



Technical Report 1
Bed Tower Addition at
Appleton Medical Center
Appleton, WI

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Table of Contents

Executive Summary.....	3
Introduction	4
Code	6
Structural System.....	6
Bracing	6
Foundation.....	7
Floor Construction	9
Construction Materials and Building Loads	11
Floor System Analysis.....	12
Existing Composite Metal Deck with Beams.....	13
One Way Prestressed Hollowcore Planks on Steel Framing.....	14
One Way Slab and Beam.....	15
Two Way Post Tension Flat Plate.....	17
System Comparison	18
Conclusion.....	19
Appendix	20
Appendix A: Gravity Spot Checks – Beam.....	21
Appendix B: Gravity Spot Checks – Girder	23
Appendix C: Gravity Spot Checks – Composite Deck.....	25
Appendix D: One Way P.S. Hollowcore Planks on Steel	26
Appendix E: One Way Slab and Beam.....	28
Appendix F: Two Way Post Tension Flat Plate.....	34
Appendix G: Estimated Cost Calculations	35

Executive Summary

This report was used to study alternative floor systems which could be used for the design of the Bed Tower Addition at Appleton Medical Center. Because they are different systems, a pro-con analysis approach was conducted comparing and contrasting each systems weaknesses and strengths.

Three alternate systems chosen were: 1) One Way Prestressed Hollowcore Plank on Steel Framing, 2) One Way Slab and Beam, and 3) Two Way Post Tension Flat Plate. Each system was analyzed for a typical bay size of 22' x 29'. The existing structural system is a composite metal deck with beams.

Using Nitterhouse Concrete Specifications, the hollowcore plank system resulted in a slab which was 6" in depth in and 4' in width along with a 2" concrete topping and 2 hour fire rating. The AISC Steel Construction Manual was used to design the support for the hollowcore planks and that turned out to be a W24x26 girder. ACI318-08 was used to design both the one-way reinforced concrete slab and beam and two-way post tension system. The design produced a 9" slab thickness with reinforcement being (1) #5 bar per square foot running in the short direction (22'). The beam was designed for a 31" overall depth including the 9" slab. Reinforcement included (15) #5 bars in the top and (10) #5 bars in the bottom each running the long direction (29'). Design for the two-way post tension system resulted in a 7" slab thickness and reinforcement being 14" on center in the E-W direction (long span) and 17" on center in the N-S direction.

It was seen that the geometry of the building made a big impact in choosing which design method was chosen originally. Two systems chosen were ruled unusable due to this, the hollowcore plank and two-way post tension systems. The one way slab and beam would be the best alternative but it would still be difficult to construct. However, if the other two systems could be used for the geometry of this building, the two way post tension would be the best alternative. It proved to have the lowest deflection due to live load of 0.124" and second lowest cost at \$15.15/sq.ft.

Introduction

The Bed Tower Addition at Appleton Medical Center, owned by ThedaCare is located in Appleton, Wisconsin approximately two hours northeast from Madison, Wisconsin. The building was measured at a height of 107'-3" above grade to the highest occupied floor, which entails 9 stories including a basement and the total size is at 152,330 sq. ft. including the renovation which was done on the existing hospital it is attached to.

Picture 1: *Bird's eye view of Appleton Medical Center*



The addition of the bed tower was put into place in order to accommodate more patients for the hospital. Because of its size, it stands out amongst the rest of the complex. It has a unique triangular shape layout which is carried throughout all the

floors of the building. The horizontal streaks of CMU along the exterior make the addition look very sleek and long. Accommodating the long streaks are large areas of glass. Both materials work together to show floor separation and this gives the perception that the addition is taller than it actually is.

The first floor is the lobby area which consists of the registration and waiting area along with a mini coffee shop.

The second floor is the office area which is a very large space and has movable partitions. The third through eighth floors consist of patient rooms, waiting rooms, and floor manager offices. The second to fourth



Picture 2: *Perspective view of Bed Tower Addition entrance*

floors connect to the original hospital with the fourth floor extended into the original building, which is the emergency and surgery center.

The building façade was very simple and consists of two essential components which are a stone façade and large areas of glazing. Limestone and Cast Stone make up the entire exterior with the limestone making up the crown running along the bottom of building. The cast stone is what is seen throughout the rest of the exterior which makes up the vertical façade.

Glazing makes up the other half of the exterior. There are three kinds of glazing. They are: 1) Clear Vision Glass; 2) Tinted Vision Glass; and 3) Spandrel Glass. The clear vision glass is used on the first floor where the lobby is located to allow the most daylight and energy. The tinted vision glass and spandrel glass work together to shade the patient rooms and stairwells and they don't transmit as much sunlight or energy as the clear vision glass.



Picture 3: Bed Tower Addition

Structurally, the addition is made up of a system of steel framing and composite



deck. The foundation is a mat padding. On top of the roof, there is a large penthouse which holds the mechanical equipment which is all supported by the steel framing of the building. For lateral loads, cross bracing is integrated within the frame.

Picture 4: Construction of the addition

Code

International Code

- 2006 International Building Code
 - Live load reduction used for typical floor loads and corridors above the first floor.

Design Codes

- ASTM International
 - Concrete and testing of masonry
- ACI 318
 - Reinforced concrete design and construction
- AISC
 - Structural steel - Designed for “in place” loads
- SDI
 - Steel roof decking
 - Steel composite floor deck - Designed as unshored
- OSHA Safety Standards
 - Steel erection
 - Steel joist erection
 - Metal Decking erection
- ASCE 7-05
 - Wind loads

Structural System

Bracing

Steel braced frames in each direction resist the lateral loads while the concrete slabs act as the diaphragm which transfers the loads to the braced frames. There are 8 sections where the braced frames run vertically throughout the building. The typical frame runs from the top of the foundation to the top of the 10th level penthouse. Two others run to the top of the 9th

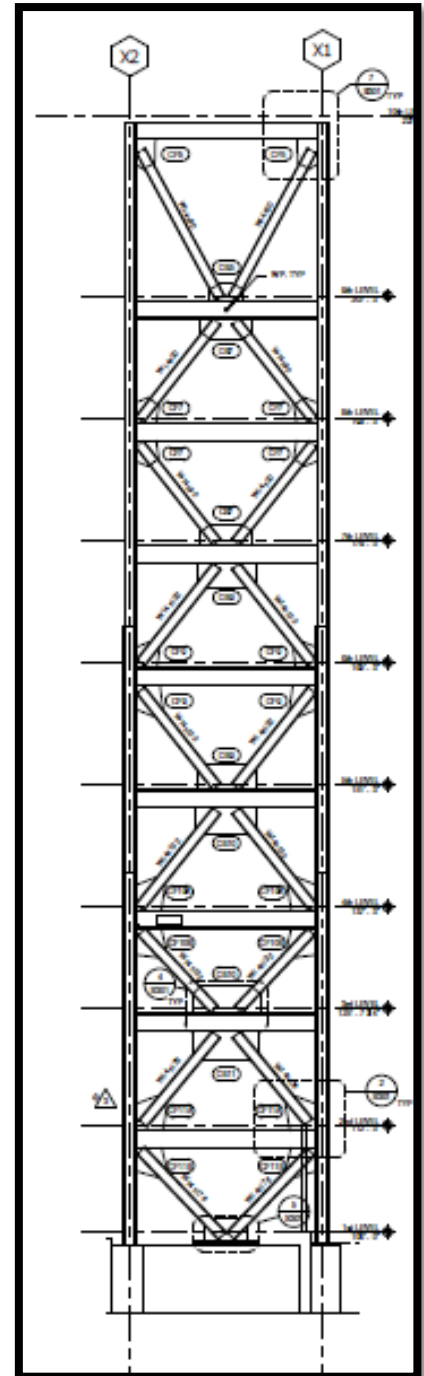


Figure 1:
Elevation of a
braced frame
system

level and the last one runs just between the 9th and 10th level. The locations of the braced frames help resist lateral loads from all directions. These braced frame locations can be found in figure 6 in the foundation section.

Connection to the mat foundation, explained later in the foundation section, help transfer the lateral loads to the base. The braced beams are connected to the columns

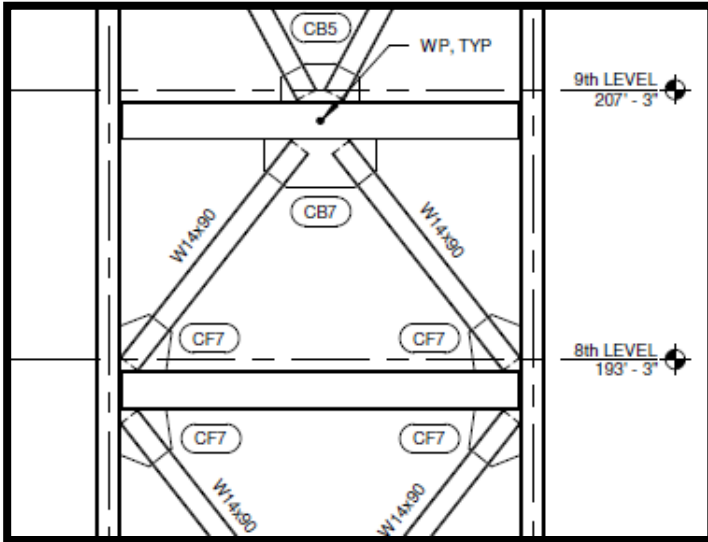


Figure 2: Close-up of the braced frame system

To the right are construction photos of the gusset plates used and connection to the foundation for the braced frames in Figures 3 and 4, respectively.

