Technical Report II

Roberts Pavilion

Camden, NJ

Andrew Voorhees | Structural

Alternative Floor Systems

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EXECUTIVE SUMMARY

The Roberts Pavilion is a patient care center located in Camden, NJ. It is part of the Cooper University Hospital and serves a large range of patient needs. Standing 10 stories above grade, it is a noticeable landmark when entering Camden. The pavilion was built between two existing hospital buildings and now serves to connect them. During construction, renovations updated the façades on the adjacent buildings to give a sense of uniformity to the complex. Aluminum and glass panels make up the main façade and give patients excellent views to the outside. Structurally, the building is framed in steel, with composite deck flooring. Lateral loads are resisted by ordinary steel concentrically braced frames.

Purpose and Scope

The purpose of this report is to provide an analysis of the Roberts Pavilion floor framing system, as well as to propose and study three alternative floor systems. The scope of this will include the design and analysis of a one-way slab with beams, a two-way flat plate system, and a precast hollow core plank system.

One of the main functions of this report is to provide a thorough comparison between the different systems; keeping in mind that they may be considered in the future for redesigning purposes. Each system has been designed under the same conditions as the existing structure. After the design, each was analyzed based on cost, depth, weight, and impact on the structure and the architecture.

Comparing the results, it was found that the most viable floor framing options, in order of most desired first, were the two-way flat plate system, the existing composite system, and the one way slab with beams. The precast hollow core plank system proved to be very inefficient for the typical bay size in the pavilion. As a result, it will not be considered in the future, based on a large cost and floor depth. Proving to be most economical, based on cost and floor depth, the two-way flat plate system should be seriously considered as an alternative system.

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BUILDING INTRODUCTION

The Roberts Pavilion, as shown in red in Figure 1, is a recently constructed patient care center at the Cooper University Hospital in Camden, New Jersey. Completed in December 2008, the project cost about $220 million. The pavilion is approximately 320,000 GSF and occupies 10 stories above grade as well as one basement level. Additionally, during construction, the adjacent Kelemen and Dorrance Buildings, shown in Figure 1 in blue and purple respectively, underwent 51,000 GSF of renovations.

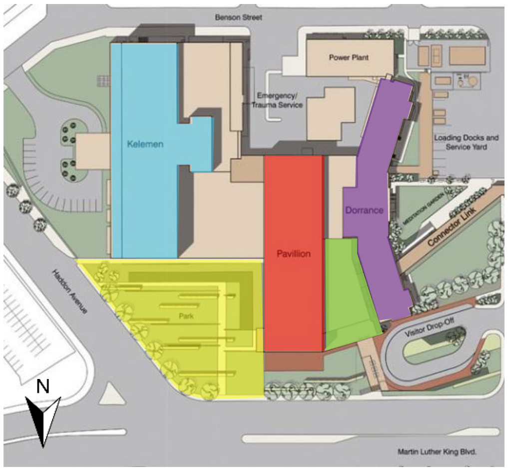


Figure 1 : Site plan (Courtesy of EwingCole)

Cooper has been a leading medical institution in southern New Jersey for many years. The Roberts Pavilion establishes Cooper's presence in Camden and upon entering the city, it is easily visible. Architecture and engineering systems were designed by EwingCole. They designed the façade, as shown in Figure 2, to be composed mostly of glass and aluminum panels. During renovations, façades of the adjacent buildings were updated to give the complex a sense of uniformity. The master plan also called for the demolition of the parking garage on the corner of Haddon Avenue and Martin Luther King Boulevard, as shown in yellow in Figure 1, and for the space to be turned into a park to improve the surrounding landscape.

Figure 2 : Roberts Pavilion (Courtesy of Halkin photography, LLC)

The lobby, shown in green in Figures 1 and 3, is a grand, open space with an abundance of natural light and warm colors. It also acts as a link between the new pavilion and the existing Dorrance Building which is shown in puple in Figure 1. Bamboo plantings and natural materials give the space a garden-like feel. Cooper wanted the pavilion to feel like a "healing garden" where patients experience a calm and peaceful atmosphere seemingly distant from the city outside. This idea is evident in the design from the lobby to the upper floors.

Figure 3 : Lobby (Courtesy of Eduard Hueber/Arch Photo, Inc.)

Each floor maintains a different function. The second floor houses clinical cardiology, while the third floor houses surgical suites, and the fourth and fifth floors hold the intensive care units. Typical patient rooms are located on floors six through ten.

STRUCTURAL OVERVIEW

Foundation

URS Corporation investigated the Roberts Pavilion site conditions by performing nine test borings. The top layer of soil in most of the drillings consisted of silty sand with some gravel and fragments of brick and concrete. This fill layer was classified as poorly to well-graded sand (SP-SW). Soil under the fill layer was classified as loose to dense silty sand with layers of clay becoming more firm with depth. 16" diameter reinforced piles were cast with a depth of -68' below the basement slab to reach firm soil. A minimum compressive strength of 4000 PSI concrete was used along with ASTM A615 Grade 60 reinforcement. Pile caps required concrete with minimum compressive strength of 5000 PSI and range in thickness from 3'-6" to 6'-0". The stratum layer under the footings was compacted to reach a bearing capacity of 4000 PSF.

The main basement will have an elevation of +8' above sea level (being about 5' above the water table), but elevator pits and mechanical space will be about +2' (1' below the water table). This means that the lower slab and walls will require waterproofing. Additionally these areas should be designed for hydrostatic uplift pressures. A permanent pump-operated subsurface drainage system was added to control the water level.

The main basement level is a 5" concrete slab, with a 16" slab poured in the north end under the mechanical room. Structural fill was placed for support under the foundations and used as backfill for the walls and footings. Soil pressures will need to be calculated when designing foundation walls.

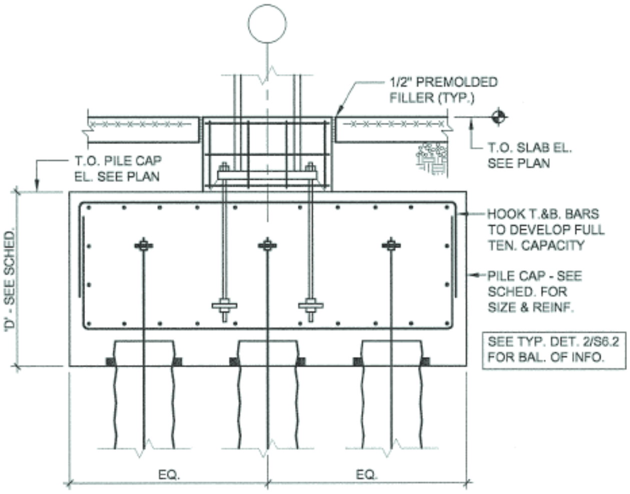


Figure 4 : Typical pile cap without pedestal

Floor System

Typical floor framing in the pavilion consists of a composite system. It incorporates a 2", 18-gauge steel deck with a 3¼" lightweight concrete topping reinforced with WWF (welded-wire-fabric). The Decking runs perpendicular to the beams and shear studs transfer the load to the beam to allow for composite behavior.

Framing System

All steel wide flange members in the building are A992 grade 50. Columns are typically spaced 30' on center in the North-South direction. In the East-West direction there are typically three bays; the interior span being 23', and the two exterior spans being 29'-6". Column spacing is shown in Figure 5 Column weights vary; with the heaviest being a W14x426. However, all columns have a 14" web.

Beams on floors 4 - 10 are typically wide flange members W16x26 and W14x22 spaced at 10' (See Figure 6). Floors 1 (ground) - 3 have larger beams, being that they are supporting heavier equipment. The 3rd floor holds the operating suites and part of the trauma unit thus it supports larger dead and live loads than most of the floors. It uses mostly W21x44 beams spaced at 7'-6".

Figure 5 : Typical bay (See Appendix A for full framing plan)

Roof System

The roof of the pavilion supports mechanical equipment; specifically three cooling towers, an air cooled chiller, and three air handling units. It has two different levels, where the center level rises 3' above the main level to support the AHU's. Composite steel decking is also used on the roof, with the exception of the elevator core roof which is a poured slab. Wide flange members in the raised level are spaced at 6'-6" maximum to support the load from the mechanical units. In the south-west corner of the roof there is a small mechanical room with the roofing material being 1½", 20 gauge roof galvanized metal roof decking. All the mechanical systems on the roof are hidden by a 19' parapet.

Lateral System

The lateral resisting system in the pavilion consists of ordinary steel concentrically braced frames (OSCBF). There are four frames in each direction of the building as shown in Figure 6. Each frame extends through one full bay and through the full height of the building. Two typical frames are shown below in Figure 8. They consist of a variety of square HSS members with the most common being HSS10x10x1/2.

Figure 6 : Braced frame locations

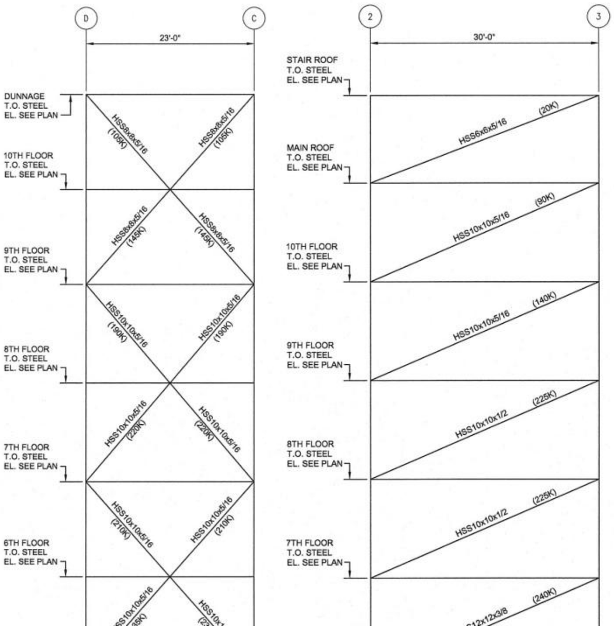


Figure 7 : Two typical braced frames (OSCBF)

Design Codes

Below is a list of the codes and standards applicable to the design of the Roberts Pavilion as used by the design team. Codes that were utilized in this report for analysis are listed separately.

Codes Used In Design:

- IBC 2000 (New Jersey Edition)
- ASCE 7-02 (Minimum Design Load for Buildings and Other Structures)
- ACI 318-02 (Building Code Requirements for Structural Concrete)
- PCI (Manual for Structural Design of Architectural Precast Concrete)
- AISC 12th Edition (Manual of Steel Construction)
- AWS D1.1 (Structural Welding Code for Steel
- ASTM (American Society for Testing and Materials)

Codes Used In Analysis:

- ASCE 7-05 (Minimum Design Load for Buildings and Other Structures)
- AISC 14th Edition (Manual of Steel Construction)

Materials

Below are listed the typical materials used in the construction of the Roberts Pavilion.

*Material strengths based on ASTM rating

Structural Steel	
Member Type	**Strength**
Wide Flange Member	A992 Grade 50
HSS Pipes	A500 Grade 46
Base Plates	A572 Grade 50
Lateral Moment Plates	A572 Grade 50
Splice Plates	A572 Grade 50
Angles	A36
Channels	A36
Anchor Bolts (1" and 2" Ø)	F1554 Grade 105
Bolts (¾" Ø)	A325 - X
Concrete Reinforcement	A615 Grade 60

Concrete	
Location	**Compressive Strength, f'_c (PSI)**
Slab on Grade	3000
Foundation Walls	4000
Piers	4000
Structural Slabs	4000
Beams	4000
Pedestals	4000
Equipment Pads	4000
Sidewalks	4000

Masonry	
Masonry	**Compressive Strength, f'_c (PSI)**
CMU	1500
Masonry Mortar	1500

Steel Deck		
Location	**Thickness (in)**	**Gauge**
Floor (composite)	2	18
Roof (composite)	2	18
Penthouse Roof	1.5	20

GRAVITY LOADS

Dead and Live Loads

Live load values were given on the structural drawings. These were similar to the values in ASCE 7-05 with the exception of several that aren't specified in the code. These values are denoted on the tables below with the value that was assumed. For spaces such as the operating rooms, that have a large difference between the code value and the value used for design, these calculations have used the value given in the drawings. This is because the live load may have been estimated larger because of specialized equipment, and it would be more conservative to use the larger value.

Dead loads are also shown below. An average value of 6.5 PSF for framing was calculated by summing the weight of framing on a given floor and dividing by the floor area. However, some floors are framed with larger members than the average floor (See Figure 26, Appendix A), thus 10 PSF was estimated as the maximum value. Although the value is larger than average, it provides a more conservative analysis.

Live Loads (PSF)		
Occupancy or Use	As Designed	ASCE 7-05
Lobby/Public Areas	100	100
1st Floor Corridor	100	100
Corridors above 1st Floor	80	80
Patient Rooms + Partitions	40+20	40+20
O.R.	100	60
O.R. Core	125	*60
Medical Equipment Rooms	100	*100
Stairways	100	100
Mechanical Rooms	150	*150
Conference Rooms	100	*100
Kitchen	125	*125
Roof	30	20

*Assumed Value

Dead Loads (PSF)	
System	As Designed
Framing	*10
Superimposed	*10
MEP	*5
Composite Floor	42

*Assumed Value

Snow Loads

Snow loads were calculated using ASCE 7-05. The ground snow load was given in the code as 25 PSF. Calculations in Appendix B show that the maximum design value for snow drift is approximately 93 PSF (94 PSF given in the drawings). Values used to calculate the flat roof snow load are shown to the right.

Flat Roof Snow Load	
Variable	Value
P_g (PSF)	25
C_e	1
C_t	1
I	1.2
P_f (PSF)	24

FLOOR SYSTEM ANALYSIS

The Roberts Pavilion framing system is composed of 10 bays in the North-South direction and 3 bays in the East-West direction, as shown in Figure 18 in Appendix A. In the 3 span (East-West) direction, the typical exterior bay, as shown in Figure 8, is 30' x 29'-6". This bay size varies slightly at the South end of the building; however, the majority of the bays have equal column spacing. The exterior bay was picked for analysis because it is larger than the interior bay and thus it will control the design of concrete systems.

The current floor is composed of a composite steel system with wide flange beams and girders, and composite steel decking. This technical report will cover the analysis of the existing system as well as the design and analysis of three alternative systems. These include a one-way slab with beams, a two-way flat plate slab, and a precast hollow core plank system. This report will go into detail about the effects of each system on the structure and the architecture, as well as provide a cost and feasibility analysis of each system.

