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TECHNICAL
REPORT II

FAIRFIELD INN & SUITES, MARRIOT
PITTSBURGH, PA

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Structural Option

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EXECUTIVE SUMMARY

In this technical report, the existing floor system of the Fairfield Inn and Suites is analyzed, and alternative floor systems are designed and discussed to determine the viability of each system. Currently, the floor system used in the Fairfield Inn and Suites is a hollow-core precast concrete plank floor which is adequately designed to handle the criteria for the building. The technical report looked at the following alternative floors systems for the Fairfield Inn and Suites:

1. Hollow-core precast concrete plank floor on steel framing
2. Non-composite steel system
3. Two-Way post tensioned slab

The existing hollow-core precast plank system sits on load bearing masonry which allows for an 8” slab thickness, assumed to be designed by the PCI Design Handbook. This system is light weight and takes advantage of using larger spans without the use of columns throughout the entire building. The hollow-core precast plank system on steel framing was designed using the PCI Design Handbook to determine a 6” concrete slab with 2” topping was necessary to carry the loads of the building. The steel girders were designed by taking into account the deflection caused by the live loads of building and using the AISC Steel Manual to determine the W18x35 sized members. The non-composite steel system was designed using the AISC Steel Manual and Vulcraft Steel Floor Deck guide. The preliminary design consists of a 2C18 metal deck under a 4.5” concrete slab. The supporting girders and beams were determined to be W21x48 and W10x12. The two-way post tensioned slab was determined to have a preliminary 7” slab with 12 tendons uniformly distributed in the long direction and 13 tendons banded in the short direction. Due to a small number of tendons in each direction, it may suggest that the slab thickness is conservative and in further investigation may find an even thinner slab thickness.

The advantages and disadvantages are discussed for each floor system, and ultimately the existing floor system is the best choice for this type of construction. But, through comparison of the alternate systems it was determined that the two-way post tensioned slab may be the most feasible system under further investigation, as it would hardly alter the existing building conditions and gives a slab thickness of 7”, thinner that the existing. The steel framing system and hollow-core plank system on steel would alter the floor-to-floor height too drastically, as the new floor depth for each system would be approximately 25”. Each of these alternative systems and the structural system of the building, as a whole, can be seen through detailed descriptions and diagrams, as well as, the materials and codes used in the actual design of the floor systems. Building layout and detailed calculations for each analysis performed can be found in an Appendix at the end of the report.

INTRODUCTION: Fairfield Inn & Suites

Fairfield Inn and Suites is a 10-story hotel. The hotel is located in the heart of Pittsburgh within walking distance to downtown Pittsburgh, Heinz Field (football stadium), the new Rivers casino, plus many other Pittsburgh attractions. The hotel's closest attraction, directly across the street, is the Pittsburgh Pirates baseball stadium, PNC Park. Being in such a prime location, this hotel with accommodate thousands of guests visiting the area throughout the year making it an essential addition to the community.

The hotel occupies 135 guest rooms in addition to an indoor pool and fitness center for its guests. There will be a variety of typical king/queen size rooms to king/queen suites to satisfy the needs of all guests. Guests to the hotel will enter into an 18' lobby off of Federal St. where the main entrance exists. The lobby consists of a large reception desk for check-in/out, a breakfast area, and a large seating area featuring a cherry finished wood fireplace. The hotel holds a basement below grade that consists of the electrical, mechanical, and maintenance rooms, along with the laundry room and break room for employees.

The façade of the building is similar for all views. Cast-stone decorates the exterior levels one thru four. Brick veneer than extends to the roof of the building. As one approaches the 18' lobby entrance a glass curtain wall system surrounds the entrance doors and extends above the entrance two stories adding verticality to the building. The entrance is then emphasized by a large steel supported, tempered glass awning shading the lobby. On street level, the lobby is lined by additional high glass windows also shaded with smaller glass awnings. From the highway that passes the buildings north façade, one will notice the hotel by its large illuminated sign placed inside a 56'x18' bond-face brick detailed rectangle accenting this view.

The structural system for the hotel is primarily hollow-core precast concrete plank floors on load bearing masonry walls, while shear walls resist the lateral forces against building. Steel transfer beams at the second floor transfer the loads of the load bearing walls to columns supporting the 18' lobby. The ground floor is a concrete slab on grade that transfers the gravity loads of the building to a foundation system that is composed of auger cast piles and steel grade beams.

