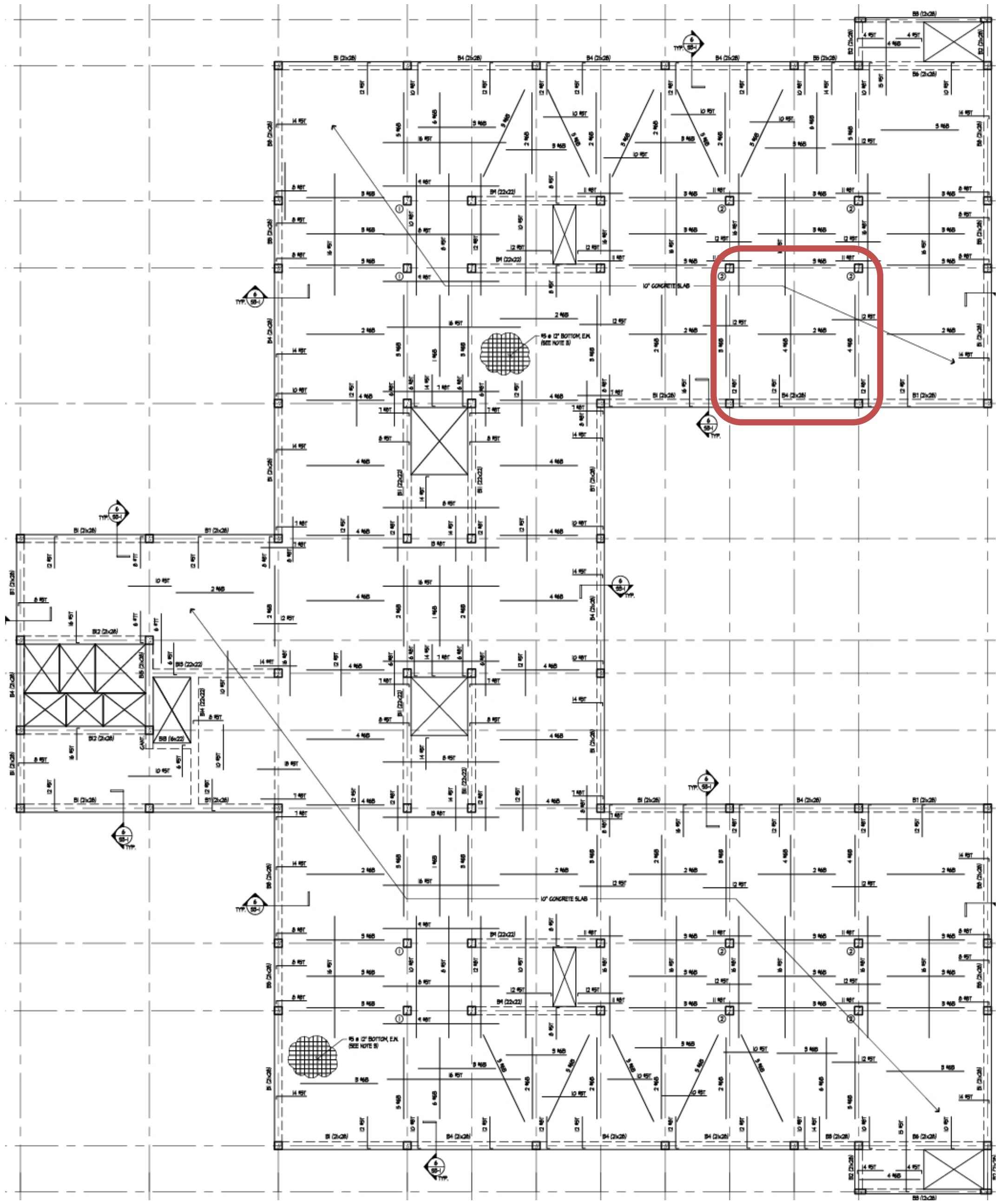


Figure 1: Typical Structural Floor Plan

The area in Figure 1 circled in red represents a typical bay. The bay is 30'x30' with adjacent bays of 30'x30' and 30'x15.' The existing floor system will be discussed and checked, along with three alternative floor systems listed below.

- Alternative Floor System 1: Composite Deck on Composite Beam
- Alternative Floor System 2: Composite Joist
- Alternative Floor System 3: Two-Way Post-Tensioned Slab

Each floor gravity system was designed based upon preliminary calculations of stresses, moment, shear, and deflection requirements along with common rules of thumb. Table 1 below is a quick summary of this report's findings.

Table 1: Comparison of Floor Systems Summary

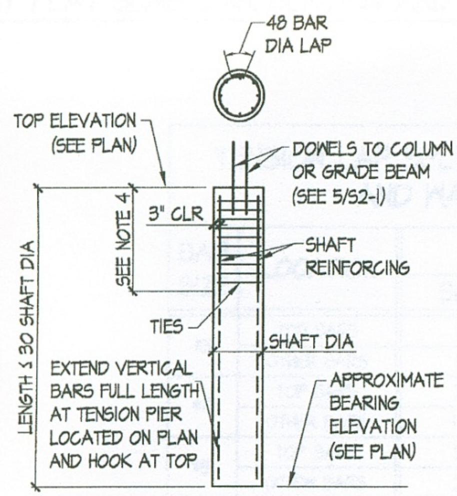
Floor System	**Existing**	**Alternative 1** (Composite Beam)	**Alternative 2** (Composite Joist)	**Alternative 3** (Two-Way Post Tensioned Slab)
Slab Depth	10"	5.25"	5.25"	9"
Total Depth	10"	23.25"	23.25"	9"
Estimated Cost	\$15.53 / ft^2	\$22.97 / ft^2	\$23.96 / ft^2	\$17.86 / ft^2
System Self Weight	121 PSF	49 PSF	48 PSF	109 PSF
Lead Time	Short	Long	Long	Short
Fireproofing	Built-In	Spray-On	Spray-On	Built-In
Vibration Concerns	Minimal	Moderate	Moderate	Minimal
Viable Option	Yes	No	No	Yes

From the above research and comparisons, it was determined that while the Composite Deck on Composite Beam and Composite Joist systems were appealing at first due to their simplicity and ease of construction, they are not appropriate for use in the Franklin Square Hospital Patient Tower. The two systems that will be further developed and reviewed are the existing Flat Plate and the proposed Two-Way Post-Tensioned Slab.

Structural Systems

Foundation System

The foundation system of the Franklin Square Hospital Patient Tower consists of drilled piers or caissons 4 feet in diameter and centered under columns or slightly offset under perimeter grade beams. The piers range in size from 1.5 feet in diameter to 5 feet in diameter. They are embedded a minimum of 20 feet into bedrock. The total typical depth of the piers is around 42 feet below grade pending geotechnical engineer inspection. See Figure 2, "Drilled Pier Reinforcing."



SHAFT DIAMETER	REINFORCEMENT	
	VERTICAL	TIES
1'-6"	6 #5	#3 @ 10"
2'-0"	6 #6	#3 @ 12"
2'-6"	6 #7	#3 @ 14"
3'-0"	7 #8	#3 @ 16"
3'-6"	7 #9	#4 @ 18"
4'-0"	7 #10	#4 @ 18"
4'-6"	9 #10	#4 @ 18"
5'-0"	9 #11	#4 @ 18"

NOTES:
1) SEE PLANS FOR DIAMETER OF SHAFT.
2) SHAFT AXIS SHALL NOT BE OUT OF PLUMB MORE THAN 3" IN 10'-0".
3) THE BOTTOM BEARING SHALL BE LEVEL AND ALL DISTURBED MATERIAL REMOVED.
4) EMBED ≥ 3x SHAFT DIAMETER (20'-0" MINIMUM).

Figure 2: Drilled Pier Reinforcing

The piers are required to be a normal weight concrete with a concrete compressive strength (f'_c) of 3000 psi. As previously mention, the piers directly support interior columns. See Figure 3, "Column Caisson Connection and Column Reinforcing."

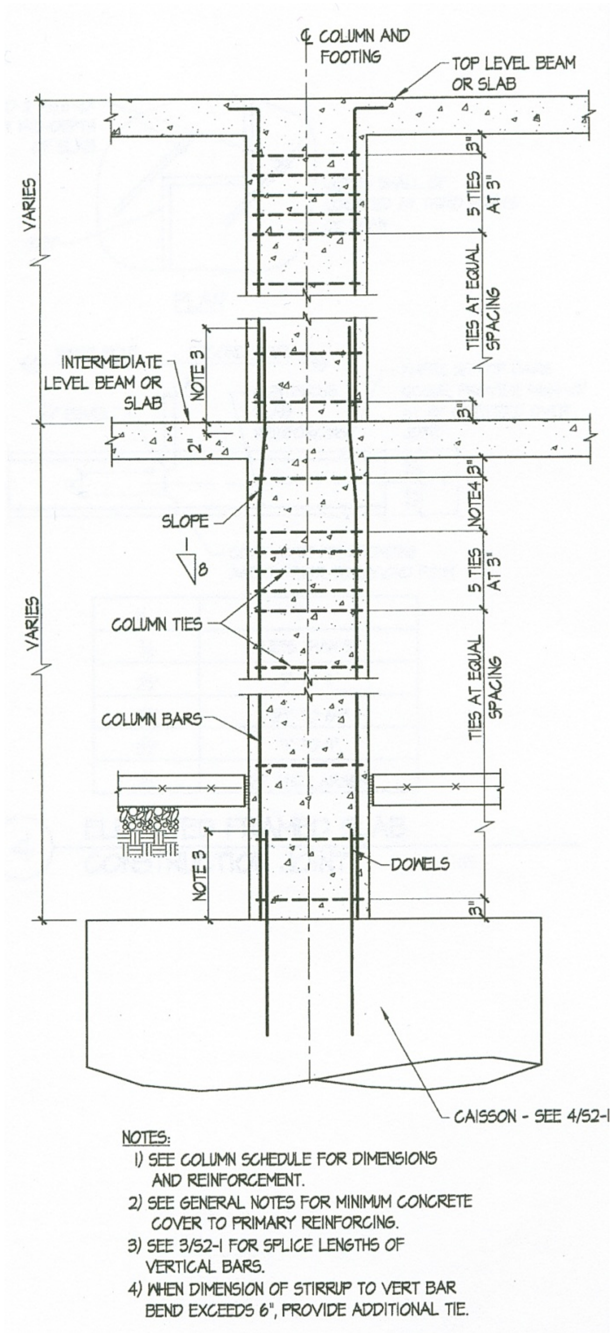


Figure 3: Typical Column Caisson Connection and Column Reinforcing

The piers also directly support perimeter grade beams. The typical grade beam is 24”x24” with some that are 36”x24”. See Figure 4, “Typical Grade Beam Caisson Connection.”

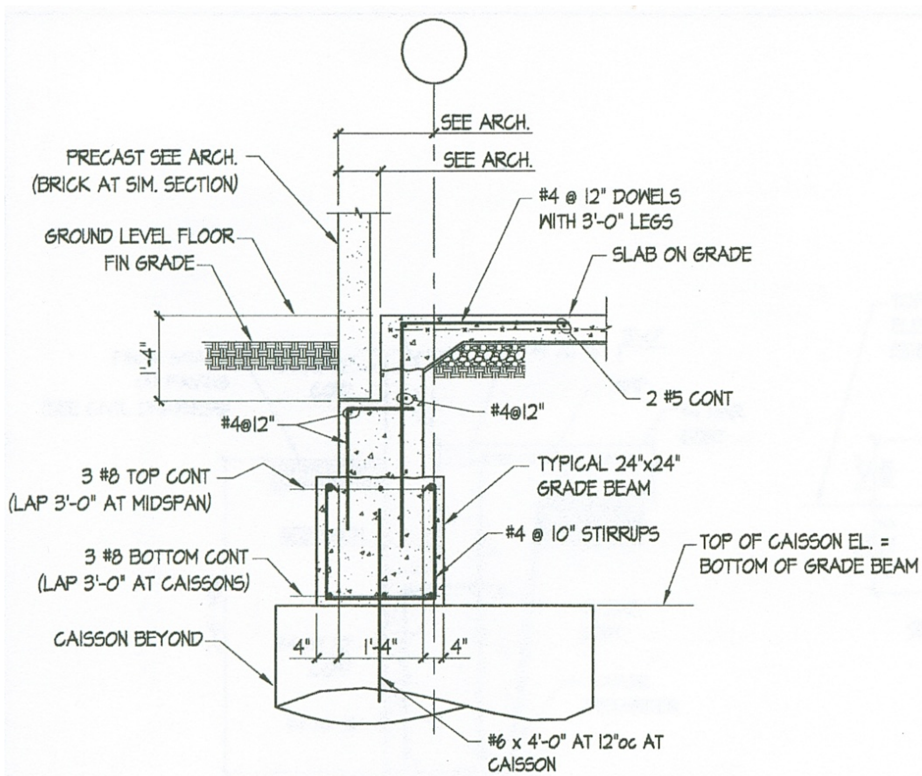


Figure 4: Typical Grade Beam Caisson Connection

While there are no sub grade levels in the structure, the west side of the ground floor can be considered below grade because the ground has been filled to provide on grade access to the first floor lobby. The existing hospital ground floor also resides on the level corresponding to the patient tower’s first floor. Lateral soil pressures from the foundation of the existing building are resisted by a 16” thick foundation wall in these areas. See Figure 5, “Typical Foundation Wall Section.”

