TECHNICAL REPORT 2

Structural Study of Alternate Floor Systems

Penn State Hershey Medical Center Children's Hospital

Hershey, Pennsylvania

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Structural Option

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October 27, 2010

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Figures and Tables

Sources:

Cover page: exterior rendering was provided by the architectural team at Payette Associates Inc.

All structural plans and detailed sections of the existing system used in this report were taken from the structural drawings provided from the structural engineers at Gannett Fleming Inc.

The framing rendering in Figure 9 was provided from a Autodesk Revit model of the structural system provided by the construction team at L.F. Driscoll Company LLC.

The figures for the hollow core plank system were provided from the *Nitterhouse Pre-stressed Concrete Products* catalog.

Figures 14 and 15 diagraming the structural system layouts are from the *Structure Point* software solutions website.

Executive Summary

The objective of Technical Report 2 is to investigate the alternative floor systems for the Penn State Hershey Medical Center Children's Hospital. To achieve this objective, this report will focus on various criteria to determine which alternative floor systems are feasible. The existing floor system is composite slab supported by composite beams. The three alternative floor systems this report will focus on include:

- Pre-Cast Hollow Core Planks
- One Way Pan Joist System
- One Way Slab and Beam

An introduction to the structural systems is provided to summarize some of the existing conditions and structural concepts. These conditions are subdivided into separate sections to explore the foundation, floor, roof, and lateral systems. A list of building codes and materials used in the design is also provided for reference in the analysis that follows.

The existing system consists of a 2" deep, 20-gage composite metal deck with a 4 ½" topping thickness. Supporting the slab are typical W16x26 composite steel beams welded with ¾" diameter shear studs. An average bay size of 19' x 34.5' was considered in designing the alternative floor systems. The pre-cast hollow core planks were designed using the *Nitterhouse Pre-stressed Catalog* under the applied superimposed loads. A 6" x 4' span plank with 7 – ½" diameter strands was determined to be sufficient. The one way pan joist system was designed using ACI 318-05 – "Building Code Requirements for Structural Concrete." A 66|6 skip joist with a depth of 14" was initially selected with a 4.5" slab thickness. The beams were designed for a width of three feet and a depth of 14". The last system was selected to be a one way slab and beam system. Using ACI 318-05, it was determined that a 6' wide beam that was 10" deep could support a 6" slab under the applied loads. All hand calculations that were performed for this report are included in the appendix.

Each of the floor systems were then compared to one another with regards to depth, cost, weight, as well as other determining factors. A comparison chart can be found on page 19 of this report. It was determined through this investigation that both the one way pan joist and the one way slab and beam systems were the most feasible. Since this report will consider gravity loading only, a lateral study for both these systems is needed to determine which system is the more viable alternative. Technical Report 3 will focus on the lateral system analysis which will provide more insight into the feasibility of these structural systems.

Building Overview

The new Penn State Hershey Medical Center Children's Hospital is located at 500 University Drive in Hershey, Pennsylvania. The Children's Hospital is an expansion project on the existing Cancer Institute and Main Hospital. The overall project plan calls for a five story, 263,556 square-foot addition which will contain a number of operating rooms, offices, and patient rooms specializing in pediatric care. The exterior of the building utilizes vision glass and an aluminum curtain wall system. The main curve of the façade helps to tie the building into the existing curve along the Cancer Institute. A vegetated roof garden will be situated on the third level above the existing Cancer Institute. See Figure 1 for a site plan of the Children's Hospital.

The dates of construction for the Children's Hospital are scheduled for March 2010 to August 2012. The drawing specifications for the Children's Hospital note that an additional two floors of occupancy are intended for a later date. The range of this thesis project will be limited to the structural analysis of the Children's Hospital.

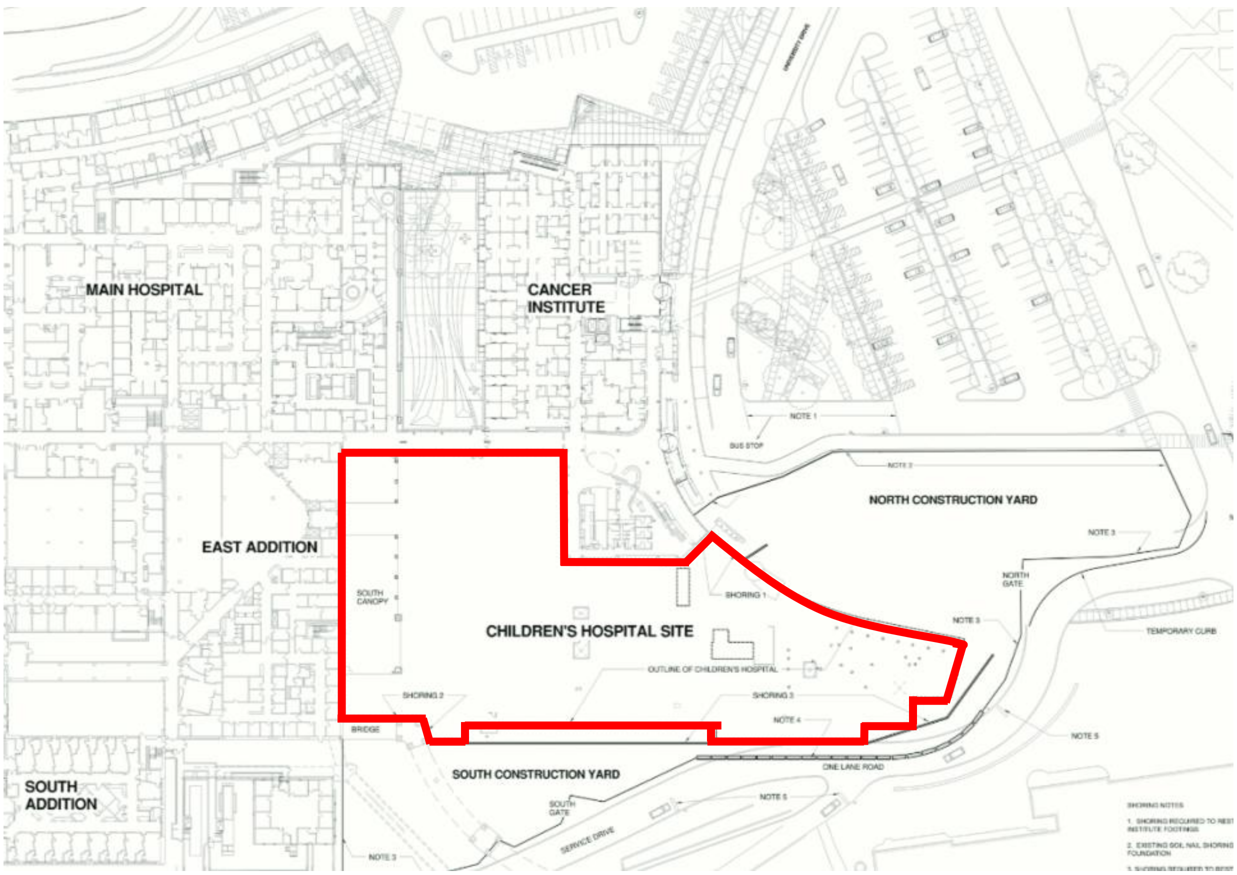


Figure 1 – Site Plan

Introduction to Structural System

The primary structural system comprises of structural steel framing integrated with a composite floor system. The composite floor consists of metal decking with normal weight concrete topping. Shear studs are welded to the supporting beam and embedded into the slab allowing interaction between the two elements. Transfer girders help to transmit the gravity loads from the beams to the columns. All of the columns consist of W14 members which allow for easier constructability. The lateral force resisting system consists of moment connected frames along the East-West direction while diagonal bracing members assist in North-South bracing.

Foundation

Due to the potential for excessive settlement, micropiles were utilized as recommended in the Geotechnical Report provided by CMT Laboratories. Micropiles consist of a casing that is injected with grout to create a friction bond within the bond zone. The piles that are used in the design are specified for a compression load of 280kips and a tension capacity of 170 kips. There are over 600 micropiles that were used in the foundation of the structure. See Figure 2 for a detail section of a typical micropile.

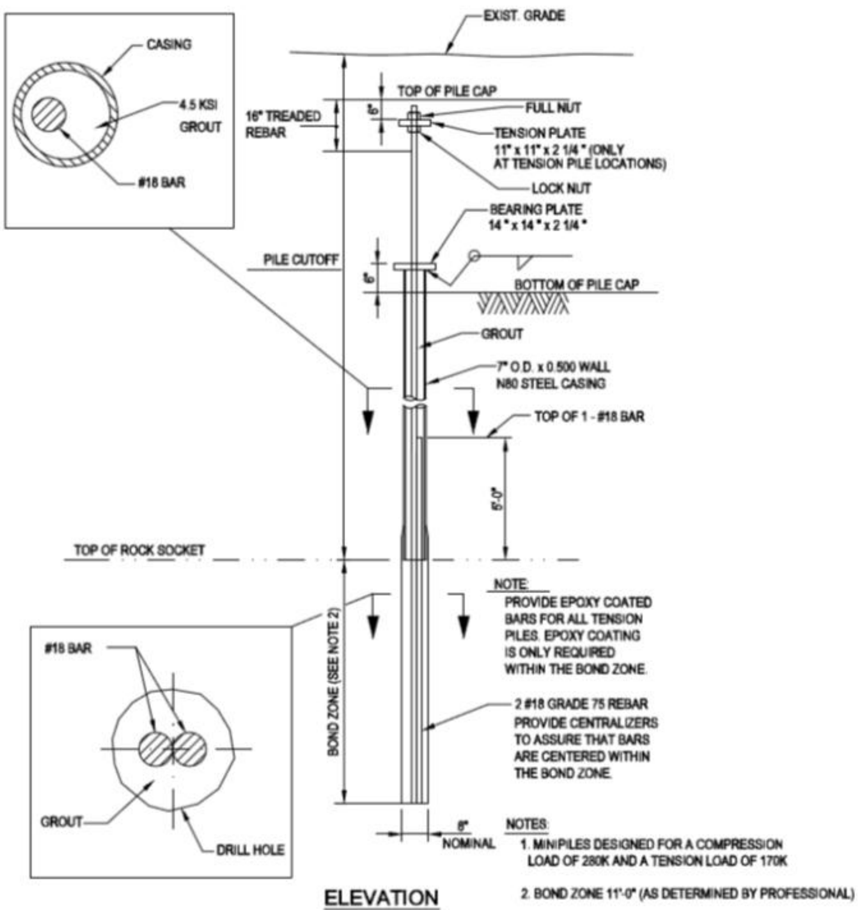


Figure 2 - Micropile Detail

The micropiles are grouped into various sizes of pile caps ranging from 3'0" x 3'0" to 10'0" x 15'0" with a depth ranging from 3' 6" to 6' 0". An example of a typical pile cap can be seen in Figure 4. Typical strut beams of 1' 6" wide by 2' 8" deep span between all pile caps to provide resistance to lateral column base movement. See "Figure 3 – Typ. Strut Beam" below.

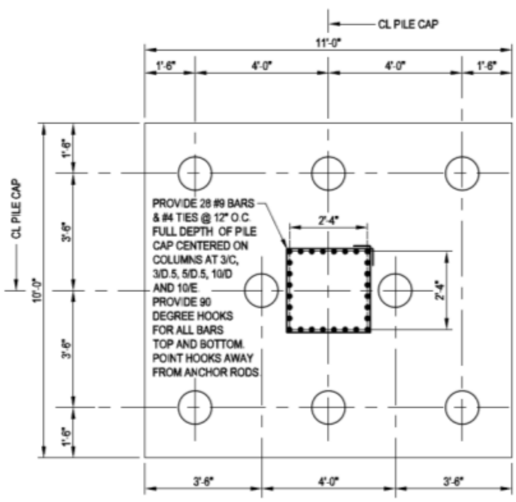


Figure 4 - P8 Pile Cap Plan

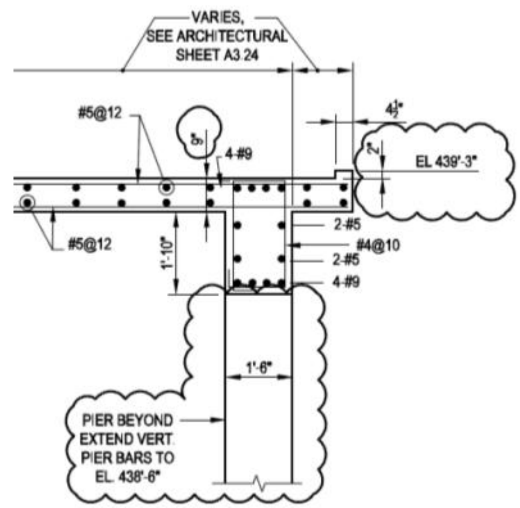


Figure 3 - Typ. Strut Beam

The floor at the ground level is a 5" concrete slab while in heavier load areas such as elevator pits and mechanical rooms a slab thickness of 6" is used. Below is an overview of the West End foundation plan.