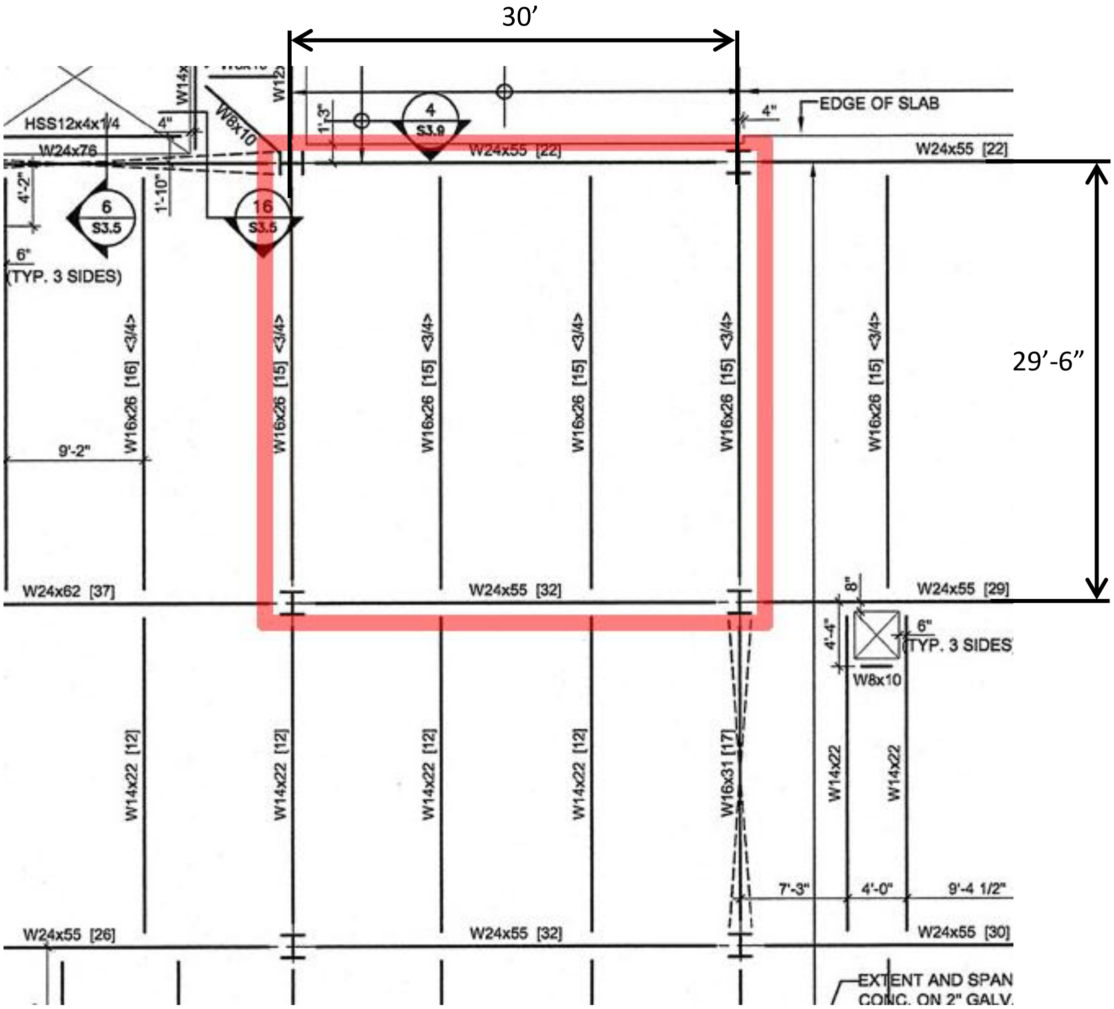


Figure 8 : Typical bay

Existing: Composite Floor System

The existing floor system in the Roberts Pavilion consists of a composite beam and decking system. A 2" composite steel deck was chosen from the manufacturer, Vulcraft, with a gauge of 18 and a 3¼" lightweight concrete topping. Topping thickness was determined by the required fire rating of 2 hours. The deck was checked and verified for the applicable loading, then beams were sized and shear studs were calculated. Beams and girders were verified for their design loads, however, shear stud counts differed from those in the drawings. In the case of this discrepancy, the member size and number of studs shown on the drawings were used. From there a detailed estimate was calculated, as shown in Appendix F, and cost per square foot was able to be determined. **Detailed calculations are shown in Appendix B.**

System Summary

- Beams: W16x26, 15 studs
- Girders: W24x55, 22-33 studs
- Deck: 2VLI18
- Topping: 3¼" LTWT Concrete

Advantages:

Framing with steel allows for larger spans with less area occupied by columns. This allows for a more open floor plan. Additionally, a composite system is more economical. Allowing the deck to take some of the load allows for smaller beams to be considered. The fire rating may be achieved by deck and topping alone, therefore, fireproofing is only needed on steel beams, and not the entire deck. From a construction standpoint, steel frames can be erected more quickly and thus lowers the cost and shortens schedule time. Cost can also be decreased by designing the deck to be unshored during construction; and in this analysis the deck was designed for this capability.

Disadvantages:

Costs associated with labor involved in a composite system may be a disadvantage. Welding of shear studs and installation of fireproofing may raise the cost. However, the overall cost of a composite steel system is roughly competitive with other systems.

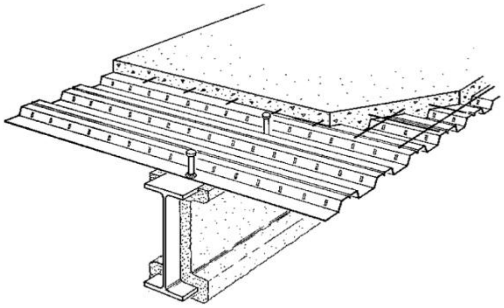


Figure 9 : Composite system

Viability:

Using the members and deck specified in the drawings, the weight of the system was determined to be approximately 48 psf. Cost was approximated using a detailed estimate of the system and was found to be $16.88/S.F. This price includes cost of material as well as labor and equipment. System depth is governed by the girders plus the decking and comes to 29.5".

The weight of the composite system is the lowest of all of those compared. Along with ease of constructability, and a cost that is competitive with the alternatives, the composite system is a very viable option. Vibrations in a steel system are of more concern than a concrete system, and would need to be studied in more depth when considering this option.

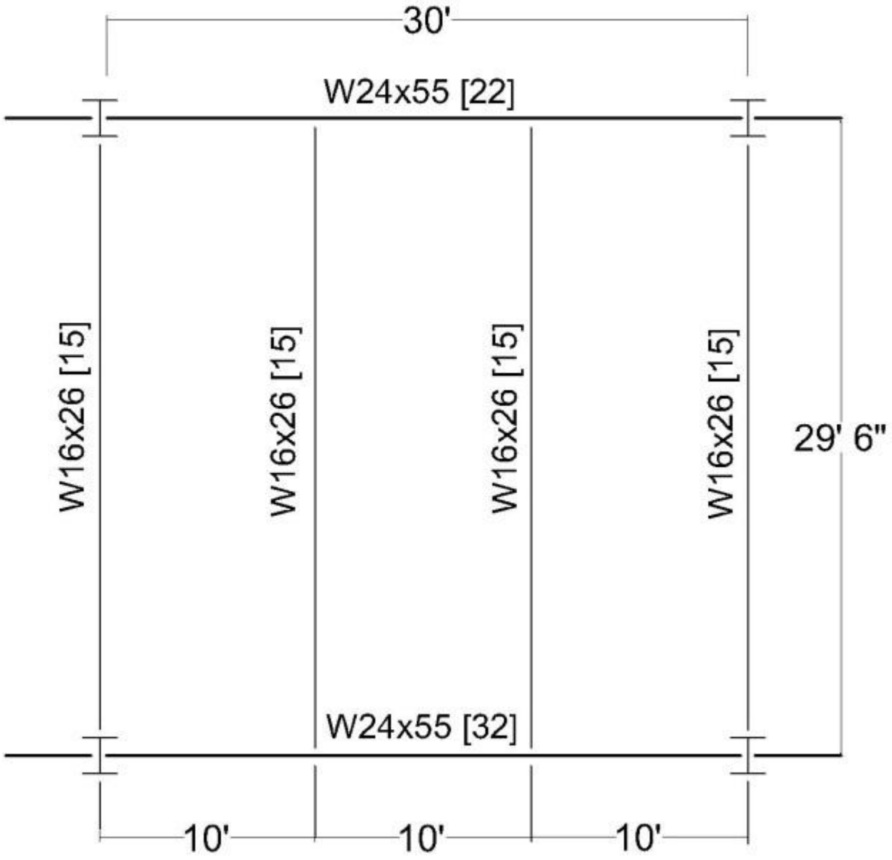


Figure 10 : Typical bay of existing composite system

One-Way Slab with Beams

The second system to be considered was a one-way slab with beams. All concrete was assumed to be normal weight with a compressive strength of 4000 psi. It was determined that a 5" slab with intermediate beams would be sufficient to carry the load. Rebar in the slab was designed to use #4 bars spaced at 12" on center. Beams were sized to be 16"x20", requiring bottom and top reinforcement of bars ranging from #7 to #9.

Design moments were determined based on the continuity of the span in question. It should be noted that the beams have the same dimensions as the exterior girder. This is because the beams are continuous at one end, while the girder is continuous in both directions. This gives the girder a lower design moment than the beams. In contrast, the interior girder, although it requires a lower design moment, is dimensioned larger than the beams because it is carrying the load from two spans. The dimensions of the interior girder are 24"x22". A plan view, specifying member dimensions, is shown in Figure 12 on the next page. **Detailed calculations, reinforcement designs, and member dimensions are shown in Appendix C.**

System Summary:

- Beams: 16"x20", #7-#9
- Ext. Girder: 16"x20", #7-#10
- Int. Girder: 24"x22", #7-#10
- Slab: 5", #4 bars

Advantages:
A one-way slab with beams has several advantages. The cost is often lower than that of a steel system, and normally system depth is lower. Additionally, the system is very good choice if vibrations are an issue. Slab depth also meets fire rating requirements, making additional fireproofing unnecessary.

Disadvantages:
One of the major disadvantages of a one-way slab with beams is column size. Concrete columns will take up more space than steel columns and will largely affect the architecture. Foundations would also need to be redesigned to support the additional weight of the system. The one-way slab is lighter than the two-way, but is twice as deep. Another consideration to take into account is formwork and labor requirements. Forming beams takes longer and will most likely increase construction time, meaning a greater cost as well.

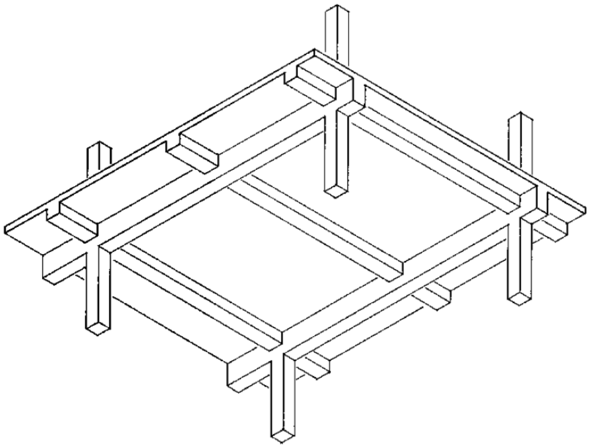


Figure 11 : One way slab with beams

Viability:

The one-way system with beams was estimated to cost approximately $16.21/S.F. This was lower than the steel system, but higher than the two-way slab cost. System depth is lower than the steel by 2”. Deflection control is good at a maximum deflection of 0.62”. The major difference between this system and the existing system would be column sizes. A column size of 24”x24” was estimated using the column’s axial load calculated in Technical Report I. Bay sizes could be maintained, however the concrete columns would be much larger and floor plans may need to be rethought.

Overall, the one-way slab system is a good option to consider. However, if cost and floor depth are the major considerations, the two-way slab would be a better choice. This system would have a large impact on the foundations, and they would need to be reevaluated for the increased weight. Constructability is also an important consideration in this system. Formwork and labor will increase the price because of the beams. Therefore, this is probably not the best option to consider as an alternative because of cost.

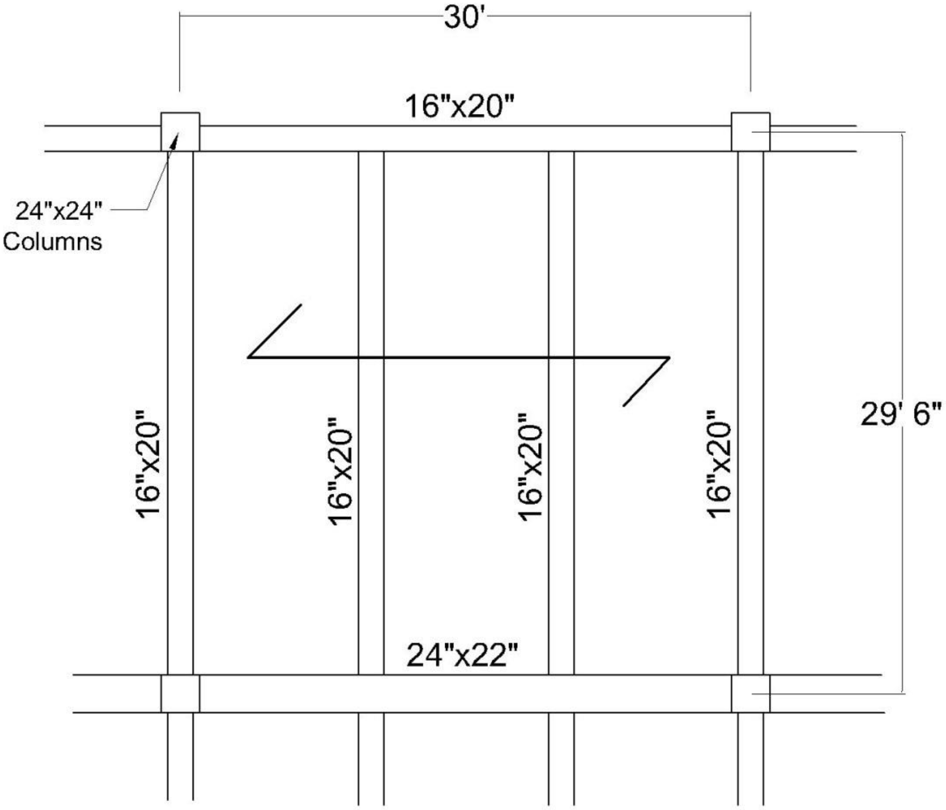


Figure 12 : Typical bay for one-way slab with beams

Two-Way Flat Plate

Next, a two-way flat plate system was designed. Concrete was assumed to have a compressive strength of 4000 psi. Slab thickness required by code to resist deflections was 11”. Reinforcement was assumed to be consistently #5 bars, and the number of bars was determined based on column strip and middle strip moments. The slab alone was close to being able to resisting punching shear; therefore shear caps were designed to resist the shear at critical sections. Drop panels could have been designed to reduce the moment; however this analysis did not consider them. **Detailed design calculations and rebar requirements are shown in Appendix D.**

System Summary:

- Slab: 11”, #5 bars
- Shear caps: 4’x4’

Advantages:
The major advantage of the two-way flat plate system is depth. This is even more advantageous because the building is a hospital. Here there will be a larger amount of MEP systems between floors. The more shallow the floor system, the more equipment can be fit into the ceiling space without increasing story height. Lowering floor-to-floor height will lower the cost. Without drop panels, the total depth of the system is about half that of the steel and one-way slab systems. Square footage cost for this system is also very low compared to the others.

Disadvantages:
This system is heavier than both the steel and one-way systems, meaning foundations will need to be redesigned to support the added weight. Architecturally, floor plans might need to be adjusted in order to account for increased column dimensions. Deflections are also higher in this system than in the others compared.

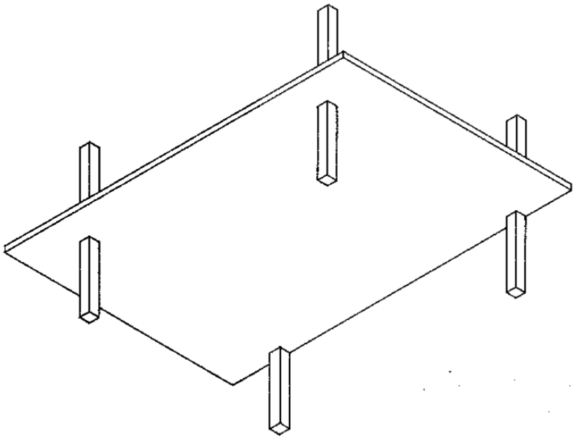


Figure 13 : Two-way flat plate

Viability:

The cost for the flat plate system came out to be $13.72/S.F. This is the lowest cost of the four systems studied. System depth is also the lowest at 13”. Column size would be the same as the one-way slab, 24”x24”. This system would be the best if floor-to-floor height is an issue. Foundations would also need adjusted as this is one of the heaviest systems. Lateral systems would also need updated, as they would change from braced frames to shear walls.

Overall, this system is probably the best alternative considered. The depth, along with the cost, makes it an extremely viable system, and one that should be seriously considered for redesign in the future. Deflections and vibrations should be studied more in depth.