Foundation

The geotechnical report was completed by River Valley Testing Corporation. Originally, the foundation was designed with spread footing in mind, but after investigation by RVT they recommended three alternatives which included the currently used mat foundation. Tests indicated that the natural soils on the site were able to hold bearing pressures ranging from 1,500 psf to more than 6,000 psf. The footings were then designed for a maximum soil bearing pressure of 3500 psf for just gravity loads and 4200 psf for gravity plus lateral loads. Spread footings range from 6 ft

and floor beams by gusset plates for ease of construction and transfer of loads. Close-up of the braced frames are pictured on the left in Figure 2.



Figure 3 (Above): Close-up of gusset plate construction for the braced frame



Figure 4 (Above): Picture of the braced frame connection to the foundation

x 6 ft to 9 ft by 9 ft with depths being 1 to 2 ft. Maximum allowable interior column loads were to be 1,500 kips and the maximum allowable perimeter wall load be 3 kips per lineal foot.

Typical reinforcement for the mat slab includes the use of #7, #9, and #11 bars. The thickness of the mat slab is 3'6" throughout the entire foundation under the triangular side of the addition. The area where the addition connects to the original part of the building has various thicknesses with 12" being the typical.

Most importantly, the braced frames are connected at the foundation. The concrete bases. Typical thicknesses of these are 4 ft and stretch as long as the column line width. The columns are connected to the bases by plates which are then connected to the top of the concrete by 6 #6 hooks. The bases are reinforced by 5 #5 bars running horizontally and #5 bars running vertically spaced at 12" O.C. Pictured below in Figure 5 is a section and elevation of the braced frame to foundation connection with reinforcement.

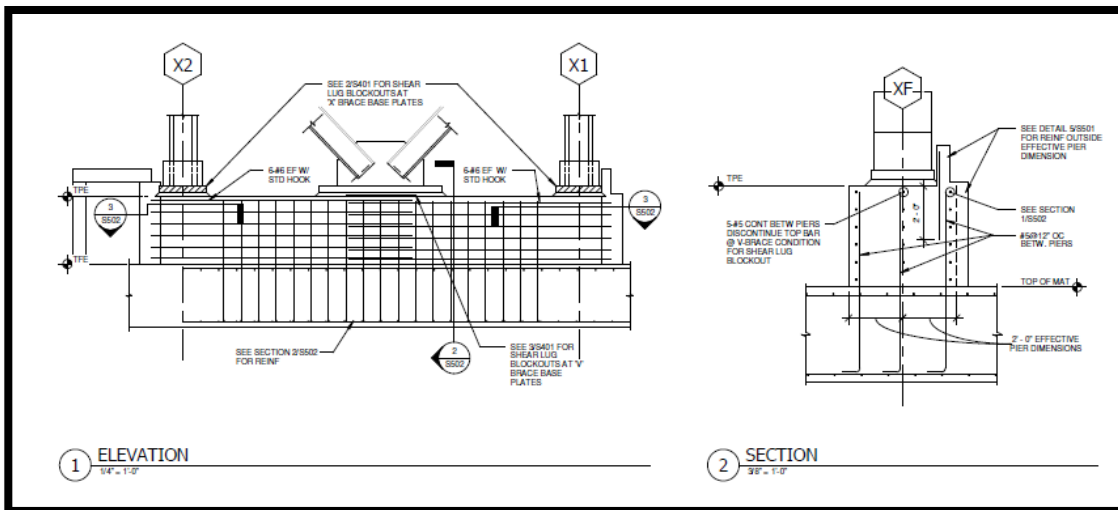


Figure 5: Detail of Typical Foundation Connection for the Braced Frames

Figure 6 shows where the braced frames are connected at the foundation level in green. There is one more braced frame, but as stated earlier in the bracing section, this one is located on the top level.

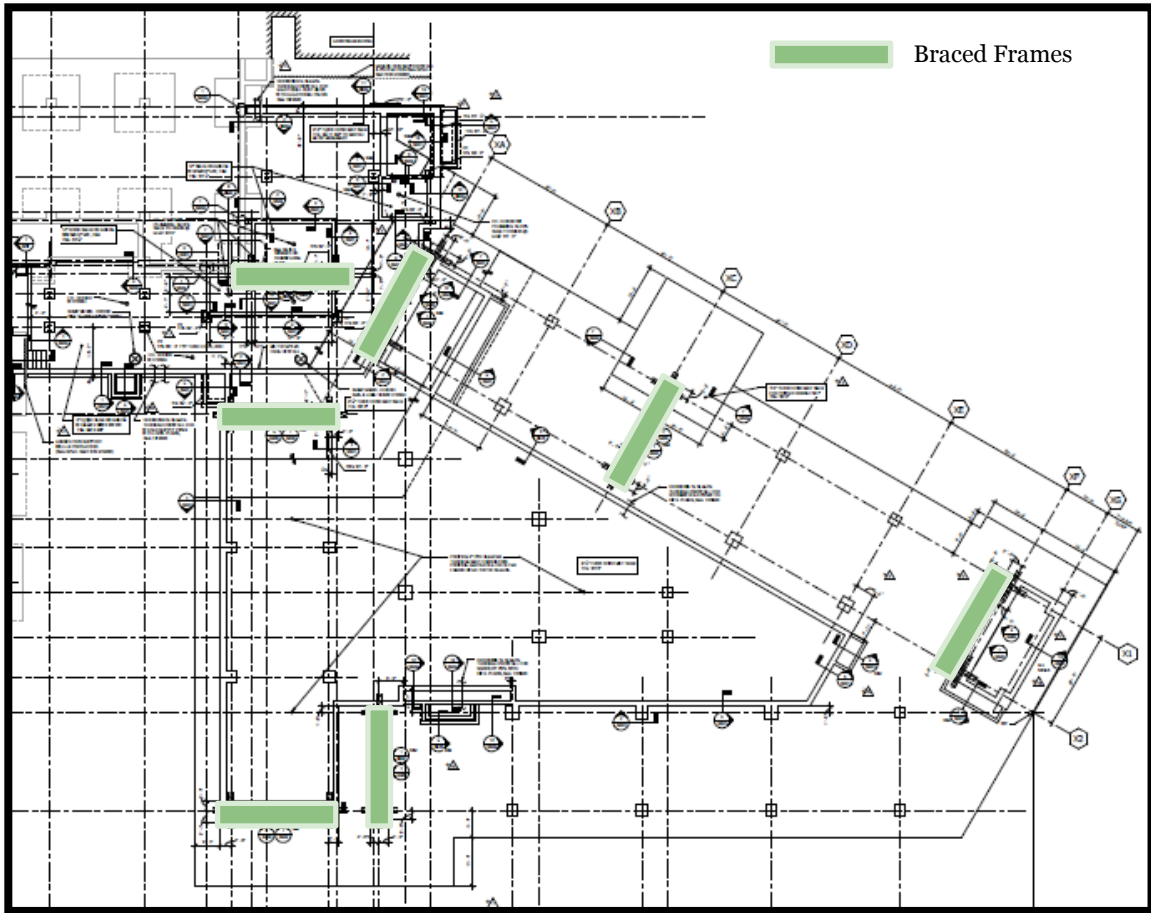


Figure 6: Location of braced frames

Floor Construction

Typical floor construction for the addition included the use 4 types of “deck.” Most floors were constructed of 3”, 18 gage galvanized steel deck with a 4-1/2” normal weight concrete topping, making it a total thickness of 7-1/2” reinforced with 6x6 WWF. One floor was a combination of two decks. One “deck” was a 10” light weight concrete slab which was reinforced with #4 @ 18” O.C. running longitudinally. The other deck was a 2”, 18 gage galvanized steel deck with a 3-1/2” light weight concrete topping making it a total thickness of 5-1/2” and reinforced with 6x6 WWF. Both the galvanized decks are composite and require a stud length of 5” for the 7-1/2” deck and 4” for the 5-1/2” deck. The roof deck was just a 1-1/2” 20 gage galvanized steel decking.

Bay sizes were typically set at 30’, especially on the outer spans of the building where the patient rooms are located. But, due to the irregular shape of the addition, column lines were hard to align so bay sizes within the middle area of the building

ranged in various lengths but came to an average of around 30'. Decking typically spanned 10' and was supported by beams ranging from W14's to W21's with the typical being W16's. Lengths of the beams were typically 22' and were supported by girders ranging from W18's to W24's, but some exterior girders were W30's. Below in Figure 7 is a typical floor plan for floors 4 through 8.