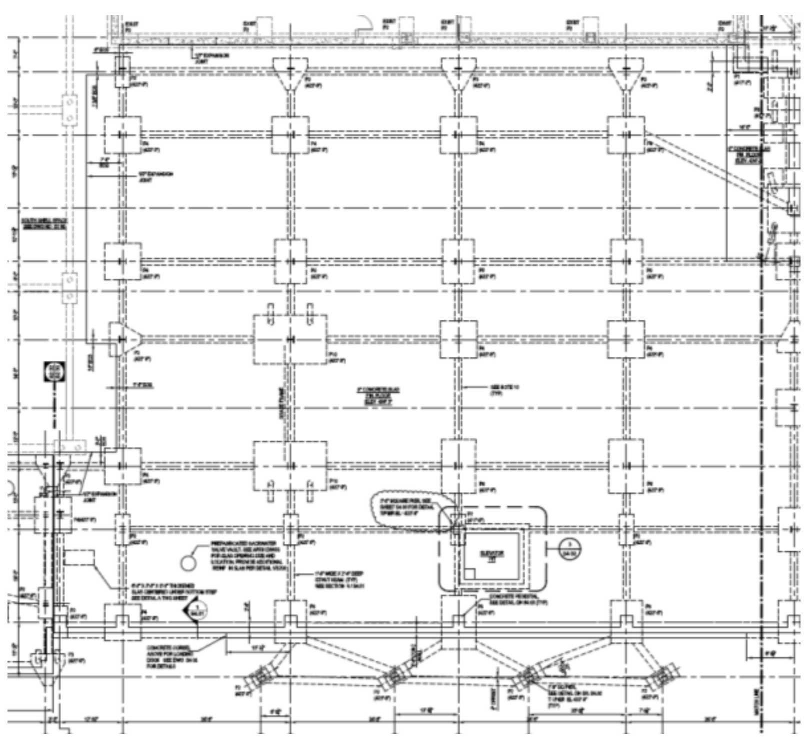


Figure 5 - West End Foundation Plan

Floor System

The typical floor slab throughout all five stories consists of a composite floor system denoted on structural drawings as S1 TYP. This slab type is comprised of a 2" deep, 20-gage composite metal deck with a 4 ½" topping thickness. The reinforcement within the slab is 6x6 W2.1xW2.1 Welded Wire Fabric. The only change in slab thickness occurs at an area on Level 2 marked as having a slab type of S2 TYP (see Figure 6). Here, a 6" concrete slab sits on a 2" deep, 20 gage composite deck with 6x6 W2.9xW2.9 Welded Wire Fabric. The main reason behind increasing the slab thickness in this area is to account for a future MRI space where the live load is considered to be 215 PSF. All floor slabs are connected to wide flange beams using ¾" diameter shear studs where the number of studs is listed on each beam in the framing plans. The typical span for a wide flange beam is 34' 6".

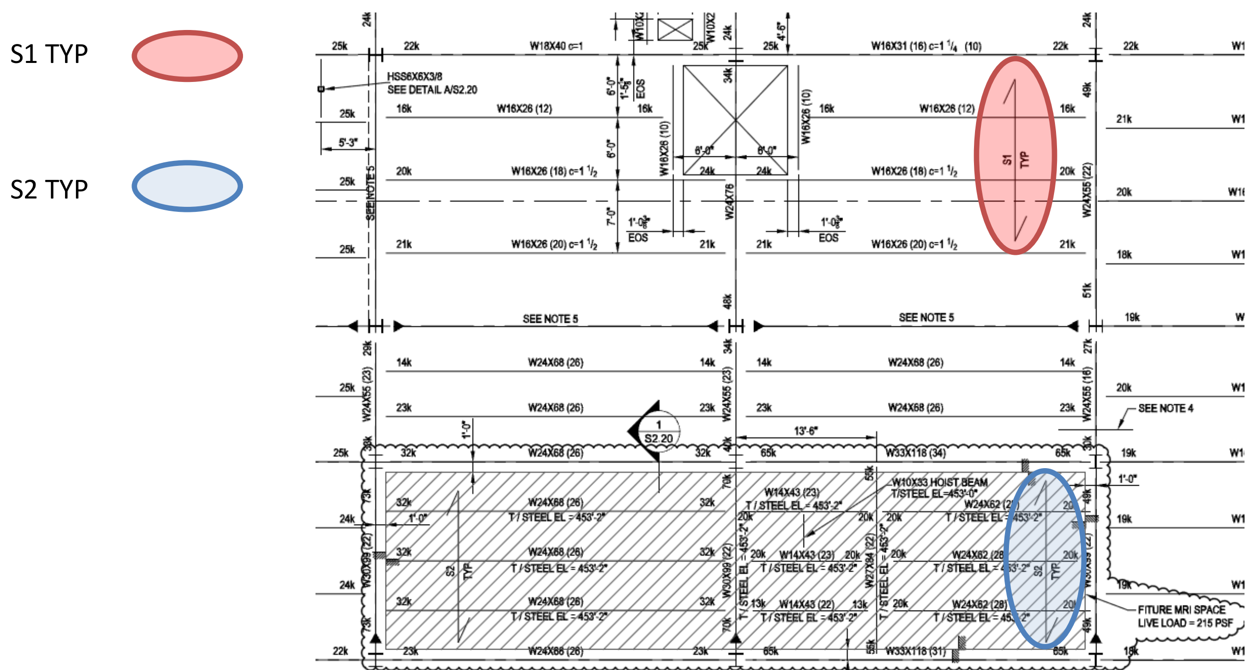


Figure 6 - Level 2 Framing Plan

Roof System

The roof system for the Children's Hospital utilizes the same construction as the S1 TYP floor designation. Future plans call for an additional two stories of occupiable space to be constructed above the current roof level. Figure 7 shows how the columns for the future sixth floor are to be attached to the existing columns. The roofing material consists of a multiple-ply built-up roofing membrane on top of insulation. Surrounding the roof is an 8" thick parapet wall that rises 1' 4" above the top of the composite slab.

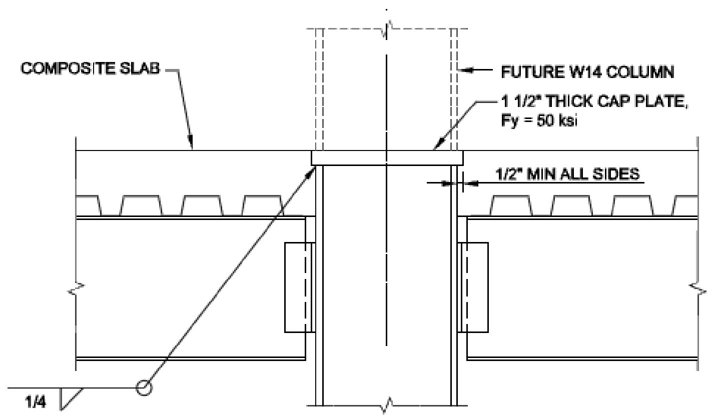


Figure 7 - Top of Column at Future Sixth Floor

Lateral System

The main lateral force resisting system is composed of several moment frames located at the interior of the floor plan. These moment frames run in the East-West direction along the floor plan and are represented in Figure 8 with red. The purpose in placing the moment frames in these locations is to allow for a consistent and open floor space which is important for the functionality of a hospital. Running perpendicular to the moment frames are diagonally braced frames which are represented with blue in Figure 8. The locations of these braced frames are set in locations where space requirements are not as significant such as partitions to the elevator banks.

The main lateral members used in the moment frame system are wide flange sections, primarily W24x229 and W24x176 while the columns are W14x342 and W14x283. The braced frames used in the structure are comprised of W10x112 and W10x88 bracing members.

Conclusions on Structural System

The structural system for the Children's Hospital allows for optimal use of space and provides room for future expansion when the need arises. The importance of using a composite floor system is that it allows for smaller framing members to be used. By using shallower members, the floor to floor height can be increased. Another benefit of using a composite floor system is that it assists in providing additional lateral resistance by creating a stiffer structure. This along with the moment frames allow for larger spaces that are necessary for daily operations of the Children's Hospital.