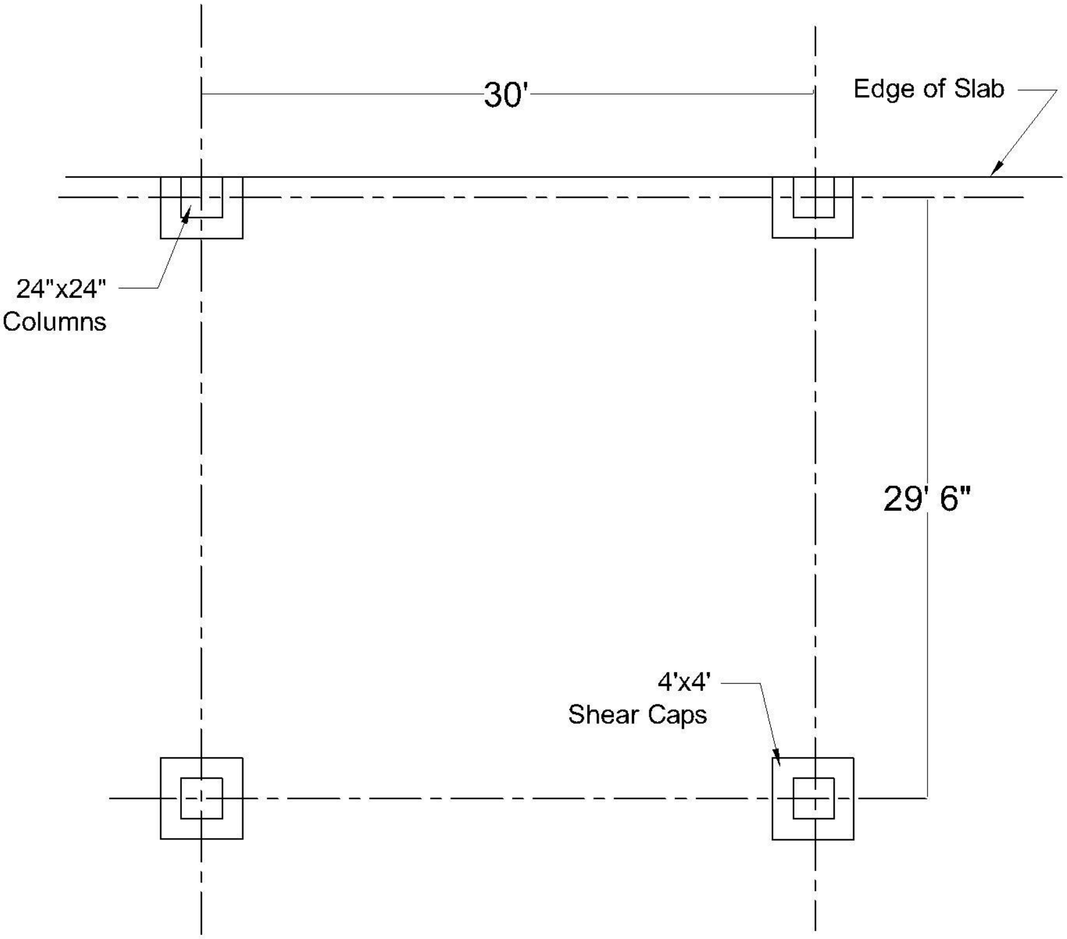


Figure 14 : Typical bay of two-way flat plate

Precast Hollow Core Planks

Finally precast hollow core planks were considered as the final alternative floor system. Using the manufacturer Nitterhouse, a 12" thick plank with 2" concrete topping was picked. The 12" plank was the smallest that would support the applicable loads. The table given for the 12" planks did not include a span of 30'. Therefore, calculations were performed to determine if the plank was adequate to support the given loading, which it was. From there, a prestressed inverse tee-beam was picked to serve as the girder supporting the planks over the 30' span. The girder picked has the smallest available width that was also capable of carrying the load. This turned out to be 40" wide. In place of the prestressed member, a wide flange member could have been used. However, as a girder this would be very inefficient, because it would add the plank thickness to the depth of the girder, giving a very large floor depth. Based on this decision, the precast inverse tee-beam should be used, although the connection to the columns will be abnormal. Using a prestressed concrete beam would also require concrete columns, and therefore, 24"x24" or larger should be used as appropriate, in order to connect the tee-beam. **Detailed calculations are shown in Appendix E.**

System Summary:

- Planks: 12"x4', 2" topping
- Girder: 40IT28-A prestressed inverse tee-beam

Advantages:
The planks offer good deflection control and are able to meet fire rating requirements.

Disadvantages:
The planks are thicker and heavier than other systems. The Prestressed beam has "awkward" connection with columns, and to attach without an overhang on the edges, the columns would need to be enlarged. If column sizes changed too much, that would create a problem with the architecture.

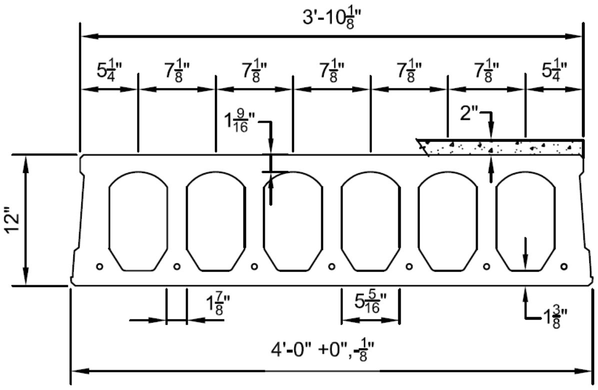


Figure 15 : Hollow core plank section

Viability:

This system would not make a good choice for an alternative system. The cost was approximately $27.61/S.F. making it the most expensive of the compared systems. Weight for this system was also calculated as the highest of the alternatives at 142 psf. Depth was also the greatest at 36". An additional issue also could arise when placing the planks around the columns. They would most likely be cut by the manufacturer, and would probably add additional cost. Finally, foundations and lateral systems would need to be redesigned to correspond to this system because of the weight.

Overall, the planks prove to be the least viable system and will not be considered as an alternative system in the future.

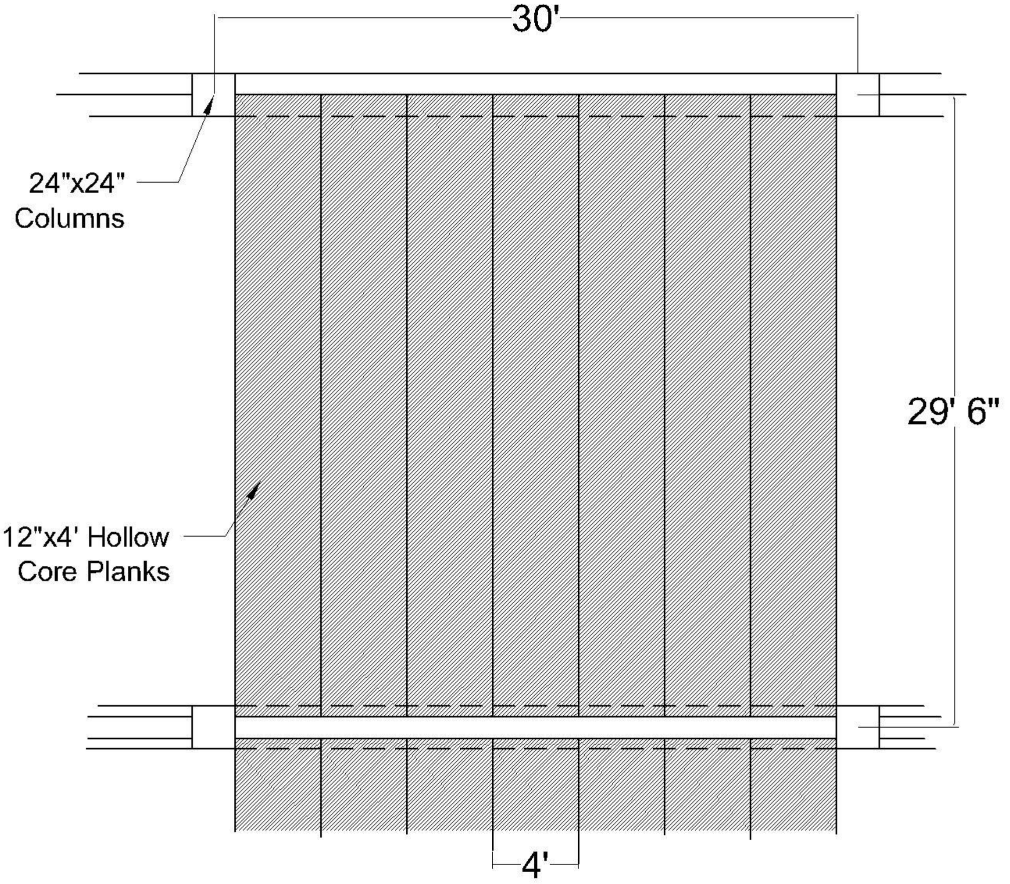


Figure 16 : Typical Bay of Hollow core planks

FLOOR SYSTEMS SUMMARY

Shown below is a comparison of the alternative flooring systems considered. See Appendix F for full cost breakdown.

Consideration		System			
		Composite Steel (Existing)	**One-Way Slab with Beams**	**Two-Way Flat Plate**	**Pre-Cast Hollow Core Planks**
General	**System Cost ($/S.F.)**	16.88	16.21	13.72	27.61
	System Weight (psf)	48	126.4	137.8	142
	System Depth (in)	29.25	27	13	36
Architectural	**Bay Size**	30' x 29'-6"	30' x 29'-6"	30' x 29'-6"	30' x 29'-6"
	Fire Rating (hr)	2	2	2	2
	Floor-to-Floor Height	N/A	Decreased	Decreased	Increased
Structural	**Foundation Impact**	Existing piles	Foundation capacity will need increased	Foundation capacity will need increased	Foundation capacity will need increased
	Lateral System Impact	Existing Braced Frames	Changed to concrete shear walls	Changed to concrete shear walls	Changed to concrete shear walls
Serviceability	**Maximum Deflection (in)**	0.74	0.619	1.1	0.616
	Vibration Control	Average	Very Good	Very Good	Fair
Construction	**Schedule Impact**	N/A	Increased slightly due to beam formwork	Increased slightly due to formwork	Shortened slightly due to easier constructability
	Constructability	Easy	Moderate	Easy	Moderate
Viability		High	Moderate	High	Low

*Cost/S.F. includes material, labor, and equipment (RS Means 2012)

CONCLUSION

This report designed and analyzed three alternative floor systems, and compared them with the existing composite system in the Roberts Pavilion. These alternatives included a one-way slab with beams, a two-way flat plate system, and a precast hollow core plank system. Each system was analyzed based on cost, depth, weight, and impact on the architectural and structural systems.

The existing composite system was found to be a viable option. The cost of the system is competitive with the comparable concrete systems. Steel is a good choice because of the spans achievable, and the economic benefits of using a composite system. Space occupied by columns is also an important consideration when thinking about the architecture. Two issues with the steel are deflection control and the impact of vibrations which would need to be studied further. Still, the composite system remains a good choice.

A one-way slab with beams system is an option to keep in mind. Although it is not as shallow or cost effective as the two-way slab, it does provide good control over vibrations and does not require fireproofing. However, coordination of trades and cost implications make it a less desirable system than two-way system or steel construction.

The two-way flat plate is the most economical alternative that was analyzed and should be seriously considered for the future redesign. It is the most cost efficient, and has the lowest depth, which is good for achieving more space in the ceiling for mechanical and electrical systems, which is very important to consider in a hospital. If this system is designed in more depth, foundations will need to be designed for increased building weight, and deflections will need to be calculated more accurately.

The precast hollow core plank was the least feasible system that was studied. It is very heavy and expensive, in addition to having the largest depth out of the systems as well. Studying the design of this system was beneficial even though this alternative will not be considered for a structure redesign.

Appendix A: Typical Plans

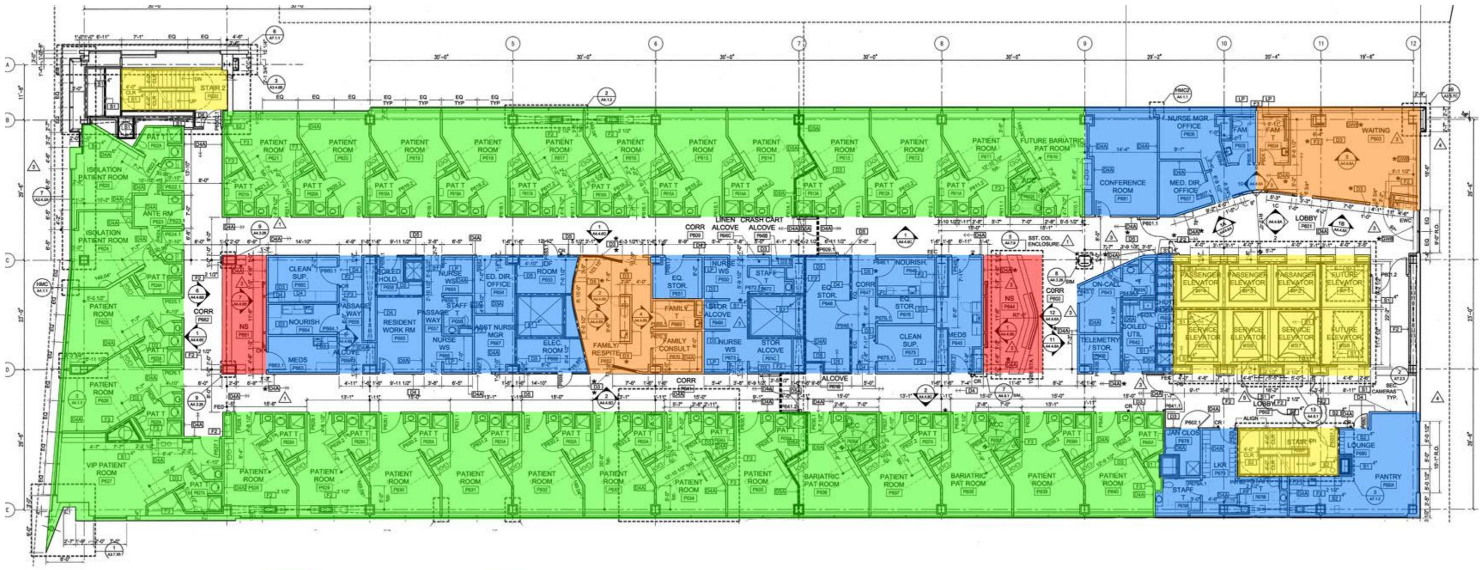


Figure 17 : Typical patient room floor plan

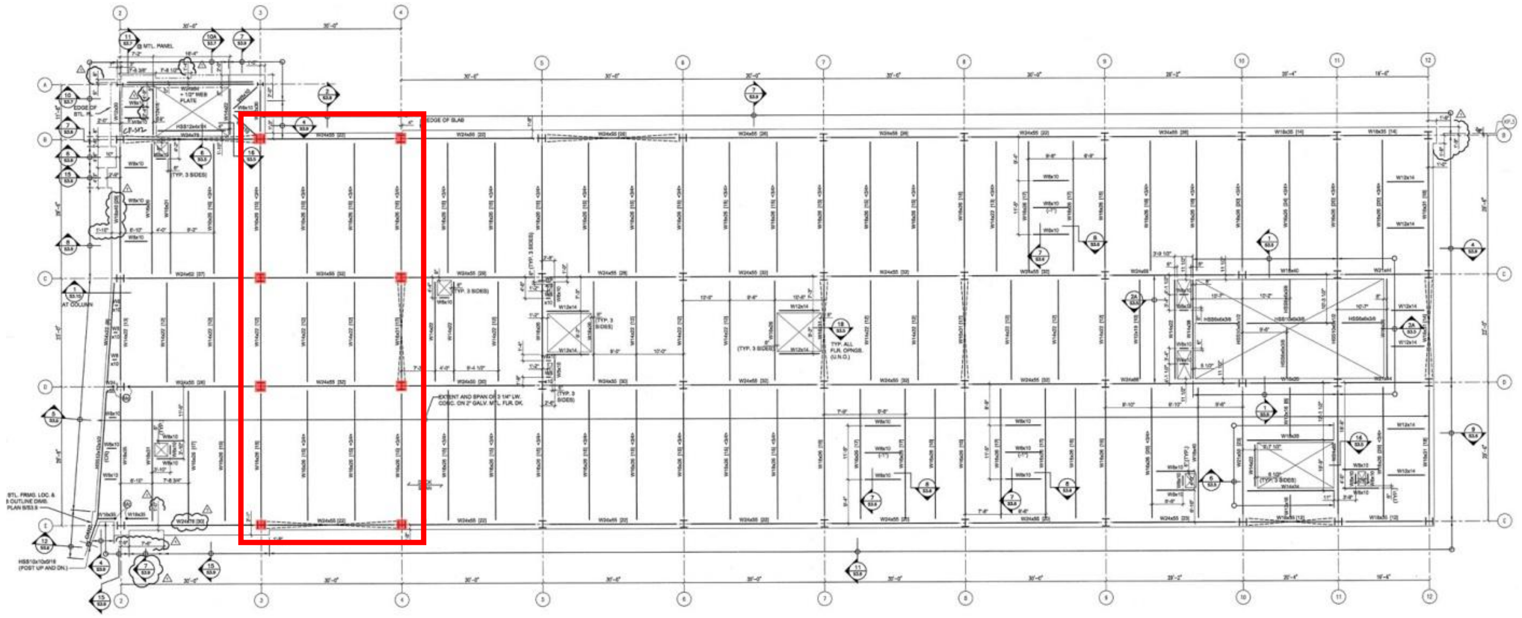


Figure 18 : Typical floor framing plan (typ bay shown)