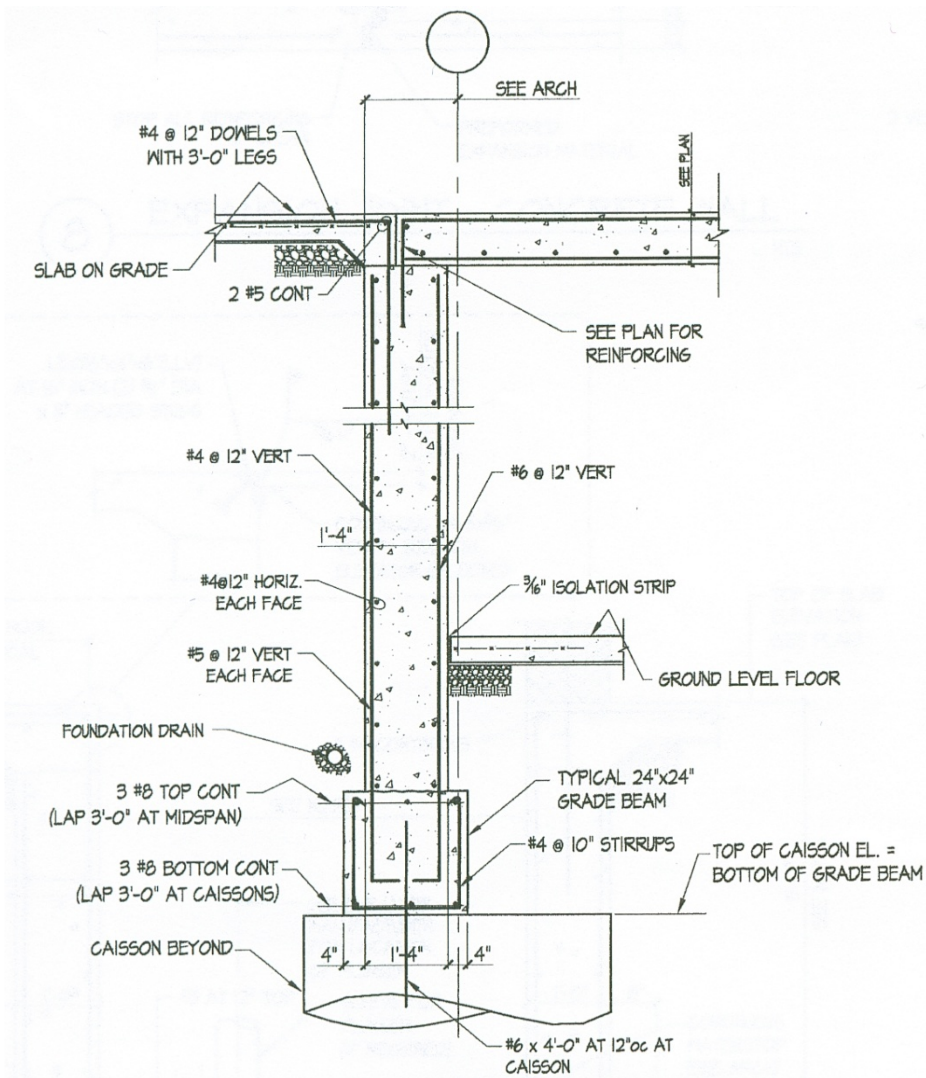


Figure 5: Typical Foundation Wall Section

The rest of the foundation consists of a 5 inch ground floor slab on grade of compressive strength equal to 3000 psi. The slab on grade is reinforced with 6x6-W2.9xW2.9 welded wire fabric over a 4 inch layer of clean, well-graded gravel or crushed stone.

Floor System

The building's typical floor system is a 10" reinforced two way slab, or flat plate, spanning a typical 30'x30' bay. The reinforcing varies a great deal depending on location and span but for the most part there is a continuous bottom mat of #5 or #6 bars at 12" each way with continuous top reinforcing within the column strips with mostly #6 or #8 bars. See Figure 6, "Slab Reinforcing Detail."

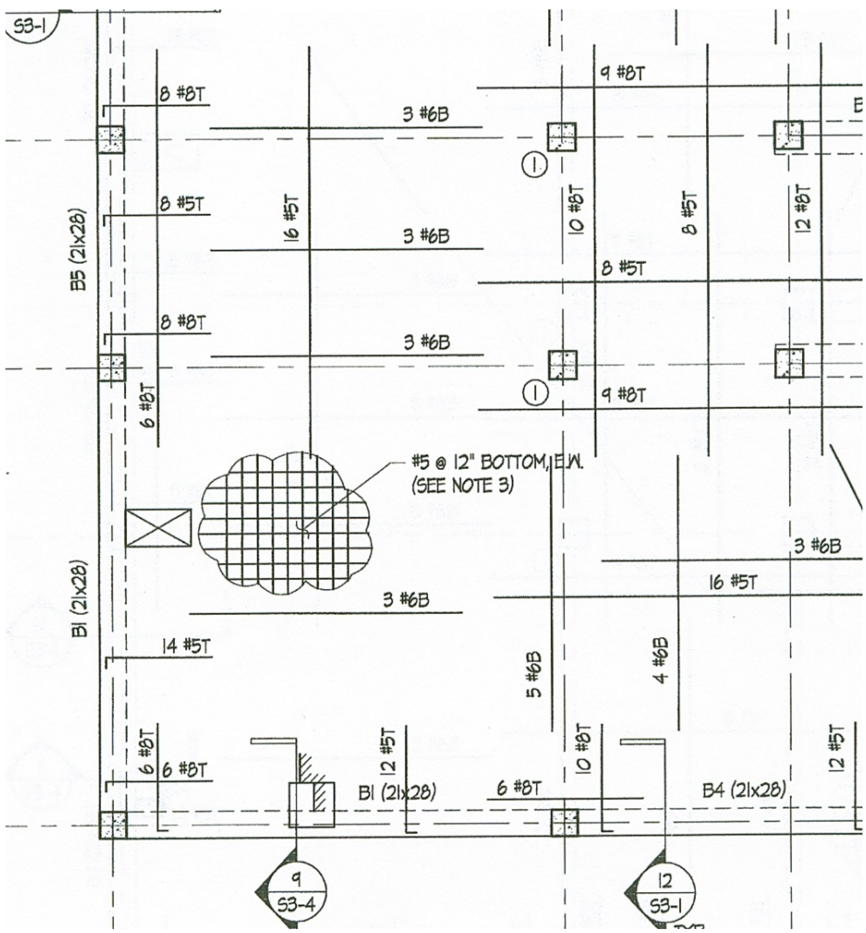


Figure 6: Slab Reinforcing Detail

The floor system also consists of edge beams that wrap the perimeter of the slab and surround openings such as stairs, elevators, and mechanical shafts. The typical edge beam is 21"x28" reinforced with #9 bars top and bottom. See Figure 7, "Portion of Concrete Beam Schedule."

CONCRETE BEAM SCHEDULE

MARK	SIZE		REINFORCING				STIRRUPS				REMARKS
	W (INCHES)	D (INCHES)	BOTTOM BARS	TOP BARS LE	TOP BARS FL	TOP BARS RE	SIZE	TYPE	SPACING (INCHES)	END	
B1	21	28	3#9	-	2#9	-	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B2	12	28	3 #9	-	3#9	-	#4	S2	1@2, R@10	EE	
B3	10	28	3 #8	-	3#8	-	#4	S2	1@2, R@12	EE	
B4	26	20	3 #9	-	3#9	-	#4	S3	1@2, R@8 CANT. 1@2, R@8	EE	
B5	21	28	2#9	-	2#9	-	#4	S2	1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B6	21	28	4#9	-	3#9	-	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B7	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 18@8, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B8	21	28	3#9	-	2#9	3#9	#4	S2	1@2, 16@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B9	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B10	22	20	4#9	5#10	2#10	5#10	#4	S3	1@2, 12@4, R@6	EE	
B11	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B12	21	28	3#9	2#9	2#9	2#9	#4	S2	1@2, 14@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B13	26	20	5#9	5#9	-	7#10	#4	S3	1@2, 12@4, R@8	EE	
B14	20	20	3#9	6#9	-	6#9	#4	S3	1@2, R@6	EE	
B15	12	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 6@8, R@12 CANT. 1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B16	20	20	2#9	-	2#9	-	#4	S2	1@2, 6@8, R@12	EE	
B17	12	20	2#9	3#9	-	3#9	#4	S2	1@2, 16@6, R@12	EE	
B18	22	24	4#9	1#9	2#9	1#9	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B19	22	24	4#9	-	2#9	-	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B20	22	24	3#9	-	2#9	-	#4	S2	1@2, 5@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B21	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B22	21	28	5#9	-	2#9	-	#4	S2	1@2, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B23	21	16	2#9	-	2#9	1#9	#4	S2	1@2, 16@6, R@12	EE	
B24	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, R@12	EE	
B25	30	28	3#9	4#9	4#9	-	#4	S3	1@2, 12@12, R@18 CANT. 1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B26	21	28	5#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B27	21	28	3#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B28	21	28	2#9	-	2#9	2#9	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B29	21	28	5#9	1#9	2#9	1#9	#4	S2	1@2, 12@6, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B30	21	28	3#9	5#9	2#9	-	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B31	21	28	3#9	-	2#9	5#9	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B32	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B33	26	22	2#9	3#9	-	3#9	#3	S2	1@2, R@6	EE	

Figure 7: Portion of Concrete Beam Schedule

Columns

The columns are for the most part 21"x21" and 22"x22 with (8) #9 bars. Instead of changing column sizes as the building rises, the engineers specified different concrete compressive strengths for different levels and reduced the reinforcing to (8) #8's in spots. The ground to 3rd floor columns have a 28 day compressive strength of 7000 psi and the columns from the 3rd floor to the roof have a 28 day compressive strength of 5000 psi.

Portions of the penthouse are supported by steel columns. For continuity and moment resisting strength, these steel columns are embedded in the full length of the concrete columns from the floor below. This results in steel columns that are 2 levels tall and fully integrated in the moment frame of the rest of the building.

The portion of the tower that does not rise past the ground floor has oversized columns designed for future expansion. The Franklin Square Hospital Center Patient Tower was realized because the existing hospital had no capacity left for additional floors. Desperately needing space, the hospital commissioned the Patient Tower and supporting spaces. In the future when such a situation arises, the new Patient tower will be able to grow with the needs of the hospital. See Figure 3, "Typical Column Caisson Connection and Column Reinforcing" and see Figure 8, "Portion of Concrete Column Schedule."