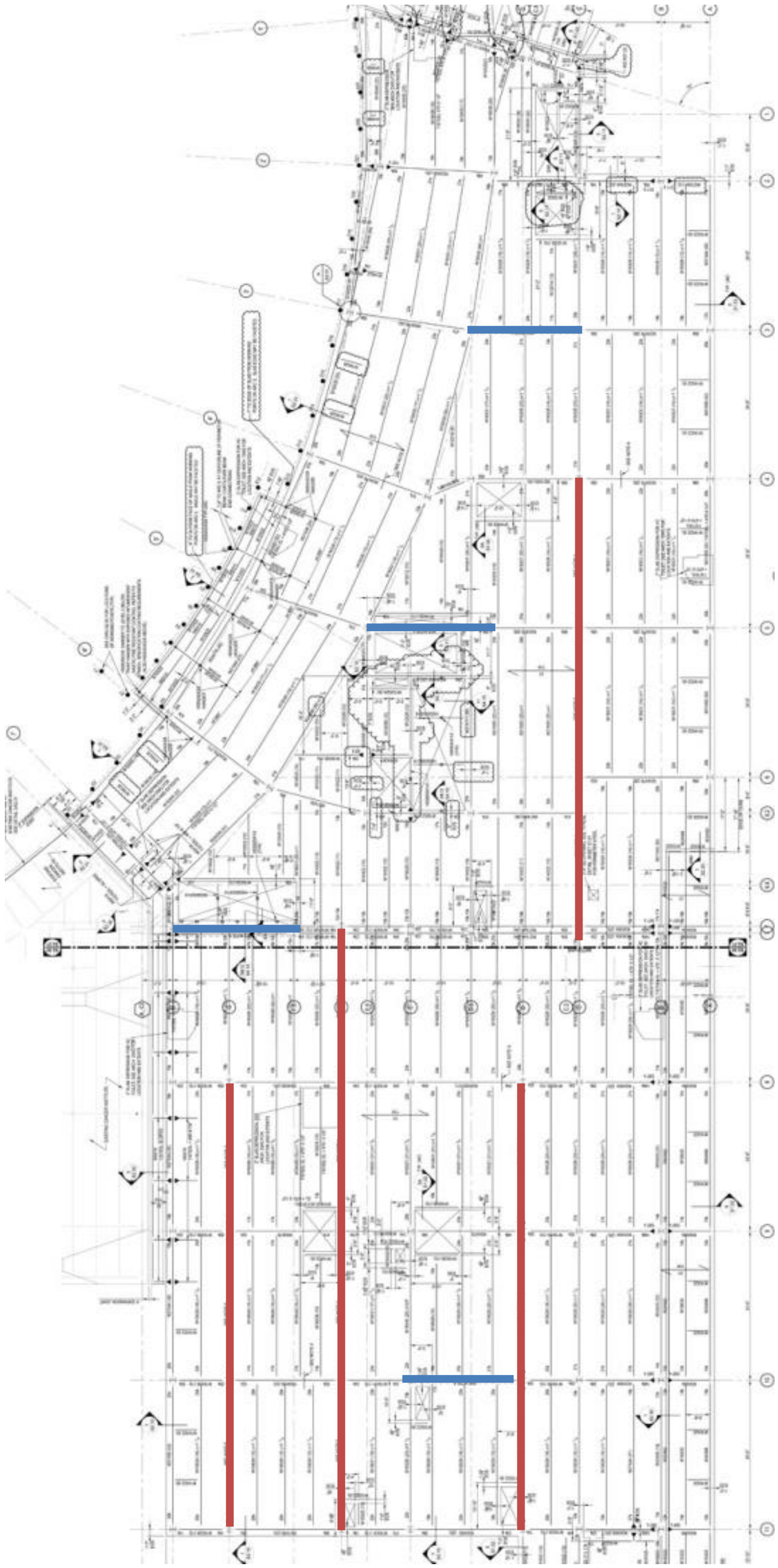


Figure 8 – Framing Plan

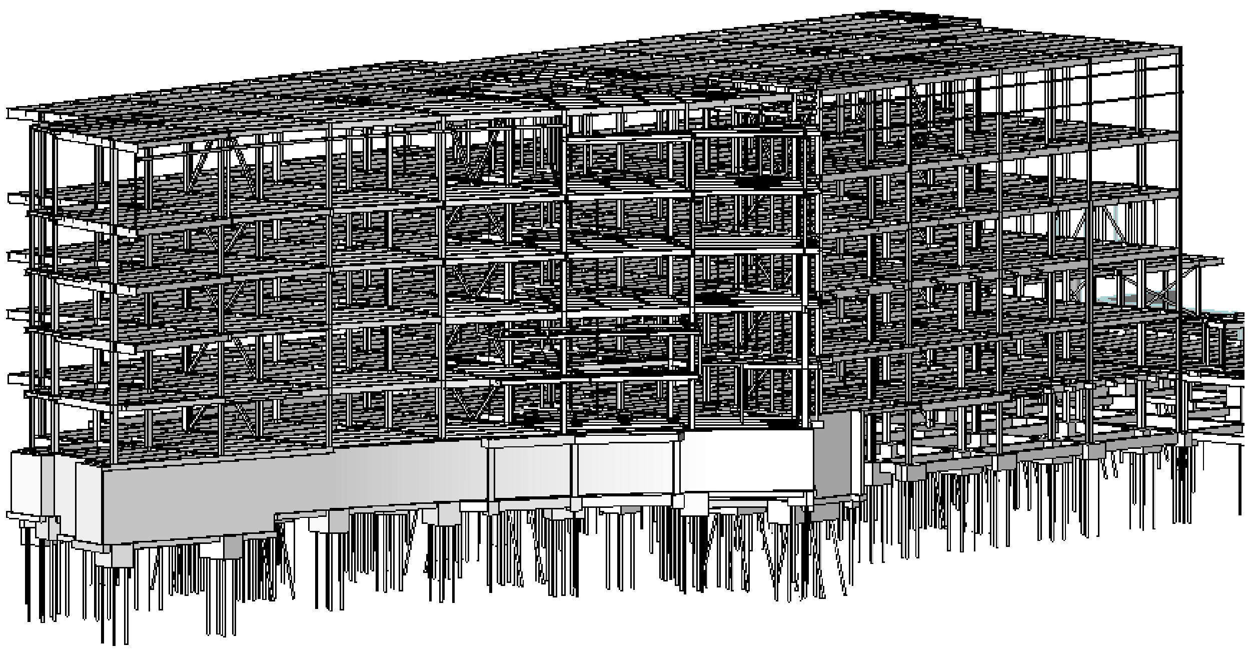


Figure 9 - Framing Rendering

Building Codes

The building codes used by the structural engineer in the design of the structural system as listed in the specifications are listed as the following:

"International Building Code, 2006 Edition"

SEI/ASCE 7-05, Third Edition – "Minimum Design Loads for Buildings and Other Structures"

AISC – "Manual of Steel Construction – Load and Resistance Factor Design"

AISC 360-05 – "Specification for Structural Steel Buildings"

AISC 303-05 – "Code of Standard Practice for Steel Buildings and Bridges"

ACI 318-05 – "Building Code Requirements for Structural Concrete"

The building codes that will be referenced throughout the research, calculations, and findings of this report are as follows:

"International Building Code, 2009 Edition"

SEI/ASCE 7-10 – "Minimum Design Loads for Buildings and Other Structures"

AISC – Steel Construction Manual, 13th Edition

ACI 318-05 – "Building Code Requirements for Structural Concrete"

Materials

Structural Steel	
Wide Flanges	ASTM A992 Grade 50
Plates, Bars, and Angles	ASTM A36
HSS Rectangular Members	ASTM A500 Grade B
HSS Round Members	ASTM A500 Grade B
Anchor Rods	ASTM F1554 Grade 36
¾" High-Strength Bolts	ASTM A325-X
Welding Electrode	E70XX
Concrete	
Pile Caps	f'c = 4000 psi
Slab on Grade	f'c = 4000 psi
Foundation Walls	f'c = 4000 psi
Column Pedestals	f'c = 4000 psi
Strut Beams	f'c = 4000 psi
Note: all concrete is normal weight concrete (145 pcf)	
Reinforcement	
Reinforcing Bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185
Decking	
Floor Deck	2" Composite Metal Deck, 20 Ga.
Roof Deck	1 ½" Metal Roof Deck, 20 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Grout (micropiles)	f'c = 4500 psi

Table 1 - Material Specifications

Dead and Live Loads

The following live loads were determined using ASCE 7-10 while most of the dead loads are assumed based on the industry standard. The design loads sited in the drawing specifications are also listed to provide comparison between those that the design team used and what the code provides. Where specific gravity loads could not be determined, estimation was made with basic research.

Dead Loads		
Normal Weight Concrete	145 pcf	
Structural Steel	490 pcf	
2" Deep Metal Deck	69 psf	
Superimposed Dead Load	30 psf	
Aluminum Cladding	0.75 psf	
Note: Superimposed Dead Load includes MEP systems, ceiling weights, and finishes		
Live Loads		
Occupancy or Use	**Original Design**	**ASCE 7-10**
Lobbies/Moveable Seat Areas	100 psf	100 psf
Corridors (First Floor)	100 psf	100 psf
Corridors (Above First Floor)	80 psf	80 psf
Classrooms, Scientific Labs, Offices, Etc.	80 psf	60 psf
Electrical and Mechanical Rooms	250 psf	N/A
Stairs and Landings	100 psf	100 psf
Storage Areas: Light Storage	125 psf	125 psf
Storage Areas: Heavy Storage	250 psf	250 psf
Computer Rooms	100 psf	100 psf
Courtyards	100 psf	100 psf
Future MRI Space	215 psf	N/A

Table 2 - Dead and Live Loads

Analysis of Floor Systems

The following is a comparison analysis between the existing floor system and the three alternative floor systems. The existing floor system is composite metal deck on composite steel beams and girders. The alternative floor systems include: pre-cast hollow core planks on steel beams, concrete one-way pan joists, and concrete one-way slab and beams. The typical bay that was considered in the analysis was a 19' x 34.5' interior span, see Figure 11

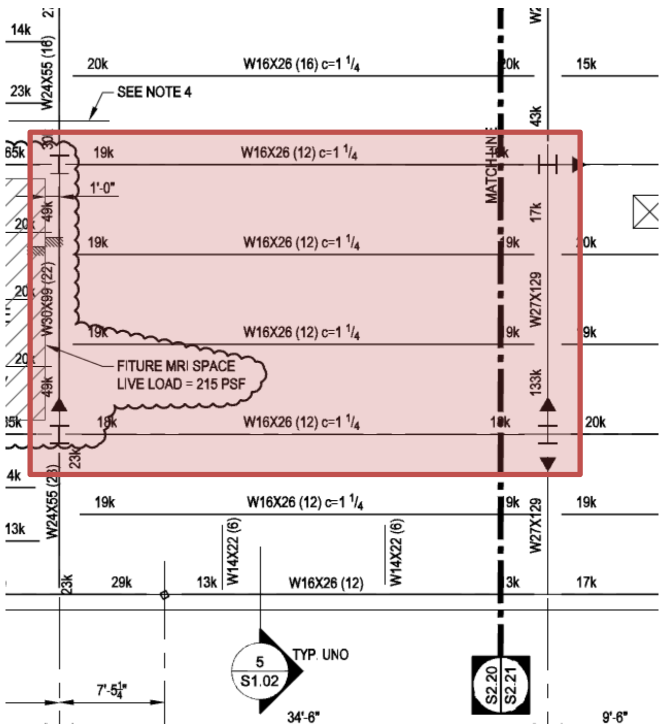


Figure 10 - Typical Layout

It is necessary to note that all of the floor systems were designed under gravity loads only. Additional consideration would have to be made into the effects of lateral forces to obtain a more accurate comparison of the floor systems. The existing system utilizes moment connections which are designed to provide lateral support. The member sizes designed for the alternative systems would eventually need to be analyzed with lateral forces which cause P Delta effects. All hand calculations performed while designing each system can be found in the appendix of this report.

Composite Metal Deck with Beams

Description:

The existing floor system utilizes a 2” deep, 20-gage composite metal deck with a 4 ½” topping thickness. The reinforcement within the slab is 6x6 W2.1xW2.1 Welded Wire Fabric. Supporting the slab are typical W16x26 composite steel beams welded with ¾” diameter shear studs. As stated earlier, the floor system layout is shown in Figure 10. The W16x26 beams span the 34.5’ direction while larger girders span the 19’ direction. A detailed section cut of how these structural elements are connected can be seen in Figure 11. Hand calculations can be found in Appendix A.

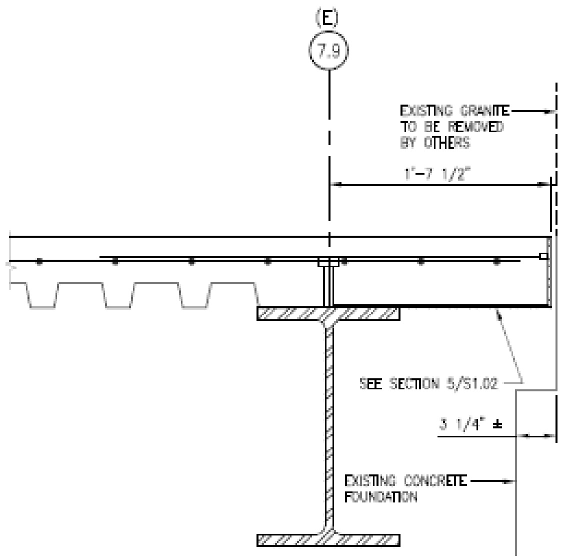


Figure 11 - Composite Beam Connection

Advantages:

There are many advantages to using a composite metal deck with beam system. The metal deck provides the necessary formwork while placing concrete. The composite action between the beam and the slab allow for the use of shallower members and slab thicknesses. Depending on the spacing for the beams, shoring may not be required during construction.

Disadvantages:

While the system allows for shallower beams, the overall system depth can still be rather large. Routing mechanical and electrical systems through the building can cause a decrease in the floor to ceiling height. Since shear studs need to be field welded, there is an increase in labor and cost for the connection. Additional cost needs to be taken into account for required fireproofing on exposed structural steel members.

Pre-Cast Hollow Core Planks on Steel Beams

Description:

The hollow core planks are pre-cast members that are pre-stressed to allow for longer spans and higher loads. For the typical bay design, the planks were chosen to run the shorter span while steel wide flanges provide end support transferring load into the columns. From the *Nitterhouse Pre-stressed Catalog*, a 6" x 4' span plank with 7 – ½" diameter strands was determined to provide sufficient support across the 19' span. This plank also accounts for a 2" cast-in-place topping that provides for a two hour fire rating. Figure 12 shows a detailed cut section of the selected hollow core plank. The columns were assumed to be the existing column layout and sizes of the existing floor system.

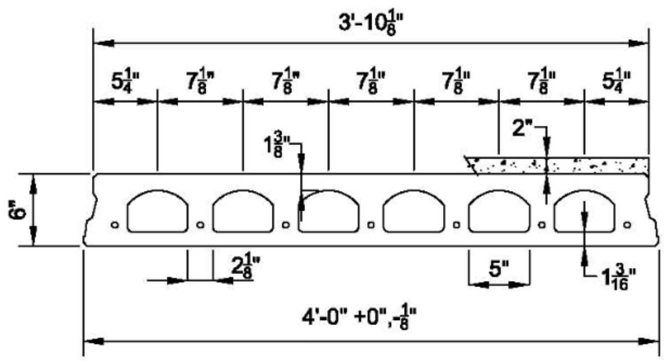


Figure 12 - Hollow Core Plank Detail

The capacity of the hollow core system allows for a 275 psf service load. The service load for the typical bay was calculated as 110 psf. For a future MRI space which was labeled to have a live load of 215 psf, the hollow core still provides enough capacity to withstand the calculated service load of 245 psf. The supporting girder was determined to be a W24x76. All hand calculations for this system can be found in Appendix B.

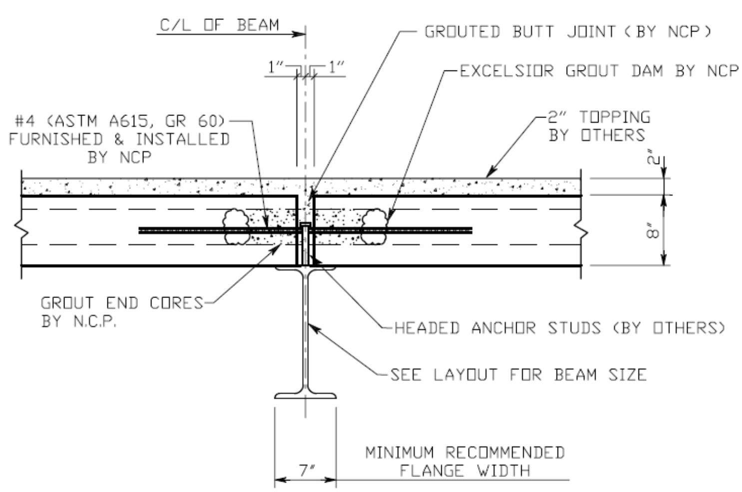


Figure 13 - Detailed Connection Bearing on Steel Beam

Advantages:

Since this system utilizes pre-cast members, much of the preliminary construction can be done in advance at a concrete plant. Factors that would slow down cast-in-place work such as climate and temperature are eliminated allowing for faster occupancy by the owner. The pre-stressed strands allow for longer spans while maintaining a shallower overall thickness. The system weight was also determined to be less than that of the existing composite system. Changes to the foundation are not as necessary as for other alternative systems that were considered.