The purpose of Technical Report 2 is to take a closer look at the existing floor system of the Fairfield Inn and Suites. Alternative floor systems were also designed and analyzed to fit into the existing building conditions. A comparison is given in regards to each floor system's framing and structural slabs designed to determine which floor system is best suited for the building's structural system by weighing the pros and cons of each floor.

STRUCTURAL SYSTEM

Foundation

A geotechnical soils report was conducted for the Fairfield Inn and Suites site on November 27, 2007 by Construction Engineering Consultants. In the study, it was found that the typical soil found on site is brown silt, clay, and sand. The reported water level was approximately 25’-0” on site. The depth of the basement is 12’-8” below grade, therefore there should not be a concern regarding the uplift pressures on the foundation due to the water level. Due to the moderate depth to bedrock and precaution taken in regards to water level, the deep foundation system consists of auger cast friction piles and grade beams. With the foundation not extending below 33 ft., the net allowable bearing pressure on site is 200 psf.

The ground floor rest on a 6” concrete slab which is 5 ksi normal weight concrete (NWC). The slab increases in thickness from 6” to 12” within the core shear walls where the elevator pit and area well are located. The slab reinforcement consists of W/ 6x6-W1.2xW1.2 welded wire fabric and #5 bars located 12” o.c. top and bottom and each way. The slab depth is approximately 12’-8” below grade, while the elevator pit extends to 17’-5” below grade.

The piles extend 12’-8” deep below grade and are spaced approximately between 26’ to 31’ apart (refer to Appendix A). The typical size of the pile caps is a 7’-6” square approximately 4’ deep with four 16” diameter piles per cap. The core shear walls incasing the stairs and elevator have additional rectangular pile caps and piles for more support. Pile caps are reinforced with #8 bars at 6” o.c. The typical column piers extending from the pile caps are composite 24”x24” columns with horizontal ties and vertical bar reinforcement. (see Figure 1.1)

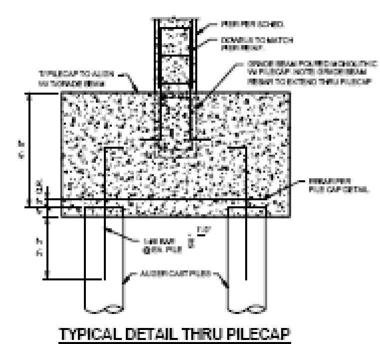


Figure 1.1

Grade beams run between pile caps transferring the loads from the façade and interior shear walls to the foundation (refer to Figure 1.2). Depth of beams ranges between 36” and 48” depending on location. Reinforcement and size varies per grade beam.

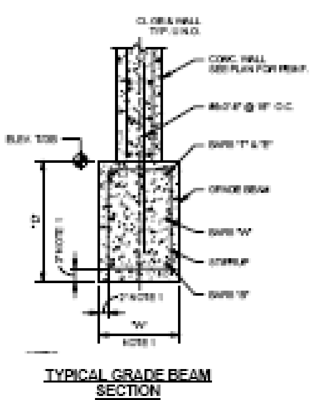


Figure 1.2

Floor System

Fairfield Inn and Suites typical floor system is a precast concrete plank floor with a thickness of 8" untopped. The hollow core concrete plank floor allows for the building to be supported without the use of columns on floors two thru ten and longer plank spans. Concrete compressive strength for floors is f'c=5000 psi. The typical span of the precast plank floors are 31'-0" and 26'-0". The floor systems supported by load bearing concrete masonry walls.

The floor system for the first floor is a combination between 4" slab on grade and the 8" precast concrete plank floor. There is no basement below the first floor running along the south wall and the entrance on the west wall of the building (see Figure 2.1). Due to a pool being located in this area, the hollow core of the typical plank floor would not be sufficient in supporting the weight of the pool and lobby live loads. Therefore, the floor system is a 4" slab on grade with W/6x6-W1.4xW1.4 weld wire fabric reinforcement.