LEVEL	COLUMN	L-1 M3-1 P-1	K-2 L-2 K-12.4 L-12.4	J-7, J-8 K-7, K-8 L-7, L-8	M-3 N-3	M-6 M-7 M-8 M-9	M-4, M-5 M-10, M-11 N-4, N-5	N-12 P-6	N-6 N-7, N-8 N-9, N-10 N-11	P-3 P-4 P-5	M-12	J-9, L-6 K-9, L-9 H-6, J-6 K-6	F-4, F-5 F-6, F-10 F-11	G-4, G-5 G-6, G-10 G-11
PENTHOUSE ROOF	SIZE													
	VERTICAL BARS													
	TIES													
	REMARKS													
MAIN ROOF/ SEVENTH FLOOR	SIZE		30x12											
	VERTICAL BARS		6#8											
	TIES													
	REMARKS													
SIXTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FIFTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FOURTH FLOOR	SIZE		30x12	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
THIRD FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
SECOND FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
FIRST FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
GROUND FLOOR	SIZE	21x21	30x12	22x22	22x22	22x22	22x22	21x21	21x21	21x21	21x21	21x21	22x22	22x22
	VERTICAL BARS	12#10	6#10	8#10	8#10	8#9	8#10	8#11	8#11	8#11	8#10	8#9	8#9	8#9
	TIES	4#8		4#8	4#8	4#8	4#8	4#8	4#8	4#8	4#8			
	REMARKS													
DOWELS		12#7	6#7	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#7	8#7	8#7

Figure 8: Portion of Concrete Column Schedule

Roof System

The main roof system consists of cambered steel beams ranging from W12x14 to W21x73 and 1.5" deep, wide rib, 20 gauge galvanized metal deck with 3 ¼" lightweight concrete. Many of these beams are moment connected to the steel columns supporting them. A center portion of the roof contains a 10" reinforced concrete slab with concrete columns extending 2' above the surface for future placement of the helipad deck.

Wall System

The exterior façade is for the most part 7" precast concrete panels. Loads bearing connections occur at each level, with two per panel. The connections permit horizontal movement parallel to the panel except for a single non-load bearing connection which is fixed. Precast panel loads are supported only by the columns.

Lateral System

The Franklin Square Hospital Center Patient Tower utilizes the entire structure to resist lateral forces. Every column, slab and beam acts as an ordinary reinforced concrete moment frame resisting forces in both the North-South direction and the East-West direction. The large moments are carried down the building through the columns and directly into the drilled piers. The piers, with depths of 42 feet, are quite substantial and help greatly to give the building a rigid, fixed base.

In the case of wind, the force exerted on the precast panels is directly transferred to the columns and not the floor diaphragm. Once this occurs, the force is carried down the column and across the floor diaphragm to the remaining columns. The columns are expected to resist the lateral force through their moment capacity. The perimeter edge beams are stiffer than the diaphragm and are therefore expected to function as more efficient moment frames.

Codes and Design Standards

General Codes and Standards

- "International Building Code 2006", International Code Council with Baltimore County Amendments
- "Minimum Design Loads for Buildings and Other Structures, ASCE 7-05", American Society of Civil Engineers

Concrete

- "Building Code Requirements for Reinforced Concrete, ACI 318", American Concrete Institute
- "ACI Manual of Concrete Practice – Parts 1 through 5"
- "Manual of Standard Practice", Concrete Reinforcing Steel Institute
- "PCI Design Handbook – Precast and Prestressed Concrete", Prestressed Concrete Institute

Structural Steel

- "Manual of Steel Construction – Allowable Stress Design", Ninth Edition
- "Manual of Steel construction – Load and resistance Factor Design", Third Edition
- "Manual of Steel Construction, Volume II Connection", ASD 9th Edition/LRFD 3rd Edition
- "Detailing for Steel construction", American Institute of Steel Construction
- "Structural Welding Code ANSI/AWS D1.1, American Welding Society

Steel Deck

- "Design Manual Floor Decks and Roof Decks", Steel Deck Institute

Material Specification

Concrete

Application	f'c @ 28 days	Weight (PCF)
Slabs-On-Grade (Interior)	3000	145
Slabs-On-Grade (Exterior)	3500	145
Reinforced Slabs	5000	145
Reinforced Beams	5000	145
Fill on Metal Deck	4000	110
Columns (Ground to 3rd Floor)	7000	145
Columns (3rd Floor to Roof)	5000	145
Walls	4000	145
Grade Beams	3000	145
Footings	3000	145
Caissons	3000	145
Topping	3000	145

Structural Steel

Application	
Deformed Reinforcing Bars	ASTM A615, Grade 60
Rolled Shapes	ASTM A992, Grade 50
Channels, Angles and Plates	ASTM A36
Structural Pipe	ASTM A53, Grade B, F_y = 35 ksi
Round HSS Shapes	ASTM A500, Grade B, F_y = 42 ksi
Structural Tubing (Square and Rectangular HSS)	ASTM A500, Grade B, F_y = 46 ksi
High Strength Bolts	ASTM A325-N typical
Anchor Rods	ASTM F1554 Grade 36
Smooth & Threaded Rod	ASTM A36
Headed Shear Studs	ASTM A108
Welding Electrodes	AWS A5.1 OR A5.5, E70XX
Galvanized Metal Deck	ASTM A653
Painted Phosphated Metal Floor Deck	ASTM A611

Gravity Live and Dead Loads

Live Loads (LL)		
Area	**ASCE 7-05 Load**	**Design Load**
Patient Rooms	40 PSF	40 PSF
Lobbies and 1st Floor Corridors	100 PSF	100 PSF
Corridors above 1st Floor	80 PSF	80 PSF
Stairs and Exits	100 PSF	100 PSF
Mechanical	-	As Noted On Plans
Partitions	20 PSF	20 PSF
Roof	20 PSF	30 PSF Minimum (Snow Load is used when greater than 30 PSF)

Dead Loads (DL)		
Material	**ASCE 7-05 Load**	**Design Load**
Superimposed	-	20 PSF
Normal Weight Concrete	-	145 PCF
Lightweight Concrete		110 PCF
Concrete on Metal Deck	-	63 PSF
Precast Façade	-	85 PSF
Curtain Wall	-	3 PSF

Existing Floor System

Flat Plate

The buildings typical floor system, as detailed in Figure 9, "Flat Plate Floor System Design", is a 10" reinforced two way slab, or flat plate, spanning a typical 30'x30' bay. The reinforcing varies a great deal depending on location and span but for the most part there is a continuous bottom mat of #5 or #6 bars at 12" each way with continuous top reinforcing within the column strips with mostly #6 or #8 bars. The floor system also consists of edge beams that wrap the perimeter of the slab and surround openings such as stairs, elevators, and mechanical shafts. The typical edge beam is 21"x28" reinforced with #9 bars top and bottom. Although the perimeter beams are part of the floor system, their main purpose is in resisting lateral loads.

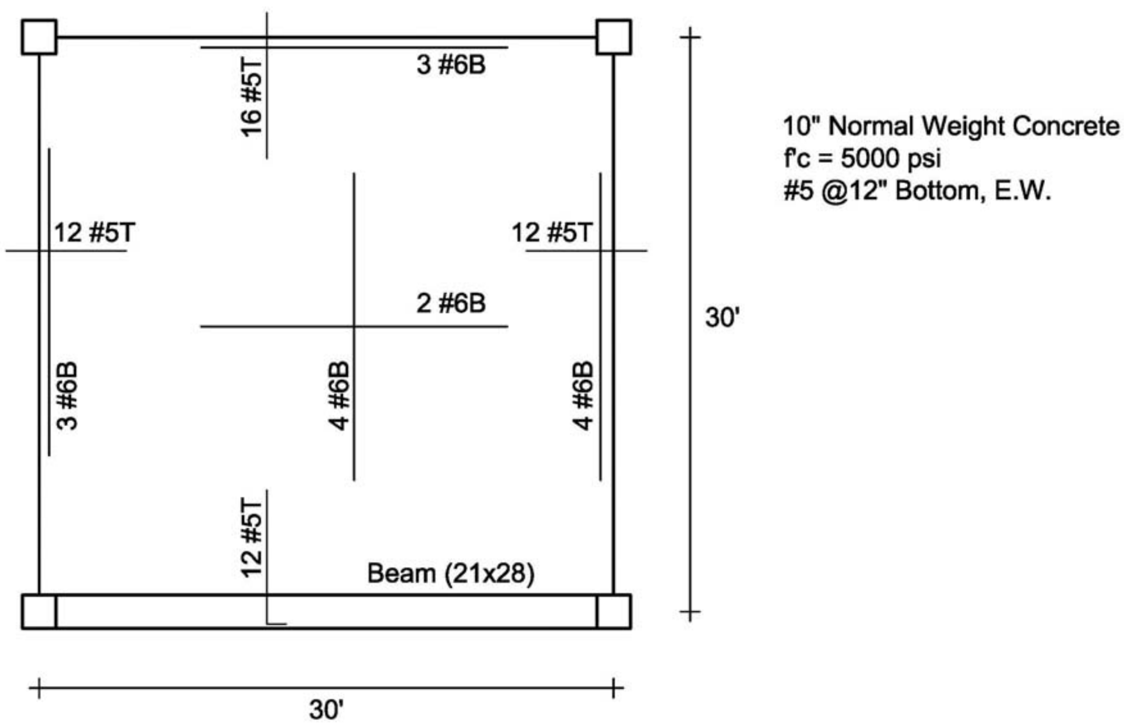


Figure 9: Flat Plate Floor System Design

Advantages:

- Vibration and acoustic control through mass of slab
- Fire protection is inherent providing an adequate fire protection rating of 2 hours
- Material availability is quite good

Disadvantages:

- Formwork and shoring is required in slab construction thereby lengthening construction time

Design Considerations:

Structural:
Deflection and vibration calculations have been omitted for the floor system due to its complexity. However, the designer most likely met the criteria for live load deflection of L/360, which in this case is ~1". However, this somewhat thick slab incurs large weight penalties which drive the seismic loading up.

Construction:
Construction companies in the DC/Baltimore area are very experienced with flat plate construction therefore this system should not be of any concern to construction companies in the area.

Architectural:
Due to the lack of drop panels, there are nice flat ceilings to work with for mechanical, lighting, and ceiling system installation.

Alternative Floor Systems

Composite Deck on Composite Beam

The first alternative floor system proposed is a composite deck on composite beam system. This system has advantages over more common non-composite beam and deck floor systems with fewer intermediate beams and smaller system depths. Composite action of the steel deck and lightweight concrete slab allows the deck to span further than in conventional systems permitting fewer beams while composite action of the steel beams and the lightweight concrete, through the use of shear studs, allows smaller steel members to be used and limits deflection. Figure 10, "Composite Deck on Composite Beam Floor System Design," shows the proposed composite deck on composite beam design of a typical bay. Decking is 18 gage 2" Lok-Floor from United Steel Deck, Inc. with 3 ¼" 4000 psi lightweight concrete. This deck and slab combination easily spans the 10' beam spacing as seen in Figure 11, "2" Lok-Floor Metal Deck". Completing the floor system, W12x22 beams with 28 shear studs and 1" of camber are used while the girders are W18x55's with 32 shear studs and 1" of camber. With this system the column framing will need to be changed to steel and minor changes will be needed to the column layout. See Appendix A for hand calculations.