Disadvantages:

The hollow core planks are pre-cast into four foot sections which mean that modification to the column layout would be needed. All columns and openings would have to be designed based on this module. Irregularities including curved perimeters would need specially designed planks which would increase the system cost. The steel members would require additional labor to account for fireproofing and connection detailing.

One Way Pan Joist System

Description:

The one way pan joist system is a cast in place concrete system with joists spanning in one direction. This type of construction allows for the slab to be cast integrally with the joists forming a monolithic structure. Wider beams run normal to the joists transferring load into the columns (see Figure 14). This system is ideal for rectangular bays where one dimension is significantly different than the other. For economy, the joists were designed to span the short direction while beams run perpendicularly between the columns.

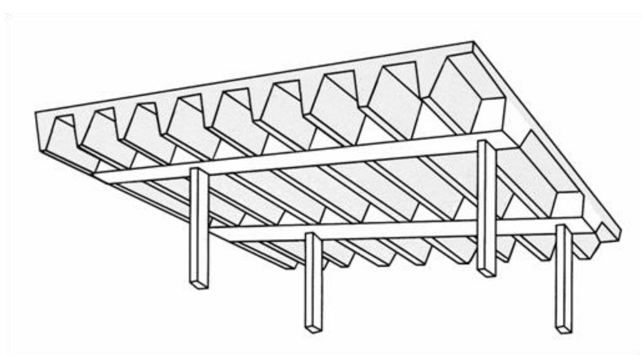


Figure 14 - One Way Pan Joist System

The pan joists that were selected are a 66|6 skip joist, which is the clear span from one face of the joist to the next is 66 inches while the web of the joist is 6 inches wide. Reinforcement for the joists was determined to be 2 #5 bars for top reinforcing and 1 #6 bar for bottom reinforcing. The slab was designed for a thickness of 4.5" with #4 reinforcing bars. The beam was designed to be 3 feet wide and have the same 14 inch depth as the joists. To resist negative moment at the column faces, 8 #9 bars should be used. Conversely, 5 #9 bars should be used for bottom reinforcement at mid-span to resist positive moment. The column layout was taken to be the same as the existing system with 24" x 24" square columns. Hand calculations can be found in Appendix C.

Advantages:

This one way system is economical for the rectangular bay size shown in Figure 10, allowing joists to span the 19' direction and beams to span the 34.5' direction. Since the depth of the beams matches that of the joists, the system can be cast monolithically. A major advantage of using this system is that the redundancy of the system allows for formwork to be recycled, reducing construction costs. Since this system utilizes concrete rather than steel, there is no need to factor in costs and labor needed for fireproofing. The layout and increased mass of the structure allow for this system to have an inherent vibration resistance which is important for sensitive hospital equipment.

Disadvantages:

Although the increase in mass of the system benefits vibration resistance, it will also be necessary to make alterations to the foundation. Larger columns would be needed to carry the increased weight of the structure. Since the one way pan joist system is a cast in place system, longer lead times will be necessary to account for curing of the concrete. This in turn will have an effect on the overall cost of the structural system.

One Way Slab and Beams

Description:

Similar to the pan joist system, the one way slab and beam system is a cast in place concrete system. The one way slab and beam system however does not use intermediate joists. This means that in order to maintain deflection limits, the slab thickness would need to be increased. Wider beams are also used to transfer loads to the columns. Figure 15 shows a typical layout of a one way slab and beam system.

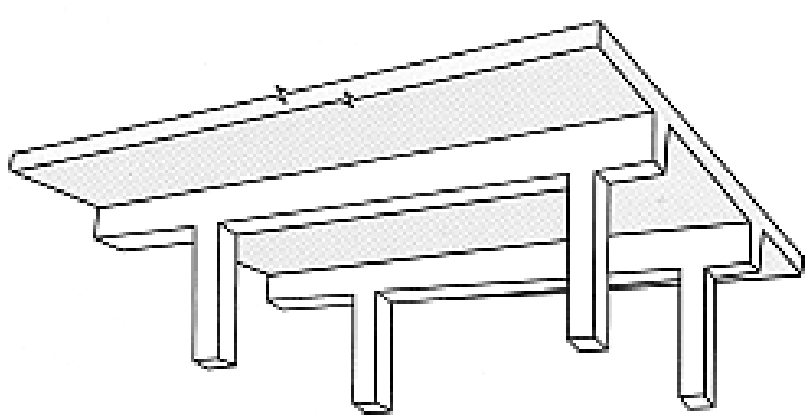


Figure 15 - One Way Slab and Beam System

The alternative system that was designed utilizes a 6 inch deep slab that spans between the beams. Reinforcement for the slab consists of #6 bars spaced at 12 inches on center from each other. The beams were designed to be six feet wide with a depth of 10 inches. To resist negative moment at the column faces, it was determined that 23 #6 bars would be necessary. For the mid span positive moment, 16 #6 reinforcing bars were initially selected. After calculating the total load deflection however, it was determined that significantly more reinforcing was necessary. Calculations in Appendix D show that 28 #9 reinforcing bars are required to provide enough capacity to resist deflection. Although increasing the depth of the beam would also have had a similar influence, the full width of the beam was utilized while maintaining a shallower system depth.

Advantages:

The one way slab and beam system has many of the same advantages as the one way pan joist system. Since it is a concrete system, the formwork can be reused and there is no additional cost for fireproofing. The overall system depth is also significantly smaller than that of systems utilizing steel beams which are deeper to resist the same loads. The additional mass of the system allows for relative vibration resistance as opposed to lighter steel framing.

Disadvantages:

Similar to the one way pan joist system, there is a relatively large increase in system weight as opposed to systems utilizing steel construction. Due to the increase in weight, the foundation would need to be increased to account for greater member weights. Larger column sizes would also be needed to transfer loads to the foundation. Since it is a cast in place system, longer lead times would have to be considered to account for curing of the concrete.

Comparison of Floor System

Various factors were taken into account when comparing the existing floor system with those of the alternative systems. Table 1 shows how these factors compare with the different systems and which systems are feasible.

Design Concern	Existing *Composite Metal Deck with Beams*	Alternative 1 *Pre-cast Hollow Core Planks*	Alternative 2 *One Way Pan Joist System*	Alternative 3 *One Way Slab and Beams*
Slab Depth	4.5"	8"	4.5"	6"
System Depth	34.1"	31.9"	18.5"	16"
Beam Deflection (D+L)	1.73"	1.58"	1.35"	1.72
System Cost	$18.40/sq. ft.	$32.34/sq. ft.	$17.73/sq. ft.	$19.41/sq. ft.
System Weight	76.8 psf	52.8 psf	88.3 psf	90.8 psf
Fire Protection	Spray-On	Spray-On	Inherent	Inherent
Formwork	No	No	Yes	Yes
Lateral System Alterations	No	Yes	Yes	Yes
Foundation Alterations	No	No	Yes	Yes
Feasibility	Yes	No	Yes	Yes

Table 3 - System Comparison

Slab Depth / System Depth

The two one way concrete systems have shallower depths than the existing and hollow core plank systems. The advantage in this is that it is possible to attain shallower plenum spaces while maintain the same MEP systems. Connection of the MEP systems to the floor system and allowing for openings would need to be considered for each system. It is possible to drill into the skip joists in pan joist system for MEP systems as long as they do not go through a joist.

Deflection (D+L)

It was determined that all the floor systems meet the necessary deflection requirements. Deflection is an important consideration when comparing systems for the Children's Hospital. There is a lot of equipment must be kept as precise as possible for doctor and patient needs. The system with the least amount of deflection was determined to be the one way pan joist system. Both the existing and one way slab and beam systems had the greatest amount of deflection.

System Cost

The system cost was roughly determined using RS Means *Assemblies Cost Data 2011* for each floor system. To obtain a more accurate cost analysis, it would be necessary to perform a unit cost analysis. A location factor of 96.1% for Harrisburg was taken as being closest city listed to Hershey. The total assemblies cost was then adjusted for this location. For this analysis, all the systems had roughly the same cost per square foot. The cost of the pre-cast hollow core planks was significantly higher simply because of the specialty of the construction specific to the manufacture's cost.

System Weight

The system weight is significant in affecting the design of the columns and foundation systems. Hollow core planks were determined to have the least weight per square foot. This would allow for a greater reduction in dead weight when compared to the existing system. It was fairly apparent by inspection that the two concrete structures would have the greatest system weight. For consideration of either the one way concrete systems as a viable alternative floor system, the dimensions of the foundation would have to be increased.

Fire Protection

By the code considerations, it is necessary to provide all structural elements with a two hour fire rating. Since both the one way systems are concrete, they inherently provide allowance for a two hour fire rating. The hollow core planks also provide for fireproofing, however the exposed steel beams would need to be sprayed with fire resistant material. Similarly, the exposed beams and underside of the metal deck would need to be sprayed with fireproofing for the existing system.

Formwork

Formwork would only be needed for those members that are cast in place concrete. This includes both the pan joist system which requires specific formed pans and the slab and beam system. The cost of labor and time necessary for the concrete to cure are external factors that would have to be taken into account.

Lateral System Alterations

The comparison of the alternative systems only considers gravity loading in the sizing of members. It is assumed that all three alternative systems will need to be increased to account for lateral loads on the structure. It can safely be assumed that the existing structure does not need to be altered for lateral loads since it utilizes moment connections and braced frames. The two concrete one way systems have the greatest mass and would be a more rigid structure. This increase in rigidity would translate into a reduction in vibration and seismic frequency. Additional calculations will need to be performed to determine which system provides more lateral resistance.

Foundation Alterations

The foundation system would need to be altered for all the alternative systems. The hollow core plank system could potentially use the existing column sizes since the system weight is less than that of the existing system. Since the planks come in four foot sections, the column lines would need to be shifted to allow for constructability of the system. For the one way joists and one way slab and beams, the size of the foundation would certainly need to be increased to account for the additional self-weight of the system. The advantage however with these two systems is that column lines can be shifted 10% from floor to floor, allowing for some variation in floor layout.

Evaluations and Summary

After comparing the floor systems based on the criteria listing in Table 1, it was necessary to choose which systems were feasible. The pre-cast hollow core planks were determined to not be feasible. Despite the advantage of pre-cast members being constructed off site allowing for faster construction, there were other criteria that made it not feasible for this project. The cost factor was a large determination in eliminating this system. The hollow core planks cost about 50% more than the existing system. For that amount of increase in cost, it would be expected that the hollow core planks provide some benefit to decreasing the system depth or deflection. However the system depth of the hollow core planks only saves the design about two inches over the existing system. Once lateral forces are to be considered, additional cost would be needed for special connections to resist additional loads.

The other two concrete one way systems are fairly similar with each other. Both provide fireproofing, have a system depth considerably less than that of the existing system, and cost relatively the same. Since both are cast in place, they have the ability to easily form the existing geometry of the building plan. The disadvantage to being cast in place is that additional time must be allowed for the concrete to cure. These systems both decrease the existing floor depth by about 47% allowing for greater floor to ceiling heights even after considering allowable space for MEP systems. The weight of both systems is about 14% more than that of the existing system. This increase however would allow the building to act more rigid when considering story displacements. This will be important when limiting story drift under lateral loads. A more thorough analysis would provide more insight into which system has a higher performance.

The additional cost of time and money in constructability for these two alternative systems may appear to be a negative factor in selecting these either of these systems. However without a lateral analysis, it is difficult to determine which one of the systems is more feasible. Under the given criteria listed in this report, both the one way pan joist system and the one way slab and beam system are both feasible for further study.

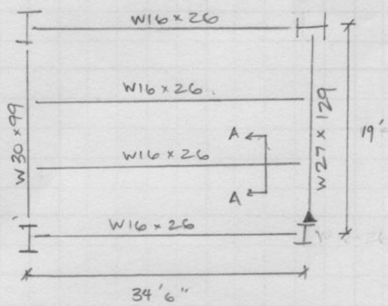
Technical Report 3 is to follow which will focus on lateral system analysis and confirmation design study.

