Technical Report #2

Pro-Con Structural Study of Alternate Floor Systems

Three PNC Plaza

Pittsburgh, PA

R. Bryan Peiffer

The Pennsylvania State University

Architectural Engineering

Structural Option

Faculty Adviser: Dr. Ali M. Memari

October 27th, 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The purpose of this technical report is to analyze four different floor systems for Three PNC Plaza. The systems being looked at are the existing composite deck system along with the following alternative floor systems:

- Concrete Two-Way Flat Slab System with Drop Panels
- Hollow Core Planks
- Two Way Post Tension Slab

The calculations for each system were performed on a typical 30' by 42.5' floor bay located on the 7th floor. The alternative floor systems do vary from the existing system due to system limitations and for simplification of hand calculations. For example the Two-Way Flat Plate System would not be able to span the existing 42.5' therefore columns were added at the half way point for the calculations.

Sizes were found for the various systems by hand calculations using ACI 318-08 and the 13th edition of the AISC Steel Manual. Many other categories were evaluated for comparison such as serviceability, building weight, cost, and architectural impact among others. It was determined that the existing system is the most ideal system due to its depth and easily changed to deal with different loadings. Systems such as the Hollow Core Plank would not be a plausible system due to the uniform layout of 4' increments that would hinder the architecture of the building.

INTRODUCTION

Three PNC Plaza is a 23 story, 780,000 square foot, mixed use high-rise building located in the heart of downtown Pittsburgh, Pennsylvania as seen in figure 2 highlighted in red. The erection of this building was a significant part to revitalizing the downtown area and marked the first new high-rise built in the city in the last 20 years.

Figure 1- Three PNC Occupancy Layout

The building is mixed-use and allows for several different tenants occupy the building as seen in figure 1. Fairmont Hotels and Resorts move into the building in March, 2010 with 185 rooms that are located on floors 14 through 23. Along with the Fairmont Hotels, 28 Residences condominium units will occupy floors 14 through 23 in the fall of 2010. The building has 10 floors of office space located from the 3rd through 13th floor. These office spaces are home to PNC Bank and the REED Smith Law Firm. The lower floors of the building house several different retail stores, restaurant, and wine bar.

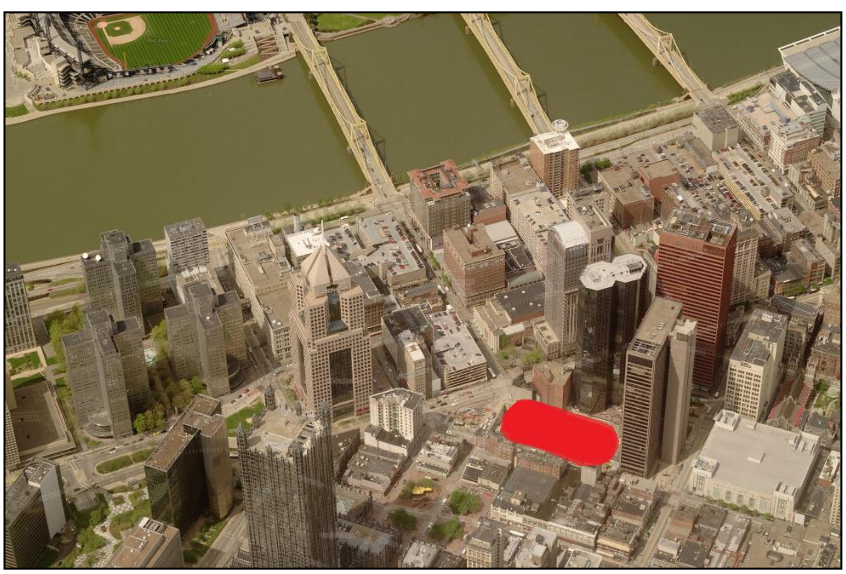


Figure 2- Three PNC Site Location

STRUCUTURE OVERVIEW

Foundation System

Pittsburgh is known for alluvial deposits which mean shallow foundations were not possible and deep foundations were required for Three PNC Plaza. Also, the Pittsburgh area soil overburden is 60’ to bedrock. This means that after the 30’ of excavation for the buildings parking garage structure, 30’ of soil would still remain until the bedrock would be reached. Several different options for the foundation of the building were considered such as; auger cast pile, piles, H-piles, and caissons. Ultimately, the foundation system chosen for Three PNC Plaza were caissons bearing on bedrock to achieve maximum axial capacity. Four different size caissons were chosen for the foundation as seen in the Caisson Schedule in figure 3. The caissons were designed for a typical column reaction of 3500 kips. Brayman Construction Corporation was in charge of the installation of the 121 caissons for the building. A typical caisson detail has been provided in figure 2. The caissons bearing value is 15 tons per square foot and were drilled to auger refusal or socketed into the bedrock.

CAISSONS $F_{BR.}$ =30K/SQ. FT.

MARK	SIZE Ø	VERT.REINF. length=3 X DIA.	TIES	DOWELS
A.	48”	7–#10	#4@18” O.C.	4–#8 X 8’–0” DEVELOP INTO PEDESTAL
B.	54”	9–#10	#4@18” O.C.	
C.	42”	7–#9	#3@18” O.C.	
D.	60”	9–#11	#3@18” O.C.	

Figure 3- Caisson Schedule

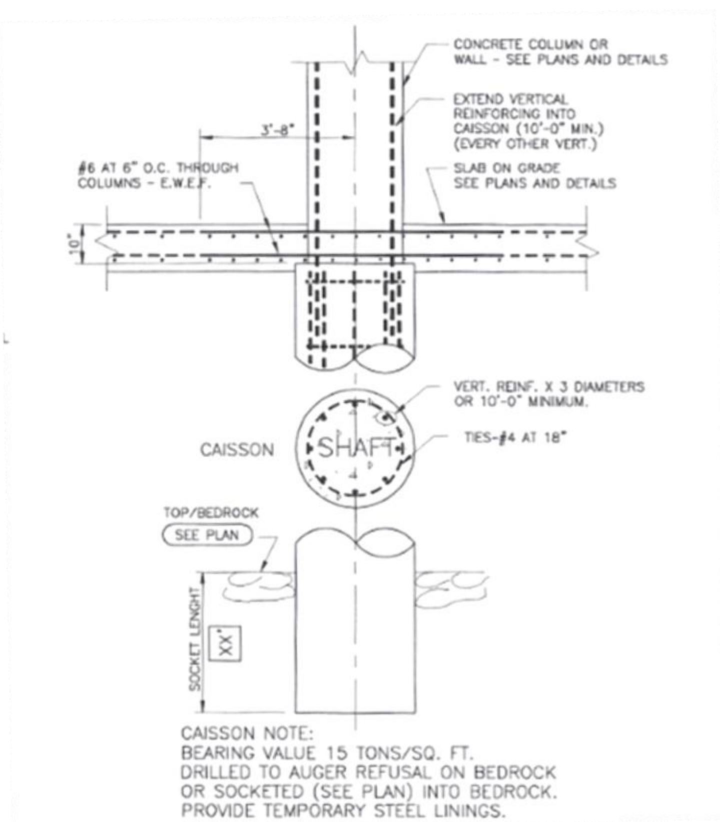


Figure 4- Caisson Detail

Columns

Three PNC Plaza uses a variety of steel columns and concrete shear walls to support the gravity load of the building. The size of these columns can range in sizes from W14x68 all the way to a W14x740 in some cases. The core of the building is supported by concrete shear walls up until the 14th floor which they then switch over to steel columns. The remainder of the building is supported by steel columns from the ground floor that attach to concrete columns located in the parking garage. The steel columns attach to the concrete shear wall via reinforced corbels. The steel columns in the building are spliced together at a typical distance of 24'-0" as see in figure 8.