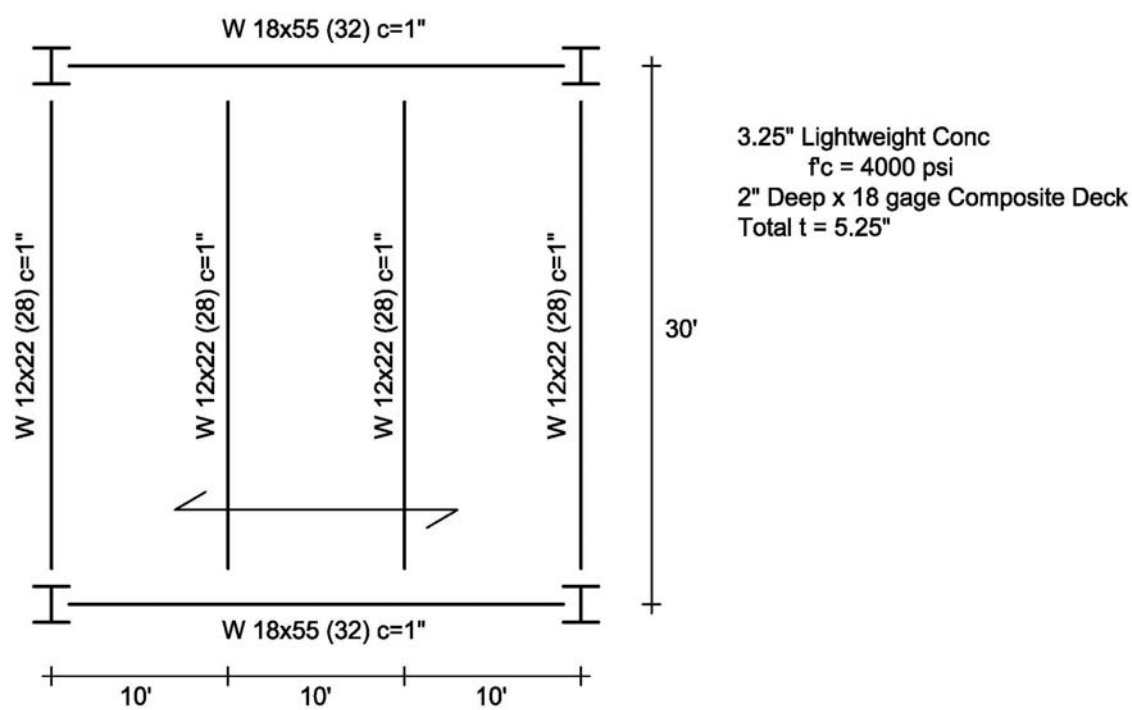


Figure 10: Composite Deck on Composite Beam Floor System Design

COMPOSITE PROPERTIES													
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max Unshored Span, ft. 1 span	2 span	3 span	A_{wwf} in²/ft
22 gage	4.50	48.06	32.6	0.292	34	0.99	4.4	34.01	4440	6.60	8.81	8.96	0.023
	5.00	55.54	37.5	0.333	38	1.17	6.0	40.07	4780	6.29	8.43	8.56	0.027
	5.25	59.28	40.0	0.354	41	1.26	6.9	43.18	4960	6.16	8.25	8.39	0.029
	5.50	63.02	42.6	0.375	43	1.36	7.8	46.33	5140	6.03	8.08	8.22	0.032
	6.00	70.50	48.0	0.417	48	1.55	10.1	52.76	5510	5.80	7.78	7.92	0.036
	6.25	74.24	50.8	0.438	50	1.64	11.3	56.01	5710	5.69	7.64	7.78	0.038
	6.50	77.98	53.6	0.458	53	1.74	12.7	59.30	5900	5.59	7.51	7.65	0.041
	7.00	85.46	59.5	0.500	58	1.94	15.7	65.93	6320	5.41	7.26	7.41	0.045
20 gage	4.50	57.78	32.6	0.292	34	1.19	4.8	40.93	4560	7.86	10.12	10.46	0.023
	5.00	66.96	37.5	0.333	38	1.41	6.5	48.24	5240	7.48	9.68	10.01	0.027
	5.25	71.55	40.0	0.354	41	1.52	7.4	52.00	5590	7.31	9.48	9.80	0.029
	5.50	76.14	42.6	0.375	43	1.63	8.5	55.81	5910	7.15	9.29	9.60	0.032
	6.00	85.32	48.0	0.417	48	1.86	10.9	63.60	6280	6.87	8.95	9.25	0.036
	6.25	89.91	50.8	0.438	50	1.98	12.2	67.55	6480	6.74	8.79	9.08	0.038
	6.50	94.50	53.6	0.458	53	2.10	13.6	71.53	6670	6.62	8.64	8.92	0.041
	7.00	103.68	59.5	0.500	58	2.34	16.9	79.60	7090	6.39	8.35	8.63	0.045
19 gage	4.50	66.15	32.6	0.292	34	1.37	5.1	46.87	4560	8.98	11.21	11.58	0.023
	5.00	76.86	37.5	0.333	38	1.62	6.9	55.29	5240	8.53	10.72	11.08	0.027
	5.25	82.21	40.0	0.354	41	1.74	7.9	59.63	5590	8.33	10.50	10.85	0.029
	5.50	87.57	42.6	0.375	43	1.87	9.0	64.03	5950	8.15	10.29	10.64	0.032
	6.00	98.28	48.0	0.417	48	2.14	11.5	73.03	6700	7.81	9.91	10.24	0.036
	6.25	103.63	50.8	0.438	50	2.27	12.9	77.60	6960	7.66	9.74	10.06	0.038
	6.50	108.99	53.6	0.458	53	2.41	14.5	82.21	7150	7.52	9.57	9.89	0.041
	7.00	119.70	59.5	0.500	58	2.69	17.9	91.55	7570	7.26	9.26	9.57	0.045
18 gage	4.50	73.29	32.6	0.292	34	1.52	5.4	52.11	4560	9.82	11.95	12.35	0.023
	5.00	85.36	37.5	0.333	38	1.79	7.2	61.48	5240	9.32	11.43	11.82	0.027
	5.25	91.39	40.0	0.354	41	1.94	8.3	66.32	5590	9.10	11.20	11.57	0.029
	5.50	97.43	42.6	0.375	43	2.08	9.4	71.24	5950	8.90	10.98	11.35	0.032
	6.00	109.50	48.0	0.417	48	2.38	12.1	81.29	6700	8.53	10.57	10.93	0.036
	6.25	115.53	50.8	0.438	50	2.53	13.6	86.41	7090	8.36	10.39	10.73	0.038
	6.50	121.57	53.6	0.458	53	2.68	15.2	91.57	7490	8.21	10.21	10.55	0.041
	7.00	133.64	59.5	0.500	58	2.99	18.7	102.02	8020	7.92	9.88	10.21	0.045

Figure 11: 2" Lok-Floor Metal Deck

<u>Advantages:</u>

- Span lengths can be increased beyond 30' if needed
- Less mass therefore reduced building weight and seismic loads
- Formwork and shoring are not necessary making it easier and faster to assemble

<u>Disadvantages:</u>

- Increased structural floor depth ~ 23 ¼"
- Vibration will be an issue with lighter floor system
- Requires fire proofing (typically spray-on), which requires additional labor and cost
- Fabrication of steel members requires lead time

Design Considerations:

Structural:

Vibration analysis of this system is complex in nature and therefore was not assessed under the scope of this report. The existing column layout would still be feasible but minor changes would be necessary. The moment frame lateral system of the building will need to be investigated for feasibility with the composite beam floor system and might need a change to either shear wall or brace frame although neither is ideal given the architectural requirements. The seismic loads for the Franklin Square Hospital Center Patient Tower are higher than similarly sized buildings in the area due to the enormous self weight of the existing slabs. Any reduction in floor system weight will dramatically reduce seismic loading.

Construction:

Composite steel construction is quick to construct, however a proper amount of lead time must be determined for the fabrication of the beams and girders. Additionally, fireproofing would need to be added during construction preventing other trades from working in the same area at the same time.

Architectural:

The outward appearance of the building would be for the most part similar except the overall height of the building would rise over 7 ½ feet. Given that the Franklin Square Hospital Patient Tower has already received a variance to exceed the height limitation of 50 feet, the increase in height of 7 ½ feet over the current 106 feet would likely not matter much. Inside, there would not be much change as only a few column locations would need changing.

Composite Joist

The second alternate floor system proposed is a composite joist system with metal deck and lightweight concrete. The top chord of the joist is connected in shear to the deck and concrete slab through the use of shear studs providing composite action. This system works in a similar manner to the composite beam system but allows more of the mechanical systems to run through the joists instead of below steel beams, helping to reduce overall depth of the floor/ceiling system. Steel Joist Institute provided design aids and design examples that were followed and used in the design of this system. Figure 12, “Composite Joist Floor System Design”, shows the proposed composite joist design of a typical bay. CJ-Series composite joists spaced 5 feet on center with a depth of 12” and camber of 1.73” were utilized along with 40 ½” diameter shear studs. See Figure 13, “SJI Design Guide LRFD Light Weight Tables”. Composite deck was utilized once again along with 3 ¼” 4000 psi concrete. The girders supporting the joists are W18x50’s with 30 shear studs and a camber of ¾”. Once again, with this system the column framing will need to be changed to steel and minor changes will be needed to the column layout. See Appendix B for hand calculations.

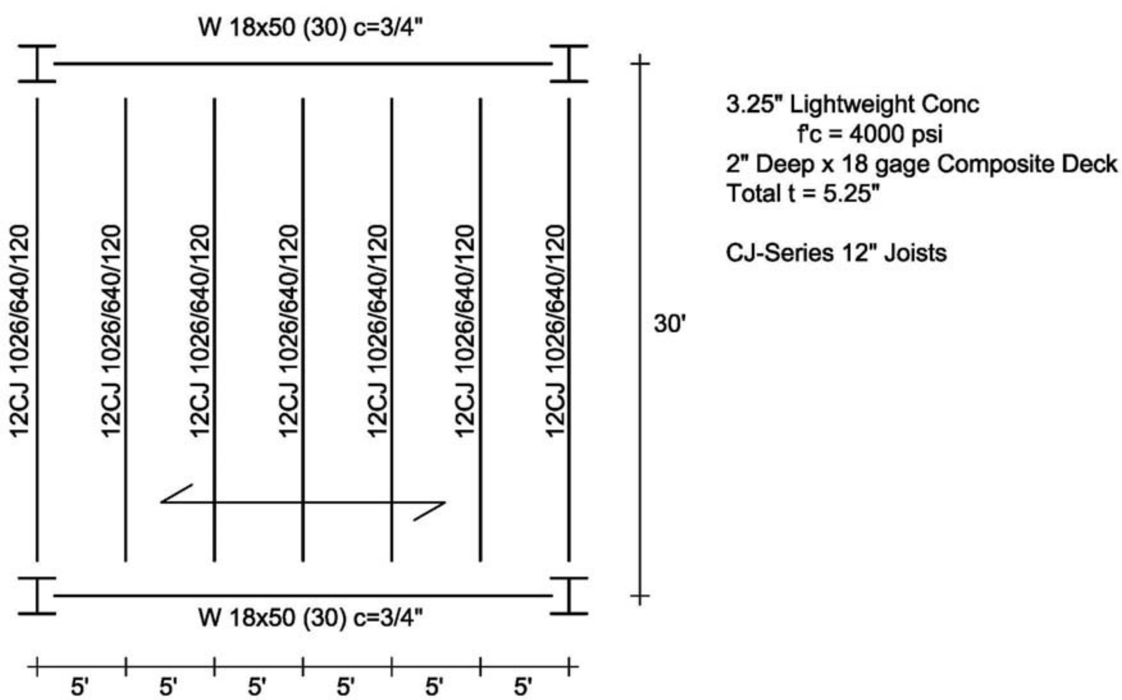


Figure 12: Composite Joist Floor system Design

LIGHT WEIGHT CONCRETE

DESIGN GUIDE LRFD LIGHT WEIGHT TABLE FOR COMPOSITE STEEL JOISTS, CJ-SERIES

	Based on a 50 ksi Maximum Yield Strength										
	BEARING HEIGHT		2 1/2"	5"	7 1/2"						
		Concrete Slab Parameters									
		Light Weight Concrete (110 pcf) f'c = 4.0 ksi									
		hr (in.)	1	1	1	1	1	1	1	1	1
		tc (in.)	2	2	2	2	2	2	2	2	2
		Js (ft.)	3	3	3	3	3	3	3	3.5	4
Joist Span (ft.)	Joist Depth (in.)	Total Safe Factored Uniformly Distributed Joist Load in Pounds Per Linear Foot									
		TL	300	400	500	600	700	800	900	1000	1200
	12	Wt(plf)	6.1	7.2	7.9	8.7	10.1	11.4	13.5	14.0	15.5
		W360(plf)	148	194	218	249	291	316	359	390	444
		N-ds	18-3/8"	24-3/8"	28-3/8"	32-3/8"	40-3/8"	46-3/8"	32-1/2"	34-1/2"	40-1/2"
		Ieff(in4)	93	122	137	157	183	198	226	245	279
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(3)H	(2)H	(2)H	(2)H	(2)H
	14	Wt(plf)	6.2	6.9	7.7	8.4	10.5	11.3	12.3	13.1	15.4
		W360(plf)	175	218	253	284	334	371	387	433	525
		N-ds	16-3/8"	22-3/8"	26-3/8"	30-3/8"	22-1/2"	24-1/2"	26-1/2"	30-1/2"	36-1/2"
		Ieff(in4)	110	137	159	179	210	233	243	272	330
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H	(2)H	(2)H
	16	Wt(plf)	6.0	6.8	7.4	8.4	9.9	11.2	12.5	13.1	16.0
		W360(plf)	190	242	287	311	360	406	461	492	590
		N-ds	16-3/8"	20-3/8"	24-3/8"	26-3/8"	18-1/2"	22-1/2"	24-1/2"	26-1/2"	22-5/8"
		Ieff(in4)	120	152	180	195	226	255	290	309	370
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H