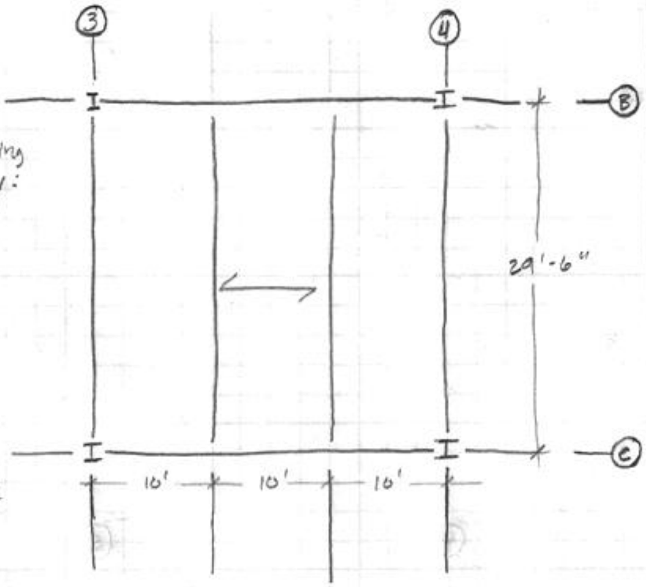
Appendix B: Existing Composite System

SPOT CHECKS: Deck | Tech 1 Report | Andrew Voorhees | 1

Floor Decking

Location: 5th floor
East-side exterior

Typ. Framing Bay:



- 2 hr Fire rating req'd
- composite action

- Live Load
  - patient room: 40 psf
  - corridors above First Floor: 80 psf
  - Moveable Partitions: 20 psf
- Dead Load
  - Framing: 10 psf
  - SDL: 10 psf
  - MEP: 5 psf

$L = \max \left| \begin{array}{l} 40+20 \\ 80+20 \end{array} \right. = 100$ psf

$D = 10 + 10 + 5 = 25$ psf

$1.2D + 1.6L = 1.2(25) + 1.6(100) = 190$ psf

* Assuming sprayed Fiber Fireproofing @ 2hrs. + 3 1/4" LW CONC.

⇒ composite 2VLI deck

Clear span = 10'

2VLI 18 table value = 205 > 190 psf ✓ ok

∴ Use 2VLI 18 decking with 3 1/4" LTWT concrete

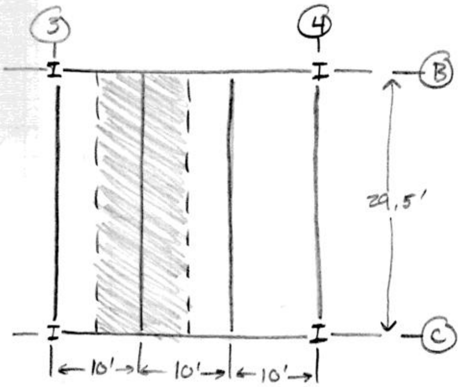
Drawings spec 2" deck, 18 gauge, 3 1/4" LTWT concrete ✓

slab + deck weight = 42 psf

3 span unshored clear span = 12'-7" > 10' ✓ ok
1 span unshored clear span = 10'-6" > 10' ✓ ok

Existing: Composite | Tech Report 2 | Andrew Voorhees | 1

Check Beams



Live Load = 80 psf

Partitions = 20 psf ← consider w/ DL

SDL = 10 psf

MEP = 5 psf

Slab + Deck = 42 psf

self weight = 5 psf

$w_u = 10'[1.6(80) + 1.2(35 + 42 + 5)]$

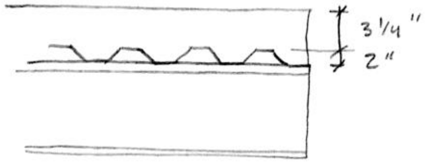
$= 2.264$ klf

$M_u = \frac{w_u l^2}{8} = \frac{2.264(29.5^2)}{8}$

$= 246.28$ k.ft

Assume $a = 0.5''$, $y = 5''$

Assume 1 weak stud / rib $f'_c = 3$ksi LtWT



W16x26 (used in drawings)

$\phi M_n = 274$ k.ft

$\Sigma Q_n = 145$ k

$\frac{145}{17.2} = 8.43 \rightarrow 18$ studs

Check a: $b_{eff} = 2 \times \min \begin{cases} 5 \times 12' = 60'' \\ \frac{29.5 \times 12''}{8} = 44.25 \rightarrow \times 2 = 88.5 \end{cases}$

$a = \frac{154.8^k}{0.85(88.5)(3)} = 0.69'' > 0.5''$ **N.G.**

$\Sigma Q_n = \frac{18}{2} \times 17.2 = 154.8$ k

Try $a = 1'' \Rightarrow Y = 4.75''$

W16x26

$\phi M_n = 271$

$\Sigma Q_n = 145 \rightarrow 18$ studs

check a: $\frac{154.8}{0.85(88.5)(3)} = 0.69'' < a = 1''$ ✓ ok

Existing : Composite | Tech Report 2 | Andrew Voorhees | 2

Check unshored strength

W16 x26 $\quad \phi M_p = 166$

$w_u = 1.4(26) + 1.4(42\times10) = 624.4 \text{ plf}$

$w_u = 1.2(26 + 42\times10') + 1.6(20\times10') = 771.2 \text{ plf}$ ← controls

↑ constr. Load

$$M_u = \frac{(0.7712)(29.5)^2}{8} = 83.9 \text{ kft} < 166 = \phi M_p$$

✓ ok for unshored

Check wet conc. Defl:

$w_{wc} = 42\times10' + 26 = 446 \text{ plf}$

$$\Delta_{wc} = \frac{5}{384}\frac{w_{wc}\ell^4}{EI_x} = \frac{5}{384}\frac{(0.446)(29.5)^4(1728)}{(29{,}000)(301\text{in}^4)} = 0.871''$$

$$\Delta_{wc} = \frac{\ell}{240} = \frac{29.5\times12}{240} = 1.475''$$

0.871" < 1.475" ✓ defl. ok

Check LL defl:

$w_{LL} = 80 \text{ psf } (10') = 0.80 \text{ klf}$

$I_{LB} = 635.5 \text{ in}^4$ @ $Y_2 = 4.75''$ (pt. 6, $\Sigma Q_n = 145$)

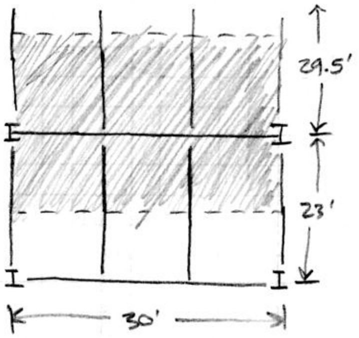
$$\Delta_{LL} = \frac{5}{384}\frac{w\ell^4}{EI} = \frac{5}{384}\frac{(0.80)(29.5^4)(1728)}{(29{,}000)(635.5)} = 0.740''$$

$$\Delta_{LL\,max} = \frac{\ell}{360} = \frac{29.5\times12''}{360} = 0.983''$$

0.983" > 0.740" ✓ ok

✓ LL Defl.s ok

Check Girder



$w_u = 26.25[226.4] + 2(26.25)(26)/30$

$\uparrow$ q_u $\uparrow$ plf

$= 6.00$ klf $+ 50$ plf $= 6.05$ klf

↖ self weight

$M_u = \frac{6.05(30^2)}{8}$

$= 680.63$ k.ft

Assume $a = 1.5$, $Y = 4.5$

W 24×55

$\phi M_n = 713$ k.ft

$\Sigma Q_n = 203^k$

$\frac{203}{17.2} = 11.8 \rightarrow 24$ studs

check a : $b_{eff} = 2\times \text{min} \left| \frac{30\times 12}{8} = 45 \right.$ $= 90''$ ← controls

or $26.25\times 12 = 315$

$a = \frac{206.4}{0.85(90)(3)} = 0.899''$

$\Sigma Q_n = \frac{24}{2}\times 17.2 = 206.4$

$a < 1.5''$ ✓ ok

Check unshored strength

W 24×55 $\phi M_p = 503$

$w_u = 1.4(55) + 1.4(42)(26.25) = 1620.5$ plf

$w_u = 1.2(55 + 42\times 26.25) + 1.6(20\times 26.25) = 2229$ plf ← controls

$M_u = \frac{(2,229)(30^2)}{8} = 250.76$ k.ft < 503

✓ ok for unshored

Existing: Composite	Tech Report 2	Andrew Voorhees	4

Check Wet Conc. Defl.

$W_{WC} = 42 \times 26.25 + 55 = 1157.5 \text{ plf}$

$\Delta_{WC} = \frac{5}{384} \frac{(1.1575)(30^4)(1728)}{(29,000)(1350)} = 0.539''$

$\Delta_{WC} = \frac{L}{240} = \frac{30' \times 12''}{240} = 1.5''$ $1.5'' > 0.539''$ ✓ ok

Check LL Defl.

$w_{LL} = 80(26.25) = 2.1 \text{klf}$

$I_{LB} = 2210$ @ $Y_2 = 4.5$ (pt 7, $\Sigma Q_n = 203$)

$\Delta_{LL} = \frac{5}{384} \frac{(2.1)(30^4)(1728)}{(29,000)(2210)} = 0.597''$

$\Delta_{LL\,max} = \frac{L}{360} = \frac{30 \times 12}{360} = 1''$ $1'' > 0.597''$

✓ LL Defl. ok

For Estimating Purposes:

Use members + studs given in Drawings

Beams: W 16×26 — 15 studs

Girder Int.: W 24×55 — 33 studs

Girder Ext.: W 24×55 — 22 studs

Appendix C: One-Way Slab with Beams

One Way Slab w/ Beams | Tech Report 2 | Andrew Voorhees | 1

Maintain 2 hr. Fire rating → IBC 2009

Min floor thickness (conc. slab) = 4.6" minimum ↓

Min cover for rebar = 3/4" → assuming carbonate aggregate concrete

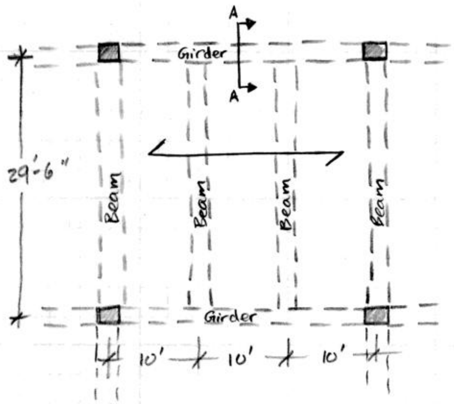
Minimum slab thickness via ACI 318 Table 9.5a

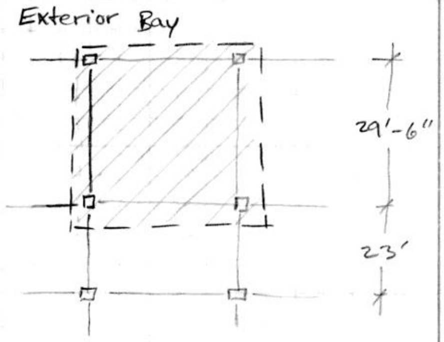
Both ends continuous $\frac{l}{28} = \frac{10' \times 12''}{28} = 4.28 \Rightarrow 5''$

Assume beams at 10' o.c.

Try 5" slab > 4.6" ✓ ok

slab weight $= \frac{5''}{12} \times 150\text{ pcf} = 62.5\text{ psf}$





Live Load : 80 psf
partitions : 20 psf ← More economical to consider as part of DL Low chance that partitions will ever be moved
SDL : 10 psf
MEP : 5 psf
Slab weight : 62.5 psf

Use $f'_c = 4000$ psi
$f_y = 60{,}000$ psi

Estimate Column size

Int. P_u eg from Tech 1 analysis = 1210 k

Take ρ_g as 2% → economical

$$A_g \geq \frac{P_u}{0.4(f'_c + f_y\rho_g)}$$

$$\geq \frac{1210\text{ k}}{0.4(4\text{ k} + 60(0.02))} = 581.7\text{ in}^2$$

Assume cols as 24" × 24"
$A_g = 576\text{ in}^2$

One Way Slab w/ Beams | Tech Report 2 | Andrew Voorhees | 2

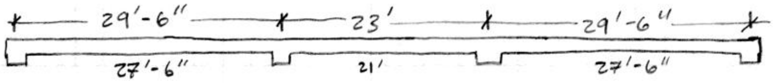
Design Beams : Exterior Bay

DL = 20 + 10 + 5 + 62.5 = 97.5 psf
LL = 80 psf

$q_u = 1.2(97.5) + 1.6(80) = 245$ psf
$w_u = \frac{245}{1000} \times 10' = 2.45$ klf

Section A-A (see previous page)

* Assuming Girders are 24" wide



Critical Design Moments

Pos. Moment End span $= \frac{w_u l_n^2}{11}$

Neg Moment at Ext. Face of First Int. Support $= \frac{w_u l_n^2}{10}$

Neg Moment at Int Face of Ext. support $= \frac{w_u l_n^2}{16}$ or $\frac{w_u l_n^2}{24}$

self weight estimate

Pos Moment : $M_u = \frac{w_u l_n^2}{11} = \frac{2.45(27.5')^2}{11} \times 1.1 = 185.3$ kft

Estimate size:

$bd^2 = 20 M_u$ try $b = 4/5\, d$

$\frac{4}{5} d^3 = 20(185.3) \Rightarrow d = 16.67 \rightarrow 17"$

$h = 17 + 2.5 = 19.5 \rightarrow$ use $h = 20"$, $d = 17.5"$, $b = 16"$

cover + stirrups

$bd^2 = 16(17.5)^2 = 4900$ in^3

Self weight :

$= \frac{16 \times 20}{144} \times 150 = 333.33$ plf

$w_u = 2.45 + 1.2(0.333) = 2.85$ klf

one way slab w/ Beams | Tech Report 2 | Andrew Voorhees | 3

$$M_u^+ = \frac{w_u l_n^2}{11} = \frac{2.85\,(27.5)^2}{11} = 196 \text{ kft}$$

$$M_{u\,int}^- = \frac{w_u l_n^2}{10} = \frac{2.85}{10}\left(\frac{27.5+21}{2}\right)^2 = 168 \text{ kft}$$

Into girder

$$M_{u\,ext}^- = \frac{w_u l_n^2}{24} = \frac{2.85\,(27.5)^2}{24} = 90 \text{ kft}$$

Into col.

$$M_{u\,ext}^- = \frac{w_u l_n^2}{16} = \frac{2.85\,(27.5)^2}{16} = 135 \text{ kft} \leftarrow$$

for simplification purposes design all beams for 135 kft at ext support

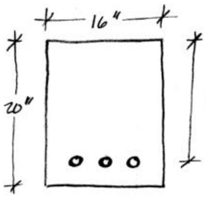
Required Steel : Bottom Reinf.