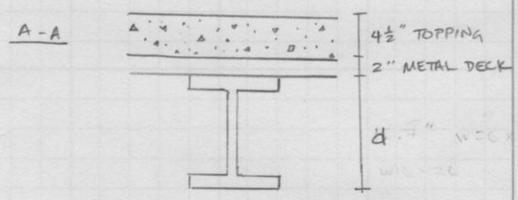
APPENDIX

Appendix A: Existing System

MATT VANDERSALL	TECH REPORT #2	EXISTING SYSTEM	1/2

COMPOSITE METAL DECK





FOR W16×26: d = 15.7 in
FOR W27×129: d = 27.6 in

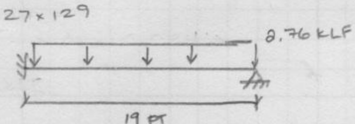
SEE TECH REPORT #1 SPOT CHECKS FOR PRELIMINARY WORK

LOAD ON W27×129:

SLAB WT = 69 PSF (34'6") = 2380.5 PLF
W16×26 = 26 PLF (34'6") = 897 lbs. (POINT LOADS AT 3RD POINTS)
W27×129 = 129 PLF
S.D.L. = 30 PSF (34'6") = 1035 PLF
L.L. = 80 PSF (34'6") = 2760 PLF

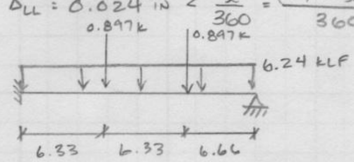
DEFLECTION ON W27×129

LIVE LOAD:



$\Delta_{LL} = 0.024 \text{ in} < \frac{l}{360} = \frac{(19\text{ FT})(12)}{360} = 0.63 \text{ in} \therefore \text{OK}$

TOTAL LOAD:



$\Delta_{TL} = 0.056 \text{ in} < \frac{l}{240} = \frac{(19\text{ FT})(12)}{240} = 0.95 \text{ in} \therefore \text{OK}$

DEFLECTIONS ARE SMALL BECAUSE GIRDERS SPAN THE 19 FT SPAN

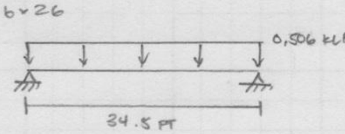
LOAD ON W16×26:

SLAB WT: 69 PSF (6.33 FT) = 436.8 PLF
W16×26: 26 PLF
S.D.L. = 30 PSF (6.33 FT) = 189.3 PLF
L.L. = 80 PSF (6.33 FT) = 506.4 PLF

MEMBER PROPERTIES:
$A_x = 7.68 \text{ in}^2$
$I_{LB} = 595 \text{ in}^4$

DEFLECTION ON W16×26

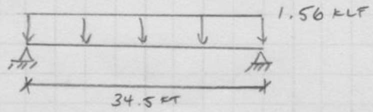
LIVE LOAD:



$\Delta_{LL} = 1.86 \text{ in} - 1.25 \text{ in (CAMBER)} = 0.61 \text{ in} < \frac{l}{360} = \frac{(34.5\text{ FT})(12)}{360} = 1.15 \text{ in} \therefore \text{OK}$

MATT VANDERSALL | TECH REPORT # 2 | EXISTING SYSTEM | 2/2

TOTAL LOAD:



$\Delta_{TL} = 2.88\,\text{in} - 1.25\,\text{in} = 1.63\,\text{in} < \frac{l}{240} = \frac{(34.5\,\text{ft})(12)}{240} = 1.73\,\text{in}$ ∴ OK
(CAMBER)

COST ANALYSIS : REF – RS MEANS ASSEMBLIES COST DATA 2011

LOCATION FACTOR = 0.961 (HARRISBURG)

AREA = 655 SQ FT → ASSUMED 25 × 30 BAY : AREA = 750 SQ FT

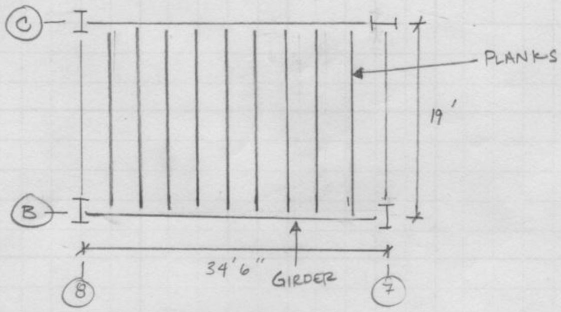
SUPERIMPOSED LOAD = 80 + 30 = 110 PSF → ASSUMED 125 PSF

COMPOSITE BEAMS, DECK & SLAB (PG 94)

MAT	INST	TOTAL
12.90	6.25	\$ 19.15 /SQ FT (0.961) = $\boxed{\$\,18.4\,/\text{SQ FT}}$

Appendix B: Pre-Cast Hollow Core Planks

MATT VANDERSALL	TECH REPORT #2	HOLLOWCORE PLANKS	1/2



S.D.L = 30 PSF
L.L. = 80 PSF
} 110 PSF SUPERIMPOSED LOAD

FROM NITTERHOUSE PRESTRESSED CATALOGE

SPAN = 19 FT , 110 PSF

TRY : 6" x 4' SPAN DECK , 2" C.I.P. TOPPING ← PROVIDES 2-HR FIRE RATING
W/7 - 1/2 ⌀ STRANDS

ALLOWABLE SUPERIMPOSED LOAD = 274 PSF > 110 PSF ∴ OK

NOTE: SOME SPACES SPECIFY A LIVE LOAD OF 215 PSF FOR FUTURE MRI SPACE

SUPERIMPOSED LOAD = 215 + 30 = 245 PSF < 274 PSF ∴ OK

SELF WEIGHT = 48.75 PSF

TOTAL LOADS

D.L. = 48.75 + 30 = 78.75 PSF (4 FT) = 315 PLF

L.L. = 80 PSF (4 FT) = 320 PLF

LOAD COMBINATION:

$w_u = 1.2D + 1.6L = 1.2(315) + 1.6(320) = 0.89$ KLF

CHECK DEFLECTION:

NOTE: DELECTION LIMITS WERE NOT CONSIDERED WHEN DETERMINING ALLOWABLE LOADS IN NITTERHOUSE CATALOGE

$$\Delta_{LL} = \frac{5 w_u l_n^4}{384 EI} \qquad I = 1519 \text{ in}^4, \; E_c = 33(150)^{1.5}\sqrt{6000 \text{ PSI}} = 4695.98 \text{ KSI}$$

$$= \frac{5(0.32 \text{ KLF})(19 \text{ FT})^4(1728)}{384(4695.98 \text{ KSI})(1519 \text{ in}^4)} = 0.13 \text{ in}$$

$$\frac{l}{360} = \frac{(19 \text{ FT})(12 \text{ IN/FT})}{360} = 0.63 \text{ in} > 0.13 \text{ in} \therefore \text{OK}$$

GIRDER DESIGN

TOTAL LOAD

D.L. = 78.75 PSF (19 FT) = 1.50 KLF

L.L. = 80 PSF (19 FT) = 1.52 KLF

LOAD COMBINATION:

$w_u = 1.2D + 1.6L = 1.2(1.50) + 1.6(1.52) = 4.23$ KLF

MATT VANDERSALL | TECH REPORT #2 | HOLLOWCORE PLANKS | 2/2

$$V_u = \frac{w_u l_n}{2} = \frac{(4.23\,KLF)(34.5\,FT)}{2} = 72.97\,K$$

$$M_u = \frac{w_u l_n^2}{8} = \frac{(4.23\,KLF)(34.5\,FT)^2}{8} = 629.3\,FT\cdot K$$

L.L. DEFLECTION: $w_{LL} = 80\,PSF\,(19\,FT) = 1.52\,KLF$

$$\frac{l}{360} = \frac{5 w_{LL} l_n^4}{384 EI}$$

$$\frac{(34.5\,FT)(12\,^{IN}/_{FT})}{360} = \frac{5(1.52\,KLF)(34.5\,FT)^4(1728)}{384\,(29000\,KSI)\,I_{min}} \longrightarrow I_{MIN} \geq 1452.8\,in^4$$

TOTAL DEFLECTION: $w_T = 158.75\,PSF\,(19\,FT) = 3.02\,KLF$

$$\frac{l}{240} = \frac{5 w_T l_n^4}{384 EI}$$

$$\frac{(34.5\,FT)(12\,^{IN}/_{FT})}{240} = \frac{5(3.02\,KLF)(34.5\,FT)^4(1728)}{384\,(29000\,KSI)\,I_{MIN}} \longrightarrow \boxed{I_{MIN} \geq 1924.3\,in^4}$$

TRY: W24 x 76

$I = 2100\,in^4 > 1924.3\,in^4 \therefore$ OK

$\phi M_p = 750\,K\cdot FT > 629.3\,K\cdot FT \therefore$ OK

$\therefore$ USE 6" x 4' HOLLOWCORE PLANK W/2" TOPPING, 7 - ½ϕ STRANDS AND W24 x 76 GIRDER

COST ANALYSIS: REF - RS MEANS ASSEMBLIES COST DATA 2011

LOCATION FACTOR = 0.961 (HARRISBURG)

ASSUMED 25 x 30 BAY

125 PSF SUPERIMPOSED LOAD

MAT	INST	TOTAL
8.50	5.05	\$13.10/SQ FT (0.961) = \$12.59/SQ FT (PLANKS)
14.80	5.75	\$20.55/SQ FT (0.961) = \$19.75/SQ FT (BEAMS)

+

TOTAL = $\boxed{\$32.34/SQ\,FT}$

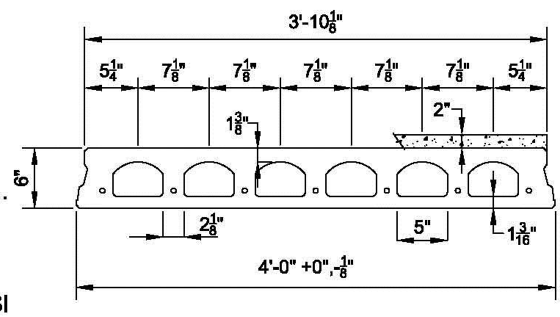
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast b_w = 16.13 in.
I_c = 1519 in.^{4}	Precast S_{bcp} = 370 in.^{3}
Y_{bcp} = 4.10 in.	Topping S_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 3.90 in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 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 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   - 4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   - 6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   - 7-1/2"Ø, 270K = 95.3 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. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. 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)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	17[illegible]	149	[illegible]27	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	26[illegible]	237	[illegible]15	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	29[illegible]	274	[illegible]42	214	190	167	144	124	107	91	77	64	53

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

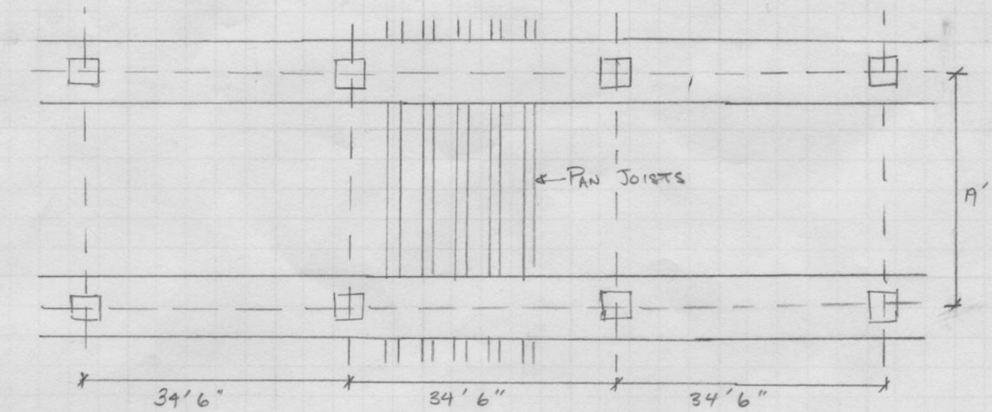
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads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

11/03/08

6F2.0T

Appendix C: One Way Pan Joist System

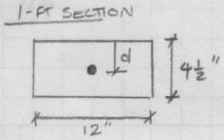
MATT VANDERSALL | TECH REPORT #2 | PAN JOIST SYSTEM | 1/6



SLAB DESIGN

- USE 4 1/2" SLAB DEPTH FOR 2HR FIRE RATING
- S.D.L. = 30 PSF
- LL = 80 PSF
- ASSUME $f'_c = 4$ ksi , $f_y = 60$ ksi

1-FT SECTION



d = 2.25"

MINIMUM REINFORCING FOR SHRINKAGE AND TEMPERATURE (ACI 7.12.2.1)

$0.0018 A_g$

$= 0.0018(4\frac{1}{2}\text{ in})(12 \text{ in}/\text{ft}) = 0.097 \text{ in}^2/\text{ft}$