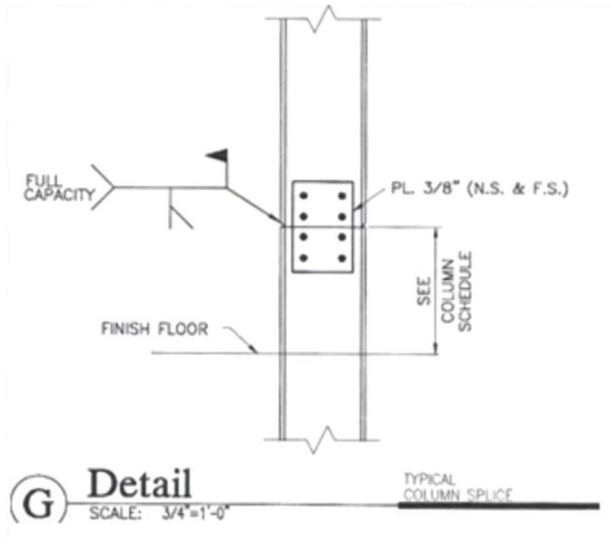


Figure 5- Splice Detail

Roof System

The roof structural system is very similar to the floor structural system used throughout the building. It utilizes the same composite deck and slab configuration along with same typical bay dimensions. However, the fill beams are spaced closer together, at a typical spacing of 7.5 feet. These fill beams can differ in size from a W21x44 to a W27x129.

Lateral System

The main lateral resistant system used in Three PNC Plaza is a combination of several concrete shear walls. These shear walls are located throughout the core of the building and encase the stairwells and elevators as seen in figure 9 highlighted in red. The shear walls start at the lowest level of the parking garage structure and extend up until the 14th floor where they are met with steel columns. All of the shear walls used a concrete with a compressive strength of 5000 ksi. The reinforcement for the shear walls changed depending on the location and can be seen in the shear wall Reinforcement schedule located in Appendix D. A more detailed view of the shear walls at key locations of the wall can be in figures 10-13.

Figure 6- Shear Wall Layout

CODES AND REFERENCES

Design Codes Used:

1. International Building Code 2003
2. AISC Manual of Steel Construction Ninth Edition (ASD)
3. AISC Manual of Steel Construction Load and Resistance Factor Design Second Edition
4. ACI 318 American Concrete Institute Building Code Requirements for Structural Concrete
5. ASCE 7-98 Minimum Design Loads for Buildings and Other Structures

Thesis Codes Used:

1. International Building Code, IBC 2010
2. American Society of Civil Engineers, ASCE 7-10
3. AISC Manual of Steel Construction Thirteenth Edition (LRFD)
4. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary

MATERIAL STRENGTHS

Concrete

Location	Strength f'c (ksi)
Columns	10000 psi
Interior Slab on Grade	5000 psi
Caissons and Grade Beams	5000 psi
Retaining Walls	5000 psi
Post Tension Slabs	5000 psi
Beams with PT Slab	5000 psi
Core Walls	5000 psi
Exterior Slab on Grade	4000 psi
Exterior topping Slabs	4000 psi
Composite Slab Fill	3000 psi
Footings and Misc.	3000 psi

Structural Steel

Type	Standard	Grade
W Shapes	ASTM A992	50 ksi
S,M, and HP Shapes	ASTM A36	
Tubes	ASTM A500	Class B
Channels	ASTM A36	
Angles	ASTM A36	
Plates	ASTM A36	

LOADINGS

Location	Design (IBC 2003)	Thesis (ASCE 7-10)
Retail	100 psf	100 psf
Office	50 psf	50 psf
Library	150 psf	150 psf
Hotel	40 psf	40 psf
Condominium	40 psf	40 psf
Ballroom	100 psf	100 psf
Garage	40 psf	40 psf
Mechanical Rooms	200 psf	-
Assembly Areas	100 psf	Depends on Area
Balconies	100 psf	1.5*Live Load
Restaurants	100 psf	-
Roof	30 psf	20 psf
Stairs and Lobby	100 psf	100 psf
Corridors	80 psf	80 psf

Floor Dead Loads	
Composite Decking	44 psf
Superimposed Dead Load	30 psf
Total	**74 psf**

Curtain Wall Dead Load:

Assumed curtain wall was 8" thick and that the material weighted 40psf. This resulted in a load of 60plf.

EXISTING FLOOR SYSTEM: COMPOSITE DECK

Three PNC Plaza utilizes a composite floor with a typical bay size of 30'-0" x 42'-6". The composite slab is composed of 2" 18-gauge metal floor deck with 3-½" light weight concrete, netting a total thickness of 5-½". The concrete is reinforced with one layer of 6x6-W2.1xW2.1 wielded wire fabric. The composite deck transfers its load to fill beams that are placed at 10'-0" on center and primarily W21X44 beams with W24X62 girders. This floor design is used throughout the structure and different sized fill beams are used to deal with higher load areas. See appendix A for calculations.

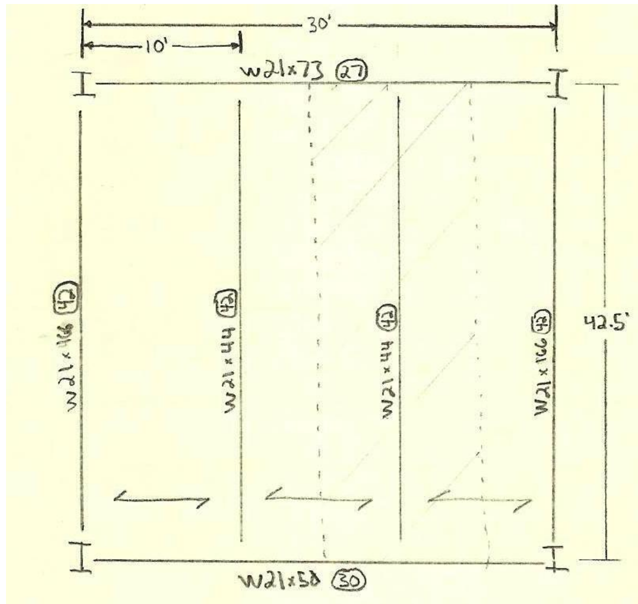


Figure 7 - Typical Bay

Pro-Con Analysis:

Composite deck systems are typically known for their ability to keep the total weight of the building relatively low. They can accomplish this by utilizing lightweight concrete and the added strength composite beams versus non-composite construction allows for smaller required sizes of beams. The current system has a dead load of only 44 psf which is very low and appealing to designers.

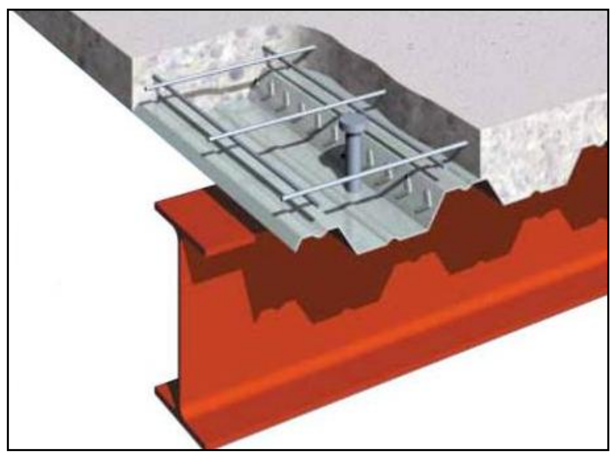


Figure 8 - Composite System

Composite construction has three main serviceability considerations associated with the design. These considerations are deflection during construction, deflection under service loads, and vibration under service loads. For this report vibration considerations were not calculated for the existing structure. Deflection were calculated and found to be within the allowable limits.