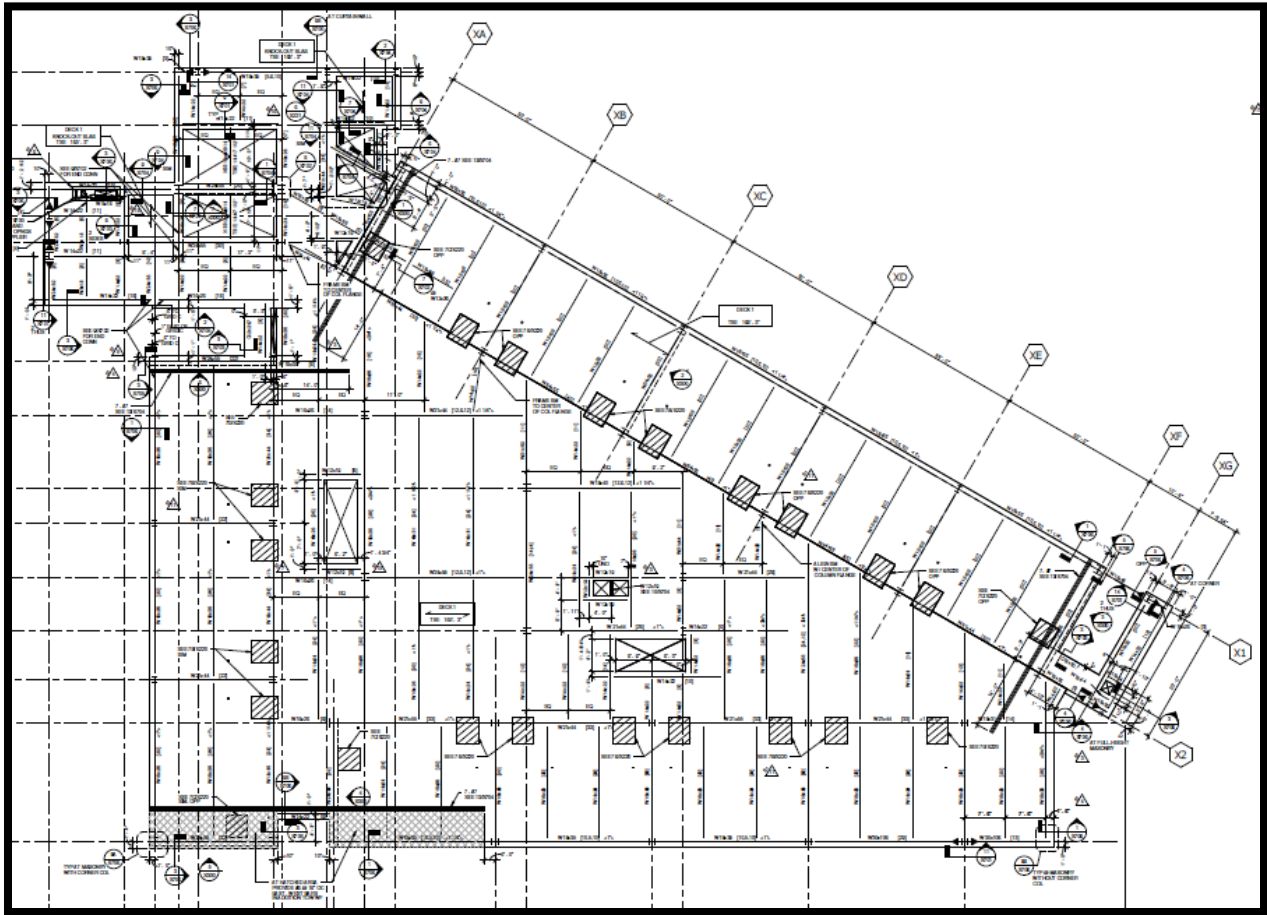


Figure 7: Typical Floor Plan

Construction Materials and Building Loads

Materials used in construction were specified in the general structural notes on Sheet SO01. More information on the materials was found on the floor plans and detailed sections and elevations as well.

Dead Loads	
Material	Load (psf)
Superimposed	30
Composite Deck	
7.5" Thick 3" Steel	75
5.5" Thick 2" Steel	57
Roof	2.14
10" Slab	120 pcf

Figure 8: Dead Loads

Dead loads used for calculations were found in various ways. The composite deck and roof deck were found using the Vulcraft Roof and Steel Deck manual. The weight of the 10" light weight concrete slab was known and it was then assumed a superimposed dead load of 30 psf was used.

Live loads were found using ASCE7-05. Just a quick note on the lives

loads. When doing research, typical hospital floors for patient rooms were found to be 40 psf but it is believed that 80 psf was used because corridors (above 1st floor) with a load of 80 psf controlled. Because the patient rooms were found above the 1st floor, 80 psf was used for ease of calculations although it is a conservative approach to the design.

Properties of Materials		
Material		Strength
Concrete		Weight
Composite Deck	145	f'c (psi)
All other concrete	145	3500
Slabs	115	4000
Steel		Grade
Reinforcing Bars	A615	fy (ksi)
W Shapes	A992	60
Other Shapes	A36	50
Rectangular HSS	A500 - B	36
Round HSS	A500 - B	46
Bolts	A325/A490	42
Studs	A108	60
		50

Figure 9: Properties of Materials

Live Loads		
Occupancy	Design (psf)	Thesis (psf)
Typ. Hosp. Floor	80	80
Corridors (Above 1st Floor)	80	80
Corridors (1st Floor)	100	100
Lobby Floor	100	100
Stair and Exits	100	100
Storage	125	125
Mechanical Room	125	125
Snow Load	34	34

Figure 10: Live Loads

Floor System Analysis

An analysis of various floor systems was used to compare the existing structural system with three others. During the analysis, research and calculations were conducted. All calculations were done by hand and followed ACI 318-08 for concrete and AISC for steel. Assumptions were made for the design of the other three systems to come to their respective solutions. The typical bay used was a 22' x 29' exterior span. Gravity loads were used during calculations. The floor systems which were analyzed for this report include:

- Composite Metal Deck with Beams
- One-way Prestressed Hollowcore Planks on Steel Framing
- One-way Slab and Beam
- Two-way Post Tension

The hand calculations for all systems can be found in the appendix. References to the appendix will be made during the explanation of each individual system. RS Means Cost Works Online was used to estimate approximate costs for each system. Calculations for those costs are located in appendix G.

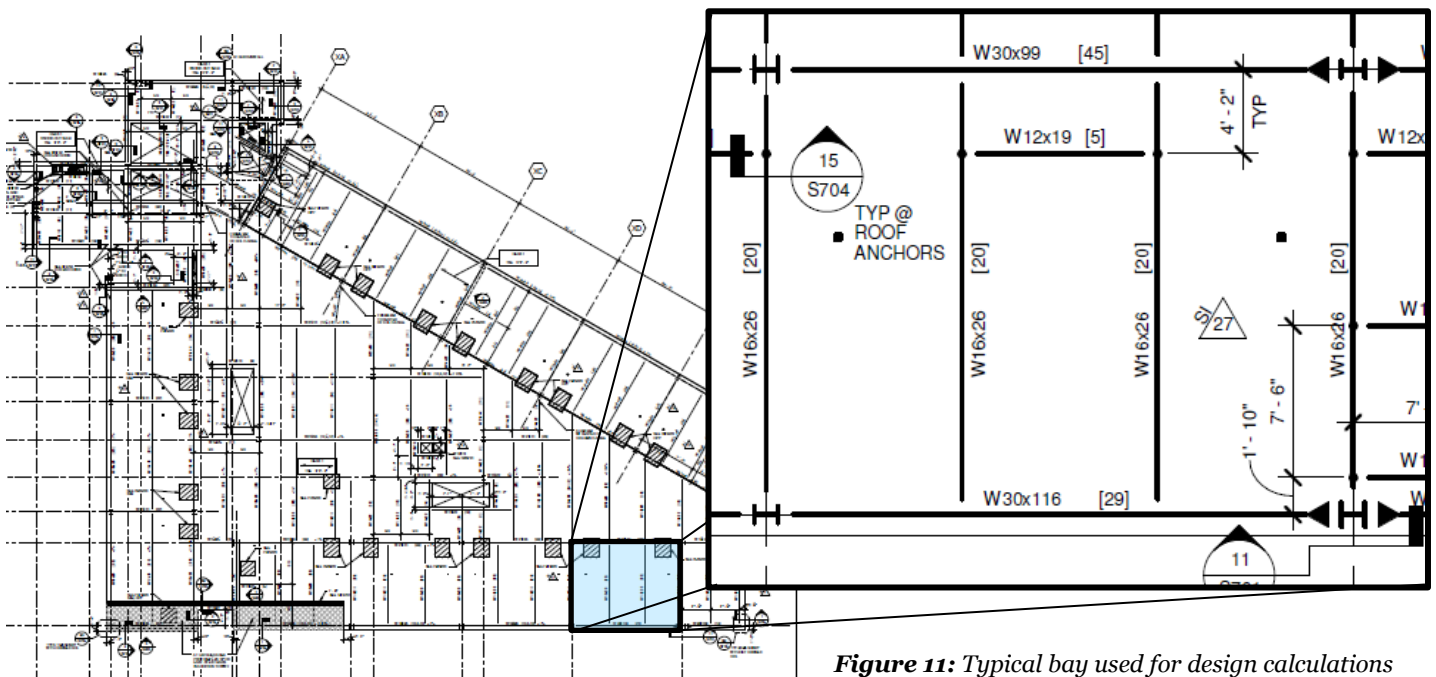


Figure 11: Typical bay used for design calculations

Existing Composite Metal Deck with Beams

The current system in the building is a composite metal deck with beams. As stated earlier in the report there are 4 different decks which were utilized. For the purpose of analyzing this system, the most typical deck and beam where used. The deck is a 3", 18 gage galvanized steel deck with a 4-1/2" normal weight concrete topping making it a total thickness of 7-1/2" reinforced with 6x6 WWF. The beam used during the analyzing process was a W16 x 26 and the girder that was also analyzed was a W21x44. Hand calculations for the beam, girder, and deck can be found in appendices A, B, and C respectively.