#4 BAR — $A_s = 0.20 \text{ in}^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.2 \text{ in}^2/\text{ft})(60 \text{ ksi})}{0.85(4 \text{ ksi})(12 \text{ in}/\text{ft})} = 0.294 \text{ in}$$

$\phi M_n = A_s f_y (d - a/2)$

$= 0.9(0.2 \text{ in}^2)(60 \text{ ksi})(2.25 - \frac{0.294}{2}) = 22.71 \text{ in}\cdot\text{k}/\text{ft} = 1.89 \text{ ft}\cdot\text{k}/\text{ft}$

LOADING ON SLAB:

SELF WEIGHT $= (4\frac{1}{2}")(\frac{1}{12}\frac{\text{ft}}{\text{in}})(150 \text{ PCF}) = 56.3 \text{ lb}/\text{ft}^2$

$w_u = 1.2(56.3 + 30) + 1.6(80) = 0.232 \text{ k}/\text{ft}$ FOR A 12 in STRIP

FROM TABLE 9.5A - $h \geq \frac{l_n}{21} \rightarrow l_n \leq (4\frac{1}{2}")(\frac{1}{12}\frac{\text{ft}}{\text{in}})(21) = 7.88$ ft

NEGATIVE MOMENT:

$$\frac{w_u l_n^2}{11} = \frac{(0.232 \text{ k}/\text{ft})(7.88 \text{ ft})^2}{11} = 1.31 \frac{\text{ft}\cdot\text{k}}{\text{ft}} < 1.89 \frac{\text{ft}\cdot\text{k}}{\text{ft}} \quad \therefore \text{OK}$$

POSITIVE MOMENT:

$$\frac{w_u l_n^2}{16} = \frac{(0.232 \text{ k}/\text{ft})(7.88 \text{ ft})^2}{16} = 0.9 \frac{\text{ft}\cdot\text{k}}{\text{ft}} < 1.89 \frac{\text{ft}\cdot\text{k}}{\text{ft}} \quad \therefore \text{OK}$$

PAN JOIST DESIGN

- USE 66 | 6 SKIP JOIST FOR 19 FT SPAN
- ASSUME 24" GIRDER WIDTH

FROM TABLE 9.5A — $h \geq \frac{l_n}{21} = \frac{19\text{ ft} - 2\text{ ft}}{21} = 0.81 \text{ ft}(12 \text{ in}/\text{ft}) = 9.71$ in

$\therefore$ TRY PAN DEPTH = 14 in

MATT VANDERSALL | TECH REPORT #2 | PAN JOIST SYSTEM | 2/6

LOADING ON PAN JOIST:

SLAB WEIGHT $= 56.3\ \text{lb/ft}^2 (6\ \text{FT}) = 337.8$ PLF

S.D.L $= 30\ \text{lb/ft}^2 (6\ \text{FT}) = 180$ PLF

PAN WEIGHT $= \frac{(6\ \text{in})(14\ \text{in})}{144} (150\ \text{PCF}) = 87.5$ PLF

L.L. $= (80\ \text{PSF})(6\ \text{FT}) = 480$ PLF

$w_u = 1.2(337.8 + 180 + 87.5) + 1.6(480) = \underline{1.49\ \text{k/ft}}$

NEGATIVE MOMENT:

$$\frac{w_u l_n^2}{11} = \frac{(1.49\ \text{k/ft})(19\ \text{FT} - 3\ \text{FT})^2}{11} = 34.7\ \text{k}\cdot\text{FT}$$

POSITIVE MOMENT:

$$\frac{w_u l_n^2}{16} = \frac{(1.49\ \text{k/ft})(19\ \text{FT} - 3\ \text{FT})^2}{16} = 23.9\ \text{k}\cdot\text{FT}$$

DESIGN FOR TOP REINFORCEMENT:

$A_s \approx \frac{M_u}{4d}$ REINFORCING AT MIDDEPTH OF SLAB: $d = 14\ \text{in} + 4.5\ \text{in} - 2.25\ \text{in} = 16.25\ \text{in}$

$A_s \approx \frac{34.7\ \text{k}\cdot\text{FT}}{4(16.25\ \text{in})} = 0.54\ \text{in}^2 \longrightarrow$ TRY 2 #5 ($A_s = 0.62\ \text{in}^2$)

$\rho = \frac{0.62\ \text{in}^2}{(6\ \text{in})(16.25\ \text{in})} = 0.006 < 0.018$ ∴ TENSION CONTROLLED SECTION ($\phi = 0.9$)

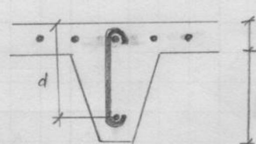
$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.62\ \text{in}^2)(60\ \text{ksi})}{0.85(4\ \text{ksi})(6\ \text{in})} = 1.82\ \text{in}$

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.62\ \text{in}^2)(60\ \text{ksi})(16.25 - \frac{1.82}{2})(\frac{1}{12}) =$

$= 42.8\ \text{k}\cdot\text{FT} > 34.7\ \text{k}\cdot\text{FT}$ ∴ OK

$\boxed{\text{USE 2 \#5 FOR TOP REINFORCEMENT}}$

DESIGN FOR BOTTOM REINFORCEMENT:



COVER = 1.5 in
#3 STIRRUP = 0.375 in
#9 BAR = 1.128 in

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

$A_s \approx \frac{23.9\ \text{k FT}}{4(16.06\ \text{in})} = 0.37\ \text{in}^2$

$d = 18.5'' - 1.5'' - 0.375'' - \frac{1}{2}(1.128'') = 16.06\ \text{in}$

TRY 1 #6 ($A_s = 0.44\ \text{in}^2$)

$\rightarrow d = 16.25\ \text{in}$

$d_t = 18.5'' - 1.5'' - 0.375'' = 16.63\ \text{in}$

CHECK IF TENSION CONTROLLED:

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.44\ \text{in}^2)(60\ \text{ksi})}{0.85(4\ \text{ksi})(6\ \text{in})} = 1.29\ \text{in}$

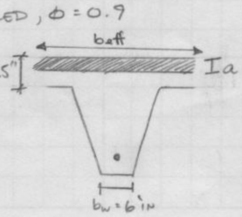
$c = a/\beta = 1.29\ \text{in}/0.85 = 1.52\ \text{in}$

$\varepsilon_s = \frac{0.003}{c}(d_t - c) = \frac{0.003}{1.52\ \text{in}}(16.63 - 1.52) = 0.03 > 0.005$

∴ TENSION CONTROLLED, $\phi = 0.9$

DETERMINE b_{eff}

$b_{eff} = \min \begin{cases} \frac{1}{4} l = \frac{1}{4}(19\ \text{FT})(12\ \text{in/FT}) = \boxed{57\ \text{in}}\ \text{CONTROLS} \\ 8h_{f\,left} + b_w + 8h_{f\,right} = 8(4.5\ \text{in}) + 6\ \text{in} + 8(4.5\ \text{in}) = 78\ \text{in} \\ b_w + l_n = 6\ \text{in} + 66\ \text{in} = 72\ \text{in} \end{cases}$



MATT VANDERSALL | TECH REPORT #2 | PAN JOIST SYSTEM | 3/6

ASSUME NEUTRAL AXIS IS IN FLANGE ($a \le h_f$)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.44\,in^2)(60\,ksi)}{0.85(4\,ksi)(57\,in)} = 0.136\,in < h_f = 4.5\,in \quad \therefore OK$$

CHECK MINIMUM STEEL:

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60000}(6\,in)(16.06) = 0.305\,in^2 < 0.44\,in^2 \quad \therefore OK$$

$$A_{s\,min} = \frac{200}{f_y} b_w d = \frac{200}{60000}(6\,in)(16.06\,in) = 0.32\,in^2 < 0.44\,in^2 \quad \therefore OK$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.44\,in^2)(60\,ksi)(16.06\,in - \frac{0.136}{2})(\tfrac{1}{12})$$
$$= 31.66\,k\cdot FT > 23.9\,k\cdot FT \quad \therefore OK$$

$\boxed{\text{USE 1 \#6 FOR BOTTOM REINFORCEMENT}}$

DESIGN FOR SHEAR REINFORCEMENT

$$V_u = \frac{w_u l_n}{2} = \frac{(1.49\,k/FT)(16\,FT)}{2} = 11.92\,k$$

$$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d$$
$$= 0.75(2)\sqrt{4000}(6\,in)(16.06\,in^2)/1000 = 9.14\,k$$

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{11.92\,k - 9.14}{0.75} = 3.71\,k$$

$V_s = A_v f_y d / s_{max}$ WHERE $S_{max} \le d/2 \therefore \frac{16.06}{2} = 8.03\,in \rightarrow$ USE 6" SPACING

$$3.71\,k = \frac{A_v(60\,ksi)(16.06\,in)}{6\,in}$$

$A_v = 0.023\,in^2 \quad \therefore$ USE #3 BARS @ 6" SPACING

$$\phi V_c + \phi V_s = 9.14\,k + \frac{(0.75)(0.11\,in^2)(60\,ksi)(16.06\,in)}{6\,in}$$
$$= 22.4\,k > 11.92\,k \quad \therefore OK$$

GIRDER DESIGN

$$w_{D\,FLOOR} = \frac{(337.8\,PLF + 180\,PLF + 87.5\,PLF)}{6\,FT}(16\,FT) = 1.62\,KLF$$

$$w_{SELF} = (150\,PCF)(3\,FT)(18.5\,in)(\tfrac{1}{12}\tfrac{FT}{in}) = 0.69\,KLF$$

$w_{@\,GIRDER}$: $D = 30\,PSF\,(3\,FT) = 90\,PLF$
$L = 80\,PSF\,(3\,FT) = 240\,PLF$

$$w_{L\,FLOOR} = \frac{480\,PLF}{6\,FT}(16\,FT) = 1.28\,KLF$$

$$w_u = 1.2(1.62 + 0.69 + 0.09) + 1.6(0.24 + 1.28) = \underline{5.31\,KLF}$$

ACI MOMENT COEFFICIENTS (8.3.3)

NEGATIVE MOMENT AT OTHER FACES OF INTERIOR SUPPORTS:

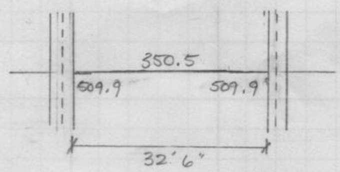
$$\frac{w_u l_n^2}{11} = \frac{(5.31\,KLF)(34.5-2)^2}{11} = 509.9\,k\cdot FT$$

POSITIVE MOMENT AT INTERIOR SPANS:

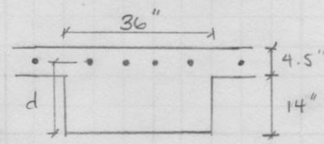
$$\frac{w_u l_n^2}{16} = \frac{(5.31\,KLF)(34.5-2)^2}{16} = 350.5\,k\cdot FT$$

MATT VANDERSALL | TECH REPORT # 2 | PAN JOIST SYSTEM | 4/6

MOMENT DIAGRAM (INTERIOR SPAN):



COLUMN DIMENSIONS: 24" × 24"
COLUMN ℄ = 34.5 FT



TOP REINFORCEMENT (INT. SPAN | INT. SUPPORT)

$A_s = \frac{M_u}{4d} = \frac{509.9\ k\cdot FT}{4(16.1\ in)} = 7.92\ in^2$

$d = 18.5" - 1.5" - 0.375" - .563" = 16.1\ in$
(DEPTH) (CLR) (#3 STIRRUP) (#9 BAR)

TRY 9 (#9) − $A_s = 9.0\ in^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(9.0\ in^2)(60\ ksi)}{0.85(4\ ksi)(36\ in)} = 4.41\ in$

$c = a/\beta = 4.41/0.85 = 5.19\ in < 0.375d = 6.04\ in$ ∴ TENSION CONTROLLED $\phi = 0.9$

$\varepsilon_t = \frac{0.003}{5.19}(16.1 - 5.19) = 0.0063 < 0.0075$ ∴ CANNOT REDUCE MOMENTS BY ACI 8.4

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.9(9\ in^2)(60\ ksi)(16.1 - \frac{4.41}{2})(\frac{1}{12})$
$= 562.7\ k\cdot FT > 509.9\ k\cdot FT$ ∴ OK

$\boxed{\text{USE 8 \#9 BARS FOR TOP REINFORCING}}$

BOTTOM REINFORCEMENT (INT. SPAN)

$A_s = \frac{M_u}{4d} = \frac{350.5\ k\cdot FT}{4(16.1\ in)} = 5.44\ in^2$

TRY 6(#9) − $A_s = 6.0\ in^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(6.0\ in^2)(60\ ksi)}{6.85(4\ ksi)(36\ in)} = 2.94\ in$