The composite floor system allows for shallower depths of members and with only 5-1/2" total thickness it achieves a very low profile. This was especially important during the construction of the building due to building height limits.

Conclusion:

<u>Advantages</u>

- Smaller Beam Sizes
- 2 Hour Fire Rating Easily Achievable (Slab)
- Low Building Weight Impact
- Quick Constructability

<u>Disadvantages</u>

- Steel Requires Spray-on Fireproofing

Not one floor system is perfect in every way, however the existing composite decking system used in Three PNC Plaza was an excellent choice for the project. It can easily adept to different loadings required throughout the building due to its mixed use occupancy. It allows for great flexibility with the floor plans from the ability to span long distances not achievable by other systems. Finally, it allowed for minimum impact to height restrictions due to its lower depths requirements.

ALTERNATIVE SYSTEMS: Precast Hollow Plank

The first alternative system looked at for Three PNC Plaza is precast hollow core planks. The system was designed referencing the PCI load tables provided in their handbook. The planks were sized according to the safe superimposed service loads provided PCI, it was determined that a 4'-10" Normal Weight Concrete Hollow Core Plank would be used. After the plank size was found the steel girders to support the planks needed to be calculated. They were found to be W30x148's. A detailed picture of a typical setup can be seen in figure 9. The 42"-6" dimension of the existing bay had to be changed to 40' for this system. See appendix B for calculations and preliminary design.

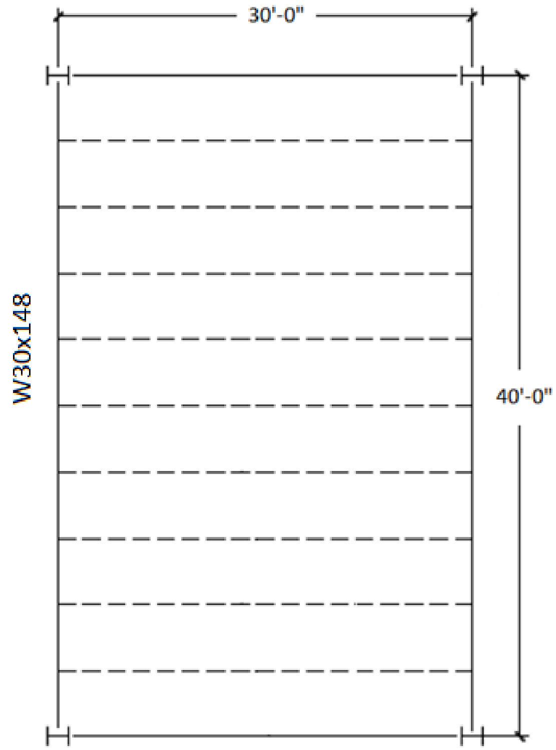


Figure 9 - Plank Layout

Pro-Con Analysis:

While hollow planks could provide a reduction in overall weight over other systems, it does not provide a reduction when compared to the existing composite system. This can be attributed to the hollow planks normal weight concrete construction versus the lightweight construction of the composite system. Also, large girders are required to deal with the loads from the planks. Overall, this system does not provided much if any savings from a building weight aspect.

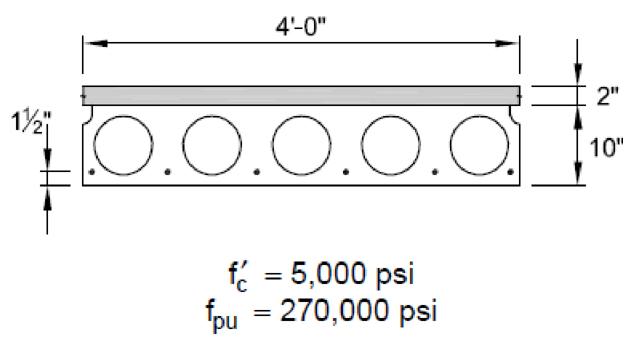


Figure 10 - Plank Details

Deflection limits were checked for the W30x148 used to support the hollow planks. The girder passed both the live load deflection limit of L/360 and the total deflection limit of L/240. Additional deflection limits may be required and resizing for special cases in the building would have to be performed. Vibration studies would also have to be performed if constructing in vibration sensitive areas.

The Hollow-Core planks present themselves with several challenges from an architectural stand point. Due to the modular 4' sizes all spans would have to be resized in multiples of 4. This would result in several changes to the current bay dimensions.

Conclusion:

Advantages

- Ease of constructability
- 2 Hour Fire Rating
- Low noise transmission
- Pre-manufactured

Disadvantages

- Lead Time may be required
- Column Grid Alteration Needed
- Leveling required with planks

Several advantages can be achieved by the use of Precast Hollow Core Planks such as fast and easy construction. The disadvantages out weight the few advantages the building would get from the system. The architecture would have to change to allow for this system to work, which makes this an easy system to rule out.

ALTERNATIVE SYSTEMS: Two Way Flay Slab

The second alternative system looked at for Three PNC Plaza is a Two Way Flat Slab System. The system was designed referencing the ACI 318-08 Building Code Requirements for Structural Concrete. The system utilizes a two-way reinforced concrete slab to transfer loads to columns. The typical bay for Three PNC Plaza was 42.5' by 30' and was too large of a span for this system. Because of this columns were added

Figure 11 - Flat Slab with Drop Panels

at the half way point in the 42.5' direction to convert the bay size to a more manageable 21.25' by 30'. It was assumed that all bay sizes of the building would be no greater then these sizes to ensure a consistent slab thickness. This system will provided the building with a 2 hour fire rating cover with proper clear cover and concrete materials.

The Flat Slab system was designed with a 9.5" thick slab, however this had to be modified during calculations due to punching shear and drop panels were added to handle these loads. The drop panels were designed with an additional 3" netting a total thickness of 12.5" at the drop panel regions. See appendix C for calculations and preliminary design.

Pro-Con Analysis:
With the slab thickness of 9.5" and additional 3" for the drop panels it was found that the self weight of the system would be approximately 124.5 psf. This value is significantly larger than the current composite floor system being used in the building. With all the additional extra weight of the flat slab system the lateral system of the building would be affected greatly and need to be reexamined. The total slab thickness of 12.5" is a very attractive aspect of this system because it will allow for floor to floor height to be increased or the ability to decrease the overall height of the building if desired. Additional space would have to be provided for the MEP systems.

The architectural impacts of this system can be fairly large due to the addition of columns into the existing floor plan lay out. These columns were sized at 18" x 18" to try and minimize the impact they would have along the building. This will dramatically change the possible layouts for the office floors of the building and how the condos and hotel rooms would be arranged.

This system excels in the typical interior bay for the building; however complications arise at key points in the structure such as the ballroom. A large ballroom span was accomplished using deep beams for the composite floor system. The two way slab would not allow for this very large open space, columns would have to be provided to support the system throughout the room.