$$A_s = \frac{M_u}{4d}$$

Bottom Reinf:

$$A_s = \frac{196 \text{ kft}}{4(17.5)} = 2.8 \text{ in}^2$$

try (3) #9 = 3.0 in² = A_s



$$d = 20 - \overset{\text{clear cover}}{1.5''} - \underset{\text{stirrups}}{3/8''} - \overset{d_b}{(1.128/2)} = 17.6''$$

Determine M_n

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{3.0\,(60{,}000)}{0.85\,(4000)(16)} = 3.31''$$

$$c = \frac{a}{\beta_1} = \frac{3.31}{0.85} = 3.89''$$

check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{3.89}(17.6 - 3.89)$$

$$\varepsilon_s = 0.0106 > \varepsilon_y \quad \checkmark \text{ ok}$$

Since $\varepsilon_t > 0.005$
$\phi = 0.9$

$$\phi M_n = \phi A_s F_y (d - a/2) = 0.9(3)(60)\left(17.6 - \frac{3.31}{2}\right) \times \frac{1}{12} = 215 \text{ kft}$$

$$= 215 \text{ kft} > 196 \text{ kft}$$

✓ ok

one way slab w/ beams | Tech Report 2 | Andrew Voorhees | 4

Check Min Steel Area

$$A_{s\,min} \geq \begin{cases} \dfrac{3\sqrt{f'_c}\,bd}{f_y} = \dfrac{3\sqrt{4000}(16)(17.6)}{60\,000} = 0.89 \text{ in}^2 \\ \dfrac{200\,bd}{f_y} = \dfrac{200(16 \times 17.6)}{60\,000} = \boxed{0.94 \text{ in}^2} \end{cases}$$

$A_s > A_{s\,min}$ √ ok

Check Max Reinf.

$$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85(0.85)\frac{4}{60}\,\frac{0.003}{0.007} = 0.0206$$

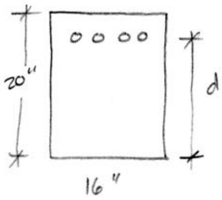
$$\rho = \frac{A_s}{bd} = \frac{3}{16(17.6)} = 0.0107 < 0.0206 \quad \text{√ ok}$$

$\boxed{\therefore \text{ use (3) \#9 bars (see page 14 for summary)}}$

Required steel : Top Reinforcement (Int. support)

$M_u = 168$ kft

$$A_s = \frac{M_u}{4d} = \frac{168}{4(17.5)} = 2.4 \text{ in}^2 \Rightarrow \text{try (4) \#7} \quad A_s = 2.4$$



$$d = 20 - 1.5'' - 3/8'' - \frac{0.875}{2}'' = 17.7''$$

M_n :

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.4(60)}{0.85(4)(16)} = 2.65''$$

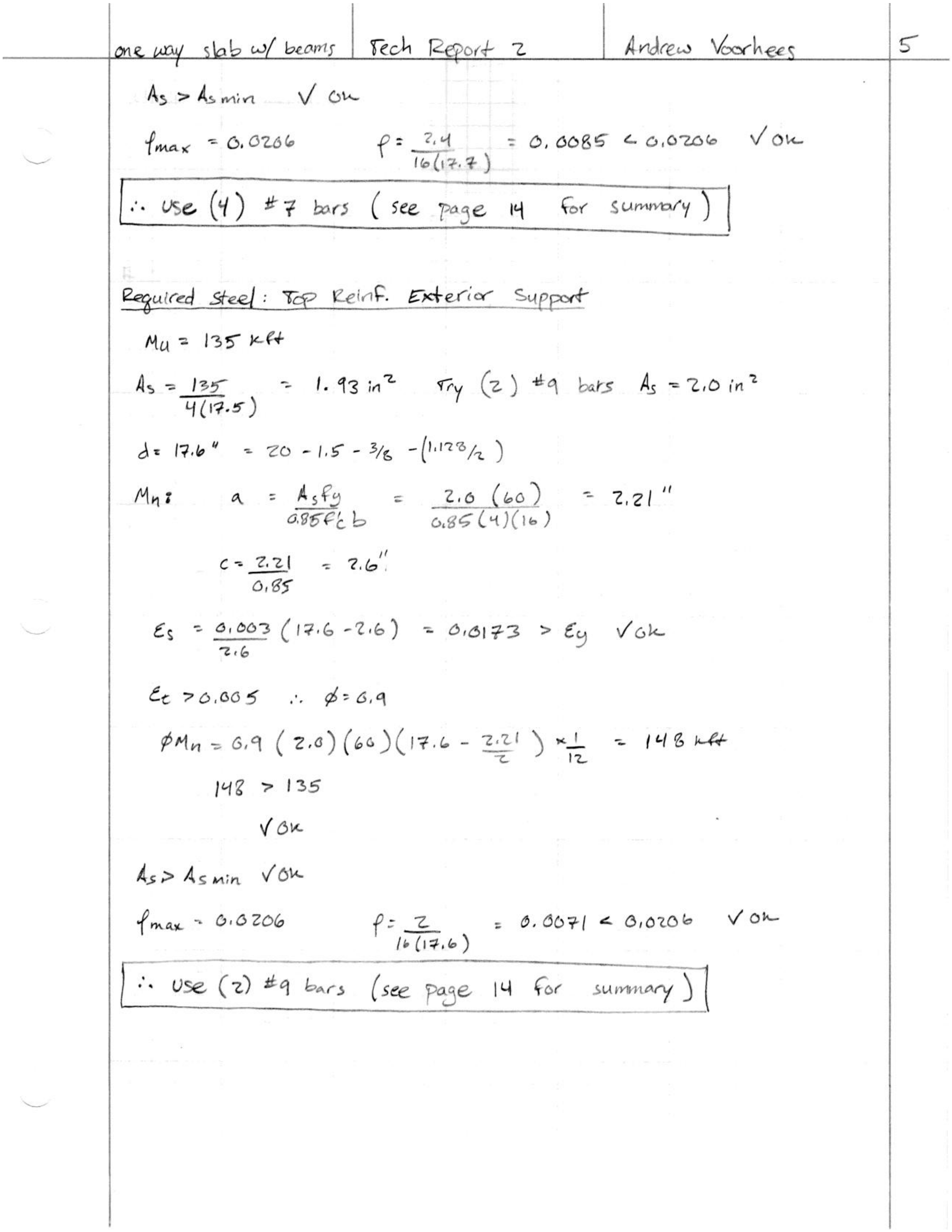
$$c = \frac{2.65}{0.85} = 3.11''$$

check ε_s : $\varepsilon_s = \dfrac{0.003}{3.11}(17.7 - 3.11) = 0.014 > 0.0027 = \varepsilon_y$ √ ok

$\varepsilon_t > 0.005 \;\therefore\; \phi = 0.9$

$$\phi M_n = 0.9(2.4)(60)\left(17.7 - \frac{2.65}{2}\right) \times \frac{1}{12} = 177 \text{ kft} > 168 \text{ kft}$$

√ ok



one way slab w/ beams | Tech Report 2 | Andrew Voorhees | 5

$A_s > A_{s\,min}$ ✓ ok

$\rho_{max} = 0.0206$ $\quad \rho = \frac{2.4}{16(17.7)} = 0.0085 < 0.0206$ ✓ ok

$\boxed{\therefore \text{ Use (4) \#7 bars (see page 14 for summary)}}$

Required Steel: Top Reinf. Exterior Support

$M_u = 135$ kft

$A_s = \frac{135}{4(17.5)} = 1.93\ in^2$ $\quad$ Try (2) #9 bars $\quad A_s = 2.0\ in^2$

$d = 17.6'' = 20 - 1.5 - 3/8 - (1.128/2)$

M_n: $\quad a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.0\ (60)}{0.85(4)(16)} = 2.21''$

$c = \frac{2.21}{0.85} = 2.6''$

$\varepsilon_s = \frac{0.003}{2.6}(17.6 - 2.6) = 0.0173 > \varepsilon_y$ ✓ ok

$\varepsilon_t > 0.005 \quad \therefore \quad \phi = 0.9$

$\phi M_n = 0.9\ (2.0)(60)(17.6 - \frac{2.21}{2}) \times \frac{1}{12} = 148$ kft

148 > 135

✓ ok

$A_s > A_{s\,min}$ ✓ ok

$\rho_{max} = 0.0206$ $\quad \rho = \frac{2}{16(17.6)} = 0.0071 < 0.0206$ ✓ ok

$\boxed{\therefore \text{ Use (2) \#9 bars (see page 14 for summary)}}$

one Way slab w/ Beams | Tech Report 2 | Andrew Voorhees | 6

Size Beams : Interior Bay

M_u @ interior face = $\frac{w_u \ell_n^2}{11}$ controls over midspan = $\frac{w_u \ell_n^2}{16}$

$$\frac{w_u \ell_n^2}{11} = \frac{2.45}{11}\left(\frac{21+27.5}{2}\right)^2 \times 1.1 = 144 \text{ kft}$$

Estimate size:

$bd^2 = 20M_u$ try $b = 4/5\, d$

$\frac{4}{5} d^3 = 20(144) \rightarrow d = 15.3$

$h = 15.3 + 2.5 = 17.8 \rightarrow h = 18''$

$d = 15.5''$

$b = 14''$

other options

b = 16"	h = 20"	h = 18"
h = 16"	d = 17.5"	d = 15.5"
d = 13.5"	b = 10"	b = 14"
A_g = 256 in²	A_g = 200 in²	A_g = 252 in²

Least amount of concrete and most economical

Use d = 17.5"
h = 20"
b = 10"
} Matches depth of exterior bay's beams

self weight = $\frac{10 \times 20}{144} \times 150 = 208.33$

$w_u = 2.45 + 1.2(0.208) = 2.70$ klf

Check deflections of Beams

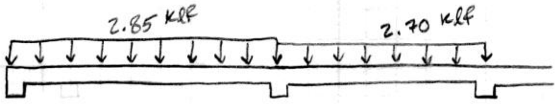
Table 9.5(a)

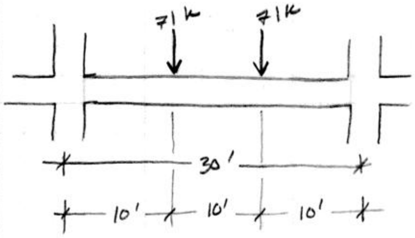
- one end continuous, min $h = \frac{\ell}{18.5} = \frac{29.5}{18.5} \times 12 = 19.14''$

$h = 20'' > 19.14''$

✓ deflections ok

Interior Girder





$2.85 \times \frac{29.5}{2} = 40\text{ k}$

$2.70 \times \frac{23}{2} = 31\text{ k}$

71 k

Equivalent distributed Load

$= \frac{71 \times 2}{28'} = 5.07\text{ klf}$

↑ ln assuming 24" columns

Interior span

$M_u^+ = \frac{w_u\, ln^2}{16}$

$M_u^- = \frac{w_u\, ln^2}{11}$ ← controls

$M_u = \frac{(5.07)(28)^2}{11} \times 1.1 = 398\text{ kft}$

Estimate size

$bd^2 = 20\, M_u$ Columns were taken as 24" therefore take girders at 24" as well.

Try $b = 24"$ $d^2 = \frac{20(398)}{24} \Rightarrow d = 18.2$

use $h = 22"$
$d = 19.5$

$bd^2 = 24\,(19.5^2) = 9126\text{ in}^3$

Self weight

$\frac{24 \times 22}{144} \times 150 = 550\text{ plf}$

$w_u = 5.07\text{ klf} + 1.2\,(0.55) = 5.73\text{ klf}$

$M_u^- = \frac{w_u\, ln^2}{11} = \frac{5.73\,(28)^2}{11} = 408\text{ kft}$

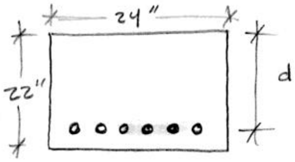
$M_u^+ = \frac{w_u ln^2}{16} = \frac{5.73\,(28)^2}{16} = 281\text{ kft}$

one way slab w/ Beams | Tech Report 2 | Andrew Voorhees | 8

Required Reinforcement : Bottom

$A_s = \frac{M_u}{4d} = \frac{281}{4(19.5)} = 3.6 \text{ in}^2 \Rightarrow$ Try (6) # 7 bars

$A_s = 3.6 \text{ in}^2$



$d = 22 - 1.5" - 3/8" - \frac{0.875"}{2} = 19.7"$

M_n: $a = \frac{3.6(60)}{0.85(4)(24)} = 2.65"$

$c = \frac{2.65}{0.85} = 3.12"$

$\varepsilon_s = \frac{0.003}{3.12}(19.7 - 3.12) = 0.016 > \varepsilon_y$ ✓ ok

since $\varepsilon_t > 0.005$ $\phi = 0.9$

$\phi M_n = 0.9(3.6)(60)(19.7 - \frac{2.65}{2}) \times \frac{1}{12} = 298$ kft

$\phi M_n = 298 > 281$ ✓ ok

$\boxed{\therefore \text{ use (6) \# 7 bars}}$

Required Reinforcement: Top

$A_s = \frac{M_u}{4d} = \frac{408}{4(19.5)} = 5.23 \text{ in}^2$ Try (2) # 10 + (3) # 9 bars

$A_s = 2(1.27) + 3(1.0) = 5.54 \text{ in}^2$

$d = 22 - 1.5 - 3/8 - \left(\frac{1.27}{2}\right) = 19.49" \rightarrow 19.5"$

M_n: $a = \frac{5.54(60)}{0.85(4)(24)} = 4.07"$

$c = \frac{4.07}{0.85} = 4.8"$

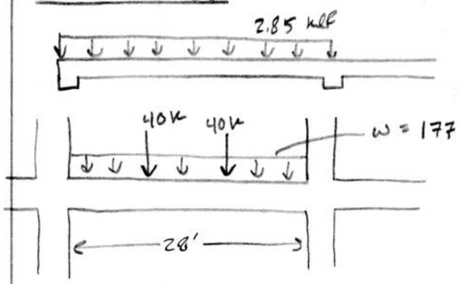
$\varepsilon_s = \frac{0.003}{4.8}(19.5 - 4.8) = 0.009 > 0.0027 = \varepsilon_y$ ✓ ok

$\varepsilon_t > 0.005 \rightarrow \phi = 0.9$

$\phi M_n = 0.9(5.54)(60)(19.5 - \frac{4.07}{2}) \times \frac{1}{12} = 435 \text{ kft} > 408 = M_u$ ✓ ok

$\boxed{\therefore \text{ use (2) \# 10 \& (3) \# 9 bars}}$

Exterior Girder



$w = 177$ lb/ft (curtain wall weight)

$w_u = 1.2(.177) + \frac{20(40)}{28} = 3.07$ klf

$Mu^+ = \frac{wu\,ln^2}{16}$

$Mu^- = \frac{wu\,ln^2}{11} = \frac{3.07(28)^2}{11} \times 1.1 = 241$ kft

Estimate size

$bd^2 = 20\,Mu$

Try $d = 17.5$ $\quad b \rightarrow \frac{20(241)}{(17.5)^2} = 15.7 \rightarrow$ use $b = 16''$

$d = 17.5''$

$h = 20''$

Self weight

$\frac{16 \times 20}{144} \times 150 = 333.33$ plf

$w_u = 3.07 + 1.2(0.333) = 3.47$ klf

$Mu^+ = \frac{3.47(28^2)}{16} = 170$ kft

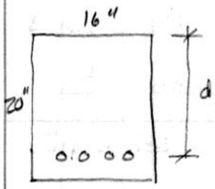
$Mu^- = \frac{3.47(28^2)}{11} = 247$ kft

Required Reinforcement : Bottom

$A_s = \frac{Mu}{4d} = \frac{170 \text{ kft}}{4(17.5)} = 2.43 \text{ in}^2$

Try (4) # 7 bars $\quad A_s = 2.40 \text{ in}^2$

One Way slab w/ Beams | Tech Report 2 | Andrew Voorhees | 10



$d = 20 - 1.5 - \frac{3}{8}'' - \frac{0.875}{2}'' = 17.7''$

$M_n:\quad a = \frac{2.4\,(60)}{0.85\,(4)(16)} = 2.65''$

$c = \frac{2.65}{0.85} = 3.12''$

$\varepsilon_s = \frac{0.003}{3.12}\,(17.7 - 3.12) = 0.014 > \varepsilon_y$ ✓ ok

$\varepsilon_t > 0.005 \rightarrow \phi = 0.9$

$\phi M_n = 0.9\,(2.4)(60)\left(17.7 - \frac{2.65}{2}\right) \times \frac{1}{12} = 171$ kft