$c = a/\beta = 2.94\ in/0.85 = 3.46\ in < 6.04\ in$ ∴ TENSION CONTROLLED $\phi = 0.9$

$\varepsilon_t = \frac{0.003}{3.46\ in}(16.1 - 3.46) = 0.011 > 0.0075$ ∴ CAN REDUCE MOMENTS BY ACI 8.4

MOMENT REDISTRIBUTION:

$1000\ \varepsilon_t = 11.0 \rightarrow$ REDUCE BY 11%

$M_u = 350.5\ k\cdot FT(1 - 0.11)$
$= \underline{311.95\ k\cdot FT}$

$A_s = \frac{311.95\ k\cdot FT}{4(16.1\ in)} = 4.84\ in^2 \longrightarrow$ TRY 5(#9) − $A_s = 5.0\ in^2$

$a = \frac{(4.84\ in^2)(60\ ksi)}{0.85(4\ ksi)(36\ in)} = 2.37\ in$

$\phi M_n = 0.9(5.0\ in^2)(60\ ksi)(16.1 - \frac{2.37}{2})(\frac{1}{12}) = 335.6\ k\cdot FT > 311.95\ k\cdot FT$ ∴ OK

$\boxed{\text{USE 5 \#9 BARS FOR BOTTOM REINFORCING}}$

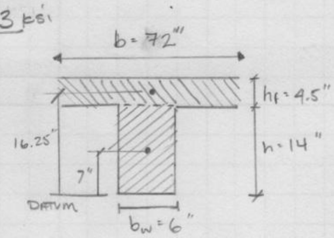
MATT VANDERSALL | TECH REPORT #2 | PAN JOIST SYSTEM | 5/6

PAN JOIST DEFLECTION:

$E_c = 33(w_c)^{1.5}\sqrt{f'_c} = 33(150)^{1.5}\sqrt{4000} = \underline{3834.3 \text{ ksi}}$

$\bar{y} = \frac{\Sigma A d_y}{\Sigma A} = \frac{(6")(14")(7") + (72")(4.5")(16.25")}{(6")(14") + (72")(4.5")}$

$\bar{y} = 14.35 \text{ in}$



$I_s = \frac{bh^3}{12} + Ay^2 = \frac{(72")(4.5")^3}{12} + (72")(4.5")(16.25 - 14.35)^2$

$= 1716.39 \text{ in}^4$

$I_j = \frac{(6")(14")^3}{12} + (6")(14")(14.35" - 7")^2$

$= 5909.9 \text{ in}^4$

$I_{TOTAL} = 5909.9 + 1716.39 = 7626.3 \text{ in}^4$

LOAD ON PAN JOIST:

D.L. – SLAB WT: 337.8 PLF
JOIST WT: 87.5 PLF
S.D.L = 180 PLF

L.L – 480 PLF

$w_{LL} = 0.48$ KLF
$w_{D+L} = 1.09$ KLF

LIVE LOAD DEFLECTION:

$\frac{5 w_u l_n^4}{384 EI} = \frac{5(0.48 \text{ KLF})(19 \text{ FT} - 3 \text{ FT})^4 (1728)}{384(3834.3 \text{ ksi})(7626.3 \text{ in}^4)} = 0.024 \text{ in}$

$\frac{l}{360} = \frac{(16 \text{ FT})(12 \text{ in/FT})}{360} = 0.53 \text{ in} > 0.024 \text{ in} \therefore$ OK

D+L DEFLECTION:

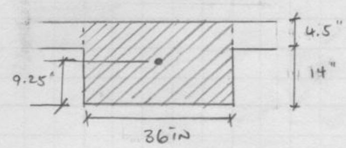
$\frac{5 w_u l_n^4}{384 EI} = \frac{5(1.09 \text{ KLF})(16 \text{ FT})^4 (1728)}{384(3834.3 \text{ ksi})(7626.3 \text{ in}^4)} = 0.055 \text{ in}$

$\frac{l}{240} = \frac{(16 \text{ FT})(12 \text{ in/FT})}{240} = 0.8 > 0.055 \text{ in} \therefore$ OK

GIRDER DEFLECTION:

$E_c = 3834.3$ ksi

$I_G = \frac{(36 \text{ in})(18.5 \text{ in})^3}{12} = 18994.9 \text{ in}^4$



LOAD ON GIRDER:

DL – JOIST + SLAB WT: 1.13 KLF
S.D.L: (30 PSF)(19 FT) = 0.57 KLF
GIRDER WT: 0.69 KLF

L.L – (80 PSF)(19 FT) = 1.52 KLF

$w_{LL} = 1.52$ KLF
$w_{D+L} = 3.91$ KLF

MATT VANDERSALL | TECH REPORT #2 | PAN JOIST SYSTEM | 6/6

LIVE LOAD DEFLECTION:

$$\frac{5w_L l_n^4}{384EI} = \frac{5(1.52\text{ KLF})(34.5-2)^4(1728)}{384(3834.3\text{ ksi})(18994.9\text{ in}^4)} = 0.52\text{ in}$$

$$\frac{l}{360} = \frac{(32.5\text{ FT})(12\text{ in/FT})}{360} = 1.08\text{ in} > 0.52\text{ in} \therefore \text{OK}$$

D+L DEFLECTION:

$$\frac{5w_u l_n^4}{384EI} = \frac{5(3.91\text{ KLF})(32.5\text{ FT})^4(1728)}{384(3834.3\text{ ksi})(18994.9\text{ in}^4)} = 1.35\text{ in}$$

$$\frac{l}{240} = \frac{(32.5\text{ FT})(12\text{ in/FT})}{240} = 1.63\text{ in} > 1.35\text{ in} \therefore \text{OK}$$

COST ANALYSIS: REF - RS MEANS ASSEMBLIES COST DATA 2011.

LOCATION FACTOR = 0.961 (HARRISBURG)

ASSUMED 25 × 30 BAY

125 PSF SUPERIMPOSED LOAD

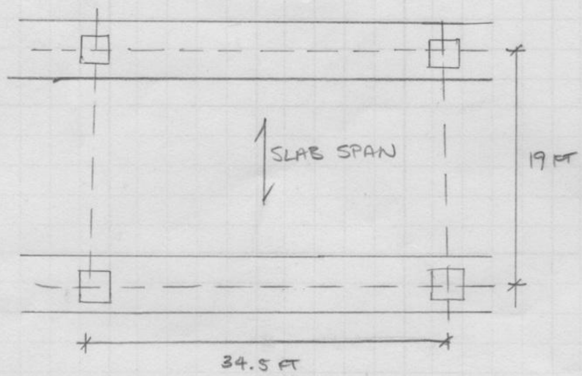
CAST IN PLACE MULTISPAN JOIST SLAB (PG 65)

MAT	INST	TOTAL
7.15	11.30	\$18.45/SQ FT

$18.45/SQ FT (0.961) = $\boxed{\$17.73/\text{SQ FT}}$

Appendix D: One Way Slab and Beam

MATT VANDERSALL | TECH REPORT #2 | ONE WAY SLAB AND BEAM | 1/6



- ASSUME $f'_c = 4$ ksi
  $f_y = 60$ ksi
- ASSUME 24"×24" COLUMNS

$E_c = 33(150)^{1.5}\sqrt{4000 \text{ PSI}}$
$= 3834.3$ KSI

- ASSUME 6' WIDE BEAM

SLAB DESIGN

- S.D.L = 30 PSF
- L.L. = 80 PSF

FROM TABLE 9.5a - MINIMUM THICKNESS OF ONE-WAY SLABS (ACI 9.5.2.2)
SOLID ONE WAY SLABS, BOTH ENDS CONTINUOUS:

$h \geq l/28 = \frac{(13 \text{ FT})(12 \text{ IN/FT})}{28} = 5.57$ in $\longrightarrow$ TRY 6" SLAB DEPTH

MINIMUM REINFORCING FOR SHRINKAGE & TEMPERATURE (ACI 7.12.2.1)

$0.0018 A_g$
$= 0.0018(6 \text{ in})(12 \text{ in/ft}) = 0.13 \text{ in}^2/\text{ft}$

TRY #6 BAR - $A_s = 0.44 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.44 \text{ in}^2/\text{ft})(60 \text{ ksi})}{0.85(4 \text{ ksi})(12 \text{ in/ft})} = 0.647$ in

d, 6", 12 in

$d = 6'' - \underset{\text{COVER}}{\tfrac{3}{4}''} - \tfrac{1}{2}(0.75) = 4.88$ in

$c = 0.647 \text{ in}/0.85 = 0.76 \text{ in} < 0.375(4.88) = 1.83$ in

$\therefore$ TENSION CONTROLLED, $\phi = 0.9$

$\phi M_n = A_s f_y (d - a/2)$
$= 0.9(0.44)(60 \text{ ksi})(4.88 - \frac{0.647}{2}) = 108.3 \frac{\text{in}\cdot\text{k}}{\text{ft}} = 9.02 \text{ ft}\cdot\text{k/ft}$

LOADING ON SLAB:

SELF WT: (150 PCF)(6 in)(12 in)/144 = 75 PLF
SDL = 30 PSF (1 FT) = 30 PLF
L.L. - (80 PSF)(1 FT) = 80 PLF

$w_u = 1.2(0.105 \text{ KLF}) + 1.6(0.08 \text{ KLF}) = 0.254$ KLF

NEGATIVE MOMENT: $\frac{w_u l_n^2}{11} = \frac{(0.254 \text{ KLF})(13 \text{ FT})^2}{11} = 3.9 \frac{\text{K}\cdot\text{FT}}{\text{FT}} < 9.02 \frac{\text{FT}\cdot\text{K}}{\text{FT}}$ $\therefore$ OK

POSITIVE MOMENT: $\frac{w_u l_n^2}{16} = \frac{(0.254 \text{ KLF})(13 \text{ FT})^2}{16} = 2.68 \frac{\text{K}\cdot\text{FT}}{\text{FT}} < 9.02 \frac{\text{FT}\cdot\text{K}}{\text{FT}}$ $\therefore$ OK

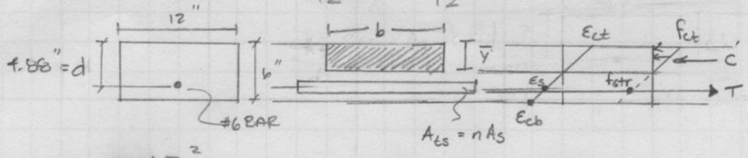
MATT VANDERSALL	TECH REPORT # 2	ONE WAY SLAB AND BEAM	2/6

DEFLECTION:

ACI 9.5.2.3 – EFFECTIVE MOMENT OF INERTIA

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

WHERE $I_g = \frac{bh^3}{12} = \frac{(12\text{ in})(6\text{ in})^3}{12} = 216\text{ in}^4$



$\bar{y} : \frac{b\bar{y}^2}{2} + nA_s\bar{y} - nA_s d \qquad n = \frac{E_s}{E_c} = \frac{29000}{3834.3} = 8$

$= \frac{12\bar{y}^2}{2} + 8(0.44)\bar{y} - 8(0.44)(4.88)$

$= 6\bar{y}^2 + 3.52\bar{y} - 17.18 \leftarrow$ QUADRATIC

$\bar{y} = \frac{-3.52 \pm \sqrt{(3.52)^2 - 4(6)(-17.18)}}{2(6)} = \frac{-3.52 \pm 20.61}{12} = \underline{1.42\text{ in}}$

$I_{cr} = \frac{(12)(1.42)^3}{12} + (12)(1.42)\left(\frac{1.42}{2}\right)^2 + 8(0.44)(4.88-1.42)^2$

$I_{cr} = 2.86 + 8.59 + 42.14 = \underline{53.59\text{ in}^4}$

$M_{cr} = \frac{f_r I_g}{y_t} = \left[\frac{7.5\sqrt{4000}\,(216\text{ in}^4)}{3\text{ in}}\right]\left(\frac{1}{12}\frac{\text{FT}}{\text{IN}}\right)\left(\frac{1\text{ k}}{1000\text{ lb}}\right) = 2.84\text{ k}\cdot\text{FT}$

$M_a = 2.68\ \frac{\text{k}\cdot\text{FT}}{\text{FT}}$

$I_e = \left(\frac{2.68}{2.87}\right)^3(216\text{ in}^4) + \left[1-\left(\frac{2.68}{2.87}\right)^3\right](53.59\text{ in}^4)$

$I_e = 185.8\text{ in}^4$

LIVE LOAD DEFLECTION:

$\frac{5w_L \ell^4}{385 E_c I_e} = \frac{5(0.08\text{ KLF})(19\text{ FT})^4(1728)}{385(3834.3\text{ ksi})(185.8\text{ in}^4)} = 0.33\text{ in}$