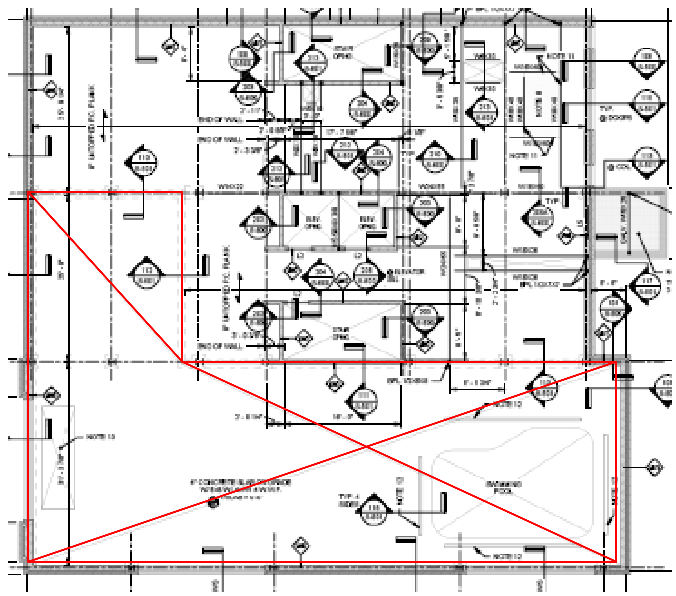


Figure 3.1: Partial First Floor Slab

Since the floor system is a precast plank floor, there are a limited number of steel beams girders throughout the structure. These transfer beams range in size from W 33x118 to W 40X149.With no columns to support floors two thru ten, the majority of the beams present are transfer beams on the second floor that transfer loads from the floors above to the columns extending from the pile caps and thus transferring all loads to the foundation system. The transfer beams run along the back of the elevator shafts from the west wall to the east wall, and along the back of south wall of stair B extending from the west wall to the east wall (see Figure 2.2). Transfer beams range in size from W 33x118 to W 40x149. Girders run along the first floor supporting mechanical equipment loads and tying into the beams and shear walls supporting the first floor. Girders and beams throughout the building are non-composite systems.

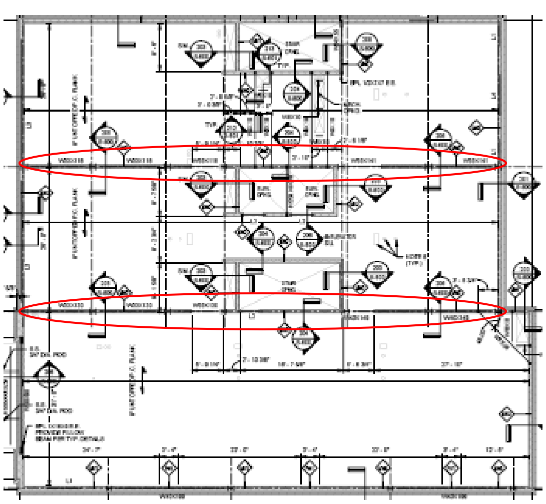


Figure 2.2: Second Floor Transfer Beams

The roof system and smaller high roof system are the same use the same 8" untopped precast concrete plank floor. W8x28 beams run along the shear walls inclosing the elevator and stair shaft while W8x18's extend outward from the corners of the shear walls inclosing the shaft. Hoist beams support the top of the elevator shaft in high roof system. There are a total of six drains located on the roof for the drainage system. (refer to Appendix A)

Columns

The only columns used in the Fairfield Inn and Suites are the ones extending from the pile caps to the second floor supporting the 18' first floor. The columns range in size from W10x100's to W 12x120's depending on location. All columns connect into the pile caps where the weight each column supports transfers the load down to the foundation (refer to Figure 3.1). The base plates are ½" thick and typically 14"x14". Each plate utilizes a standard 4 bolt connection using 1" A325 bolts.

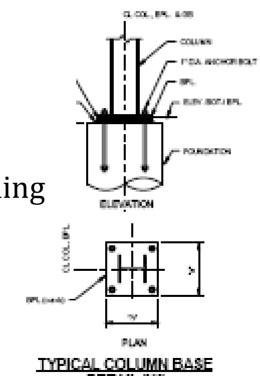


Figure 3.1

Lateral System

The lateral system for the Fairfield Inn and Suites is a combination of ordinary reinforced concrete masonry shear walls. The exterior shear walls are 10" concrete masonry and the core shear walls are 8" concrete masonry. The core shear walls surround the staircases and elevator shaft. On floors two thru ten, two additional load bearing masonry walls extend from the west wall to the east wall running along the south wall of staircase B and the north wall of the elevator shafts (see Figure 4.1). Shear walls supporting the ground floor to the fourth floor support a compressive strength of f'c=8000 psi. All other shear walls support a compressive strength of f'c=5000 psi. The typical reinforcement in both the 10" and 8" shear walls is #5 bars at 16" o.c., 24" o.c., or 32" o.c. with bars centered in wall and solid grout wall.