Figure 13: SJI Design Guide LRFD Light Weight Tables

Advantages:

- Electrical and some mechanical systems can run through joist openings
- Span lengths can be increased beyond 30’ if needed
- Less mass therefore reduced building weight and seismic loads
- Formwork and shoring are not necessary making it easier and faster to assemble

Disadvantages:

- Increased structural floor depth ~ 23 ¼”
- Vibration will be an issue with lighter floor system
- Requires fire proofing (typically spray-on), which requires additional labor and cost
- Fabrication of joist and steel members requires lead time

Design Considerations:

Structural:

Vibration analysis of this system is complex in nature and therefore was not assessed under the scope of this report. The existing column layout would still be feasible but minor changes would be necessary. The moment frame lateral system of the building will need to be investigated for feasibility with the composite joist and girder floor system and might need a change to either shear wall or brace frame although neither is ideal given the architectural requirements. The seismic loads for the Franklin Square Hospital Center Patient Tower are higher than similarly sized buildings in the area due to the enormous self weight of the existing slabs. Any reduction in floor system weight will dramatically reduce seismic loading.

Construction:

Composite joist and girder construction is quick to construct, however a proper amount of lead time must be determined for the fabrication of the joists and girders. Additionally, fireproofing would need to be added during construction preventing other trades from working in the same area at the same time.

Architectural:

The outward appearance of the building would be for the most part similar except the overall height of the building would rise over 7 ½ feet. Given that the Franklin Square Hospital Patient Tower has already received a variance to exceed the height limitation of 50 feet, the increase in height of 7 ½ feet over the current 106 feet would likely not matter much. Inside, there would not be much change as only a few column locations would need changing.

Two-Way Post-Tensioned Slab

The third and final alternate floor system proposed is a Two-Way Post-Tension Slab. With concrete lacking tensile strength, post-tensioned slabs provide pre-compression to the concrete to reduce tensile stresses that result from flexure. Figure 14, “Two-Way Post-Tensioned Floor System Design”, shows the proposed post-tensioned design of a typical bay. With a preliminary design completed, it appears a 9” slab will be required with 28 ½” diameter tendons spaced uniformly in the N-S direction and 28 ½” diameter tendons banded into the column strips in the E-W direction. Additional reinforcement is needed with 16 #4 top bars over interior supports, 13 #4 top bars at exterior supports, 13 #4 top bars in the middle 15’ span, and #8 bars at 12” o.c. in the bottom of end spans. With further development an 8” slab could be feasible. See Appendix C for hand calculations.

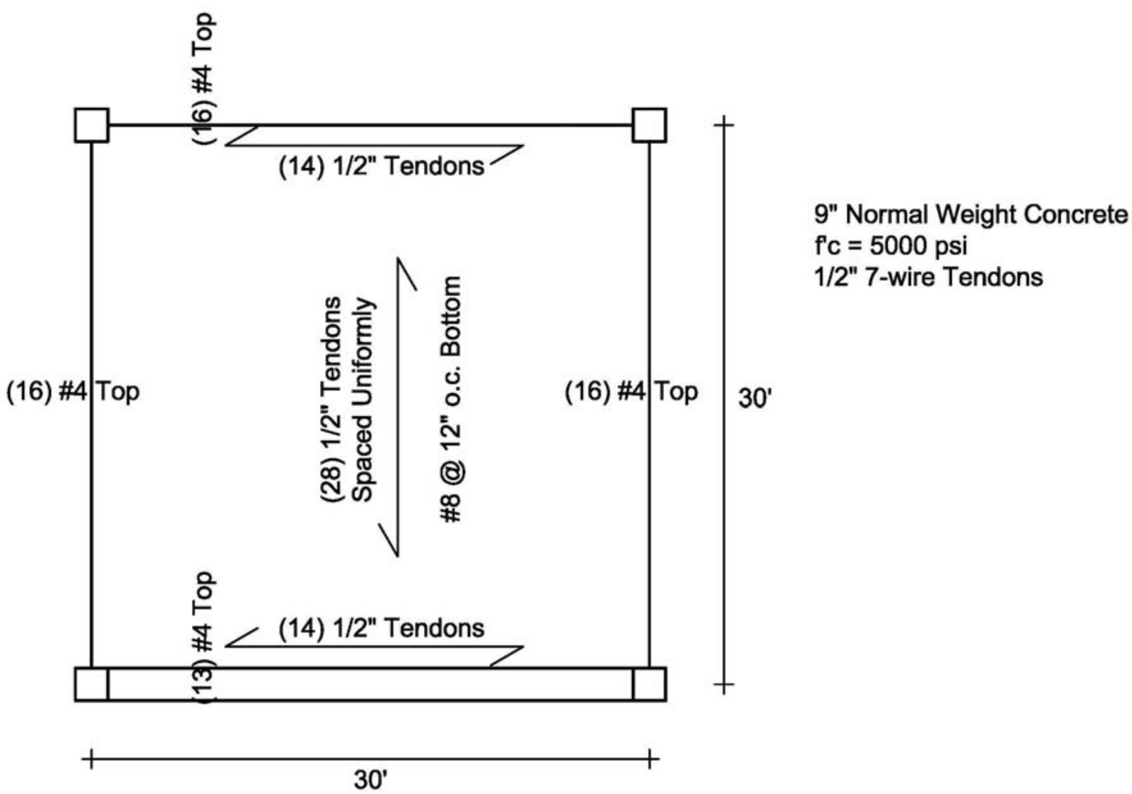


Figure 14: Two-Way Post-Tensioned Floor System Design

Advantages:

- Decreased structural floor depth
- Vibration and deflection control through post-tensioning
- Span lengths can be increased over conventional reinforced slabs
- Fire protection is inherent providing an adequate fire protection rating of 2 hours
- Less mass therefore reduced building weight and seismic loads

Disadvantages:

- Field post-tensioning can be very dangerous and extra safety measure must be taken
- Formwork and shoring is required in slab construction
- Penetrations and openings in slabs must be carefully located and designed around

Design Considerations:

Structural:

Deflection and vibration calculations have been omitted for the floor system due to its complexity. The use of post-tensioned floor slabs is more efficient than conventional reinforced slabs in terms of span capabilities and self weight. The seismic loads for the Franklin Square Hospital Center Patient Tower are higher than similarly sized buildings in the area due to the enormous self weight of the existing slabs. Any reduction in slab weight will dramatically reduce seismic loading.

Construction:

Construction companies in the DC/Baltimore area are very experienced with post-tensioned construction therefore the proposal of this system should not be of any concern to construction companies in the area.

Architectural:

The outward appearance of the building would likely not change at all with the change to a post-tensioned floor system. Inside, there would also be no change as column locations would not change.

Comparison of Floor Systems

Below is Table 2 which compares each floor system.

Table 2: Comparison of Floor Systems

Floor System	Existing	Alternative 1 (Composite Beam)	Alternative 2 (Composite Joist)	Alternative 3 (Two-Way Post Tensioned Slab)
Slab Depth	10”	5.25”	5.25”	9”
Total Depth	10”	23.25”	23.25”	9”
Estimated Cost	$15.53 / ft^2	$22.97 / ft^2	$23.96 / ft^2	$17.86 / ft^2
System Self Weight	121 PSF	49 PSF	48 PSF	109 PSF
Effect on Existing Column Grid	N/A	Minimal	Minimal	None
Construction Difficulty	Medium	Easy	Easy	Hard
Lead Time	Short	Long	Long	Short
Fireproofing	Built-In	Spray-On	Spray-On	Built-In
Durability	Great	Moderate	Moderate	Great
LL Deflection	~ 1.0”	1.0”	0.89”	Omitted
Vibration Concerns	Minimal	Moderate	Moderate	Minimal
Impact on Building Foundations	N/A	No	No	No
Viable Option	Yes	No	No	Yes

Slab Depth and Total Depth:

Each floor system has slab depths that are of no concern regarding size. However, the total system depth of the Composite Beam System and the Composite Joist System are very large. Both have total structural depths just over 23 inches. For the most part, mechanical systems must run beneath the structural members however some small mechanical equipment and electrical could run through webs of the composite joist system. Based on structural depth alone, both the existing Flat Plate System and the proposed Two-Way Post Tensioned Slab are the best options.

Estimated Cost:

The existing system, at a low $15.53 / ft^2, is the cheapest due to material cost. Both steel systems have very high material costs while having slightly lower installation costs than the concrete systems. See Appendix D for hand calculations

System Self Weight:
Self weight of the floor systems is of great importance. A system with high mass is desired for vibration control and acoustic performance but a light floor system greatly helps when designing for seismic conditions. The current flat plate floor system is the source of almost 65% of the building weight. In this case, a lighter floor system would drastically reduce the seismic loads the building experiences and allow for a more economical lateral system.

Effect on Existing Column Grid:
All of the proposed systems work very well with the existing column grid. It is very regular and repetitious with only six total perimeter columns needing replacement for the steel floor system options.

Construction Difficulty:
The steel systems are far easier to construct than their concrete counterparts. The steel systems both have metal decks meaning no forms or shoring is needed for concrete deck placement. Also, steel erection is fairly simple when compared to the placing of steel reinforcing. While the steel system can go up very quickly, the concrete systems take a great deal of time to form, reinforce, and cure which hold up the construction schedule.

Lead Time:
Long lead times were given to those systems that required fabrication prior to construction at the site. These systems included the Composite Beam System and the Composite Joist System.

Fireproofing:
Once, again two systems of differing materials required completely different additional fireproofing needs. The two steel systems would need to be spray-fire-proofed while the Flat Plate System and Post-Tensioned System would need no additional fireproofing to receive a two hour fire rating. The added task of spray-fireproofing is messy, time consuming, and prevents the use of the space by other trades, further holding up the construction schedule.

LL Deflection:
With the Flat Plate System, the Composite Beam System, and the Composite Joist System, the members can be cambered thereby negating deal load deflection. In the case of the Post-Tensioned System, dead load deflection is taken out through the pre-stressing done by the tendons. For live load, all systems meet the requirement of L/360 which is 1 inch except for the post-tensioned system whose deflection calculations were not calculated for this report but should have no trouble meeting requirements.

Conclusions:

After reviewing all of the floor systems, it can be seen why the existing system was used. It is the least expensive system of the four systems reviewed, has one of the smallest structural depths, provides great vibration and acoustic performance, has almost zero lead time, and does not require additional fire proofing.

The second floor system that has performed very well through this comparison is the two-way post-tensioned floor system. It is the second least expensive of the systems reviewed, has the smallest structural depth, provides great vibration and acoustic performance, and minimal lead time, and also does not require additional fireproofing.

While the composite joist system is easily constructed and is light, benefiting the lateral system, it has too many negatives in this comparison to make it a reasonable alternative. Along with the very immense structural depth, it is the most expensive system in this comparison, is likely too light which will cause vibration issues, and too difficult to fireproof. Therefore the Composite Joist Floor System comes in last place in this comparison and does not have potential for more in-depth investigation.