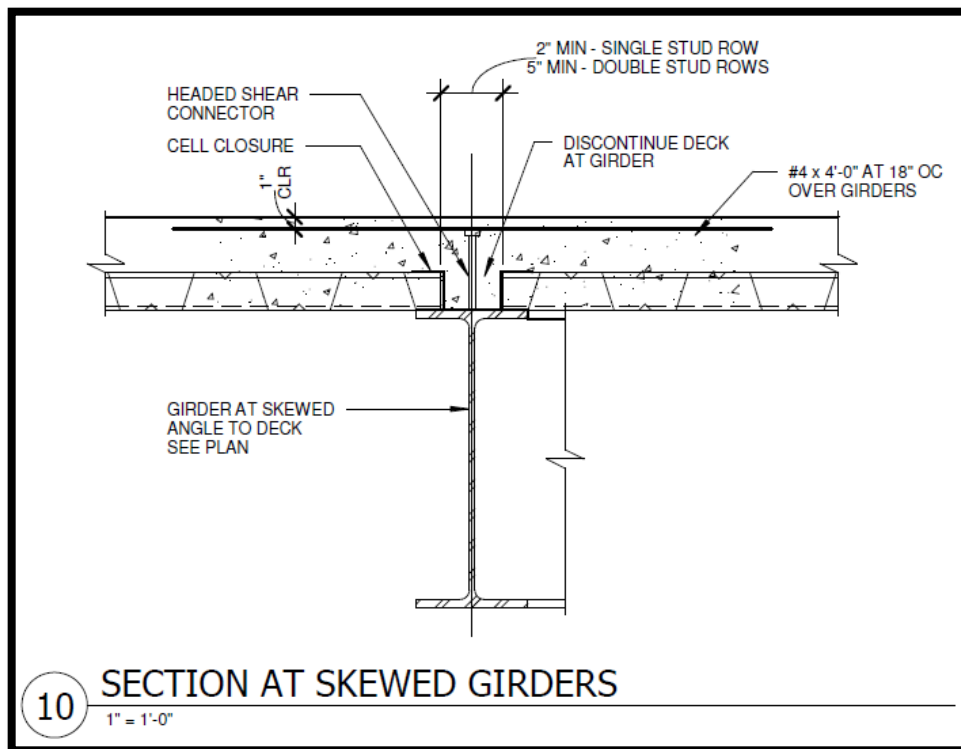


Figure 12: Cross section of a girder perpendicular to the deck

Advantages: There are many advantages to using a composite system. One advantage is they are able to be used for long spans and heavy loads. Two other benefits include the use of smaller and lighter steel beams. Smaller steel beams leads to shorter story heights and slab depths. This is very helpful when constructing a high rise-multi story building. The reduced overall beam depth also means reduction in steel weight. This saves construction cost greatly.

Disadvantages: Because the deck sits atop the steel beam, shear studs are needed in order to provide connection to the two elements. The installation of shear studs increases labor costs as well as material costs based on the number of shear studs need per beam. This could be costly in the long run. Other disadvantages include obstructions for the MEP system because of the smaller beam depth as well as cost in fireproofing all exposed steel.

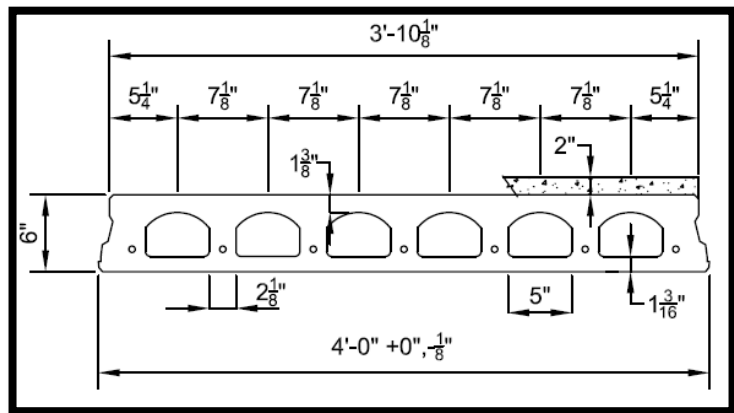
One Way Prestressed Hollowcore Planks on Steel Framing

The first alternate system to be analyzed is the one way hollowcore planks on steel beams. From Nitterhouse, a 6" x 4' (slab depth x slab width) hollowcore plank was picked. It has a 2 hour fire rating (helpful because the existing floor plan requires 2 hour fire rating), 2" topping, and has 7-1/2" diameter strands. The selected plank passed the strength requirements by Nitterhouse Concrete specifications.

The planks spanned the short direction of the bay which was 22' in length. A girder was used to then support the planks. It was designed as simply supported and ran the long direction of the bay which was 29' in length. The beam that met both strength and deflections requirements was a W24x62. The hand calculations for both the hollowcore plank and supporting girder can be found in appendix D.

Figure 13 (Right):
Cross section detail of a hollowcore plank

Figure 14 (Below):
Plank allowable load table.
Arrows point to the allowable load used in calculation



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 1/2"∅	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55	X					X									
6 - 1 1/2"∅	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39										
7 - 1 1/2"∅	LOAD (PSF)	541	492	451	410	364	331	293	274	242	219	190	167	144	124	107	91	77	64	53										

Advantages: Hollowcore planks are very beneficial to the construction process. The panels are pre-made before sent to the construction site within a plant. This then leads to faster installation and overall structural erection because they are at full strength upon arrival. Another advantage is each slab will be produced at a consistent level due to the controlled conditions while being pre-made. A few other advantages include being able to span long lengths, utilize the voids in the hollow core slab for electrical and mechanical runs, as well as using the underside as a finished ceiling.

Disadvantages: Because the slabs come pre-made to specific dimensions, layouts of a structural system would have to be planned for the use of the slabs. If they were to be installed throughout the rest of this building, columns would have to be moved around which will disrupt the existing system. Hollowcore slabs are also very light and when used as a majority of the structural system, can make a building lightweight and more susceptible to failing from an overturning moment due to lateral forces.

One Way Slab and Beam

Unlike the pre-stressed hollowcore planks, the one way slab and beam is a cast in place concrete system. The system is self-explanatory in that reinforcement runs one direction in both the slab and beam with no help from any other supports besides the columns. In this system, slabs usually span perpendicular to the direction of the beams. Because the system relies on both the slab and beam to work together to transfer load to the columns, slab thicknesses and beam widths would have to increase in order to pass deflection limits while maintaining adequate strength.

During the design process of the one way slab and beam, ACI318-08 was used to follow strength and deflections limits. A 22' x 29' bay was analyzed. The slab was calculated to be 9" according to the minimum thickness equation: $h \geq L/28$ where L was the effective length. Calculations resulted in using the 9" slab with (1) #5 rebars per square foot. The beam was assumed to be doubly reinforced requiring reinforcement in both the top and bottom. The total height of the beam was assumed to be 31" including the 9" slab and 3' wide. Top reinforcement was found to be (15) #5 bars and bottom

reinforcement was found to be (10) #5 bars. Shear reinforcement for the beam also came out to be #5 bars at 14" spacing.

Both the slab and beam passed strength requirements easily which means the design was very conservative. Design improvements could be made including reducing the slab thickness and using a lower bar size. For the beam, the slab height could be greatly reduced and the bars could be kept at the same size. Hand calculations can be found in appendix E.

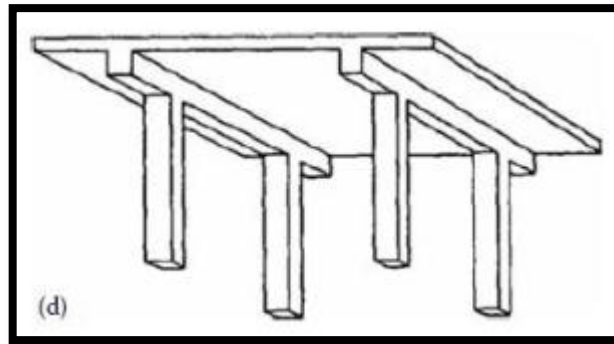


Figure 15: Picture of a typical one way beam and slab

Advantages: One way slab and beam is advantageous for large ratio bay sizes. Slab to ceiling heights are low and flat between beams. If slab and beam sizes are consistent throughout the building, formwork can be reused over and over again reducing labor and formwork costs. They are also able to span large lengths and increase usable area depending on the size of the beams and columns. Another advantage to this system is the overall weight of the building will be much larger than a steel building, resulting in needing a large overturning moment.