$\phi M_n = 171$ kft $> M_u = 170$ ✓ ok

$\boxed{\therefore \text{ use } (4)\ \#7 \text{ bars}}$

Required Reinf. Top

$A_s = \frac{M_u}{4d} = \frac{247}{4\,(17.5)} = 3.53\ \text{in}^2$

Try (2) #10 + (1) #9

$A_s = 2\,(1.27) + 1\,(1.0)$
$= 3.54\ \text{in}^2$

$d = 20 - 1.5 - \frac{3}{8} - \frac{1.27}{2} = 17.5''$

$M_n:\quad a = \frac{3.54\,(60)}{0.85\,(4)(16)} = 3.9''$

$c = 3.9 / 0.85 = 4.59''$

$\varepsilon_s = \frac{0.003}{4.59}\,(17.5 - 4.59) \sim 0.008 > \varepsilon_y$ ✓ ok

$\varepsilon_t > 0.005 \quad \therefore \phi = 0.9$

$\phi M_n = 0.9\,(3.54)(60)\left(17.5 - \frac{3.9}{2}\right) \times \frac{1}{12} = 248$ kft

$\phi M_n = 248 > M_u = 247$ ✓ ok

$\boxed{\therefore \text{ use } (2)\ \#10 + (1)\ \#9}$

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Check defl. + shear in Beams

Beams: $V_{u_{max}} = 1.15 \frac{w_u l_n}{2} = \frac{2.45(27.5)(1.15)}{2} = 38.75^k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2\sqrt{4000}\,(20'')(17.5) = 44.3^k$

$\phi V_c : 33.2^k < V_u$ ∴ shear stirrups will be needed at ext. supports and int supports

Defl: $\frac{l}{18.5} \rightarrow \frac{29.5 \times 12}{18.5} = 19.14'' < 20'' = h$ ✓ ok

Girders:

Interior: $V_u = \frac{w_u l_n}{2} = \frac{5.73\text{ klf}(28)}{2} = 80.22^k$

$V_c = 2\sqrt{4000}\,(24)(19.5) = 59^k \quad \phi V_c = 44.4^k$

$\phi V_c < V_u$ ∴ shear stirrups are needed in interior girder

Defl: $\frac{l}{21} = \frac{30 \times 12}{21} = 17.14'' < 22'' = h$ ✓ ok

Exterior: $V_u = \frac{3.47(28)}{2} = 48.6^k$

$V_c = 2\sqrt{4000}\,(16)(17.5) = 35.4^k \quad \phi V_c = 26.6^k$

$\phi V_c > V_u$ ∴ shear stirrups needed in exterior girder

Defl. : $\frac{l}{21} = \frac{30 \times 12}{21} = 17.14'' < 20'' = h$ ✓ ok

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Slab Design

$t = 5''$

$Mu^- = \frac{w_u l_n^2}{11}$ $q_u = 245 \text{ psf}$

1' thick strip

$= \frac{245 \times 1 \text{ ft} \times (10' - \frac{16''}{12''})^2}{11} = 1.67 \text{ k ft / ft}$

$Mu^+ = \frac{w_u l_n^2}{16} = \frac{245 \times 1 \times (10 - \frac{16}{12})^2}{16} = 1.15 \text{ kft/ft}$

Reinf required per foot

$A_s = \frac{Mu}{4d}$ take $d = 4''$ → Assuming #4 bars + 3/4" C.C.

Bottom reinf: $A_s = \frac{1.15}{4(4)} = 0.072 \text{ in}^2/\text{ft}$

$$A_{s\,min} \geq \begin{cases} \frac{3\sqrt{4000}}{60000}(12)(4) = 0.152 \text{ in}^2/\text{ft} \\ \frac{200(12'')(4)}{60000} = 0.16 \text{ in}^2/\text{ft} \leftarrow \text{controls} \end{cases}$$

For temp. and shrinkage requirements

$\rho_{g\,min} = 0.0018 = \frac{A_s}{bd}$ $A_s = 0.0864 \text{ in}^2$

∴ $A_{s\,min} = 0.16 \text{ in}^2/\text{ft} > \rho_{g\,min}$ ✓ ok

Try #4 bars @ 12" o.c. $A_s = 0.20 \text{ in}^2/\text{ft}$

M_n : $a = \frac{0.2(60)}{0.85(4)(12'')} = 0.29''$

$c = \frac{0.29}{0.85} = 0.34''$

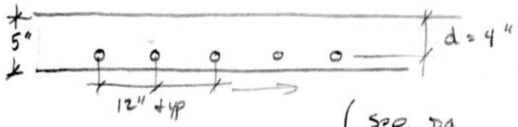
$\varepsilon_s = \frac{0.003}{0.34}(4 - 0.34) = 0.032 > \varepsilon_y$ ✓ ok

$\varepsilon_t > 0.005$ ∴ $\phi = 0.9$

$\phi M_n = 0.9(0.2)(60)(4 - \frac{0.29}{2}) \times \frac{1}{12} = 3.47 \text{ k ft/ft}$

$\phi M_n = 3.47 \text{ kft/ft} > 1.15 \text{ kft/ft}$ ✓ ok

∴ use #4 bars at 12" O.C.



(see pg for summary)

Top Reinforcement

$M_u^- = 1.67$ kft/ft

$A_s = \frac{1.67}{4(4)} = 0.104 \text{ in}^2/\text{ft}$

$A_{s\,min} \geq \begin{cases} \frac{3\sqrt{4000}}{60000}(12)(4) = 0.152 \\ \frac{200(12)(4)}{60000} = 0.16 \leftarrow \text{controls} \end{cases}$

$\rho_{g\,min} = 0.0864 \text{ in}^2/\text{ft}$

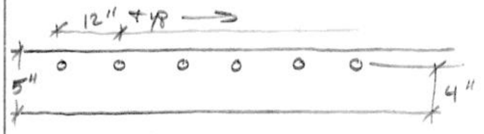
∴ $A_{s\,min} = 0.16 \text{ in}^2/\text{ft} > \rho_{g\,min}$ ✓ ok

Try #4 bars @ 12" O.C. $A_s = 0.20 \text{ in}^2/\text{ft}$

$\phi M_n = \phi M_n$ for bottom reinforcing
$= 3.47$ kft/ft

$\phi M_n = 3.47$ kft/ft > 1.67 kft/ft ✓ ok

use #4 bars at 12" O.C. where necessary for negative M_u



(see page for summary)

Check Shear in Slab

$V_u = \frac{w_u \ell_n}{2} = \frac{245 \text{ psf} \times (10 - \frac{16}{12})}{2} = 1.06^k$ per 1 ft width of slab

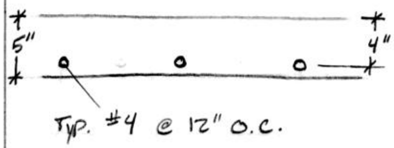
$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2 \times 1\sqrt{4000} \times 12" \times 4" = 6.07^k$ per 1' width

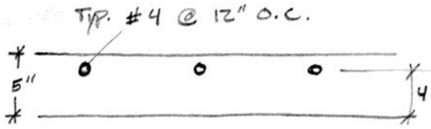
$\phi = 0.75$ $\phi V_c = 4.55 > 1.06 = V_u$ ✓ shear ok

One way slab w/ Beams | Tech Report 2 | Andrew Voorhees | 14

Summary

Slab

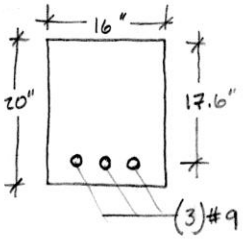




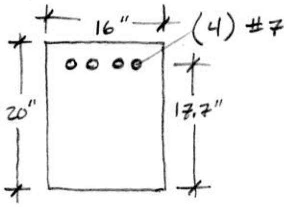
Top Reinf. at Beam intersection

Bottom Reinf. at Midspan

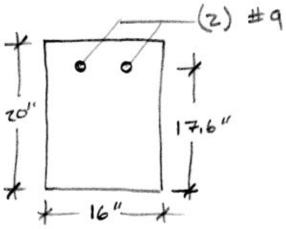
Beams



Bottom Reinf. at Midspan



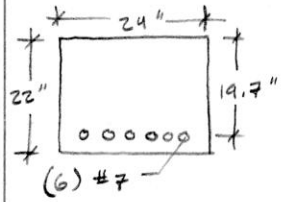
Top reinf. at Interior Support



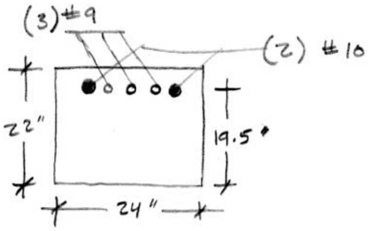
Top Reinf. at Exterior Support

Girders

Interior

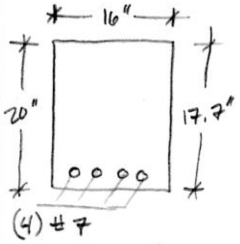


Bottom Reinf. at Midspan

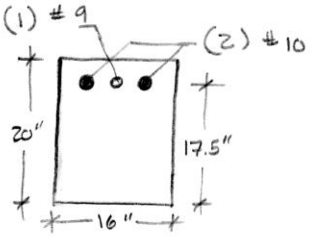


Top Reinf. at column intersection

Exterior



Bottom Reinf at Midspan



Top Reinf. at Column Intersection

Size of ext. girder
= size of beams
⇓
Due to beams are end span ∴ larger Moment
Girder is continuous ∴ lower Moment
(see report body for discussion)

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Deflections : Assume simply supported

$$\Delta_{LL} = \frac{5}{384}\frac{(0.8)(30^4)(1728)(1000)}{(57000\sqrt{4000})(\frac{1}{12}(16)(20^3))} = 0.379''$$

$$\Delta_{LL\,max} = \frac{30' \times 12}{360} = 1.0''$$

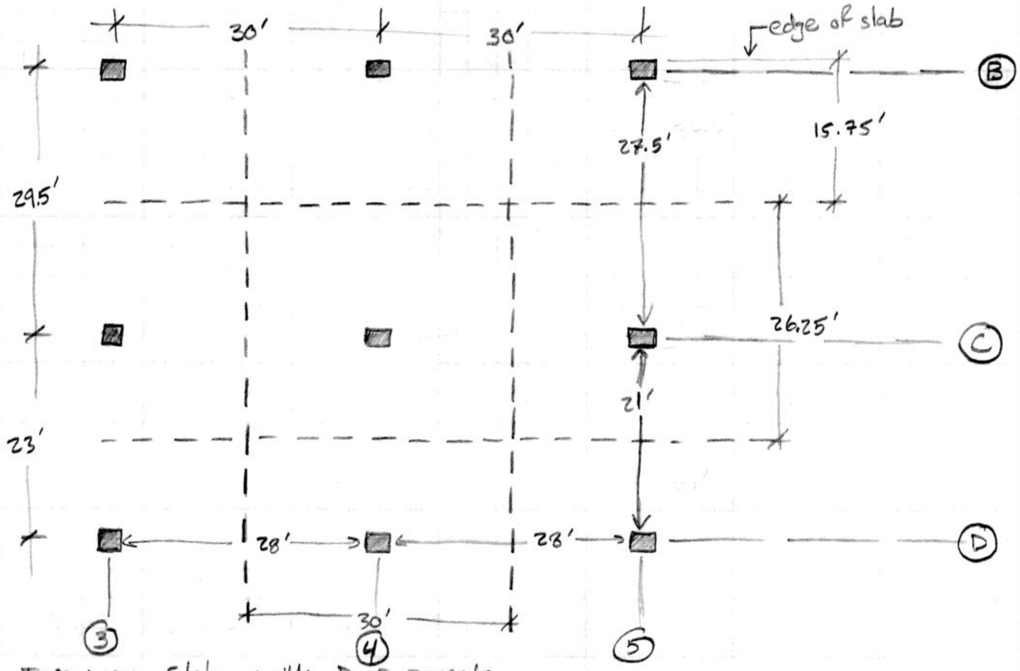
0.379 < 1.0 ✓ ok

$$\Delta_{TL} = \frac{5}{389}\frac{[10(97.5) + 333](30^4)(1728)}{(57000\sqrt{4000})(\frac{1}{12})(16)(20^3)} = 0.619''$$

$$\Delta_{TL} = \frac{30 \times 12}{240} = 1.5 > 0.619'' \checkmark \text{ok}$$

Appendix D: Two-Way Flat Plate Slab

Using the Direct Design Method



Two way slab with Drop panels

From ACI 318-11 Table 9.5 (c)

Minimum slab Thickness ($f_y = 60{,}000$) *Assuming col. dimensions 24"×24"

Ext. Panels w/o edge bms $l_n/33 = 28'/33 \times 12" = 10.2"$

Int. Panels $l_n/36 = 28 \times 12/36 = 9.33"$

controls

use $t = 11" > 4"_{min}$

Self weight from slab

$= (11/12) \times 150 = 137.5$ psf

$q_u = 1.6(80) + 1.2(10 + 20 + 5 + 137.5) = 335$ psf

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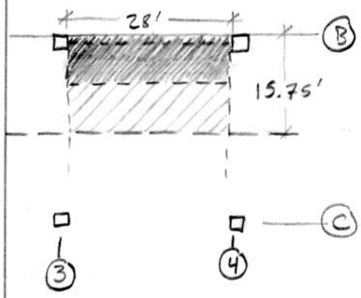


$$M_o = \frac{q_u l_2 l_n^2}{8}$$

$$= \frac{(0.335)(15.75)(28)^2}{8}$$

$$= 517 \text{ k.ft}$$

Considered an interior span

Distribute M_o :

$M_u^- = 0.65(517) = 336$ k.ft

$M_u^+ = 0.35(517) = 181$ k.ft

Distribute to Strips

Col. strip: $\frac{1}{4}(29.5) = 7.4' + 1' = 8.4'$

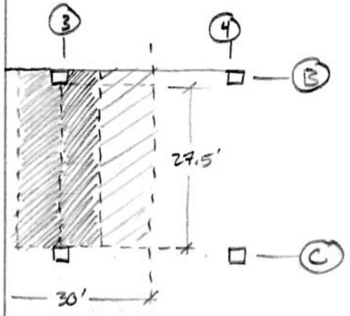
$\alpha = 0 \Rightarrow \quad M_u^- = 0.75(336) = \boxed{252 \text{ k.ft}}$

$M_u^+ = 0.60(181) = \boxed{109 \text{ k.ft}}$

Middle strip : $15.75' - 8.4' = 7.35'$

$M_u^- = 336 - 252 = \boxed{84 \text{ k.ft}}$

$M_u^+ = 181 - 109 = \boxed{72 \text{ k.ft}}$



$$M_o = \frac{(0.335)(30')(27.5)^2}{8} = 950 \text{ k.ft}$$

Distribute M_o :

Ext $M_u^- = 0.26(950) = 247$ k.ft

$M_u^+ = 0.52(950) = 494$ k.ft

Int $M_u^- = 0.70(950) = 665$ k.ft

Distribute to Strips

Col. Strip: $\frac{1}{4}(29.5) = 7.4' \times 2 = 14.75'$

Middle strip $= 30 - 14.75 = \frac{15.25}{2} = 7.6'$ on each side

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Col. Strip: Ext. $M_u^- = 1.0(247) = \boxed{247 \text{ k.ft}}$

$\alpha = 0$

$\beta_t = 0$

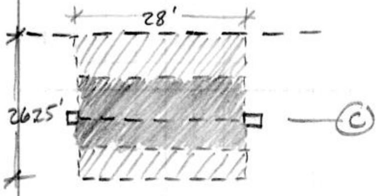
$M_u^+ = 0.6(494) = \boxed{296 \text{ k.ft}}$