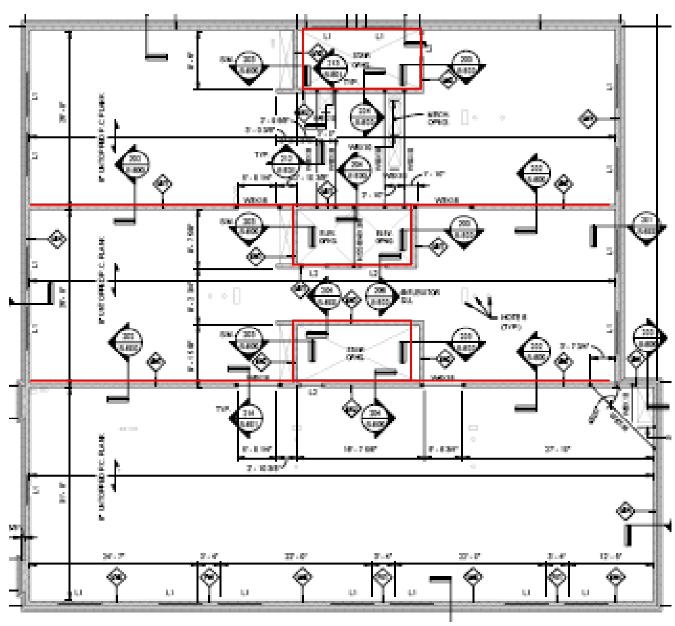


Figure 4.1: Lateral Shear Wall System

The wind and seismic loads, as well as gravity loads, reach the foundation by first traveling through the rigid building diaphragm (floor system) to the load bearing walls. From there the loads carry through the transfer beams and girders which connect to the columns at second floor. All loads travel in the columns to the basement level and into the auger cast piles and grade beam foundation. This load path is governed by the concept of relative stiffness.

CODES AND REQUIREMENTS

Various references were used by the engineer of record in order to carry out the structural design of the Fairfield Inn and Suites:

- The 2006 International Building Codes as amended by the city of Pittsburgh
- The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- Specifications for Structural Concrete (ACI 301-05), American Concrete Institute
- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute
- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute
- PCI Design Handbook – Precast/Prestressed Concrete Institute
- Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (AISC), American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers
- RS Means Assemblies Cost Data
- RS Means Facilities Construction Cost Data
- Live load deflection criteria used: $\leq \ell/360$
- Total load deflection criteria used: $\leq \ell/240$

GRAVITY LOADS

The gravity load conditions determined by ASCE 7-05 are provided for reference:

Dead Loads:

Concrete	150 pcf
Steel	490 pcf
Partitions	15 psf
MEP	10 psf
Finishes and Miscellaneous	5 psf
Roof	20 psf

Live Loads:

Description	Design Load Used By Engineer	ASCE 7-05
Public Areas	100 psf	100 psf
Lobbies	100 psf	100 psf
First Floor Corridors	100 psf	100 psf
Corridors above First Floor	80 psf	80 psf
Private Hotel Rooms	40 psf	40 psf
Stairs	100 psf	100 psf
Roof	75 psf	20 psf
Mechanical	150 psf	150 psf

FLOOR SYSTEMS

EXISTING: Hollow-core precast concrete plank on load bearing masonry

Material Properties

Concrete: 4'-0"x8" untopped
f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Tendons: 76-S
f_{pu} = 270,000 psi