Disadvantages: Concrete takes time to cure and so time to construct a one way slab and beam system would have to be considered when scheduling. A larger foundation would also have to be designed for the increased amount of weight. Other disadvantages include obstruction of the MEP systems due to the increase in slab and beam depth, and columns would need to be greatly increased to account for the larger loads.

Two Way Post Tension Flat Plate

For long spans and thinner slabs, post tensioning is one of the best systems to account for both. It has greater deflection and crack control. If done the correct way, post tensioning can ensure that the concrete takes all the compression. Pre-stressed grouped tendons are the reason for this. This method also allows for greater strength capacities.

The design of the two way post tension system was analyzed for a typical exterior bay of 22' by 29'. The tendon profile spans one way and only one bay in this design. A slab thickness was found to be 7" and depth to be 6" due to a 3/4" clear cover and 1/2" diameter duct for the 1/2" diameter 7 wire 270 k tendons used. In the E-W direction, the short direction, the tendons were to be placed at 17" apart O.C. In the N-S direction, the long direction, the tendons were to be placed at 14" apart O.C.

Like the one way slab and beam system, the post tensioning within the bay were greatly below the allowable strength capacities. It also passed the shear strength capacities but it was much closer to the allowable than the moment strengths. Hand calculations can be found in appendix F.

Advantages: Post-tensioning work wells in every area the structural field. As stated earlier, construction of a post tensioning system allows for thinner slabs, long clear spans, and much fewer if not any beams. Because thinner slabs are designed, floor to floor height greatly decreases which also leads to use of less concrete and lower construction costs. Other advantages of post-tensioning include free roam for mechanical and electrical equipment, continuous slabs, and reduced foundation load.

Disadvantages: Labor costs can become high due to the amount of equipment needed to pre-stress the tendons. They also take some time to prepare because formwork is needed and concrete needs time to cure. When taking into account a flat plate system where it be post tensioned or not, there will still be problems in controlling deflection. Punching shear is also another problem which could arise. In this case, if it is impossible to design an efficient flat plate system, drop panels could be taken into consideration.

System Comparison

Typical Bay Floor System Comparison				
Characteristics	Floor System			
	(Existing) Composite Metal Deck w/ Beams	One-Way P.S. Hallowcore Plank	One-Way Slab and Beam	Two-Way Post Tension
Overall Depth	Various	8"	31"	7"
Slab Depth	7-1/2"	6"	9"	7"
Deflection (w/L.L.)	0.227"	0.236"	0.169"	0.124"
Bay Size Impact	No	Yes	Yes	Yes
Foundation Impact	No	No	Yes	Yes
Architectural Impact	No	Yes	Yes	Yes
Add Fire Protection (slab)	Yes	No	No	No
Add Fire Protection (other)	Yes	Yes	No	No
Rire Rating (hour)	2	2	2	2
Cost (per sq. ft.)	\$17.41	\$13.71	\$17.07	\$15.15
Constructable	Easy	Easy	Moderate	Hard
Formwork Needed	No	No	Yes	Yes
Extra Time	N/A	No	Yes	Yes
Plausible	Yes	No	Yes	No

Conclusion

The three alternative systems analyzed in this report were the one way hollowcore plank on steel framing, one way slab and beam, and the two way post tension flat plate.

The one way hollowcore plank system resulted in a slab which was 6" in depth and 4' in width. The concrete topping was 2" and the fire rating was 2 hours. Specs for the hollowcore plank were designed by the help from Nitterhouse concrete. The girder used to support to hollowcore planks was a W24x62.

For the one way slab and beam it was designed for a slab depth of 9" and needed (1) #5 rebar per square foot of concrete running the short direction of 22'. The beam was assumed to be 31" in depth which also included the 9" slab leaving the depth of the beam itself to be 22". The beam needed reinforcement of (15) #5 bars for top reinforcement and (10) #5 bars for bottom reinforcement.

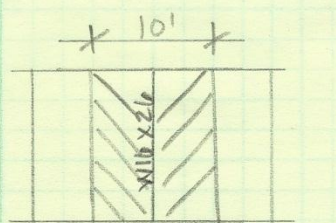
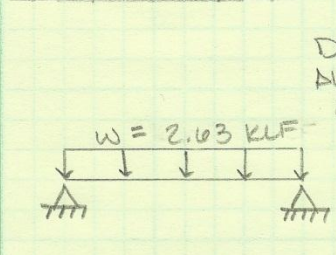
Lastly, the two way post tension system only required a slab depth of 7". Reinforcement needed for the system included tendons placed 14" on center in the E-W direction (long span) and 17" on center in the N-W direction (short span).

Once all three systems were designed, each explained the advantages and disadvantages. One factor stood out in the design for all 4 systems though including the existing one, the geometry of the building. Due to the triangular shape of the addition, it can be seen why steel was chosen as the primary design method. Typical bays were only found on the exterior of the building. On the interior of the building, bays became very sporadic and were inconsistent.

The hollowcore plank system would not be very usable. Because of the set rectangular dimensions, the building would have to move many columns in order to accommodate for the widths of the hollowcore slabs. The one way post tension system would be constructible but it would be difficult to do once the beams have to turn toward the northeast to support those patient rooms. Just like the hollowcore plank system, the two way post tension system would not be very usable either or at least this would be very difficult. The triangular shape of the building would not work for a two way slab system.

Appendix

Appendix A: Gravity Spot Checks – Beam

AE SENIOR THESIS	SPOT CHECKS	JESSE ELLIOTT										
COMPOSITE BEAM: W16 X 26 [20]												
FROM AISC STEEL MANUAL: $A_g = 7.68 \text{ in}^2$ $I_x = 301 \text{ in}^4$ $Z_x = 44.2 \text{ in}^3$												
	TRIB WIDTH: 10' SPAN: 22'											
<table border="0" style="width:100%"> <tr> <td style="text-align:center"><u>DEAD LOAD</u></td> <td style="text-align:center"><u>LIVE LOAD</u></td> </tr> <tr> <td>SD = 30 PSF</td> <td>FLOOR: 80 PSF</td> </tr> <tr> <td>DECK = 75 PSF</td> <td></td> </tr> <tr> <td>ALLOW. = 5 PSF</td> <td></td> </tr> <tr> <td>SELF = 26 PLF</td> <td></td> </tr> </table>			<u>DEAD LOAD</u>	<u>LIVE LOAD</u>	SD = 30 PSF	FLOOR: 80 PSF	DECK = 75 PSF		ALLOW. = 5 PSF		SELF = 26 PLF	
<u>DEAD LOAD</u>	<u>LIVE LOAD</u>											
SD = 30 PSF	FLOOR: 80 PSF											
DECK = 75 PSF												
ALLOW. = 5 PSF												
SELF = 26 PLF												
	$w = 1.2D + 1.6L$ $D = (30 + 75 + 5)(10') + 26 = 1126 \text{ PLF}$ $L = (80)(10') = 800 \text{ PLF}$											
$w = 1.2(1126) + 1.6(800) = 2631.2 \text{ PLF} = 2.63 \text{ KLF}$												
$V = \frac{(2.63)(22)}{2} = 28.9 \text{ K}$	$M = \frac{wL^2}{8} = \frac{(2.63)(22)^2}{8} = 159.1 \text{ K}\cdot\text{ft}$											
beff: $\frac{\text{SPAN}}{8} = \frac{22' \times 12''}{8} = 33 \text{ in}$ ← CONTRAS FOR BOTH SIDES min $\frac{1}{2}(\text{SPACING}) = \frac{10' \times 12''}{2} = 60 \text{ in}$ beff = 2(33 in) = 66 in												
FROM TABLE 3-19												
PNA = 7 $\Sigma Q_n = 96.0$												
$a = \frac{\Sigma Q_n}{0.85 f_c \text{ beff}} = \frac{96.0}{0.85(3.5)(66 \text{ in})} = 0.489 < 1.0$ use a = 1.0												
$y_2 = t_{\text{slab}} - a/2 = 7.5 - 1/2 = 7.0''$												
$\phi M_n = 259 \text{ k}\cdot\text{ft} > 159.1 \text{ k}\cdot\text{ft}$ <u>OK</u> ✓												
$Q_n = \frac{96.0}{14.6} = 6.6 \sim 7$ 14 STUDS NEEDED 20 USED <u>OK</u> ✓												
SHEAR CHECK: (TABLE 3-2) $\phi V_n = 106 > 28.9$ <u>OK</u> ✓												

CHECK DL:

$$\Delta u = \frac{l}{360} = \frac{(22')(12")}{360} = 0.73 \text{ in}$$

FROM TABLE 3-20:

W16 X 26 PNA 7, $I_{LB} = 640 \text{ in}^4$

$$\Delta u = \frac{5 w u l^4}{384 EI} = \frac{5 \left(\frac{80 \cdot 10}{1000} \right) (22)^4 (1728)}{384 (29000) (640)} = 0.227$$

$$0.227 \text{ in} < 0.73 \text{ in} \quad \underline{\text{OK}} \checkmark$$

CHECK BEAM DEFLECTIONS
UNDER WET CONCRETE

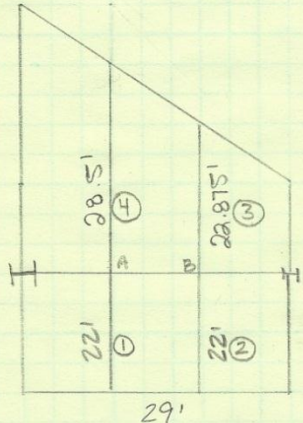
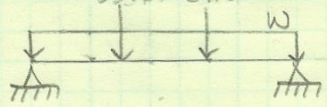
$$\Delta_{\max} = \frac{l}{240} = \frac{(22')(12")}{240} = 1.1 \text{ ''}$$

$$I = \frac{5 w l^4}{384 E \Delta_{\max}} = \frac{5 \left(\frac{.75 + 5(22) + 26}{1000} \right) (22)^4 (1728)}{384 (29000) (1.1 \text{ ''})}$$

$$= 295.1 \text{ in}^4 < I_x = 301 \text{ in}^4 \quad \underline{\text{OK}} \checkmark$$

W16 X 26 OK TO USE!