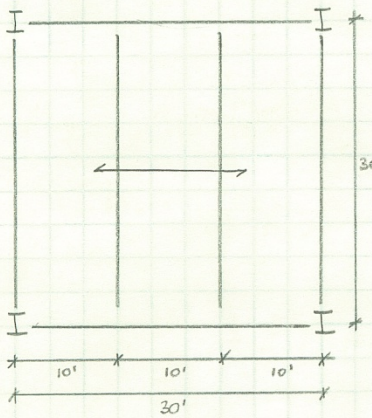
Having many of the same problems, the composite deck on composite beam floor system also has too many negatives in this comparison to make it a reasonable alternative. It also has a very immense structural depth, is the second most expensive, and is also likely too light which will cause vibration issues. While slightly better than the composite joist system, the Composite Deck on Composite Beam Floor System come in second to last place and does not have potential for more in-depth investigation.

In the future, both an in-depth analysis of the existing Flat Plate floor system and the Two-Way Post-Tensioned floor system will need to be completed for further comparison.

Appendix

Appendix A: Composite Deck on Composite Beam Floor System Calculations

COMPOSITE DECK ON COMPOSITE BEAM FLOOR SYSTEM



$3\frac{1}{4}"$ LIGHT WEIGHT CONC. $f'_c = 4000$ PSI
ON 2" DEEP x 18 GUAGE COMPOSIT DECK
TOTAL $t = 5\frac{1}{4}"$

$w_D = \underset{\text{CONC}}{39} + \underset{\text{DECK}}{3} + \underset{\text{SD}}{20} = 62$ PSF

$w_L = \frac{40+80}{2} = 60$ PSF (AVG LL) $+ 20$ PSF (PARTITION) $= 80$ PSF

$\frac{3}{4}"$ STUDS

COMPOSITE BEAMS: TRIB WIDTH = 10' → $w_D = 62(10) = 620$ lb/ft
$w_L = 80(10) = 800$ lb/ft

$w_u = 1.2(620) + 1.6(800) = 2024$ lb/ft

$M_u = 2024(30)^2/8 = 227.7$ ftk

Assume $a = 1"$ → $Y_2 = 5.25 - \frac{1}{2} = 4.75 \approx 4.5"$

TRY W12x22 $\phi M_n = 244$ ftk @ PNA 2 ⇒ $\Sigma Q_n = 281$k

$b_{eff} = \begin{cases} 10' = 120" \\ 30(12)/4 = 90" \end{cases}$ ∴ $b_{eff} = 90"$

$a = \frac{281}{0.85(4)(90)} = 0.918" < 1"$ OK✓

$Y_2 = 5.25 - 0.918/2 = 4.79" > 4.5"$ OK✓

STUDS $= \frac{281}{17.2} = 14 \times 2 = 28$ STUDS

INCLUDE SELF WEIGHT: $w_u = 1.2(620+22) + 1.6(800) = 2050$ lb/ft
$M_u = 230.6$ ftk < 244 ftk OK✓

CONSTRUCTION LOAD
$w_D = (39+3)(10) + 22 = 442$ lb/ft $w_L = 20(10) = 200$ lb/ft

$w_u = 1.2(442) + 1.6(200) = 850.4$ lb/ft $M_u = 95.7$ 'k $< \phi M_p = 110$ 'k OK✓

$\Delta = \frac{5}{384} \frac{(0.442)(30)^4(1728)}{29000(156)} = 1.78$ in $> \frac{L}{360} = 1$ in CAMBER BEAM 1"

LIVE LOAD DEFLECTION

$I_{LB} = 498 \quad \Delta_L = \frac{5}{384}\frac{(0.80)(30)^4(1728)}{29000\ (498)} = 1.0 \text{ in} \leq 1'' \quad$ OK ✓

COMPOSITE GIRDERS:

$P_{U\,total} = 2(\tfrac{1}{2}(2050 \text{ lb/ft})(30 \text{ ft})) = 61.5^k$

$M_U = P_U a = 61.5(10') = 615 \text{ ftk}$

ASSUME $a = 1'' \rightarrow Y_2 = 4.5''$

TRY W18×55 $\quad \phi M_n = 652$ ftk @ PNA BFL $\Rightarrow \Sigma Q_n = 336$ k

$b_{eff} = \begin{cases} 30' \\ 30(12)/4 = 90'' \end{cases} \quad b_{eff} = 90''$

$a = \frac{336}{0.85(4)(90)} = 1.1'' > 1''$ NOT OK

$Y_2 = 5.25 - a/2 = 4 \Rightarrow a = 2.5'' \quad \phi M_n = 640$ ftk @ BFL $\Rightarrow \Sigma Q_n = 336$ k

$a = \frac{336}{0.85(4)(90)} = 1.1'' < 2.5''$ OK ✓

$Y_2 = 5.25 - 1.1/2 = 4.7'' > 4''$ OK ✓

$\#STUDS = \frac{336}{21.2} = 16 \times 2 \Rightarrow 32$ STUDS

INCLUDE SELF WEIGHT: $M_{SELF} = \frac{1.2(0.055)(30)^2}{8} = 7.43$ ftk

$M_U = 615 + 7.43 = 622$ ftk < 640 ftk OK ✓

CONSTRUCTION LOAD

$P_U = 2\left[(850.4 \text{ lb/ft})(30 \text{ ft})/2\right] = 25.5^k \qquad M_{SELF} = 7.43$ ftk

$M_U = 25.5(10) + 7.43 = 262.4$ ftk $< \phi M_p = 420$ ftk OK ✓

$\Delta = \frac{25.5(30)^3(1728)}{28(29,000)(890)} + \frac{5}{384}\frac{(1.2(0.055))(30)^4(1728)}{(29,000)(890)} = 1.65'' + 0.05'' = 1.7''$

CAMBER BEAM 1''

LIVE LOAD DEFLECTION

$I_{LB} = 1700 \qquad P_L = 0.80(30) = 24$ k

$\Delta_L = \frac{24(30)^3(1728)}{28(29,000)(1700)} = 0.81'' < 1.0''$ OK ✓

COMPOSITE DECK

USD UNITED STEEL DECK PRODUCT

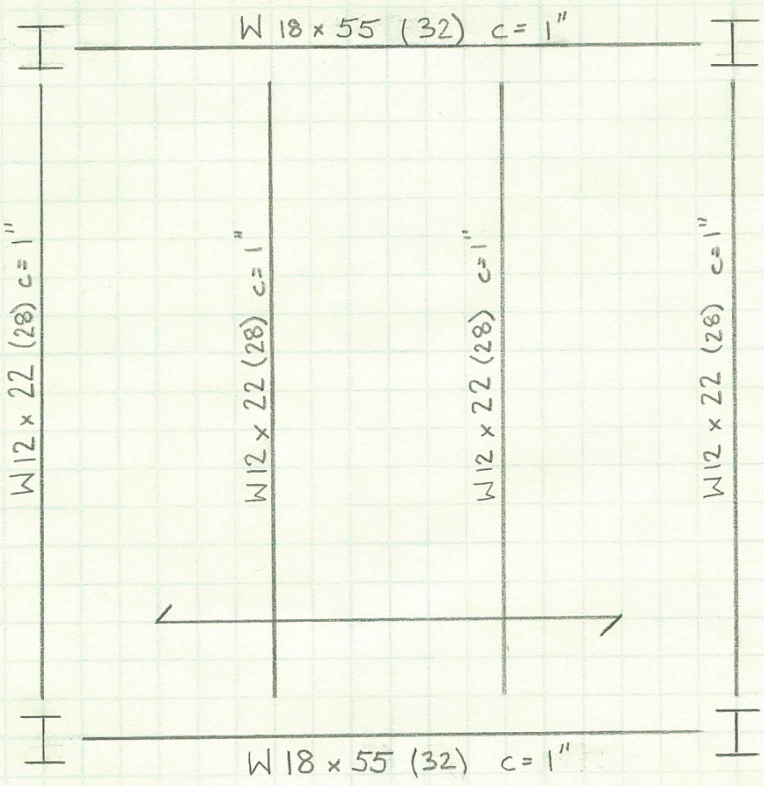
- ↳ 2" 18 GUAGE COMPOSIT DECK
- ↳ t = 5.25"
- ↳ ALL TABLES AND CHARTS CALCULATED WITH $f'_c = 3$ ksi < 4 ksi USED IN DESIGN

$\omega_D = 20$ PSF $\quad \omega_L = 80$ PSF $\quad \omega_U = 1.2(20) + 1.6(80) = 152$ PSF $= 152$ lb/ft 1' WIDTH

$M_U = 152\,(10)^2/8 = 1900$ ft lb $= 22.8$ in k $< \phi M_{no} = 82.21$ in k OK ✓

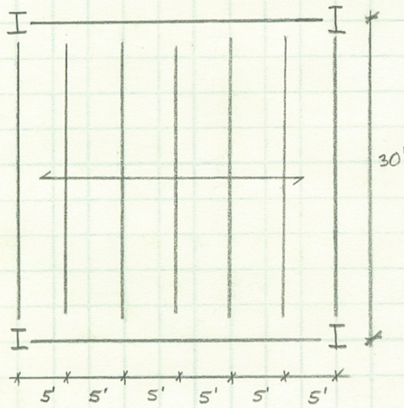
WITH 3 CONTINUOUS SPANS, MAX SPAN = 10.25 ft > 10 ft OK ✓

FINAL DESIGN



Appendix B: Composite Joist Floor System Calculations

COMPOSITE JOIST FLOOR SYSTEM



3¼" LIGHT WEIGHT CONC. $f'_c = 4000$ PSI
ON 2" DEEP x 18 GUAGE COMPOSIT DECK
TOTA t = 5¼"

STEEL JOIST INSTITUTE
CJ - SERIES

JOISTS

JOIST GEOMETRY:
DEPTH = TRY 12"
SPAN = 30 ft
SPACING = 5 ft

NOMINAL LOADS
1. NON COMPOSITE CONSTRUCTION DEAD LOAD
a) CONCRETE 39 PSF
b) DECK 3 PSF
c) JOIST AND BRIDGING 2.3 PSF
d) TOTAL 44.3 PSF = 222 lb/ft
2. CONSTRUCTION LIVE LOAD
a) DURING CONC. PLACEMENT 14PSF = 70 lb/ft
3. COMPOSITE DEAD LOAD
a) SUPERIMPOSED 20 PSF = 100 lb/ft
4. COMPOSITE LIVE LOAD
a) AVG LL + PARTITION = 60 + 20 = 80 PSF = 400 lb/ft

5. TOTAL FACTORED NON COMPOSITE DEAD LOAD
1.2D = 1.2(222) = 266.4 lb/ft
6. TOTAL FACTORED COMPOSITE DEAD LOAD
1.2D = 1.2(100) = 120 lb/ft
7. TOTAL FACTORED COMPOSITE LIVE LOAD
1.6L = 1.6(400) = 640 lb/ft
8. TOTAL FACTORED COMPOSITE DESIGN LOAD = 266 + 120 + 640 = 1026 lb/ft

CAMBER AND DEFLECTION (UNFACTORED LOAD)
1. LOADS TO CAMBER FOR
a) NON COMPOSITE DEAD LOAD 222 x 100% = 222 lb/ft
b) COMPOSITE DEAD LOAD 100 x 50% = 50 lb/ft
c) COMPOSITE LIVE LOAD 400 x 10% = 40 lb/ft
2. MAXIMUM ALLOWABLE LIVE LOAD DEFLECTION = (30x12)/360 = 1"
3. MAXIMUM DEFLECTION = 30(12)/240 = 1.5"

SELECT SPAN 30 ft, JOIST DEPTH 12 in.