Conclusion:

Advantages

- Ease of constructability
- 2 Hour Fire Rating
- Floor to Floor Height

Disadvantages

- Span Length/Addition of Columns
- Construction Time
- System Weight

The two-way flab slab with drop panels system does not seem as an appropriate alternative for Three PNC Plaza. The main reasons for this are the negative changes to floor space for offices, condos, and the hotel rooms. Also, the interior ballroom area would have to be dramatically changed to make this system work. Finally a large increase in floor weight would require a reexamination of the current lateral and foundation systems.

ALTERNATIVE SYSTEMS: Two Way Post-Tensioned

The second alternative system looked at for Three PNC Plaza is a Two Way Post tensioning Slab. The system was designed using both ACI 318-08 and Portland Cement Association time saving design aid. The existing bay dimensions of 30' by 42.5' were used for the calculations. Since the bay dimensions fall under the category of L2/L1 < 2 a two way slab was needed. The post tensioning comes from the 1/2", 7-wire tendons used throughout the design and the overall slab thickness used was 11.5". The tendons in the 30' dimension of the bay will be banded together over the column strip and the tendons in the 42.5' dimension will be placed uniformly throughout the slab. Calculations for the preliminary design can be seen in appendix D.

Figure 12 - Post Tensioned System 1

Pro-Con Analysis:

The post tensioned floor design was picked as an alternative system that would be able to span in the 42.5' direction of the existing design. This is a major advantage of this system because it will not cause any conflicts with the current floor plan if it were to be implemented. The current system design calls for an 11.5" thick slab which would be a reduction versus the current system providing greater floor to floor heights. The slab will also provide the required 2 hour fire ratings provided by its clear cover. Vibration and deflection limits were not analyzed for this system due to the complexity of the calculations.

The disadvantages involved with a post-tensioned system would the specialized construction of the system. It will require an experienced post tension contractor to accomplish. Another issue that hinders the system would be openings, they would need to be planned out in advance and have tendons placed around them. The increased weight of the system will also put a larger force on the foundation and lateral systems and may need reexamined.

Conclusion:

Advantages

- Floor Depth
- Long Spans
- 2 Hour Fire Rating

Disadvantages

- Specialized Construction
- Formwork

The two-way post-tension slab seems to be a valid alternative floor system for Three PNC Plaza. It would be able to use the existing 30' by 42.5' bay dimensions limiting architectural impact. This system would be valid for further investigation.

CONCLUSION

Floor System	Weight	Architectural Impact	Fireproofing	Fire rating	Cost	Constructability	Future Investigation
Composite Deck	44psf	No	Spray on	2 hr	33.20/sqft	Easy	**Yes**
Two-Way Flat Slab	125psf	Yes	Built In	2 hr	16.85/sqft	Medium	No
Hollow Core Planks	93psf	Yes	Built In	2 hr	23.48/sqft	Easy	No
Two-Way PT	144psf	No	Built In	2 hr	17.18/sqft	Difficult	**Yes**

In this report alternative floor systems were analyzed for Three PNC Plaza and compared to the current system used throughout the building. Systems were designed using a typical 30' by 42.5' bay from the existing building structure when applicable. Alternative floor systems were explored for several different reasons such as depth, weight, and span distance. A main complication for a floor system to be used in Three PNC Plaza is the 42.5' span to the columns or shear walls located at the center of the building.

It is evident why the engineers of Three PNC Plaza utilized the existing composite floor system. It provides a very light weight system that is capable to span long distances as needed. It also allows for easy adaptability to different loads needed throughout the building due to the buildings mixed use occupancy. The composite construction also provides a stiff diaphragm for lateral loads to be transferred. Overall, it is a very good choice for the building.

The two-way flat plate system with drop panels would not be an adequate substitute for the existing system. The system is not capable to span the needed 42.5' length and would require an additional set of columns placed at the half way point. This would disrupt the architecture of the building and ruin the large open spaced provided to the occupants of the building. It would also add a significant amount of weight to the building resulting in reexamination of both lateral and foundation systems.

The precast hollow core plank system was also ruled out as a useable alternative system. The planks are pre-manufactured offset in set dimensions of 4' which would require the building to have its bay layout be divisible by 4. This would alter the existing structure and would put added weight on the building.

The final alternative system of a two-way post-tensioned slab is the most viable alternative. It has the ability to span the existing dimensions. It also would also allow for a great floor to floor height with the overall thickness of the system being reduced. Draw backs to the system could be the increased weight and its effects on the lateral and foundation systems. If it were to be investigated further a more detailed investigation would have to be performed to see the changes to these systems. Another reason this system might not be the best to use would be the specialized construction that is involved with post-tensioned construction.

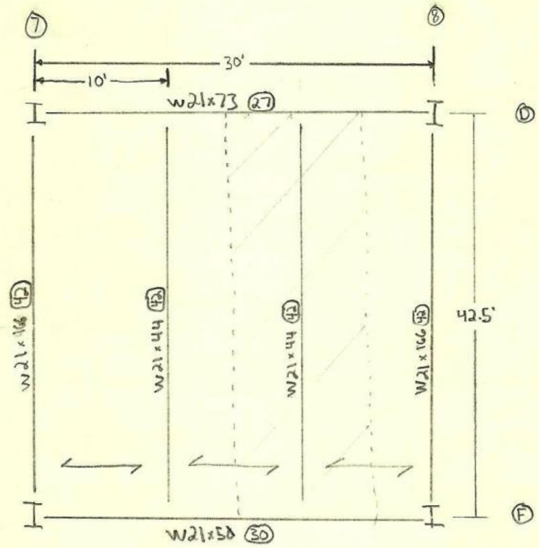
APPENDIX

APPENDIX A: Existing Systems

Decking, Fill Beam, Girder

Spot Checks

Floor 7 Typical Bay with Typical Floor



Composite Slab
2" x 18 G.A. Steel Deck } total 5 ½"
3 ½" Lightweight Concrete }
3000 psi Fill

· Use Vulcraft Decking Catalog

2VL118: Max unshored clear span, 3 span = 12'-5"

10'-0" < 12'-5" ∴ OK ✓

Superimposed Live Load: 10'-0" span = 217 psf

Loading ASCE 7-10
Office Live Load = 50
Superimposed Dead Load = 30
80 psf

80 psf < 217 psf ∴ OK ✓

Decking is ok, over designed for this loading. Not the controlling load factor.

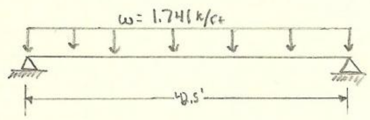
Decking provides a 2 hr fire rating.