Appendix B: Gravity Spot Checks – Girder

AE SENIOR THESIS	SPOT CHECKS	JESSEL ELLIOTT
COMPOSITE GIRDER: W18 x 35 [28]		$A = 10.3 \text{ in}^2$ $I_x = 570 \text{ in}^4$ $Z_x = 66.5 \text{ in}^3$
LOAD FROM BEAMS: [ALL W16 x 26]		
	$\textcircled{1} + \textcircled{2} \quad D = (30 + 75 + 5)(9.67) + 26$ $= 1126 \text{ PLF}$ $L = 80(10) = 800 \text{ PLF}$ $W = 1.2(1126) + 1.6(800)$ $= 2.63 \text{ KLF}$ $P = \frac{(22')(2.63 \text{ KLF})}{2} = 28.9 \text{ K}$ $PL = 8.8$ $\textcircled{3} \quad P = \frac{(22.875')(2.63 \text{ KLF})}{2} = 30.1 \text{ K}$ $PL = 9.15$ $\textcircled{4} \quad P = \frac{(28.5')(2.63 \text{ KLF})}{2} = 37.5 \text{ K}$ $PL = 11.4$	
$P_A = \frac{(28.9 + 37.5)}{2} = 33.2 \text{ K}$ $P_B = \frac{(28.9 + 30.1)}{2} = 29.5 \text{ K}$		$P_A = \frac{(11.4 + 8.8)}{2} = 10.1$
		$w = 35/1000 = .035 \text{ KLF}$
$R_1 = V_1 = \frac{P_A(l-a) + P_B b}{l} = \frac{33.2(29-9.67) + 29.5(9.67)}{29}$ $R_1 = 32.0 \text{ K}$		
$R_2 = V_2 = \frac{P_B(a) + P_A(l-b)}{l} = \frac{33.2(9.67) + 29.5(29-9.67)}{29}$ $R_2 = 30.7 \text{ K}$		
$V_u = 32.0 \text{ K}$		
$M_{max} = R_1 a = (32.0 \text{ K})(9.67) = 309.3 \text{ K}\cdot\text{ft}$		
$V_u = .035(29')/2 = 0.5 \quad V_{u_{TOT}} = 32.5$		
$M = \frac{wl^2}{8} = \frac{(.035)(29)^2}{8} = 3.7 \text{ K}\cdot\text{ft} \quad M_{u_{TOT}} = 313.0 \text{ K}\cdot\text{ft}$		

(TABLE 3-19)

$$PNA = 7 \quad \Sigma Q_n = 129$$

$$\begin{array}{l|l} \text{beff} & \frac{\text{SPAN}}{8} = \frac{29(12)}{8} = 43.5'' \leftarrow \text{CONTROLS} \\ \text{min} & \frac{\text{SPACING}}{2} = \frac{22(12)}{2} = 132'' \quad \text{beff} = 87'' \end{array}$$

$$a = \frac{\Sigma Q_n}{0.85 f_c \text{beff}} = \frac{129}{0.85(3.5)(87 \text{ in})} = 0.50 < 1.0 \quad \text{use } a = 1.0$$

$$jz = t_{\text{SLAB}} - a/2 = 7.5 - 1/2 = 7.0$$

$$\phi M_n = 382.0 \text{ k-ft} > 313.0 \text{ k-ft} \quad \text{OK} \checkmark$$

$$\text{(TABLE 3-2)} \quad \phi V_n = 159 \text{ k} > 32.5 \text{ k} \quad \text{OK} \checkmark$$

$$\text{DEFLECTION} \quad I_b = 1030 \quad \Delta_{un} = l/360 = 29(12)/360 = 0.967$$

$$\Delta_U = \frac{Pl^3}{28EI} + \frac{5w_l l^4}{384EI} = \frac{10.1(29)^3(1728)}{28(29000)(1030)} + \frac{5(1.035)(29)^4(1728)}{384(29000)(1030)}$$

$$\Delta_U = 0.527 < 0.967 \quad \text{OK} \checkmark$$

DEFLECTION W/ WET CONCRETE

$$\Delta_{\text{MAX}} = l/240 = 29(12)/240 = 1.45$$

$$I = \frac{Pl^3}{28EI} + \frac{5w_l l^4}{384EI} = \frac{10.1(29)^3(1728)}{28(29000)(1.45)} + \frac{5(1.035)(29)^4(1728)}{384(29000)(1.45)}$$

$$I = 374.8 \text{ in}^4 < 510 \text{ in}^4 \quad \text{OK} \checkmark$$

W18x35 WIPES

Appendix C: Gravity Spot Checks – Composite Deck

AE SENIOR THEISS	SPOT CHECKS	JESSEL ELLIOTT
DECK CHECK IN TYPICAL 30' BAY FLOOR 7		
		<p><u>DECK INFO FROM VULCRAFT</u></p> <p>3" DECK 4 1/2" TOPPING 18 GAUGE NW CONCRETE 75 PSF FC = 3500 PSI</p>
<p>LL = 80 PSF DL = 30 PSF ← SUPERIMPOSED</p>		
<p>3VL118 w/ UNSHORED LENGTH 13'-3" FOR 3-SPAN CONDITION</p>		
<p>13'-3" > 10' <u>OK</u> ✓</p>		
<p>SUPERIMPOSED LL (PSF)</p>		
<p>CLEAR SPAN - 10' SLL = 294 PSF > 110 PSF <u>OK</u></p>		
<p>FROM GENERAL NOTES, FLOOR CONSTRUCTION SHOULD BE 2-HR RATING.</p>		
<p>ACCORDING TO FIRE RATINGS FROM VULCRAFT DECK IS 2HR RATING</p>		
<p style="border: 1px solid black; padding: 5px; display: inline-block;">3VL118 <u>OK</u> ✓</p>		

Appendix D: One Way P.S. Hollowcore Planks on Steel

HOLLOW CORE PLANKS	AE SENIOR THESIS	JESSEL ELLIOTT	1/2
	<p>ACCORDING TO SHEET A010, FLOOR CONSTRUCTION SHOULD BE A 2-HR RATING.</p>		
	<p>CHECK IF 6" X 4' HOLLOWCORE 2 HR FIRE RATING (2" TOPPING) w/ 7-1/2" ϕ STRANDS</p>		
	<p>SPAN = 22' D.L. = 30 PSF L.L. = 80 PSF FROM NITTERHOUSE CONCRETE PRODUCTS @ 22' SPAN w/ 7-1/2" ϕ SERVICE LOAD ALLOWABLE = 190 PSF > 30+80 = 110 PSF <u>OK</u></p>		
	<p>CHECK DEFLECTION: SELF WT. 48.75 PSF (4') = 195 PLF D.L. : 30 PSF (4') = 120 PLF L.L. : 80 PSF (4') = 320 PLF $W_u = 1.2(195+120) + 1.6(320)$ $= 890$ PLF</p>		
	<p>$\Delta_u = \frac{5W_u l^4}{384EI}$ $I = 1519 \text{ in}^4$ $E = 33(150)^{1.5} \sqrt{60000}$ (FROM NITTERHOUSE) $= 4696 \text{ KLF}$ $= \frac{5(0.32)(22)^4(1728)}{384(4696)(1519)} = 0.236 \text{ ''}$</p>		
	<p><math>\Delta_{allow} = \frac{l}{360} = \frac{(22')(12'')}{360} = 0.73 \text{ ''} > 0.236 \text{ ''} \text{ <u>OK</u>}</math></p>		