$W_t = 15.5$ lb/ft
$W_{360} = 444$ lb/ft
$N\text{-}ds = 40 - \frac{1}{2}''$
$I_{eff} = 279\ in^4$
BRIDGING = 2(H)

BRIDGING $P_{br} = 600$ lb
L1.25 × 1.25 × 0.109

DEFLECTION: $I_{non\text{-}comp\ eff} = 87\ in^4$

$$\Delta_{non\text{-}comp\ DL} = \frac{5(222)(29.67)^4(1728)}{384(29{,}000{,}000)(87)} = 1.534\ in = L/235$$

$$\Delta_{composit\ DL} = \frac{100}{444}\left[\frac{29.67(12)}{360}\right] = 0.22\ in = L/1636$$

$$\Delta_{composit\ LL} = \frac{400}{444}\left[\frac{29.67(12)}{360}\right] = 0.89\ in = L/404$$

$$\Delta_{TL} = 1.53 + 0.22 + 0.89 = 2.64'' = L/136$$

CAMBER = 1.0(1.534) + 0.5(0.22) + 0.10(0.89) = 1.73 in

GIRDERS SUPPORTING JOISTS DESIGN AS COMPOSITE

APPROXIMATE LOADS AS DISTRIBUTED $P_{U\,TOTAL} = 1026(30) = 30.8^k$

$\omega_{U\,TOTAL} = 30.8(5)/30 = 5.13$ lb/ft $\quad M_U = \frac{5.13(30)^2}{8} = 577.1$ ftk

ASSUME $a = 1'' \rightarrow Y_2 = 4.5''$

TRY W18 × 50 $\phi M_n = 589$ ftk @ PNA BFL $\Rightarrow \Sigma Q_n = 306^k$

$$b_{eff} = \begin{cases} 30' \\ 30(12)/4 = 90'' \end{cases} \quad b_{eff} = 90''$$

$$a = \frac{306}{0.85(4)(90)} = 1.00'' > 1'' \quad \text{OK}\checkmark$$

$Y_2 = 5.25 - \frac{1}{2} = 4.75'' > 4.5''$ OK✓

$$\#\ \text{STUDS} = \frac{306}{21.2} = 14.4 = 15 \times 2 = 30 \text{ STUDS}$$

INCLUDE SELF WEIGHT: $M_{SELF} = \frac{1.2(0.05)(30)^2}{8} = 6.75$ ftk

$M_U = 577.1 + 6.75 = 583.9 < \phi M_n = 589$ ftk OK✓

CONSTRUCTION LOAD

$P_U = 2\left[1.2(222) + 1.6(70)\right](30\,\frac{ft}{2}) = 11.35^k$ $\quad M_{SELF} = 6.75$ ft k

$w_{U_{JOIST}} = 11.35(5)/30 = 1.89$ k/ft $\quad M_U = \frac{1.89(30)^2}{8} + 6.75 = 219.4$ ft k $< \phi M_p = 379$ <u>OK</u>✓

$w_{U_{TOTAL}} = 1.89 + 0.06 = 1.95$ k/ft

$\Delta = \frac{5}{384}\ \frac{1.95(30)^4(1728)}{29{,}000(800)} = 1.53'' > 1''$ ∴ CAMBER BEAM 0.75"

LIVE LOAD DEFLECTION

$I_{LB} = 1590 \quad P_L = 400(30) = 12^K \quad w_L = 12(5)/30 = 2$ k/ft

$\Delta_L = \frac{5}{384}\ \frac{2(30)^4(1728)}{29{,}000(1590)} = 0.79'' < 1''$ <u>OK</u>✓

<u>COMPOSITE DECK</u>

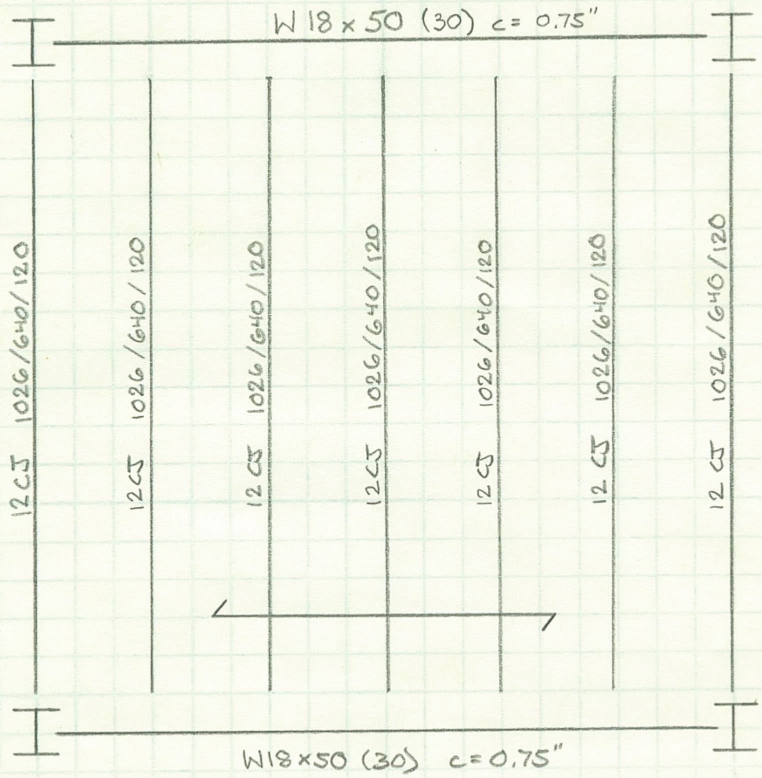
USD UNITED STEEL DECK PRODUCT
↳ 2" 18 GUAGE COMPOSIT DECK
↳ $t = 5.25''$
↳ ALL TABLES AND CHARTS USE $f'_c = 3$ ksi $< f'_c = 4$ ksi USED IN THIS DESIGN

$w_D = 20$ PSF $\quad w_L = 80$ PSF $\quad w_U = 1.2(20) + 1.6(80) = 152$ PSF $= 152$ lb/ft 1' WIDTH

$M_U = 152\,(5)^2/8 = 475$ ft lb $= 5.7$ in k $< \phi M_{n_o} = 82.21$ in k <u>OK</u>✓

WITH 3 CONTINUOUS SPANS, MAX SPAN = 10.25 ft > 5 ft USED <u>OK</u>✓

FINAL DESIGN



COMPOSITE JOIST: 12CJ 1026/640/120

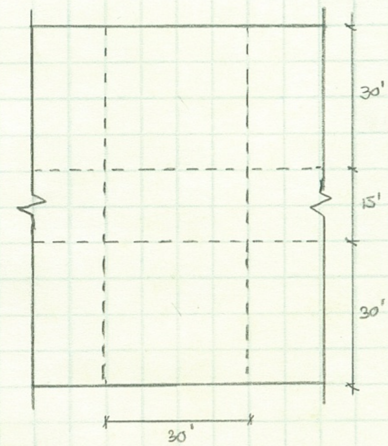
USING 2 ROWS OF 2L's 1.25 x 1.25 x 0.109

CAMBER = 1.73 in.

QUANTITY AND TYPE OF SHEAR STUDS, $N - d_s = 40 - \frac{1}{2}''$

Appendix C: Two-Way Post-Tensioned Floor System Calculations

TWO-WAY POST TENSIONED FLOOR SYSTEM



LOADS: FRAMING DEAD LOAD = SELF WEIGHT
SUPERIMPOSED DEAD LOAD = 20 PSF
LIVE LOAD = 80 PSF
ASSUME 2 HOUR FIRE RATING

MATERIALS: CONCRETE: NORMAL WEIGHT 145 PCF
$f'_c = 5{,}000$ PSI
$f'_{ci} = 3{,}000$ PSI
REBAR: $f_y = 60{,}000$ PSI
PT: UNBONDED TENDONS
1/2" ϕ 7-WIRE STRANDS $A = 0.153\ in^2$
$f_{pu} = 270$ KSI
ESTIMATED PRESTRESS LOSSES = 15 KSI (ACI 18.6)
$f_{se} = 0.7(270) - 15 = 174$ KSI (ACI 18.5.1)
$P_{eff} = Af_{se} = (0.153)(174\ ksi) = 26.6$ K/TENDON

DETERMINE PRELIMINARY SLAB THICKNESS
$L/h = 45$ LONGEST SPAN = 30 ft
$h = (30\ ft)(12)/45 = 8.0"$ PRELIMINARY SLAB THICKNESS = 9.0"

LOADING: DL = SELF WEIGHT = (9 in)(145 PCF) = 109 PSF
SIDL = 20 PSF
$LL_o = 80$ PSF
EXTERIOR BAY: $A_T = (30)(30) = 900\ ft^2$ $K_{LL} = 1.0$ $LL = 0.75 LL_o$
LL = 60 PSF
INTERIOR BAY: $A_T = (30)(15) = 450\ ft^2$ LL RED NOT ALLOWED
LL = 80 PSF

EQUIVALENT FRAME METHOD
CALCULATE SECTION PROPERTIES
$A = bh = (30 \times 12)(9) = 3{,}240\ in^2$
$S = bh^2/6 = (30 \times 12)(9^2)/6 = 4{,}860\ in^3$

SET DESIGN PARAMETERS
ALLOWABLE STRESSES: CLASS U
AT TIME OF JACKING: $f'_{ci} = 3{,}000$ PSI
COMPRESSION = $0.60 f'_{ci} = 1{,}800$ PSI
TENSION = $3\sqrt{f'_{ci}} = 164$ PSI
AT SERVICE LOADS: $f'_c = 5{,}000$ PSI
COMPRESSION = $0.45 f'_c = 2{,}250$ PSI
TENSION = $6\sqrt{f'_c} = 424$ PSI

AVERAGE PRECOMPRESSION LIMITS
P/A = 125 PSI MIN
= 300 PSI MAX

TARGET LOAD BALANCES:

60-80% OF DL (SELF WEIGHT) FOR SLABS

$0.75 w_{DL} = 0.75(109) = 82$ PSF

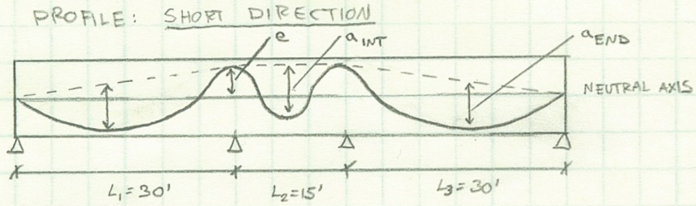
COVER REQUIREMENTS

RESTRAINED SLABS = 3/4" BOTTOM

UNRESTRAINED SLABS = 1 1/2" BOTTOM

= 3/4" TOP

TENDON PROFILE: SHORT DIRECTION



TENDON ORDINATE	TENDON (CG) LOCATION (MEASURED FROM BOTTOM)
EXTERIOR SUPPORT - ANCHOR	4.5"
INTERIOR SUPPORT - TOP	8.0"
INTERIOR SPAN - BOTTOM	6.75"
END SPAN - BOTTOM	1.75"

$a_{INT} = 8.0" - 6.75" = 1.25"$

$a_{END} = (4.5 + 8.0)/2 - 1.75" = 4.5"$

PRESTRESS FORCE REQUIRED TO BALANCE 75% OF SELFWEIGHT DL

$w_b = 0.75 w_{DL} = 0.75(109)(30) = 2453$ PLF $= 2.45$ k/ft

FORCE NEEDED IN TENDONS TO COUNTERACT THE LOAD IN END BAY

$P = w_b L^2 / 8 a_{END} = (2.45)(30)^2 / 8(4.5/12) = 735$ k

CHECK PRE COMPRESSION ALLOWANCES

DETERMINE NUMBER OF TENDONS TO ACHIEVE 735 k

TENDONS = 735 / 26.6 = 27.63 → USE 28 TENDONS

ACTUAL FORCE FOR BONDED TENDONS

$P_{ACTUAL} = (28)(26.6) = 745$ k

THE BALANCED LOAD FOR THE END SPAN IS SLIGHTLY ADJUSTED

$w_b = (745/735)(2.45) = 2.48$ k/ft

DETERMINE ACTUAL PRE COMPRESSION STRESS

$P_{ACTUAL}/A = (745)(1000)/(3{,}240) = 230$ PSI > 125 PSI MIN OK ✓

< 300 PSI MAX OK ✓

CHECK INTERIOR SPAN BALANCING

$w_b = (745\text{k})(8)(1.25/12) / (15)^2 = 2.76$

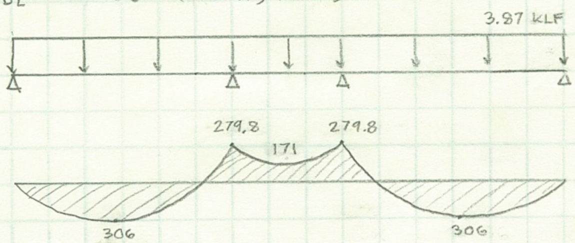
$w_b / w_{DL} = 2.76 / 3.27 = 0.84 < 1.0$ OK ✓