Int. $M_u^- = 0.75(665) = \boxed{499 \text{ k.ft}}$

Middle strip: Ext. $M_u^- = \boxed{0 \text{ k.ft}}$

$M_u^+ = 494 - 296 = \boxed{198 \text{ k.ft}}$

Int. $M_u^- = 665 - 499 = \boxed{166 \text{ k.ft}}$ → half to each middle strip



$$M_o = \frac{(0.335)(26.25)(28^2)}{8} = 862 \text{ k.ft}$$

Distribute M_o

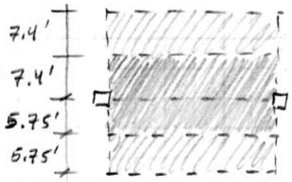
$M_u^- = 0.65(862) = 560 \text{ kft}$

$M_u^+ = 0.35(862) = 302 \text{ kft}$

Distribute to Strips

Col. strip $= \frac{1}{4}(29.5) + \frac{1}{4}(23) = 13.13'$

Middle strip $= 13.12'$



Col. Strip: $M_u^- = 0.75(560) = \boxed{420 \text{ k.ft}}$

$M_u^+ = 0.60(302) = \boxed{181 \text{ kft}}$

Middle strip: $M_u^- = 560 - 420 = \boxed{140 \text{ kft}}$ → } half to each strip

$M_u^+ = 302 - 181 = \boxed{121 \text{ kft}}$ →

For long direction Try #5 bars

$d = h - 3/4 - \frac{0.625}{2} = 9.94''$

for short direction

$d = h - 3/4 - \left(\frac{0.625}{2}\right)3 = 9.31''$

Design Reinforcement * Assuming #5 bars

$M_n = M_u / \phi$; $\phi = 0.9$

$R = \frac{M_n \times 12000}{bd^2}$

ρ (From Table A-3 : Wight & MacGregor)

$A_s = \rho b d$

$A_{s,min} = 0.0018 bt$

N (number of bars) $= \frac{A_s}{0.31}$ ← controlling value; 0.31 ← A_s for #5 bar

$N_{min} = \frac{b}{2t}$

Span in Long direction (Along Line B or C)

	Col. Strip (Edge)		Middle Strip		Col. Strip (Interior)	
	M_u^-	M_u^+	M_u^-	M_u^+	M_u^-	M_u^+
M_u (kft)	252	109	70+84 = 154	72+60.5 = 132.5	420	181
b (in)	8.4′ x 12″ = 100.8′		(7.4 + 7.35) x12″ = 177″		(7.4 + 5.75) x12 = 157.5″	
d (in)	9.94″		9.94		9.94	
M_n (kft)	280	121	171	147	467	201
R (psi)	337.4	145.8	117.3	100.9	360.12	155.0
ρ (%)	0.0059	0.0025	0.0019	0.0016	0.0064	0.0026
A_s (in²)	5.91	2.50	3.34	2.82	10.02	4.07
$A_{s,min}$ (in²)	2.00	2.00	3.50	3.50	3.12	3.12
N (# of bars)	20	9	12	12	33	14
N_{min}	5	5	9	9	8	8

$\rho_{max} = 0.0206$ ✓ ok

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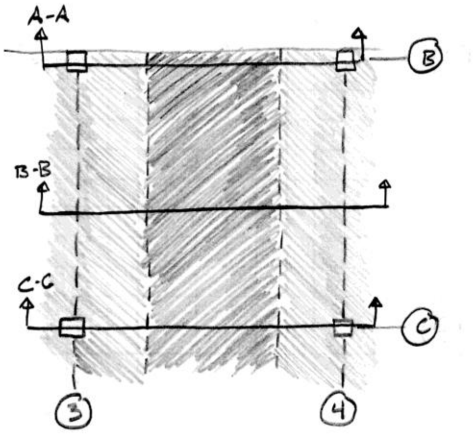
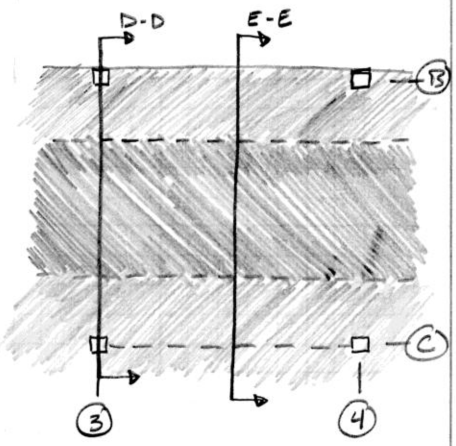
Span in Short Direction (along Line 3 or 4)

	Column Strip			Middle Strip		
	Int M_u^-	M_u^+	Ext M_u^-	Int M_u^-	M_u^+	Ext M_u^-
M_u (k-ft)	499	296	247	166	198	0
b (in)	14.75′ × 12″ = 177″			15.25′ × 12″ = 183″		
d (in)	9.31″			9.31″		
M_n (k-ft)	554	329	274	184	220	0
R (psi)	433.33	257.34	214.32	139.20	166.44	—
ρ (%)	0.0078	0.0045	0.0037	0.0023	0.0028	—
A_s (in²)	12.85	7.42	6.10	3.92	4.77	—
$A_{s,min}$ (in²)	3.50	3.50	3.50	3.62	3.62	—
N (# of bars)	42	24	20	13	16	—
N min	9	9	9	9	9	—

$\rho_{max} = 0.0206$ ✓ ok





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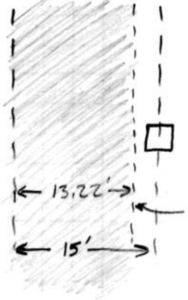
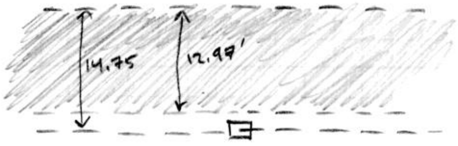
Check One-Way Shear



$l_2 = 26.25'$

← critical section at $d = 9.31''$

$b \times l_2 = 26.25 \times 13.22$
$= 347 \text{ ft}^2$



$l_2 = 30'$

$b \times l_2 = 30 \times 12.97$
$= 389 \text{ ft}^2$
↑
controls

$q_u = 0.335$

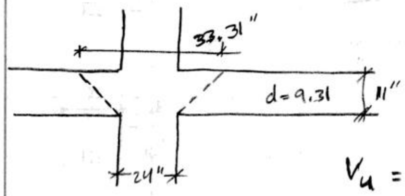
$V_u = 0.335 \times 389 = 130.3^k$

$V_c = 2\lambda\sqrt{f'c}\, bd = 2\sqrt{4000}\,(12.97 \times 12)(9.31)\ /1000 = 183.3^k$

$\phi V_c = 0.75(183.3) = 137.5^k > V_u = 130.3^k$

✓ ok

Check Two-Way Shear



b_o around the column
$= 2(24 + 24 + 2(9.31)) = 133.24 \text{ in}$

$V_u = 0.335\left[(26.25 \times 30') - \left(\frac{33.31}{12}\right)^2\right] = 261.2^k$

$$V_c \le \begin{cases} 2 + 4/\beta = 6 \\ \frac{\alpha_s d}{b_o} + 2 = \frac{40(9.31)}{133.24} + 2 = 4.8 \\ 4 \leftarrow \text{controls} \end{cases}$$

$\alpha_s = 40$ (int. col.
$\beta = 1$

$\phi V_c = 4(0.75)(1)\sqrt{4000}\,(133.24)(9.31) = 235.4^k$

$\phi V_c = 235.1^k < 261.2 = V_u$ N.G.

Try shear cap to increase shear strength

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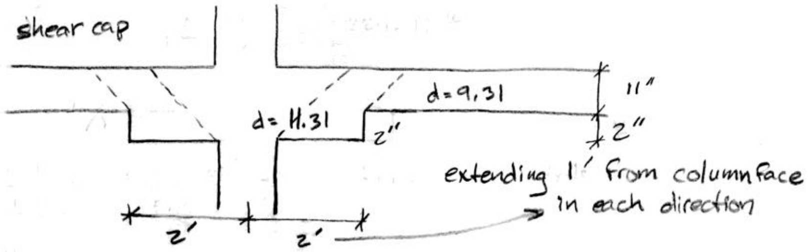
Check two-way shear with shear cap

Try a 2" shear cap



b_o around column = $2(24+24+2(11.31)) = 141.24 \text{ in} = 35.31'' \times 35.31''$

b_o around cap = $2(48+48+2(9.31)) = 229.24 \text{ in} \sim 57.31'' \times 57.31''$

q_{du} cap = $1.2\left(\frac{2''}{12''}\right)(150) = 30 \text{ psf}$

$$V_u = 0.335\left[(26.25\times 30) - \left(\frac{35.31}{12}\right)^2\right] + 0.03\left[4^2 - \left(\frac{35.31}{12}\right)^2\right]$$
$$= 261.13^k$$

$$V_c \le \begin{cases} 2 + 4/\beta = 6 \\ \frac{\alpha_s d}{b_o} + 2 = \frac{40(11.31)}{141.24} + 2 = 5.2 \\ 4 \leftarrow \text{controls} \end{cases}$$

$$\phi V_c = 4(0.75)(1)\sqrt{4000}(141.24)(11.31) = 303.10^k$$

$\phi V_c > V_u$ ✓ ok

Check shear around cap

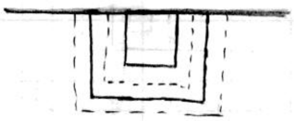
$$V_u = 0.335\left[(26.25\times 30) - \left(\frac{57.31}{12}\right)^2\right] = 256.17^k$$

$$V_c \le \begin{cases} 2 + 4/\beta = 6 \\ \frac{\alpha_s d}{b_o} + 2 = \frac{40(9.31)}{229.24} + 2 = 3.62 \leftarrow \text{controls} \\ 4 \end{cases}$$

$$\phi V_c = 3.62(0.75)\sqrt{4000}(229.24)(9.31)$$
$$= 366.47^k$$

$\phi V_c > V_u$ ✓ ok

Check exterior column w/ shear cap



Use same dimensions as interior shear cap

b_o around column $= (24 \times 3) + 2(11.31) = 94.62$

b_o around cap $= (3+3+4) \times 12 + 2(9.31) = 138.62$ in

b_o around col
$= 29.655'' \times 35.31''$

b_o around cap
$= 57.31'' \times 40.655''$

$$V_u = 0.335\left[(15.75 \times 30) - \left(\frac{29.655 \times 35.31}{12}\right)\right] + 0.03\left[(3 \times 4) - \left(\frac{29.655 \times 35.31}{12}\right)\right]$$
$$= 129.2^k$$

$$V_c \le \begin{cases} 2 + 4/\beta = 6 \\ \frac{\alpha_s d}{b_o} + 2 = \frac{30(11.31)}{94.62} + 2 = 5.58 \\ 4 \leftarrow \text{controls} \end{cases} \qquad \alpha_s = 30 \text{ : Edge column}$$

$$\phi V_c = 4(0.75)\sqrt{4000}\,(94.62)(11.31) = 203.05^k$$

$\phi V_c > V_u$ ✓ ok

Check around cap

$$V_u = 0.335\left[(15.75 \times 30) - \left(\frac{57.31 \times 40.655}{12}\right)\right] = 93.24^k$$

$$V_c \le \begin{cases} 2 + 4/\beta = 6 \\ \frac{30(9.31)}{138.62} + 2 = 4.01 \\ 4 \leftarrow \text{controls} \end{cases}$$

$$\phi V_c = 4(0.75)\sqrt{4000}\,(138.62)(9.31) = 244.87^k$$

$\phi V_c > V_u$ ✓ ok

Deflection Check:

t= 11"

67.5 % of M_u to Col. strip
32.5 % of M_u to Mid strip

DL= 172.5 psf
LL= 80 psf

Immediate Deflection due to DL

Col strip: $w_D = 172.5\text{ psf}(30)(0.675) = 3.5 \text{ k/ft}$

$$I_g = \frac{(13.13' \times 12'')(11'')^3}{12} = 17476 \text{ in}^4$$

$$E_c = 57000\sqrt{4000} = 3605 \text{ ksi}$$

$$\Delta_{D(max)} = \frac{0.0026(3.5)(30^4)(1728)}{(3605)(17476)} = 0.202''$$

Middle strip

$$w_D = (172.5)(30')(0.325) = 1.682 \text{ k/ft}$$

$$I_g = \frac{(13.13 \times 12)(11)^3}{12} = 17476 \text{ in}^4$$

$$\Delta_{D(max)} = \frac{0.0026(1.682)(30^4)(1728)}{3605(17476)} = 0.097''$$

Immediate Δ_{DL}
due to Total DL $= 0.202'' + 0.097''$ $\boxed{= 0.30''}$

Immediate defl. due to total Live Load:

Col. strip $w_L = (80)(30)(0.675) = 1620 = 1.62 \text{ k/ft}$

$$\Delta_{L max} = \frac{0.0048(1.62)(30^4)(1728)}{3605(17476)} = 0.173''$$

Middle Strip $w_L = (80)(30)(0.325) = 0.9 \text{ k/ft}$

$$\Delta_{L max} = \frac{(0.0048)(0.9)(30^4)(1728)}{3605(17476)} = 0.096''$$

Total Immediate Δ due to LL_{total}
$\Delta = 0.173'' + 0.096''$ $\boxed{= 0.27''}$

Two - way slab | Tech Report 2 | Andrew Voorhees | 10

Additional α_Δ after time

assume $\lambda = 3.0$

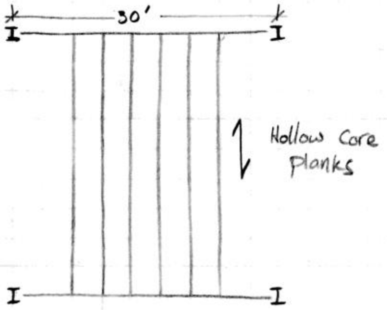
$\Delta_{max, D} = 3(0.36 + 0.25(0.27)) = 1.1025''$

check against Table 9.5b

Live Load: $l/360 = \frac{30 \times 12}{360} = 1.0'' > 0.27''$

✓ ok

Appendix E: Precast Hollow-Core Plank



Live Load = 80psf
MEP = 5 psf
SDL = 10 psf
Partitions = 20 psf

Maintain a 2 hr Fire rating

Try 12" x4' hollow core plank
w/ 2" topping

span = 30' → value not on chart
∴ calculate capacity

Panel weight = 77psf + 25 psf = 102 psf
↖ topping

use (7) 1/2"Ø Pattern

*Tables require superimosed Load to be considered as Live Load

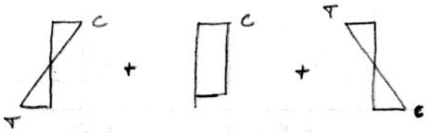
$w_u = 1.6(80) + 1.2(20+102+5+10)$

$= 292.4 \text{ psf} \times 4' = 1.17 \text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{1.17\,(30')^2}{8} = 131.63 \text{ kft} = 1579.5 \text{ kin}$

$P/\text{strand} = 0.6(270)(0.153) = 24.8 \text{ k/strand}$
↖ for 1/2" Ø

$f = \frac{M}{S} + \frac{P}{A} + \frac{Pe}{S}$



$f_{top} = \frac{1579.5}{1653} + \frac{24.8(7)}{361} - \frac{24.8(7) \times 5.51}{1653} = 0.8577$

$= 857.76 \text{ psi} < 0.60f'_c = 3600 \text{psi}$
✓ok

$f_{bottom} = \frac{-1579.5}{1081} + \frac{24.8(7)}{361} + \frac{24.8(7) \times 5.51}{1081} = -0.095 \text{ ksi} = 95.40 \text{psi}$

$95.40 < 10\sqrt{f'_c} = 774.6 \text{psi}$

✓ok

Check Deflections

$E_c = \frac{1}{1000}(57{,}000)\sqrt{6000} = 4415$

$$\Delta_{LL} = \frac{5(1.6)(0.08\,\text{ksf})(4')(30')^4(1728)}{384(4415)(7840)} = 0.270'' < \frac{L}{360} = 1'' \quad \checkmark\text{ok}$$

$$\Delta_{TL} = \frac{5\left[1.2(35+102)+1.6(80)\right]\left(\times\frac{1}{1000}\right)(4')(30')^4(1728)}{(384)(4415)(7840)} = 0.616'' < \frac{L}{240} = 1.5'' \quad \checkmark\text{ok}$$

$$M_u = \frac{\left[1.2(35+102)+1.6(80)\right]\left(\frac{1}{1000}\right)(4)(30')^2}{8} = 131.6\text{ kft}$$

131.6 kft < M_n = 235.4 kft at 60% jacking force

√ ok

* Because value for 30' span is just off the table, assume the planks will pass in shear as well.