Composite Beam W21x44

Properties: $A_g = 13.0$ $\quad$ LL = 50 psf not reducing to be conservative
$I_x = 843$ $\quad$ DL = 44 psf (Decking)
$F_y = 50$ ksi $\quad$ SDL = 30 psf (Superimposed)
SW = 44 plf

$W_u = 1.2D + 1.6L$ $\quad$ Dead = $(44+30)10' + 44 = 784$ plf
Live = $50(10) = 500$ plf

$W_u = 1.2(784) + 1.6(500) = 1740.8$ plf ≈ 1.741 K/ft



$V_u = 1.741(42.5)\left(\frac{1}{2}\right) = 36.996^k \approx 37^k$

$M_u = 1.741(42.5)^2\left(\frac{1}{8}\right) = 393.085 \approx 393.1^{1k}$

* pin supports assumed

b_{eff} = min | Span/4 = 10.625'
| Spacing = 10' * controls

$\phi V_n = 217$ K > 37K ∴ ok ✓

PNA = 7 $\quad$ $\Sigma Q_n = 162$

$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})} = \frac{162}{0.85(3)(10 \cdot 12)} = 0.53$

since a = 0.53 which is < 1.0 treat a as 1.0

$Y_2 = thickness_{slab} - \frac{a}{2} = 5.5 - \frac{1}{2} = 5$

$\phi M_n = 516^{1k}$ which is larger then $M_u = 393.1$ ∴ ok ✓

$Q_n = \frac{162}{17.2} = 9.4 \Rightarrow 10$ studs required

Deflection $\quad$ $\Delta_{LL} = \frac{l}{360} = \frac{42.5(12)}{360} = 1.4167$ in Max deflection

$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 E I}$ $\quad$ $I_B = 1450$ $\quad$ $w_{LL} = (50 \cdot 10) = \frac{500}{1000} = 0.5$

$= \frac{5(0.5)(42.5)^4}{384(29000)(1450)}(1728) = 0.8728$ in

$0.8728'' < 1.4167''$ ∴ OK ✓

Deflection wet concrete

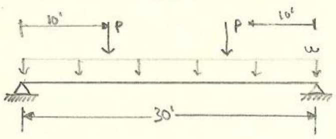
$$\Delta_{Max} = \frac{l}{240} = \frac{42.5(12)}{240} = 2.125 \text{ in}$$

$$I_{req} = \frac{5 w l^4}{384 \Delta_{max} E} \qquad w = \frac{(44 \cdot 10) + 44}{1000} = 0.484$$

$$= \frac{5(0.484)(42.5)^4}{384(2.125)(29000)}(1728) = 576.5 \text{ in}^4$$

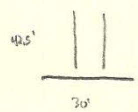
$576.5 < 843 \quad \therefore$ ok ✓

Composite Girder : W21 × 56



* Assumed Pin supports

Assume Load $P = 37^k$ due to 42.5' beam



$w_u = 50/1000 = 0.05$ klf

$$V_u = \frac{37}{2} + \frac{0.05(30)}{2} = 19.25^k$$

$$M_u = \frac{0.05(30)^2}{8} + 37(10) = 375.625^{1k}$$

PNA = 7 $\quad \Sigma Q_n = 184$

$b_{eff} = \begin{cases} 30/4 = 7.5 \text{ * controls} \\ \text{spacing} \end{cases}$

$$a = \frac{184}{0.85(3)(7.5)(12)} = 0.8 < 1.0$$

use $a = 1$

$Y_2 = 5.5 - 0.5 = 5$

$\phi M_n = 592^{1k} > 375.625^{1k}$ ∴ ok ✓

$\phi V_n = 237^k > 19.25^k$ ∴ ok ✓

Deflection

Δ_{LL} : $P_L = \frac{500(42.5)}{100} = 21.25$

Δ_{LL} : $\frac{l}{360} = \frac{(30)(12)}{360} = 1$ in

$I_{LB} = 1680$ $\quad \Delta_{LL} = \frac{Pl^3}{28EI} + \frac{5wl^4}{384EI} = \left[\frac{21.25(30)^3}{28(29000)(1680)} + \frac{5(.05)(30)^4}{384(29000)(1680)}\right]1728 = 0.75''$

$0.75'' < 1.0''$ ∴ OK ✓

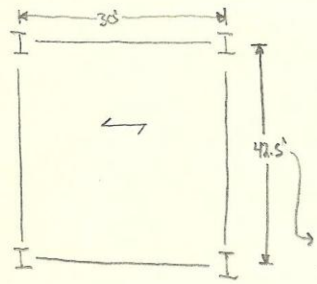
Wet $\quad \Delta_{Max} = \frac{30(12)}{240} = 1.5''$ $\quad P = 10.285$

$$I_{req} = \frac{(10.285)(30)^3(1728)}{28(29000)(1.5)} + \frac{5(0.05)(30)^4(1728)}{384(29000)(1.5)} = 250.8 \text{ in}^4$$

$250.8 \text{ in}^4 < 984 \text{ in}^4$ ∴ ok ✓

APPENDIX B: Precast Hollow Core Planks

Hollow Core Plank:



PCI Design Handbook 4'x10" NWC

SDL = 25
LL = 100 $\qquad$ 100+25 = 125

PCI uses unfactored loads

Would change 42.5' dimension to 40' to allow for easy fitting of Planks

Span of 30'
2" topping for 2-hr fire rating

58-S $\quad$ DL = 93 psf w/ 2" topping

93 + 25 = 118 psf , LL = 100

$1.2(118) + 1.6(100) = 301.6$ psf

Trib Width = $30'(301.6) = 9048 = 9.05$ K/ft

$$M_u = \frac{w_u l^2}{8} = \frac{9.05(40)^2}{8} = 1810'^K \qquad \text{Try}$$

@ 40'

$$\Delta_{LL\,max} = \frac{l}{360} = 1.33" = \frac{5(100 \cdot 30)(40)^4(1728)}{384(29000)(I)(1000)} \qquad I \geq 4980$$

$$\Delta_{D+L} = \frac{l}{240} = 2" = \frac{5(118+100)(30)(40)^4(1728)}{384(29000)(I)(1000)} \qquad I \geq 6495$$

W30x148 $\quad \phi M_n = 1880 \geq 1810 \therefore$ ok
$\quad I = 6680 > 6495 \therefore$ ok

Use W30x148 girders w/ hollow core plank
4'-0" x10" w/ 2" topping , 58-S

pg 73

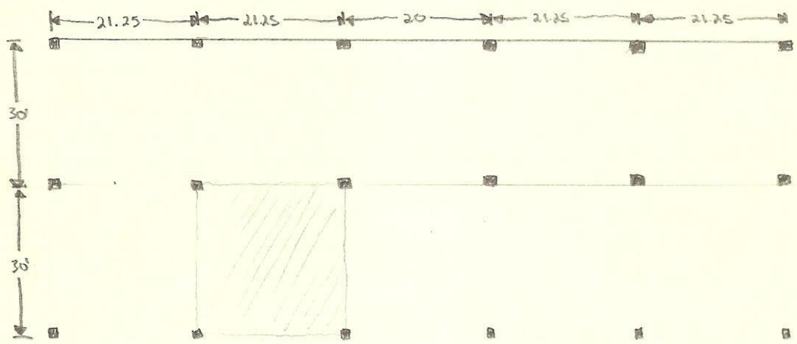
APPENDIX C: Two-Way Flat Plate System

Two Way Flat Slab Concrete Floor System with Drop Panels

Designed using ACI 318.08

Would Change Floor Plan to look as followed

assume 18" x 18" columns



Critical bay size would be 21.25 x 30 assumed 22 x 30 to simplify calcs

Min slab thickness : $t = \frac{l_n}{33} = \frac{(30 - \frac{30}{12})}{33} = 10"$ slab (Table 9.5c)

$\therefore DL = 150\left(\frac{10}{12}\right) = 125 + 25\ (SDL) = 150$ psf

$1.2(D) + 1.6(L) \Rightarrow 1.2(150) + 1.6(100) = 340 \Rightarrow .34$ ksf

$V_u = .340\left[(30 \cdot 22) - (1.5^2)\right] = 224^k$

Check Punching Shear : Assume #5 bars

$d = 10" - 0.75 - 0.625 = 8.625"$

$b_o = (18 + 8.625)4 = 106.5"$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}(106.5)(8.625) = 259^k \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1.36}\right)\sqrt{5000}(106.5)(8.625) = 321^k \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40 \cdot 8.625}{106.5} + 2\right)106.5(8.625)\sqrt{5000} = 340^k \end{cases}$$

$\beta_c = \frac{30}{22} = 1.36$

$\alpha_s = 40$ int col

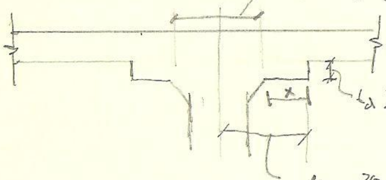
$V_c = 259^k$

$\phi V_c = 0.75(259) = 194.25^k$ Not Good!