$\boxed{P_{EFF} = 745 \text{ k}}$

CHECK SLAB STRESSES

DEAD LOAD MOMENTS

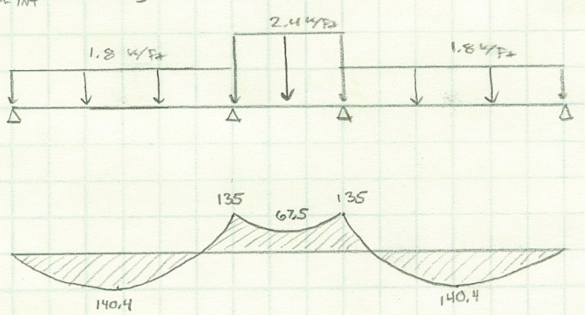
$w_{DL} = 109 + 20 = (129\ PSF)(30\ ft) = 3.87\ KLF$



LIVE LOAD MOMENTS

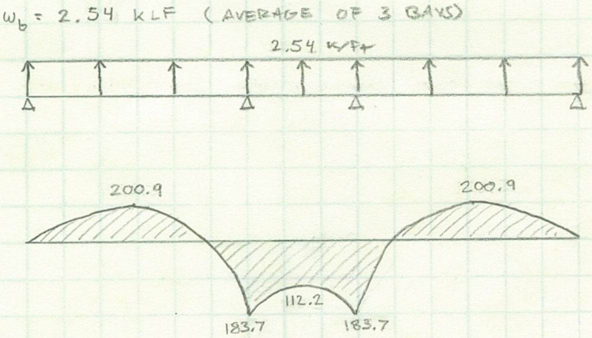
$w_{LL\ END} = 60(30) = 1.8\ KLF$

$w_{LL\ INT} = 80(30) = 2.4\ KLF$



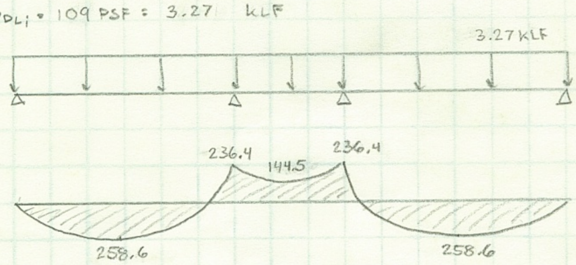
TOTAL BALANCING MOMENTS, M_{bal}

$w_b = 2.54\ KLF$ (AVERAGE OF 3 BAYS)



DEAD LOAD MOMENTS AT TRANSFER

$w_{DL_i} = 109\ PSF = 3.27\ KLF$



STAGE 1: STRESSES IMMEDIATELY AFTER JACKING (DL + PT)

INTERIOR SPAN

$f_{top} = \left[(144.5 - 112.2)(12)(1000)\right]/(4860) - 230 = -150.2$ psi $\leq 3\sqrt{f'_{ci}} = 164.3$ OK ✓

$f_{bot} = \left[(-144.5 + 112.2)(12)(1000)\right]/(4860) - 230 = -309.8$ psi $\geq -0.6f'_{ci} = -1800$ OK ✓

END SPAN

$f_{top} = \left[(-258.6 + 200.9)(12)(1000)\right]/(4860) - 230 = -372.5$ psi $\geq -0.6f'_{ci} = -1800$ OK ✓

$f_{bot} = \left[(258.6 - 200.9)(12)(1000)\right]/(4860) - 230 = -87.5$ psi $\leq 3\sqrt{f'_{ci}} = 164.3$ OK ✓

SUPPORT STRESSES

$f_{top} = \left[(236.4 - 183.7)(12)(1000)\right]/(4860) - 230 = -99.9$ psi $\leq 3\sqrt{f'_{ci}} = 164.3$ OK ✓

$f_{bot} = \left[(-236.4 + 183.7)(12)(1000)\right]/(4860) - 230 = -360.1$ psi $\geq -0.6f'_{ci} = -1800$ OK ✓

STAGE 2: STRESSES AT SERVICE LOAD (DL + LL + PT)

INTERIOR SPAN

$f_{top} = \left[(171 + 67.5 - 112.2)(12)(1000)\right]/(4860) - 230 = 81.9 \leq 6\sqrt{f'_c} = 424.3$ OK ✓

$f_{bot} = \left[(-171 - 67.5 + 112.2)(12)(1000)\right]/(4860) - 230 = -541.9 \geq -0.45f'_c = -2250$ OK ✓

END SPAN

$f_{top} = \left[(-306 - 140.4 + 200.9)(12)(1000)\right]/4860 - 230 = -836.2 \geq -0.45f'_c = -2250$ OK ✓

$f_{bot} = \left[(306 + 140.4 - 200.9)(12)(1000)\right]/4860 - 230 = 376.2 \leq 6\sqrt{f'_c} = 424.3$ OK ✓

SUPPORT STRESSES

$f_{top} = \left[(279.8 + 135 - 183.7)(12)(1000)\right]/4860 - 230 = 340.6$ psi $\leq 6\sqrt{f'_c} = 4243$ OK ✓

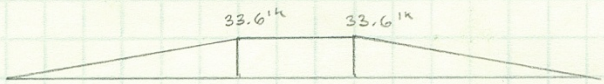
$f_{bot} = \left[(-279.8 - 135 + 183.7)(12)(1000)\right]/4860 - 230 = -800.6 \geq -0.45f'_c = -2250$ OK ✓

ALL STRESSES ARE WITHIN THE PERMISSIBLE CODE LIMITS

ULTIMATE STRENGTH

$M_1 = Pe = (745)(3.5)/12 = 217.3$ ftk

$M_{SEC} = M_{BAL} - M_1 = 183.7 - 217.3 = -33.6^k$ @ INTERIOR SUPPORT



M_u^- @ MIDSPAN $= 1.2(-171) + 1.6(-67.5) + 1.0(-33.6) = -347$ ftk

M_u^+ @ MIDSPAN $= 1.2(306) + 1.6(140.4) + 1.0(-16.8) = 575$ ftk

M_u^- @ SUPPORT $= 1.2(-279.8) + 1.6(-135) + 1.0(-33.6) = -585$ ftk

DETERMINE MINIMUM BONDED REINFORCEMENT

POSITIVE MOMENT REGION

INTERIOR SPAN: $f_t = -150.2$ psi $< 2\sqrt{f'_c} = 141$ psi

NO POSITIVE REINFORCEMENT REQUIRED

EXTERIOR SPAN: $f_t = 376.2$ psi $> 2\sqrt{f'_c} = 144$ psi

MINIMUM POSITIVE MOMENT REINFORCEMENT

$y = [376.2/(376.2 + 836.2)](9) = 2.79$ in

$N_c = [(306 + 140.4)(12)/4860](0.5)(2.79)(30)(12) = 553^k$

$A_{s,min} = 553/[(0.5)(60)] = 18.4$ in^2

$A_{s,min} = 18.4/30$ ft $= 0.61$ in^2

USE #8 @ 12 in O.C. BOTTOM $= 0.79$ in^2/ft

NEGATIVE MOMENT REGION

INTERIOR SUPPORTS

$A_{cf} = 30(12)(9) = 3240$ in^2

$A_{s\,min} = 0.00075(3240) = 2.43$ in^2 $= 13$ #4 TOP (2.6 in^2)

EXTERIOR SUPPORTS

$A_{cf} = 30(12)(9) = 3240$ in^2

$A_{s\,min} = 0.00075(3240) = 2.43$ in^2 $= 13$ #4 TOP (2.6 in^2)

INTERIOR SPAN

$A_{cf} = 30(12)(9) = 3240$ in^2

$A_{s\,min} = 0.00075(3240) = 2.43$ in^2 $= 13$ #4 TOP (2.6 in^2)

MUST SPAN A MINIMUM OF 1/6 THE CLEAR SPAN ON EACH SIDE

AT LEAST 4 BARS REQUIRED IN EACH DIRECTION

PLACE TOP BARS WITHIN 1.5h AWAY FROM THE SUPPORT ON EACH SIDE

$= 1.5(9) = 13.5$ in

MAXIMUM BAR SPACING = 12 in.

CHECK MIN REINFORCEMENT IF IT IS SUFFICIENT FOR ULTIMATE STRENGTH

AT INTERIOR SUPPORTS

$d = 9 - 3/4 - 1/4 = 8''$

$A_{ps} = 0.153\ (28) = 4.284\ in^2$

$f_{ps} = 174{,}000 + 10{,}000 + [5{,}000(30)(12)d] / [(300)(4.284)]$

$= 184{,}000 + 1401\,d$

$= 195{,}208$

$a = [(2.6)(60) + 4.284(195)] / [0.85(5)(30)(12)] = 0.65$

$\phi M_n = 0.9[(2.6)(60) + (4.28)(195)][8 - (0.65)/2]/12 = 570.2$ ftk < 585 NOT OK✗

$A_{s\,req} = 3.2\ in^2 \rightarrow \phi M_n = 590 > 585$ OK✓

↳ (16) #4

AT MID SPAN (INTERIOR SPAN)

$f_{ps} = 184{,}000 + 1401(6.75) = 193457$

$a = [2.6(60) + 4.28(193)] / [0.85(5)(30)(12)] = 0.64$

$\phi M_n = 0.9[4.28(195)][6.75 - 0.64/2]/12 + 0.9[2.6(60)][8 - 0.64/2]/12$

$= 402 + 89.9 = 491.9 > 347$ OK✓

$A_{s\,req} = 2.6\ in^2 \rightarrow (13)\ \#4$

AT MID SPAN (END SPAN)

$d = 9 - 1.5 - 0.25 = 7.25$ in

$f_{ps} = 184000 + 1401\,d = 194{,}157$ psi

$a = [18.44(60) + 4.28(194)] / [(0.85)(5)(30)(12)] = 1.27$ in

$\phi M_n = 0.9[18.44(60) + 4.28(194)][7.25 - 1.27/2]/12 =$

$= 961$ ftk > 575 ft k OK✓

16-#4	TOP AT INTERIOR SUPPORTS
13-#4	TOP AT EXTERIOR SUPPORTS
13-#4	TOP INTERIOR SPAN
#8 @ 12" o.c.	BOTTOM END SPANS

Appendix D: Floor System Cost Analysis

COST ANALYSIS

RS MEANS 2009

FLAT PLATE

25' x 25' BAY → 10" THICK → MAT: \$ 8.20
INST: \$ 8.50
\$ 16.70

LOCATON FACTOR: BALTIMORE = 0.93

TOTAL COST = (16.70)(0.93) = \$15.53/ft²

COMPOSITE BEAM / DECK

30' x 30' BAY → SUPERIMPOSED LOAD = 125 PSF → MAT: \$ 18.40
INST: \$ 6.30
\$24.70

TOTAL COST = (24.70)(0.93) = \$22.97/ft²

COMPOSITE JOIST

30' x 30' → SUPERIMPOSED LOAD = 100 PSF → TOTAL LOAD = 145 PSF

MAT = \$ 18.05 + 1.57 = \$19.62
LAB = \$ 5.75 + 0.39 = \$ 6.14
\$ 25.76

TOTAL COST = (25.76)(0.93) = \$ 23.96/ft²

POST - TENSIONED

STEEL PRESTRESS = \$3.33/lb   0.52 lb/ft = ½" STRAND

$0.52\ \mathrm{lb/ft}\,\big(28(30') + 28(30')\big) = 874\ \mathrm{lb}$

$\$3.33/\mathrm{lb}\,(874\ \mathrm{lb}) = \$2,910.42/_{30' \times 30'\ \mathrm{BAY}} = \$3.23/\mathrm{ft}^2$

CAST IN PLACE CONC: $\$575/\mathrm{yard}^3 \left(\frac{1\ \mathrm{yd}^3}{27\ \mathrm{ft}^3}\right)\left(\frac{9"}{12"}\right) = \$15.97/\mathrm{ft}^2$

\$19.20

TOTAL COST = (19.20)(0.93) = \$ 17.86/ft²