∴ use 12" hollow core with 2" topping + (7) ½" ø strands

Size Girder

*size interior girder – supports two spans

$q_u = 1.6(80\text{psf}) + 1.2(102+35) = 0.2924$ ksf

$w_u = (15' \times 0.2924) + \frac{1}{2}(23')(0.2924) = 7.75$ klf

$$I_{reqd,LL} = \frac{5\left[1.6(26.5' \times 80/1000)\right](30^4)(1728)}{384(4415)(30 \times 12/360)} = 14{,}002\text{ in}^4$$

$$I_{reqd,TL} = \frac{5(7.75)(30^4)(1728)}{384(4415)(30 \times 12/240)} = 21{,}328\text{ in}^4$$

Use Prestressed Concrete Inverted Tee Beam

From Nitterhouse 40IT28 w/ Strand pattern 16-0-0 + (4) #9 top bars

from table w_u = 8.2klf > 7.75 klf √ ok
@ 30' span

M_u = 19,237 k in = 1603 k.ft

$$M_u = \frac{(7.75\text{ klf})(30^2)}{8} = 871.38\text{ kft} \leq 1603 \quad \checkmark\text{ ok}$$

Hollow Core Conc. Planks | Tech Report 2 | Andrew Voorhees | 3

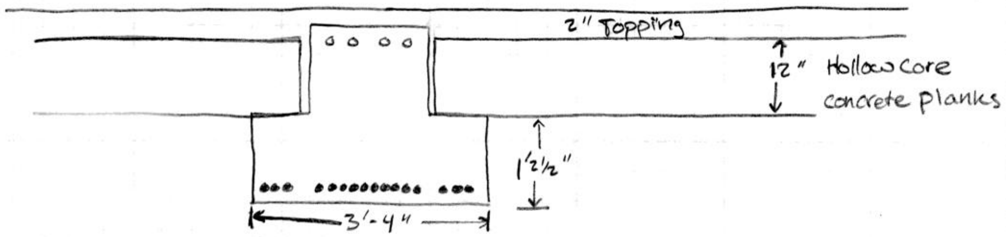
$I = 55{,}827 \text{ in}^4$ ∴ defl. ok

$$\Delta_{LL} = \frac{5[1.6(26.5 \times 0.08)](30^4)(1728)}{384(4415)(55{,}827)} = 0.251'' < 1'' \ \checkmark \text{ ok}$$

$$\Delta_{TL} = \frac{5[(7.75)](30^4)(1728)}{384(4415)(55{,}827)} = 0.573'' < 1.5'' \ \checkmark \text{ ok}$$

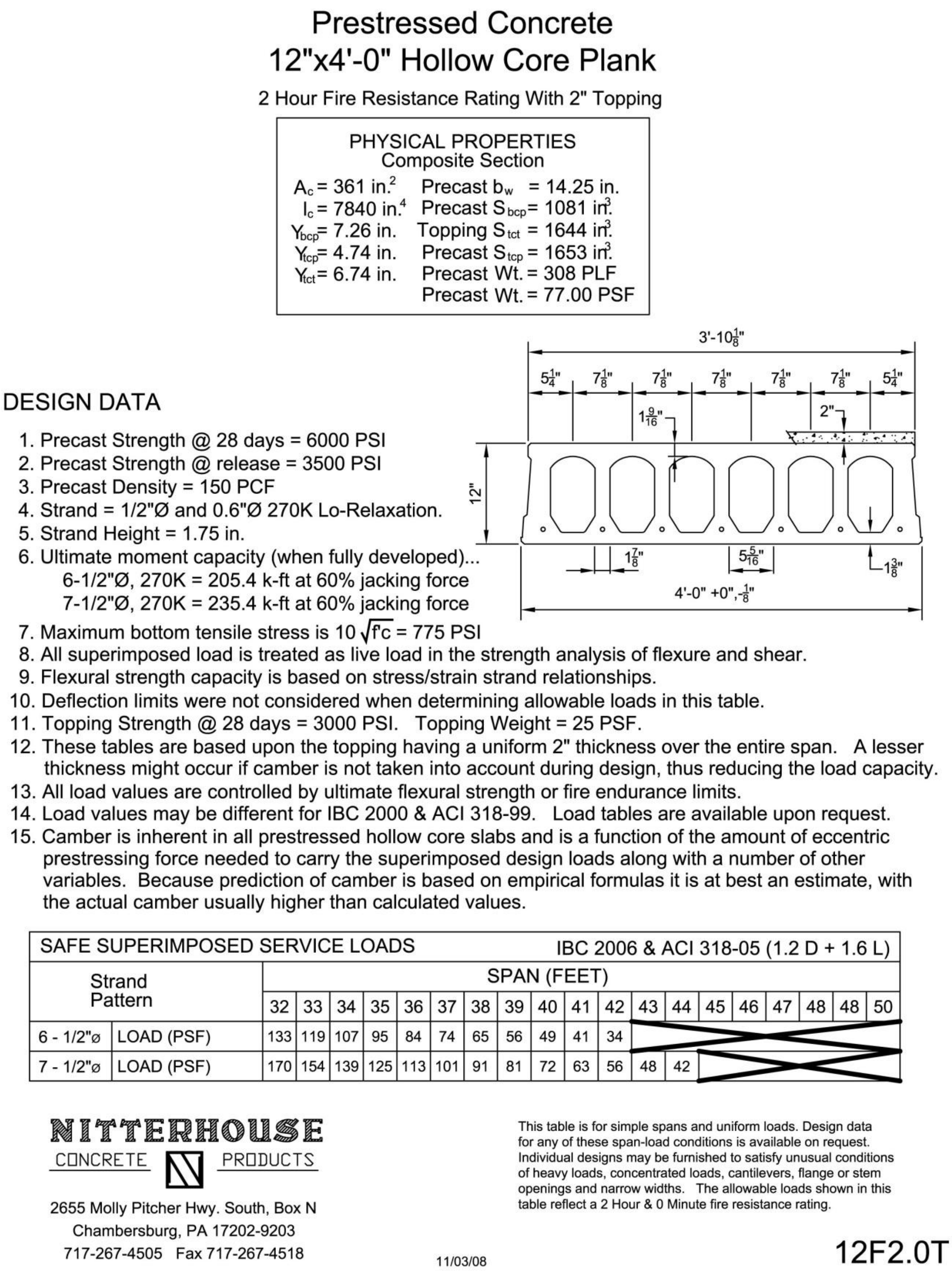
∴ Use Prestressed Concrete Inverted Tee Beam 40IT 28-A

Beams Frame directly into columns



Total system Depth

$$= (1'2\tfrac{1}{2}'') + 12'' + 2'' = \boxed{2'4\tfrac{1}{2}''}$$



Prestressed Concrete
12"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 361 in.^{2}
I_c = 7840 in.^{4}
Y_{bcp}= 7.26 in.
Y_{tcp}= 4.74 in.
Y_{tct}= 6.74 in.
Precast b_w = 14.25 in.
Precast S_{bcp}= 1081 in.^{3}
Topping S_{tct} = 1644 in.^{3}
Precast S_{tcp} = 1653 in.^{3}
Precast Wt. = 308 PLF
Precast Wt. = 77.00 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 205.4 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 235.4 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. All load values are controlled by ultimate flexural strength or fire endurance limits.
14. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
15. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	48	50
6 - 1/2"ø	LOAD (PSF)	133	119	107	95	84	74	65	56	49	41	34								
7 - 1/2"ø	LOAD (PSF)	170	154	139	125	113	101	91	81	72	63	56	48	42						

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

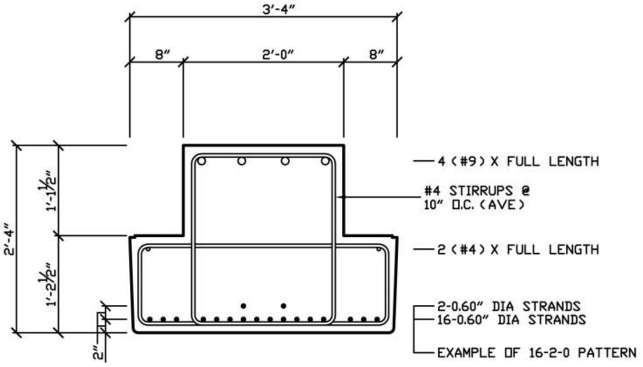
11/03/08

12F2.0T

Prestressed Concrete
Inverted Tee Beam 40IT28-A

PHYSICAL PROPERTIES

A = 904 in.^{2}
I = 55,827 in.^{4}
Y_b = 12.27 in.
Y_t = 15.73 in.
S_b = 4,551 in.^{3}
S_t = 3,549 in.^{3}
Wt.= 942 PLF



DESIGN DATA

1. Precast Strength @ 28 days = 6,000 PSI
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF
4. Strand = 0.60"Ø 270K Lo-Relaxation.
5. Ultimate moment capacity shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is $12\sqrt{f'c}$ = 930 PSI
7. Flexural strength capacity is based on stress/strain strand relationships and is slightly variable.
8. Deflection limits were not considered when determining allowable loads in this table.
9. All superimposed live loads listed are controlled by ultimate flexural strength, not allowable stresses.
10. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

11. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
12. The concrete strength at release of prestress force increases to 4,500 psi for more than 18 strands.

ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF) IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern	Top Bars	Moment Capacity	SPAN													
			16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'
8 - 0 - 0	2 - #9	10,180 "k	13.1	10.5	8.9	7.7	6.5	5.5	4.7	4.0	3.4	2.9	2.5	2.2	1.9	1.6
16 - 0 - 0	4 - #9	19,237 "k	25.5	20.6	17.4	15.3	13.0	11.1	9.5	8.2	7.1	6.2	5.4	4.8	4.3	3.8
16 - 2 - 0	4 - #9	20,952 "k	27.8	22.5	19.1	16.8	14.2	12.2	10.4	8.9	7.8	6.8	6.0	5.3	4.7	4.2
16 - 6 - 0	6 - #9	24,735 "k	33.0	26.8	22.7	20.0	16.9	14.5	12.4	10.7	9.4	8.2	7.2	6.4	5.7	5.1

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

04/04/08

40IT28-A

Appendix F: Cost Estimates

Composite Beams, Decking						
	Unit	Quantity	Material	Labor	Equipment	Cost Total
Welded wire fabric, sheets 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F.	C.S.F.	8.85	0.14	0.23	0.00	0.36
Structural concrete, placing, elevated slab, pumped, less than 6" thick	C.Y.	11.59	0.00	0.23	0.07	0.30
Structural concrete, ready mix, lightweight, 110 # / C.F., 3000 psi	C.Y.	11.59	1.74	0.00	0.00	1.74
Concrete finishing, floors	S.F.	885	0.00	0.56	0.03	0.59
Concrete surface treatment, curing, sprayed membrane compound	C.S.F.	8.85	0.07	0.06	0.00	0.13
Weld shear connector, 3/4" dia x 3-7/8" L	Ea.	100	0.06	0.10	0.05	0.21
Structural steel, A992, W24x55	L.F.	60	5.12	0.24	0.10	5.46
Structural steel, A992, W16x26	L.F.	88.5	3.60	0.27	0.15	4.02
Metal floor decking, steel, non-cellular, composite, galvanized, 2" D, 18 gauge	S.F.	929.25	2.46	0.49	0.04	2.99
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	L.F.	30	0.12	0.04	0.01	0.17
Sprayed fireproofing, cementious, normal density, beams, 2 hour rated	S.F.	660	0.40	0.44	0.07	0.90
		Total	**13.71**	**2.65**	**0.52**	**16.88**

One Way Slab						
	Unit	Quantity	Material	Labor	Equipment	Cost Total
C.I.P concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use	SFCA	90	0.08	0.68	0.00	0.76
C.I.P concrete forms, beams and girders, interior, plywood, 12" wide, 4 use	SFCA	543	0.61	3.34	0.00	3.95
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use	S.F.	765	0.89	3.18	0.00	4.07
Reinforcing steel, in place, elevated slabs, #4 to #7, A615, grade 60	Ton	0.3006	0.36	0.18	0.00	0.54
Reinforcing steel, in place, Beams & Girders #3 to #7, A615, grade 60	Ton	0.56524	0.63	0.63	0.00	1.25
Reinforcing steel, in place, Beams & Girders #8 to #18, A615, grade 60	Ton	0.35912	0.40	0.24	0.00	0.63
Structural concrete, ready mix, normal weight, 4000 psi	C.Y.	27.62	3.21	0.00	0.00	3.21
Structural concrete, placing, elevated slab, pumped, less than 6" thick	C.Y.	13.66	0.00	0.23	0.09	0.32
Structural concrete, placing, beams, small, pumped	C.Y.	13.96	0.00	0.54	0.20	0.75
Concrete finishing, floors	S.F.	885	0.00	0.56	0.03	0.59
Concrete surface treatment, curing, sprayed membrane compound	C.S.F.	8.85	0.07	0.06	0.00	0.13
		Total	**6.25**	**9.64**	**0.32**	**16.21**

Two Way Slab						
	Unit	Quantity	Material	Labor	Equipment	Cost Total
C.I.P concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use	SFCA	885	1.03	3.68	0.00	4.71
C.I.P concrete forms, elevated slab, edge forms, alternate pricing, 7" to 12", use 4	SFCA	110	0.02	0.48	0.00	0.50
Reinforcing steel, in place, elevated slabs, #4 to #7, A615, grade 60	Ton	2.04950	2.43	1.25	0.00	3.68
Structural concrete, ready mix, normal weight, 4000 psi	C.Y.	30.1111	3.50	0.00	0.00	3.50
Structural concrete, placing, elevated slab, pumped, over 10" thick	C.Y.	30.1111	0.00	0.46	0.15	0.60
Concrete finishing, floors	S.F.	885	0.00	0.56	0.03	0.59
Concrete surface treatment, curing, sprayed membrane compound	C.S.F.	8.85	0.07	0.06	0.00	0.13
		Total	**7.06**	**6.48**	**0.18**	**13.72**

Hollow Core Concrete Planks						
	Unit	Quantity	Material	Labor	Equipment	Cost Total
C.I.P concrete forms, elevated slab, bulkhead with keyway, 2 piece, 1 use	SFCA	885	1.85	4.12	0.00	5.97
C.I.P concrete forms, elevated slab, edge forms, to 6" height, use 4	SFCA	20	0.00	0.06	0.00	0.06
Welded wire fabric, sheets 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F.	C.S.F.	8.85	0.14	0.23	0.00	0.36
Structural concrete, ready mix, normal weight, 3000 psi	C.Y.	5.46	0.63	0.00	0.00	0.63
Structural concrete, placing, elevated slab, pumped, under 6"	C.Y.	5.46	0.00	0.11	0.03	0.14
Concrete finishing, floors	S.F.	885	0.00	0.56	0.03	0.59
Concrete surface treatment, curing, sprayed membrane compound	C.S.F.	8.85	0.07	0.06	0.00	0.13
Precast concrete beam, 6000 psi, T-shaped, 30' span, 28"x40"	Ea.	1	5.75	0.16	0.09	6.00
Precast concrete beam, 6000 psi, L-shaped, 30' span, 28"x32"	Ea.	1	5.00	0.16	0.09	5.25
Precast slab, roof/floor members, grouted, hollow, 12" thick, prestressed	S.F.	812	7.25	0.80	0.42	8.47
		Total	**20.69**	**6.26**	**0.66**	**27.61**