$\frac{\ell}{360} = \frac{(19\text{ FT})(12\text{ IN/FT})}{360} = 0.63\text{ in} > 0.33\text{ in} \therefore$ OK

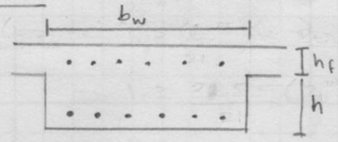
TOTAL LOAD DEFLECTION:

$\frac{5w_{D+L}\ell^4}{385 E_c I_e} = \frac{5(0.185\text{ KLF})(19\text{ FT})^4(1728)}{385\ (3834.3\text{ ksi})(185.8\text{ in}^4)} = 0.76\text{ in}$

$\frac{\ell}{240} = \frac{(19\text{ FT})(12\text{ IN/FT})}{240} = 0.95\text{ in} > 0.76\text{ in} \therefore$ OK

MATT VANDERSALL | TECH REPORT #2 | ONE WAY SLAB AND BEAM | 3/6

BEAM DESIGN



LOADING ON BEAM:

$\omega_{SLAB} = (150\text{ PCF})(6\text{ in})(\frac{1}{12}\frac{\text{FT}}{\text{IN}})(13\text{ FT}) = 975\text{ PLF}$

$\omega_{BEAM} = (150\text{ PCF})(16\text{ in})(\frac{1}{12})(6\text{ FT}) = 1200\text{ PLF}$

$S.D.L = (30\text{ PSF})(19\text{ FT}) = 570\text{ PLF}$

$L.L. = (80\text{ PSF})(19\text{ FT}) = 1520\text{ PLF}$

$\omega_u = 1.2(0.975 + 1.2 + 0.57) + 1.6(1.52) = 5.73\text{ KLF}$

ACI MOMENT COEFFICIENTS:

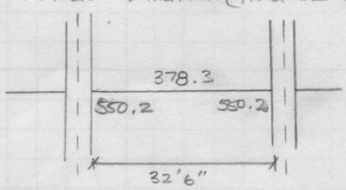
NEGATIVE MOMENT AT OTHER FACES OF INTERIOR SUPPORTS:

$$\frac{\omega_u l_n^2}{11} = \frac{(5.73\text{ KLF})(32.5\text{ FT})^2}{11} = 550.2\text{ K}\cdot\text{FT}$$

POSITIVE MOMENT AT INTERIOR SPANS:

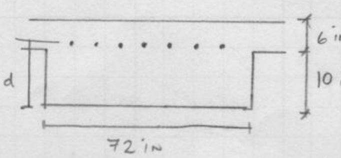
$$\frac{\omega_u l_n^2}{16} = \frac{(5.73\text{ KLF})(32.5\text{ FT})^2}{16} = 378.3\text{ K}\cdot\text{FT}$$

MOMENT DIAGRAM (INTERIOR SPAN)



TOP REINFORCEMENT (INT. SPAN | INT. SUPPORT)

$$A_s \cong \frac{M_u}{4d} = \frac{550.2\text{ K}\cdot\text{FT}}{4(13.76\text{ in})} = 9.99\text{ in}^2$$



TRY 23 #6 BARS ($A_s = 10.12\text{ in}^2$)

$$b_{eff} = \begin{cases} \frac{1}{4}l = \frac{1}{4}(32.5\text{ FT})(12) = \boxed{97.5\text{ in}} \text{ CONTROLS} \\ 8h_{f,left} + b_w + 8h_{f,right} = 8(6) + 72 + 8(6) = 168\text{ in} \\ b_w + l_n = 72 + (32.5\text{ FT})(12) = 462\text{ in} \end{cases}$$

$d = 16" - 1.5" - 0.375" - 0.375" = 13.75\text{ in}$
(DEPTH) (COVER) (#3 STIRRUP) (#6 BAR)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(10.12\text{ in}^2)(60\text{ ksi})}{0.85(4\text{ ksi})(97.5\text{ in})} = 1.83\text{ in}$$

$$c = a/\beta = 1.83\text{ in}/0.85 = 2.15\text{ in} < 0.375d = 0.375(13.75\text{ in}) = 5.16\text{ in}$$

$\therefore$ TENSION CONTROLLED, $\phi = 0.9$

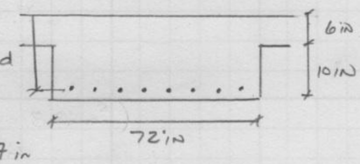
$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= 0.9(10.12\text{ in}^2)(60\text{ ksi})(13.75 - \frac{1.83}{2})/12 = 584.5\text{ FT}\cdot\text{K} > 550.2\text{ FT}\cdot\text{K}$$

$\boxed{\text{USE 23 \#6 BARS FOR TOP REINFORCING}}$

MATT VANDERSALL | TECH REPORT #2 | ONE WAY SLAB AND BEAM | 4/6

BOTTOM REINFORCING (INT. SPAN)

$$A_s = \frac{M_u}{4d} = \frac{378.3\ k\cdot ft}{4(13.75\ in)} = 6.88\ in^2$$



TRY 16 #6 ($A_s = 7.04\ in^2$)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(7.04\ in^2)(60\ ksi)}{0.85(4\ ksi)(97.5\ in)} = 1.27\ in$$

$c = a/\beta = 1.27/0.85 = 1.49\ in < 5.16\ in$ ∴ TENSION CONTROLLED, $\phi = 0.9$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= 0.9(7.04\ in^2)(60\ ksi)(13.75 - \tfrac{1.27}{2})/12 = 415.5\ k\cdot ft > 378.3\ k\cdot ft \therefore OK$$

CHECK SHEAR CAPACITY:

$$V_u = \frac{w_u l_n}{2} = \frac{(5.73\ KLF)(32.5\ ft)}{2} = 93.11\ k$$

$$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{4000}(72\ in)(13.75\ in)/1000 = 93.9\ k$$

TRY #4 BARS AT $s \le d/2 = 13.75/2 = 6\ in$

$A_v = 0.4\ in^2$

$$V_{s\,min} = \frac{A_v f_y d}{s} = \frac{(0.4\ in^2)(60\ ksi)(13.75\ in)}{6\ in} = 55\ k$$

CHECK SPACING ASSUMPTION:

$$V_s = 55\ k \le 4\sqrt{f'_c}\, b_w d = 4\sqrt{4000}(72\ in)(13.75\ in)/1000 = 250.4 \therefore OK$$

CHECK MIN. STEEL REQUIREMENTS:

$$A_{v\,min} = \max \begin{cases} \dfrac{50 b_w s}{f_y} = \dfrac{50(72\ in)(6\ in)}{60{,}000} = \boxed{0.36\ in^2} \\[2ex] \dfrac{0.75\sqrt{f'_c}\, b_w d}{f_y} = \dfrac{0.75\sqrt{4000}(72\ in)(13.75\ in)}{60000} = 0.34\ in^2 \end{cases}$$

$A_v = 0.4\ in^2 > 0.36\ in^2 \therefore OK$

$$\phi V_n = \phi V_c + \phi V_{s\,min} = 93.9\ k + 0.75(55\ k) = 135.15\ k > 93.11\ k \therefore OK$$

$\boxed{\text{USE #4 BARS AT 6 in SPACING}}$

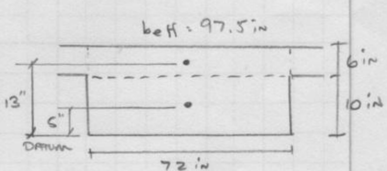
DEFLECTION:

EFFECTIVE MOMENT OF INERTIA

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

WHERE: $I_g = I_s + I_b$

$$\bar{y} = \frac{\Sigma A d_y}{\Sigma A} = \frac{(10\ in)(72\ in)(5\ in) + (6\ in)(97.5\ in)(13")}{(10)(72) + (6)(97.5)}$$
$$= 8.59\ in$$

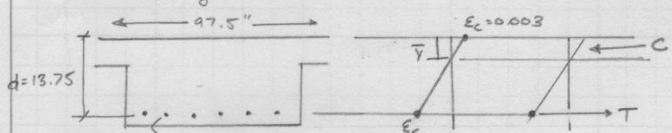


MATT VANDERSALL | TECH REPORT #2 | ONE WAY SLAB AND BEAM | 5/6

$$I_s = \frac{bh^3}{12} + Ay^2 = \frac{(97.5in)(6in)^3}{12} + (97.5in)(6in)(13-8.59)^2$$
$$= 13132.1\ in^4$$

$$I_b = \frac{(72in)(10in)^3}{12} + (72in)(10in)(8.59-5)^2$$
$$= 15279.4\ in^4$$

$$I_g = 13132.1 + 15279.4 = 28411.5\ in^4$$



$$n = \frac{E_s}{E_c} = 8$$

28 #9 (A_s: 28.0 in^2)

$$\bar{y} = \frac{b\bar{y}^2}{2} + nA_s\bar{y} - nA_s d$$
$$= \frac{(97.5'')\bar{y}^2}{2} + 8(28.0)\bar{y} - 8(28.0)(13.75)$$
$$= 48.75\bar{y}^2 + 224\bar{y} - 3080$$

$$\bar{y} = \frac{-224 \pm \sqrt{(224)^2 - 4(48.75)(-3080)}}{2(48.75)} = \frac{-224 \pm 806.7}{97.5} = 5.97\ in$$

$$I_{cr} = \frac{(97.5'')(5.97)^3}{12} + (97.5)(5.97)\left(\frac{5.97}{2}\right)^2 + 8(28.0)(13.75-5.97)^2$$
$$= 1728.8 + 5186.4 + 13558.4 = 20473.6\ in^4$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \left[\frac{7.5\sqrt{4000}\,(15279.4)}{8.59}\right]\left(\frac{1}{12}\frac{ft}{in}\right)\left(\frac{1}{1000}\right) = 70.3\ k\cdot ft$$

$$M_a = 378.3\ k\cdot ft$$

$$I_e = \left(\frac{70.3}{378.3}\right)^2(28411.5) + \left[1 - \left(\frac{70.3}{378.3}\right)^3\right](20473.6in^4)$$

$$I_e = 20527.5\ in^4 \quad 185.3$$

LIVE LOAD DEFLECTION:

$$\frac{5w_L l^4}{385 E_c I_e} = \frac{5(1.52\ klf)(34.5\ ft)^4(1728)}{385(3834.3\ ksi)(20527.5\ in^4)} = 0.61\ in$$

$$\frac{l}{360} = \frac{(34.5\ ft)(12)}{360} = 1.15\ in > 0.61\ in \quad \therefore OK$$

TOTAL LOAD DEFLECTION:

$$\frac{5w_{D+L} l^4}{385 E_c I_e} = \frac{5(4.27\ klf)(34.5\ ft)^4(1728)}{385(3834.3\ ksi)(20527.5\ in^4)} = 1.72\ in$$

$$\frac{l}{240} = \frac{(24.5\ ft)(12)}{240} = 1.73\ in > 1.72\ in \quad \therefore OK$$

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NEED TO RECHECK MOMENT CAPACITY FOR BEAM BOTTOM REINFORCEMENT SINCE BARS CHANGED FROM 16 #6 TO 28 #9 TO CONTROL DEFLECTION

28 #9 ($A_s = 28.0 \text{ in}^2$)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(28.0 \text{ in}^2)(60 \text{ ksi})}{0.85(4 \text{ ksi})(97.5 \text{ in})} = 5.07 \text{ in}$$

$$c = a/\beta = 5.07/0.85 = 5.96 \text{ in} > 0.375(13.75) = 5.16 \text{ in} \therefore \text{NOT TENSION CONTROLLED}$$

$$\varepsilon_t = \frac{0.003}{5.96}(13.75 - 5.96) = 0.004$$

ACCORDING TO R9.3.2.2, INTERPOLATE FOR ϕ

$$\frac{\phi - 0.65}{0.9 - 0.65} = \frac{0.004 - 0.002}{0.005 - 0.002} \longrightarrow \phi = 0.82$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= 0.82(28 \text{ in}^2)(60 \text{ ksi})(13.75 - 5.07/2)/12 = 1287.5 \text{ k}\cdot\text{ft} > 378.3 \text{ k}\cdot\text{ft} \therefore \text{OK}$$

COST ANALYSIS: REF – RS MEANS ASSEMBLIES COST DATA 2011

LOCATION FACTOR = 0.961 (HARRISBURG)
ASSUMED 25 × 30 BAY
125 SUPERIMPOSED LOAD

CAST IN PLACE BEAM & SLAB, ONE WAY (PG 58)

MAT	INST	TOTAL
7.50	12.70	\$20.20/SQ FT (0.961) = $\boxed{\$19.41 \text{/SQ FT}}$