Loadings: Dead (self weight) = 56 psf
Live = 40 psf
Superimposed = 25 psf

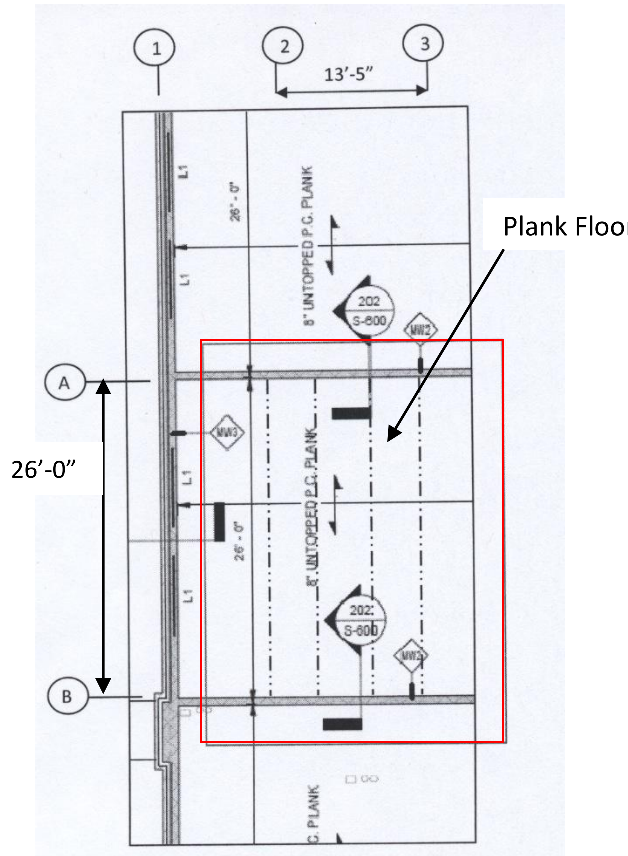


Figure 5.1 – Hollow-core plank floor

Description

The hollow core precast concrete plank system spans distance of 26'-0" for the particular section of the building shown in Figure 5.1 and the 4'-0" wide planks run the entire length of the floor. In regards to the analysis of this floor system, an interior section of 26'-0"x13'-5" bay was used as shown in Figure 5.1. The plank floor system is framed into a load bearing masonry walls that distribute the weight of the precast concrete floor.

The planks that were designed for the building are 8" thick planks un-topped. Unable to retrieve the actual design method of the planks from the manufacture, the design assumption was made that the planks were designed using the PCI Design Handbook. In order to achieve the 26'-0" span of the planks, 76-S strands were used within the hollow core panel. This relates to the designation of the number of strands (7), the diameter of the strands in 16^{th} (6), and that the strands are to be straight throughout the panel. The assembly of this panel can hold a service load of 95 psf which exceeds the total load calculated of 80 psf. The total load is a combination of live loads, superimposed dead loads, and an additional 10 psf for untopped members. Supporting calculations may be found in Appendix B.

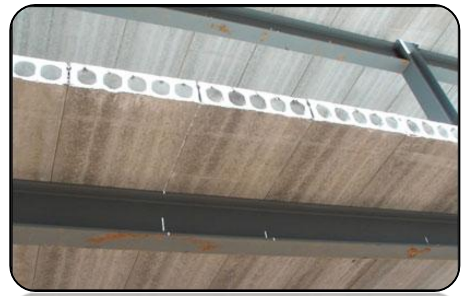
Advantages

One of the greatest advantages to using the precast hollow core plank floor system is the time efficiency that it allows. The precast concrete does not require the curing time that concrete that is cast-in-place requires, allowing for it to be installed much quicker. This leads to a faster construction schedule and ultimately lower overall project cost. The typical span of a hollow-core system tends to be greater and has a greater loading capability increasing the size of the basic structural grid. Along with a longer span, the floor depth of the precast planks is much shallower allowing for the most efficient use of the floor-to-floor heights. With the plank floor system resting on the loading bearing masonry walls, the entire system is concrete, which is a good sound-insulating material and fire-resistant without any fire proofing required.

Disadvantages

The most relevant disadvantage of using the hollow core precast plank system is that precast concrete requires more upfront planning. The faster construction schedule could be counteracted by prolonged time in the design process for precast design.

ALTERNATIVE #1: Hollow-core precast concrete plank on steel



Material Properties

Concrete: 4'-0"x6" with 2" topping
f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Tendons: 96-S
f_{pu} = 270,000 psi