HOLLOW CORE PLANKS	AE SENIOR THESIS	JESSEL ELLIOTT	2/2
GIRDER DESIGN FOR THE PLANKS			
SELF + DEAD = $(22)(48.75 + 30) = 1732.5$ PLF LIVE = $(22)(80) = 1760$ PLF			
$W_u = 1.2(1.7325) + 1.6(1.76) = 4.895$ KLF			
CALCULATE SHEAR & MOMENT			
$V_u = \frac{w_u l_0}{2} = \frac{(4.895)(29)}{2}$			
$V_u = 71.0$ K			
$M_u = \frac{w_u l_0^2}{8} = \frac{(4.895)(29)^2}{8} = 514.6$ K-FT			
FIND MIN I :			
FROM LL			
$\frac{l}{360} = \frac{S w_{LL} l^4}{384 EI}$			
$\frac{29'(12"/1')}{360} = \frac{S(80 \times 22/1000)(29)^4(1728)}{384(29000)(I)}$			
$\Rightarrow I_{min} = 999.1$ in ⁴			
FROM TL			
$\frac{l}{240} = \frac{S w_{TL} l^4}{384 EI}$			
$\frac{29(12)}{240} = \frac{S(158.75 \times 22/1000)(29)^4(1728)}{384(29000)(I)}$			
$\Rightarrow I_{min} = 1321.7$ in ⁴ ← CONTROLS			
FROM STEEL MANUAL:			
TRY : W24 x 55 (MOST ECONOMICAL)			
$I_x = 1350$ in ⁴ > 1321.7 in ⁴ <u>OK</u>			
$\phi M_p = 503$ K-FT < 514.6 <u>NG</u>			
TRY : W24 x 62 (NEXT MOST ECONOMICAL)			
$I_x = 1550$ in ⁴ > 1321.7 in ⁴ <u>OK</u>			
$\phi M_p = 574$ K-FT > 514.6 <u>OK</u>			
$\phi V_n = 306$ K >> 71 K <u>OK</u>			
USE 6" x 4" HOLLOWCORE PLANK W/ 2" TOPPING, 7-1/2" ϕ AND W24 x 62 AS GIRDER			

Appendix E: One Way Slab and Beam

ONE WAY BEAM & SLAB	AE SENIOR THESIS	JESSEL ELLIOTT	1/6
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ASSUME $f_c = 4 \text{ KSI}$
 $f_y = 60 \text{ KSI}$

3/4" COVER 24" x 24" COLUMNS.
3' WIDE BEAM

LOADS: SUPERIMPOSED D.L. = 30 PSF
LIVE LOAD = 80 PSF

* ACI 318-08 USED

$l = 22' - 2' = 20'$

DESIGN OF SLAB

FROM ACI 9.5.2.2 TABLE 9.5a - Min thickness of one-way slabs
SOLID ONE-WAY SLAB, BOTH ENDS CONTINUOUS:
 $h \geq \frac{l}{28} = \frac{(20')(12")}{28} = 8.6" \Rightarrow$ TRY 9" SLAB THICK.

MINIMUM REINFORCING FOR SHRINKAGE & TEMP (7.12.2.1)
(b) Slabs where Grade 60 is used... $0.0018 A_g$
 $= .0018(9")(12"/ft) = 0.1944 \text{ in}^2/\text{ft}$
USE #5 BAK w/ $A_s = 0.31 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.31)(60 \text{ KSI})}{(0.85)(4 \text{ KSI})(12")}$ TENSION CONTROLLED $\phi = 0.9$
 $= 0.456 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{0.456 \text{ in}}{0.85} = 0.536 \text{ in} < 0.375(9") = 3.38"$

$d = 9" - 3/4" - \frac{0.625}{2} = 7.94 \text{ in}$

9" 12"

$\phi M_n = A_s f_y (d - a/2)$
 $= (0.31)(60)(7.94 - 0.456/2) = 143.4 \frac{\text{in} \cdot \text{lb}}{\text{ft}}$
 $= 11.9 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$

LOADS

SELF: $150 \text{ PCF}(7") \times (12 \text{ in}) / 144 = 87.5 \text{ PLF}$
SDL: $30 \text{ PSF}(1') = 30 \text{ PLF}$
L.L.: $80 \text{ PSF}(1') = 80 \text{ PLF}$

$W_u = 1.2(87.5 + 30) + 1.6(80) = 269 \text{ PLF} = 0.269 \text{ KLF}$

FROM ACI 8.3.3 FIND POSITIVE & NEG. MOMENTS

INTERIOR SPAN POSITIVE MOMENT: $\frac{W_u l_n^2}{16} = \frac{(0.269)(20')^2}{16} = 6.7 \frac{\text{K} \cdot \text{ft}}{\text{ft}} < 11.9 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$ OK

NEGATIVE MOMENT: $\frac{W_u l_n^2}{11} = \frac{(0.269)(20')^2}{11} = 9.78 \frac{\text{K} \cdot \text{ft}}{\text{ft}} < 11.9 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$ OK

ONE WAY BEAM & SLAB	AE SENIOR THESES	JESSEL ELLIOTT	2/6
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DEFLECTION:
 FIND EFFECTIVE MOMENT OF INERTIA USING EQ F/ ACI 9.5.2.3

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

$$n = \frac{E_s}{E_c} \quad E_c = 33(150)^{1.5} \sqrt{4000 \text{ psi}} = 3834.3 \text{ ksi}$$

$$n = \frac{29000}{3834.3} = 8 \quad I_g = \frac{bb^3}{12} = \frac{(12'')(9'')^3}{12} = 729 \text{ in}^4$$

9"
 7.74"
 #5

12"
 STEEL

c
 d
 T

$$\bar{y} \Rightarrow \frac{b\bar{y}^2}{2} + n A_s \bar{y} - n A_s d = \frac{12\bar{y}^2}{2} + 8(0.31)\bar{y} - 8(0.31)(7.94)$$

$$0 = 6\bar{y}^2 + 2.48\bar{y} - 19.69$$

$$\bar{y} = \frac{-2.48 \pm \sqrt{(2.48)^2 - 4(6)(-19.69)}}{2(6)} = 1.62''$$

$$M_a = 6.7 \text{ K} \cdot \text{FT} / \text{FT}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{7.5 \sqrt{4000} (729)}{4.5''} \left(\frac{1 \text{ FT}}{12 \text{ in}}\right) = 6404 \text{ lb} \cdot \text{FT} = 6.40 \text{ K} \cdot \text{FT}$$

$$I_{cr} = \frac{12(1.62)^3}{12} + (12)(1.62)\left(\frac{1.62}{2}\right)^2 + 8(0.31)(7.94 - 1.62)^2$$

$$= 116 \text{ in}^4$$

$$I_e = \left(\frac{6.40}{6.70}\right)^3 (729) + \left[1 - \left(\frac{6.40}{6.70}\right)^3\right] (116) = 650 \text{ in}^4$$

DEFLECTION

LIVE LOAD = $\frac{5 w_u l^4}{385 E_c I_e} = \frac{5(.08)(22 \text{ FT})^4 (1728)}{385 (3834.3)(650)} = 0.169''$

CHECK AGAINST

$$\Delta_{l, \text{max}} = l/360 = \frac{(22')(12'')}{360} = 0.733'' > 0.169'' \text{ OK}$$

TOTAL LOAD = $\frac{5 w_u l^4}{385 E_c I_e} = \frac{5(.1975)(22)^4 (1728)}{385 (3834.3)(650)} = 0.416''$

CHECK AGAINST

$$\Delta_{t, \text{max}} = l/240 = \frac{(22')(12'')}{240} = 1.1'' > 0.416'' \text{ OK}$$

ONE WAY BEAM + SLAB	AE SENIOR THESIS	JESSEL ELLIOTT
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3/6

DESIGN OF TYP. BEAM

$h_f = 9"$
 $h = 22"$
 (ASSUMED)

DEAD LOAD

S.D.L = 30 PSF(22') = 660 PLF
 BEAM SELF = (150)(3')(22")(1/12)
 = 825 PLF
 SLAB SELF = (150)(9")(1/12)(22')
 = 2475 PLF

$W_u = 1.2(0.66 + 0.825 + 2.475) + 1.6(1.76)$
 = 7.568 KLF

LIVE LOAD

L.L. = 80 PSF(22') = 1760 PLF

FROM ACI 8.3.3 FIND POSITIVE & NEG. MOMENTS FOR INTERIOR SPAN

POSITIVE MOMENT: $\frac{W_u l_n^2}{16} = \frac{(7.568)(27)^2}{16}$
 = 344.8 K-FT

NEGATIVE MOMENT: $\frac{W_u l_n^2}{11} = \frac{(7.568)(27)^2}{11}$
 = 501.6 K-FT

$b_w + l_n = 3'(12"/1') + (27')(12"/1') = 360"$
 $b_{eff} \leq \begin{cases} 8h_{fleft} + b_w + 8h_{fright} = 8(9") + 3'(12"/1') + 8(9") = 180" \\ \frac{l_n}{4} = \frac{(27')(12"/1')}{4} = 81" \leftarrow \text{CONTROLS} \end{cases}$

$A_s = \frac{M_u}{4d} = \frac{501.6 \text{ K-FT}}{4(28.8")}$
 = 4.35 in² ⇒ TRY 15 #5 BARS $A_s = 4.65 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(4.65 \text{ in}^2)(60 \text{ ksi})}{(0.85)(4 \text{ ksi})(81")}$ = 1.01" $c = a/\beta = \frac{1.01}{0.85}$
 $c = 1.19"$
 $1.19" < 0.375d = 0.375(28.8) = 10.8"$
 so... $\phi = 0.9$