Use Drop Panels

$t_{slab\,min} = \dfrac{\left(30 - \frac{30}{12}\right)}{36} = 9.16 \approx 9.5"$ Table 9.5 C

Size Drop Panel



$0.2(\text{Avg Span}) = 0.2\left(\frac{30+22}{2}\right) = 5.2'$

$t_d \geq t/4$ but $< 1/4\, x$

$X = 5' - \frac{5.2}{2} = 2.4'$

$l/6 = \frac{30}{6} = 5' = 60"$

$t_d = 3"$

$A_{drop} = 10' \times 10' = 100\,ft^2$

Direct Design Method:

$M_o = \frac{q_u}{8} l_2 l_n^2$ $q_u = 1.2\left(\frac{9.5}{12} 150 + 25\right) + 1.6(100) = 332.5$

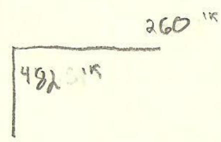
Frame A: $\frac{0.3325}{8}(22)(30-1.5)^2 = 742^{1k}$

Frame B: $\frac{0.3325}{8}(30)(22-1.5)^2 = 524^{1k}$

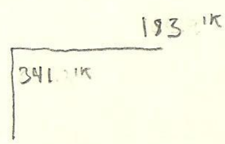
Distribution of M_o:

$-M = 0.65 M_o$ $+M = 0.35 M_o$ (13.6.3.2)

Frame A:



Frame B:



Distribution to Column Strip:

Frame A: $-M_{int} = 0.75M \Rightarrow$ 362 ft-k to CS
120 ft-k to MS

$+M_{int} = 0.6M \Rightarrow$ 156 ft-k to CS
104 ft-k to MS

Frame B: $-M_{int} \Rightarrow$ 226 ft-k to CS
115 ft-k to MS

$+M_{int} \Rightarrow$ 110 ft-k to CS
73 ft-k to MS

Summary of Moments

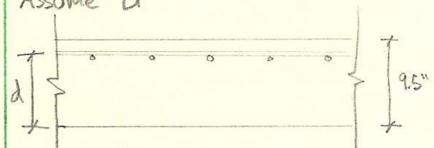
Frame A:

Total M (ft-k)	482	260
CS M (ft-k)	362	156
MS M (ft-k)	120	104

Frame B:

341	183
226	110
115	73

Assume d



* Assume #5 Bars

Frame A: $d = 9.5 - 0.75 - \frac{0.625}{2} = 8.44"$

Frame B $d = 8.44 - 0.625 = 7.82"$

Design Reinforcement

Frame A:

Description	$-M_{cs}$	$-M_{ms}$	$+M_{cs}$	$+M_{ms}$	
Moment	-362	-120	156	104	
width b	132"	132"	132"	132"	
Eff d	8.44"	8.44"	8.44"	8.44"	
$M_n = M_u/\phi$	-402	-133	173	116	
$K = M_n/d^2b \times 12000$	513	170	221	148	
ρ	0.0090	0.0029	0.0038	0.0025	
$A_s = \rho b d$	10.0	3.2	4.2	2.9	
$A_{s,min} = b\,t\,(.002)$	2.5 ✓	2.5 ✓	2.5 ✓	2.5 ✓	
$N = A_s/A_{s,bar}$	17 #7's*	11	14	10	#5 bars
$N_{min} = w/2t$	7 ✓	7 ✓	7 ✓	7 ✓	$A_s = 0.31$

Frame B

	$-M_{cs}$	$-M_{ms}$	$+M_{cs}$	$+M_{ms}$	
Moment	-226	-115	183	73	
b	180	180	180	180	
d	7.82	7.82	7.82	7.82	
M_n	-251	-128	203	81	
K	274	140	221	88	
ρ	0.0047	0.0024	0.0038	0.0015	
A_s	6.6	3.4	5.3	2.1	
$A_{s,min}$	2.8 ✓	2.8 ✓	2.8 ✓	2.8	
N	11 #7's*	11	17	10	#5 bars
N_{min}	10 ✓	10 ✓	10 ✓	10 ✓	

Check Punching Shear @ Drop Panels

$V_u = w_u A = 0.3325(22 \cdot 30 - 1.5^2) = 218.7$

$d_{avg} = 12.5 - 0.75 - 0.625 = 11.125$

$d/2 = 5.56$

$b_o = 4(18 + 11.125) = 116.5''$

$$V_c = \begin{cases} 4\sqrt{5000}(116.5)\,11.125 = 367 \;* \\ \left(2 + \frac{4}{1.36}\right)\sqrt{5000}(116.5)(11.125) = 452.8 \\ \left(\frac{40 \cdot 11.125}{116.5} + 2\right)(116.5)(11.125)(\sqrt{5000}) = 430 \end{cases} \text{min}$$

$\phi V_c = 0.75(517.6) = 275$ OK $\phi V_c > V_u$ ✓

@ Slab

$d = 9.5 - 0.75 - 0.625 = 8.125$

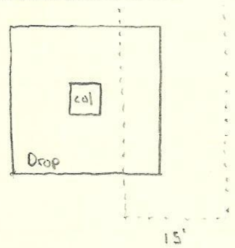
$V_u = 0.3325\left(22 \cdot 30 - \left(10' + \frac{8.125}{12}\right)^2\right) = 182$

$b_o = 4\left(10' + \frac{8.125}{12}\right) = 42.7' = 512.5''$

$$V_c = \begin{cases} 4\sqrt{5000}(512.5)(8.125) = 1178 \\ \left(2 + \frac{4}{1.36}\right)\sqrt{5000}(512.5)(8.125) = 1455 \\ \left(\frac{40 \cdot 8.125}{512.5} + 2\right)(512.5)(8.125)(\sqrt{5000}) = 776 \;* \end{cases}$$