Loadings: Dead (self weight) = 74 psf
Live = 40 psf
Superimposed = 25 psf

Description

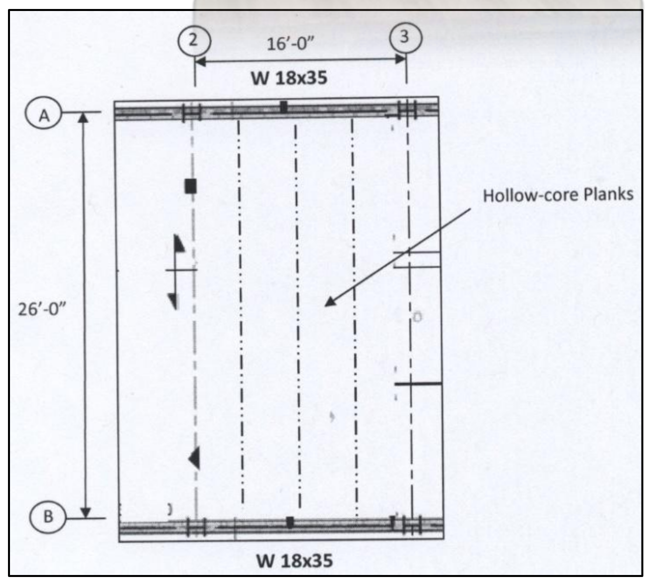


Figure 6.1 – Hollow-core planks on steel

The hollow-core precast concrete plank on steel system is very similar to the existing floor system of the building. This system would dismiss the use of the load bearing masonry walls. The existing columns that run from the foundation to the second floor would further be extended to run through all floors of the building. These columns were not analyzed and designed for the conditions of the alternative floor system in this report, as they are part of the lateral system and will be discussed at a later time.

The planks will span distances of 26'-0" and 31'-0", while the widths of the panels are in 4'-0" increments. Since the existing floor system uses load bearing masonry walls to support the panels, there is no set dimension for the size of the bays. The columns that do extend from the foundation to the second floor are spaced at a minimum 10'-5" to a maximum 13'-5" apart, which when extended through all the floors, would give the building its bay sizes. In regards to this analysis, since the panels are in 4'-0" increments, an interior bay size of 26'-0"x16'-0" is designed as seen in Figure 6.1.

In order to keep with the existing slab depth of 8", a 6" plank with 2" topping was selected using the PCK Design Handbook. In order to achieve the 26'-0" plank span, strands of 96-S were used within the hollow-core panel. The designation relates the number of strands (9) with the diameter of the strand in 16ths (6/16"). The strands are to be straight, as determined by the S. The design of this plank system is capable of holding a capacity of 82 psf. This exceeds the value of the total load 80 psf, determined by the live load, superimposed loads, and dead load of a 2" topped concrete plank member.

The steel members that the precast concrete planks will frame into were designed using the American Institute of Steel Construction manual (AISC). Girders were determined to be W18x35 members. W10x12 beams can be used parallel to panels to add stability to the floor system. Supporting calculations for this floor system can be found in Appendix C.

Advantages

The hollow-core precast concrete plank system on steel has numerous benefits. The system as a whole is recognized as a LEED rated system, which for many projects and buildings today, it is necessary to be LEED approved. The light weight of the hollow-core precast concrete allows for larger bay sizes, as well as typical girder sizes to support the live and total loads from deflection. With no curing time of the precast concrete, the floor system can be constructed year round allowing for faster construction of the project. This system is also durable and low maintenance, reducing future costs for the owner.

Disadvantages

Along with the advantages, there are several disadvantages with the plank system on steel. The main disadvantage is the decrease in floor-to-floor height. The decrease is due to the deeper floor system caused by the W18x35 steel girders that support the planks. The floor depth would increase from 8" (existing floor system) to 25.7" (the 17.7" depth of girder + 8" precast concrete). This would present a problem if the building is located in an area where building height is limited. Not only would the precast concrete produce extra lead time in the design process as mentioned previously, but the steel would need upfront planning. The fabrication, detailing, and transportation of the steel could increase the lead time. The steel also would require spray fireproofing to obtain the appropriate fire rating. All these factors could increase the cost of the overall project.

Feasibility

In Pittsburgh, the building height limit is 11 stories, and the building currently occupies 10 stories, therefore this system could still exist within the boundary conditions for this building at its current location. Depending if this system could dramatically impact the pace of the construction, leading to a faster construction schedule, this system could be a likely candidate for further investigation. With the faster construction schedule, the money saved could account for the few cost disadvantages this system posses' in its use for the Fairfield Inn and Suites.