TOP REINFORCEMENT

$\phi M_n = \phi A_s f_y (d - a/2)$
 = (0.9)(4.65)(60)(28.8 - 1.01/2)/12
 = 592 K-FT

592 K-FT > 501.6 K-FT OK

15 #5 bars
 OK FOR TOP
 REINFORCEMENT

ONE WAY BEAM + SLAB	AE SENIOR THESIS	JESSEL ELLIOTT	5/6
$\bar{y} = \frac{-23.8 \pm \sqrt{(23.8)^2 - 4(40.5)(-714.24)}}{2(40.5)}$			
$\bar{y} = 3.92''$			
$I_{cr} = \frac{(81'')(3.92'')^2}{12} + (81'')(3.92'')\left(\frac{3.92''}{2}\right)^2 + 8(3.1in^2)(8.8-3.92)^2$			
$I_{cr} = 16675 \text{ in}^4$			
$M_{cr} = \frac{f_r I_g}{y_t} = \left[\frac{7.5 \sqrt{4000} (128064 \text{ in}^4)}{18.4''} \right] / (12)(1000) = 275.12 \text{ K-FT}$			
$M_a = 344.8 \text{ K-FT}$			
$I_e = \left(\frac{275.12 \text{ K-FT}}{344.8 \text{ K-FT}} \right)^3 (128064 \text{ in}^4) + \left[1 - \left(\frac{275.12 \text{ K-FT}}{344.8 \text{ K-FT}} \right)^3 \right] (16675 \text{ in}^4)$			
$I_e = 65056.7 + 8204.1 = 73260.8 \text{ in}^4$			
$\text{LIVE LOAD DEFLECTION} = \frac{5 W_{LL} l^4}{384 E_c I_e} = \frac{5(1.76 \text{ KLF})(29')^4(1728)}{384(3834.3 \text{ KSI})(73260.8 \text{ in}^4)}$			
$= .110'' < l/360 = \frac{29(12)}{360} = 0.967''$ <p style="text-align: right;"><u>OK</u></p>			
$\text{TOTAL LOAD DEFLECTION} = \frac{5 W_{TL} l^4}{384 E_c I_e} = \frac{5(5.720)(29')^4(1728)}{384(3834.3 \text{ KSI})(73260.8 \text{ in}^4)}$			
$= 0.324'' < l/240 = \frac{29(12)}{360} = 1.45''$ <p style="text-align: right;"><u>OK</u></p>			

ONE WAY BEAM & SLAB	AE SENIOR THESIS	JESSEL ELLIOTT	6/6
CHECK SHEAR CAPACITY			
$V_u = \frac{W_u l_n}{2} = \frac{(7.568 \text{ klf})(27)}{2} = 102.2 \text{ k}$			
$\phi V_c = \phi 2 \sqrt{f'_c} b_w d = 0.75(2) \sqrt{4000} (36" \times 28.8") / 1000 = 98.36 \text{ k}$			
Try #5 BARS @ $s \leq d/2 = 28.8/2 = 14.4" \text{ SO USE } 14"$			
$A_{v \min} = \max \left\{ \begin{array}{l} 0.75 \sqrt{f'_c} b_w s / f_{yt} = 0.75 \sqrt{4000} = 47.4 \\ 50 b_w s / f_{yt} \leftarrow \text{CONTROLS} \end{array} \right.$			
$A_{v \min} = 50 (36") (14") / 60000 = 0.42 \text{ in}^2$			
$V_{s \min} = \frac{A_v f_y d}{s} = \frac{(0.42 \text{ in}^2) (60 \text{ ksi}) (28.8 \text{ in})}{14"} = 51.84 \text{ k}$			
$V_{s \min} = 51.84 \text{ k} \leq 4 \sqrt{f'_c} b_w d = 4 \sqrt{4000} (36") (28.8") = 262.3 \text{ k} \text{ OK}$			
$\phi V_n = \phi V_c + \phi V_s = 98.36 + 0.75(51.84) = 137.24 \text{ k}$			
$102.2 \text{ k} < 137.24 \text{ k} \text{ OK}$			
<u>USE #5 bars @ 14" SPACING</u>			

Appendix F: Two Way Post Tension Flat Plate

TWO WAY PT	AE SENIOR THESIS	JESSEL ELLIOTT	1/4
	<p> $f_c = 4000 \text{ psi (34.5 MPa)}$ $f'_{ci} = 3000 \text{ psi (25.9 MPa)}$ E-W max. f_c due to net prestress losses = 200 psi N-S max. f_c due to net prestress $\leq 350 \text{ psi}$ Max f_c due to combined stresses = $0.45 f_c = 1800$ $F_c = 57000 \sqrt{f_c} = 3.60 \text{ e } 6 \text{ psi}$ $f_{ps} \leq 0.70 f_{pu} = 189,000 \text{ psi}$ $f_{py} = 240,000 \text{ psi}$ $f_{pe} = 159,000 \text{ psi}$ $E_{ps} = 29,000 \text{ ksi}$ $P_y = 60 \text{ ksi}$ $E_s = 29,000 \text{ ksi}$ </p> <p style="text-align: right;"> S.D.L. = 36 PSF LL. = 80 PSF </p> <p style="text-align: center;"> $e_s = e_l = 7/2 - 1 = 2.5''$ </p> <p> CHOOSE TRIAL SLAB THICKNESS ON BASIS OF SPAN TO DEPTH RATIO ≈ 45 $h = \frac{(22' + 29') \times 12''}{2} \times \frac{1}{45}$ $h = 6.8'' \rightarrow \text{TRY } 7'' \text{ DIAMETER} = 0.5''$ AND... $d_p = 7'' - (0.5/2 + 3/4) = 6''$ </p> <p><u>BALANKING LOAD</u></p> <p> $w_D = 15 \text{ PSF} + 7/12 (150 \text{ PCF}) = 102.5 \text{ PSF}$ ASSUME $w_{BAL} = w_D = 102.5 \text{ PSF}$ BECAUSE f_c due to prestressing = 200 psi in E-W direction, the effective prestressing force in the E-W direction is $P_L = 200 (7 \times 12'') = 16,800 \text{ lb/strip}$ </p> <p> $w_{bal}(L) = \frac{8 P_L e_l}{L^2} = \frac{8 (16800) (2.5)}{(29')^2 \times 12} = 33.3 \text{ PSF}$ </p> <p> $w_{bal}(S) = w_D - w_{bal}(L) = 102.5 - 33.3 = 69.2 \text{ PSF}$ $P_S = \frac{w_{bal}(S) L_s^2}{8 e_s} = \frac{69.2 (22')^2 \times 12}{8 (2.5'')} = 20096 \text{ lb/ft after losses}$ </p> <p> f_c in N-S direction is $f_c = \frac{P_S}{bh} = \frac{20096}{12 \times 7} = 239 \text{ psi} < 350$ <u>OK</u> </p> <p> so use 1/2" ϕ 7 wire 270 K tendons with effective prestressing force $P_e = 159,000 \times 0.153 = 24,327 \text{ lb}$ </p> <p> Required spacing for N-S direction. $S_s = \frac{24327}{20096} = 1.21' = 14''$ </p> <p> Required spacing for E-W direction $S_L = \frac{24327}{16800} = 1.44' = 17''$ </p> <p> To prevent concrete splitting in anchorage zones at walls, add two #4 non prestressed bars along anchorage line. </p>		

Appendix G: Estimated Cost Calculations

COST ANALYSIS	AE SENIOR THESIS	JESSEL ELLIOTT
USE OF RS MEANS COST WORKS ONLINE		
<u>EXISTING COMPOSITE BEAM & DECK SYSTEM</u>		
B1010 FLOOR CONSTRUCTION		
B1010256 COMPOSITE BEAM, DECK & SLAB		
B10102563400		
ASSUME: BAY 25'x30'		
S.D.L 119 PSF		
<u>MAT.</u>	<u>INSTALL.</u>	<u>TOTAL</u>
11.88	5.53	\$ 17.41
<u>ONE WAY PRESTRESSED HOLLOWCORE PLANKS</u>		
B1010 FLOOR CONSTRUCTION		
B1010230 PRECAST PLANK W/ 2" TOPPING		
ASSUME: SPAN 20'		
S.D.L 75 PSF		
<u>MAT.</u>	<u>INSTALL.</u>	<u>TOTAL</u>
8.69	5.02	\$ 13.71
<u>ONE WAY SLAB & BEAM</u>		
B1010 FLOOR CONSTRUCTION		
B1010219 CAST IN PLACE BEAM & SLAB, ONE WAY		
ASSUME: BAY SIZE 25'x30'		
S.D.L 75 PSF		
<u>MAT.</u>	<u>INSTALL.</u>	<u>TOTAL</u>
5.94	11.13	\$ 17.07
<u>TWO WAY POST TENSION</u>		
B1010 FLOOR CONSTRUCTION		
B1010223 CAST IN PLACE FLAT SLAB W/ PT		
ASSUME: BAY SIZE 25'x25'		
S.D.L 75 PSF		
<u>MAT.</u>	<u>INSTALL.</u>	<u>TOTAL</u>
6.37	8.78	\$ 15.15