$\phi V_c = 0.75(776) = 582$ $\phi V_c > V_u$ ✓

Check Beam Shear



ⓐ Panel

$V_u = 0.3325\,(22 \cdot 15) = 110^k$

$\phi V_c = 0.75(2)\sqrt{5000}\,(9.5 \cdot 12)\,11.125 = 135^k$

$\phi V_c > V_u \quad \therefore$ ok ✓

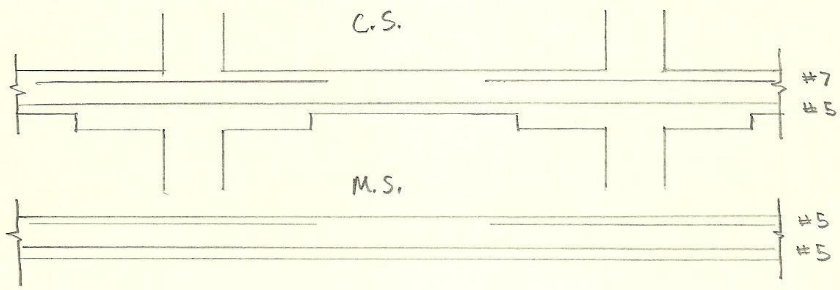
ⓑ Slab

$V_u = 0.3325\,(22 \cdot 10) = 73^k$

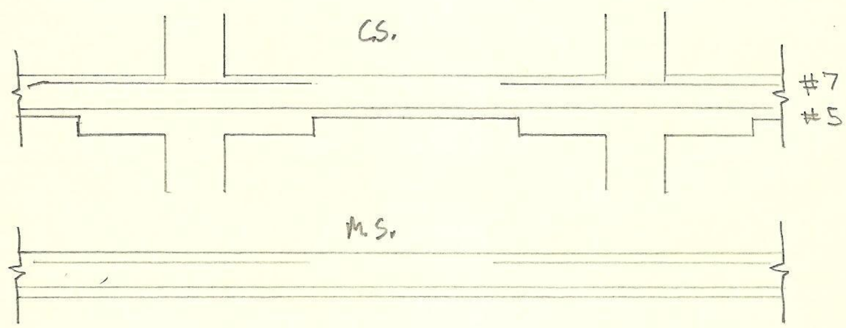
$\phi V_c = 0.75(2)(\sqrt{5000})(22)(12 \cdot 8.125) = 228$

$\phi V_c > V_u \quad \therefore$ OK ✓

Design: Frame A:



Frame B



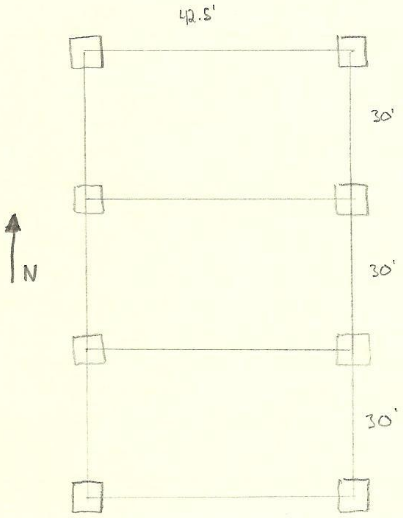
System Weight

$$\left[\frac{9.5}{12}(150)(30)(22) + \frac{3}{12}(150)(10)(10)\right] \Big/ (22)(30)$$

$$= 124.5 \text{ psf}$$

APPENDIX D: Post Tension

Two Way Post Tension with Wide Shallow Slab Beams



Rectangular Spans
- Bands in short direction over columns
- Uniform in long direction

Loads:
Self weight
SDL = 25
LL = 100

2-hr Fire rating

Normal weight concrete 150pcf
$f'_c = 5000$ psi
$f'_{ci} = 3000$ psi

Rebar: $F_y = 60{,}000$ psi

PT: unbonded tendons $\frac{1}{2}$" ϕ, 7-wire strands, ($A = 0.153 in^2$)

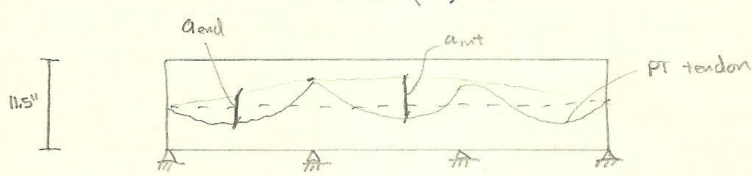
$f_{pu} = 270$ Ksi
Prestress losses $\approx$ 15 Ksi
$f_{se} = 0.7(270) - 15 = 174$ Ksi
$P_{eff} = 0.153(174) = 26.6$ K/tendon

Preliminary Slab thickness:

$L/h = 45 \qquad h = \frac{42.5(12)}{45} = 11.33 \approx 11.5''$

$DL = \frac{11.5}{12}(150) = 143.75$ psf $\qquad SDL = 25$ psf

LL reduction Exterior bay $\quad A_T = 30(42.5) = 1275$
$K_{LL} = 1$
$LL = 0.67(100) = 67$



$a_{int} = 10.5 - 1 = 9.5''$ $\qquad a_{end} = \left(\frac{5.75 + 9.5}{2}\right) - 1.75 = 5.875''$

E-W Direction

Prestress force to balance 60% of SW DL

$w_b = 0.6(143.75)(30) = 2.58\ ^k/_{ft}$

$P = \dfrac{2.58(42.5)^2}{8\left(\frac{5.875}{12}\right)} = 1189$

$\dfrac{1189}{26.6} = \boxed{45 \text{ tendons}}$

$P_{act} = 45(26.6) = 1197$ Balance load = $\dfrac{1197}{1189}(2.58) = 2.6\ ^k/_{ft}$

actual Precompression Stress : $\dfrac{1197}{(11.5 \cdot 30 \cdot 12)} = 289$ psi $125 < 289 < 300 \therefore$ ok

N-S Direction

Prestress force to balance 95% of SW DL

$w_b = 0.95(143.75)(21.25) = 2.9\ ^k/_{ft}$

$P = \dfrac{(30)^2}{8\left(\frac{5.875}{12}\right)} = 666^k$

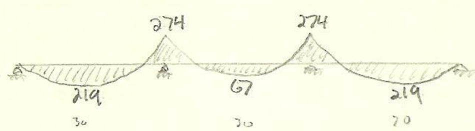
$\dfrac{666}{26.6} = \boxed{25 \text{ tendons}}$

$P_{actual} = 666$ Balance load = $2.9\ ^k/_{ft}$

actual Precompression Stress : $\dfrac{666}{(11.5 \cdot 21.25 \cdot 12)} = 227$ psi $125 < 227 < 300 \therefore$ ok

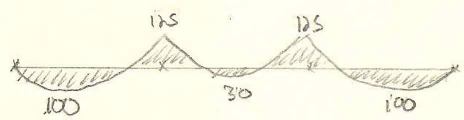
DL Moments N-S

$W_{DL} = (143.75 + 125)(21.25)/1000 = 3.05$



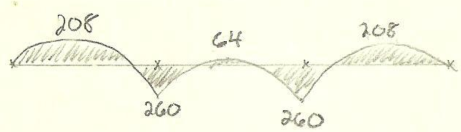
LL Moments N-S

w_{LL} = 67(20.5)/1000 = 1.4



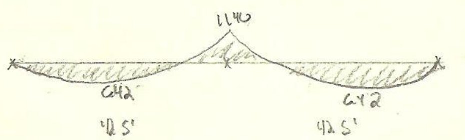
Bal Moments 2.9

w_b = 2.58



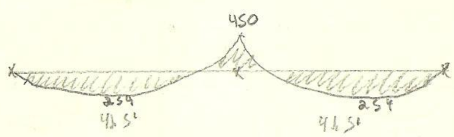
DL Moments

w_{DL} = 5.06



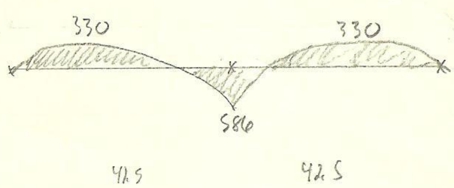
LL Moments

w_{LL} = 2



Bal Moment

w_B = 2.6



N-S
Stage 1 : Stresses Immediately after Jacking (DL + PT)

$S = \frac{21.25(11.5)^2(12)}{6} = 5620$

- Int Span :

$f_{top} = [(-67 + 64)(12)(1000)]/5620 - 227$

$\boxed{= -283 \text{ psi} \quad OK}$

$f_{bot} = [(67 - 64)(12)(1000)]/5620 - 227$

$\boxed{= -221 \text{ psi} \quad OK}$

- End Span :

$f_{top} = [(-219 + 208)(12000)]/5620 - 227$

$\boxed{= -250 \text{ psi} \quad OK}$

$f_{bot} = [(219 - 208)(12000)]/5620 - 227$

$\boxed{= -203.5 \text{ psi} \quad OK}$

- Support Stress :

$f_{top} = [(274 - 260)(12000)]/5620 - 227$

$\boxed{= -197 \text{ psi} \quad OK}$

$f_{bot} = (-274 + 260)(12000)/5620 - 227$

$\boxed{= -256 \text{ psi} \quad OK}$

N-S
Stage 2 : Stress @ Service (DL + LL + PT)

- Int Span : f_{top} $[(-67 - 30 + 64)(12000)]/5620 - 227$

$\boxed{= -297 \text{ psi} \quad OK}$

$f_{bot} = [[67 + 30 - 64](12000)]/5620 - 227$

$\boxed{= -156 \text{ psi} \quad OK}$

- End Span $f_{top} = [(-219 - 100 + 208)(12000)]/5620 - 227$

$\boxed{= -464 \text{ psi} \quad OK}$

$f_{bot} = [(219 + 100 - 208)(12000)]/5620 - 227$

$\boxed{= 10 \text{ psi} \quad OK}$

- Support Stress $f_{top} = [(274 + 125 - 260)(12000)]/5620 - 227$

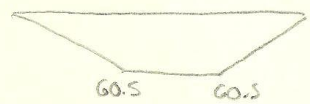
$\boxed{= 69 \text{ psi} \quad OK}$

$f_{bot} = [(-274 - 125 + 260)(12000)]/5620 - 227$

$\boxed{= -523 \text{ psi} \quad OK}$

Ultimate Strength $M_1 = P^* e = 666(3")/12 = 166.5$

$M_{sec} = 227 - 166.5 = 60.5$ at int supports



M_u @ Midspan $= 1.2(212) + 1.6(160) + 1.0\left(\frac{60.5}{2}\right) = 453^{1K}$

M_u @ Support $= 1.2(274) + 1.6(125) + 1.0(60.5) = -468.3^{1K}$

Min Bonded Reinforcement

Positive Moment : Interior Span $f_t = - \leq 2\sqrt{f'_c} = 141 psi$
None required

End Span $f_t = 69 psi \leq 141 psi$
None required

Negative Moment :

Int Supports : $A_{s\,min} = 0.00075 A_{cf}$ $\quad A_{cf} = (11.5)\left(\frac{30}{2}\right)(12) = 2070$

$A_{s\,min} = 1.5\ in^2$ $\quad$ 8 - #4 bars top $(1.6\ in^2)$

Exterior Supports : $A_{s\,min} = 1.5\ in^2$ $\quad$ 8 - #5 bars top $(2.6\ in^2)$

* • Must span 1/6 clear span on each side of support
• Atleast 4 bars in each direction
• Top bar places @ 17.25" away from face of support
• Max bar spacing 12"

Check Min Reinforcement for Ult STR

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$ $\quad d = 10.5$

$A_{ps} = 0.153(25) = 3.83$

$f_{ps} = 174{,}000 + 10{,}000 + [(5000)(42.5)(12)(10.5)] / [300 \cdot 3.83] = 207{,}302$

$a = [(3.1)(60) + (3.83)(207)] / (.85 \cdot 5 \cdot 42.5 \cdot 12) = 0.41$

$\phi M_n = 0.9(988)(10.295)/12 = 685$

N-S Summary:

@ Neg Moment 10 - #5 top bars

(25) 1/2" ø, 7-wire strands

Acting along Col strips

E-W Stage 1 (DL+PT) $w_b = 2.6$ $w_D = 5.06$ $w_L = 2$

End Span

$S = 30(12)(11.5)^2/6 = 7935$

$f_{top} = [(-642 + 330)(12000)]/7935 - 289$

$= -760$ psi tension not ok ignore for Preliminary design

$f_{bot} = [(642 - 330)(12000)]/7935 - 289$

$= 182$ psi, C, ok

Support Stress $f_{top} = [(1140 - 586)(12000)]/7935 - 289$

$= 548$, C, ok

$f_{bot} = [(-1140 + 586)(12000)]/7935 - 289$

$= -1126$ psi, T, not ok ignore for prelim design

E-W Stage 2: (DL+LL+PT)

End Span: $f_{top} = [(-642 - 254 + 330)(12000)]/7935 - 289$

$= -1144$ psi, T, not ok

$f_{bot} = [(642 + 254 - 330)(12000)]/7935 - 289$

$= 566$ psi, C, ok

Support Stress $f_{top} = [(1140 + 450 - 586)(12000)]/7935 - 289$

$= 1229$ psi, C, ok

$f_{bot} = [(-1140 - 450 + 586)(12000)]/7935 - 289$

$= -1807$ psi, T, not ok

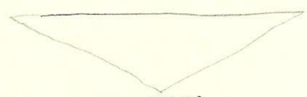
* Most tension values are off but decided it was ok for a preliminary design

Would model as a 3 span in future but was not positive on how to handle a 42.5 - 20 - 42.5 3 span w/ post-tension

Ultimate Strength

$M_1 = P * e = 1189(3)/12 = 297.25$

$M_{sec} = 586 - 297.25 = 288.75$



M_u @ Midspan $= 1.2(642) + 1.6(254) + 1.0\left(\frac{288.75}{2}\right) = 1321^{\,\text{'K}}$

M_u @ Support $= 1.2(-1110) + 1.6(-450) + 288.75 = -1799.25^{\,\text{'K}}$

Det Min Bonded Rein

Neglecting interior span for preliminary Design

End Span $\quad f_t = 566 > 2\sqrt{f'c}$

$y = 566/(566 + 1144)(11.5) = 3.8''$

$N_c = \left[(642 + 254)(12)/(2935)\right](0.5)(3.8)(30)(12)) = 926$

$A_{s\,min} = 926/(0.5 \cdot 60) = 30\ in^2$

$30/30 = 1''/ft$

use #6 @ 5'' $= 1.01\ in^2/ft$

use #6 @ 5'' oc bottom

- Min length shall be 1/3 clear span and centered in positive region

Neg Moment region

$A_{cf} = 11.5(42.5)/2\,(12) = 2932.5$

$A_{s\,min} = 2.2\ in^2$

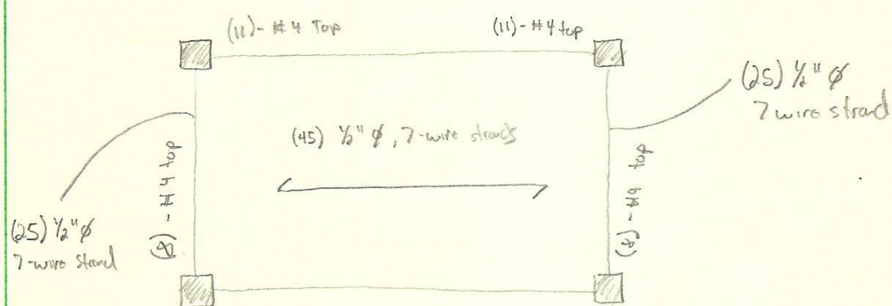
Use 11 #4 bars top ($2.2\ in^2$)

Summary E-W

11 - #4 top bars

(45) 1/2" ∅, 7-wire strands

Final Design



Slab: 11.5" thick