ALTERNATIVE #2: Non-Composite Steel Framing

Material properties

Concrete: 4.5” slab
2.5” topping
f’c = 3,000 psi

Steel: fy = 50,000 psi
Reinforcement: fy = 60,000 psi
Metal Deck: 2C18 – 3 span

Loadings: Dead Load (self weight) = 45 psf
Live load = 40 psf
Superimposed = 25 psf

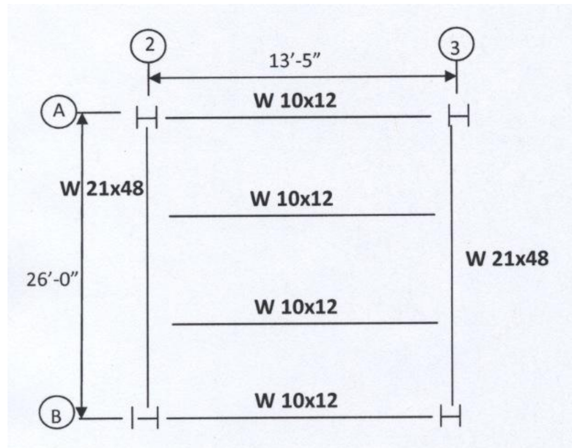
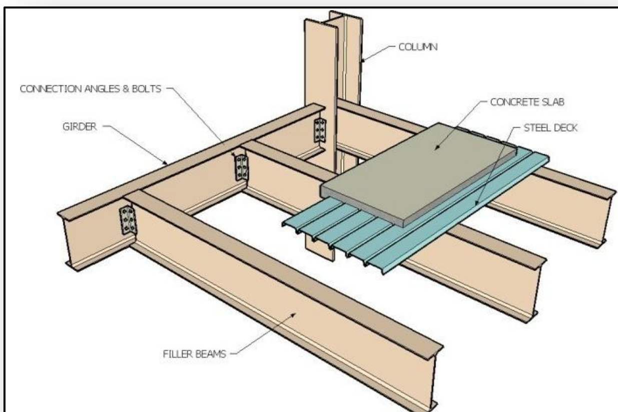


Figure 7.1 – Steel Framing



Description

The typical bay sized used to design a non-composite floor system is a 26’-0”x13’-5”. This was chosen because in order for this system to work for the building, the existing columns would need to extend to the roof. As to not alter the building too much, the spacing for the columns would remain the same for the building, although the column sizes would probably change. At this point, column design was not completed. Ultimately, this is what determined the bay size analyzed. Intermediate beams would be spaced equally at 8’-8” as seen in Figure 7.1.

A 2C18 non-composite Vulcraft deck is used to accompany a 4.5” concrete slab. For the normal weight concrete slab with a 2.5” topping, the deck is able to span 12’-4” unshored giving a 3 span condition. This well exceeds the 8’-8” spacing used for this design. The size of the steel girders and beams were designed according to the American Institute of Steel Construction manual (AISC). The determined size of the steel framing can be seen in Figure 7.1. The size of the members designed and the slab thickness satisfies the load and deflection limits of the entire system. Supporting calculations for the steel framing and concrete slab can be found in Appendix D.

Advantages

The most beneficial advantage of the non-composite steel is the quick erection of this system, speeding up the overall project construction. The non-composite system requires no formwork and therefore reducing the labor of the layout. Since the decking spans 12'-4" during un-shoring construction, no shoring is necessary. The absence of shear studs that a composite system would require lowers the cost of the project as well. Additionally, there is flexibility in the system when it comes to laying out our building systems throughout the building.

Disadvantages

Once again, the depth of the steel beams will reduce the floor-to-floor height in the building. The girder size designed is a W21x48 creating a 25.2" floor system depth including the 4.5" concrete slab on deck. This would either adjust the entire height of the building, adding additional costs to the owner, or it would reduce ceiling heights giving the hotel rooms a tighter feel. The self weight of this floor system is also substantially larger than that of the existing system. This could cause an increased loading on the framing members in flexure, which in turn could raise the cost of materials for the floor system. Lead time is also a factor in working with steel due to steel needing fabrication, detailing, and transportation to the project. With an all steel framing system, fireproofing would be necessary to obtain an approved fire rating for the building. With the occupancy of the building being a hotel, the rooms need a certain amount of privacy and steel materials are not known to be sound-insulating materials, therefore extra sound insulation may be necessary in the walls, ceilings, and floors, to keep the noise entering and exiting each guest room down.

Feasibility

Ultimately, after looking at the advantages and disadvantages of the non-composite system, it seems the disadvantages outweigh the advantages. Therefore, use of the this system in the Fairfield Inn and Suites is not likely, due to the decrease in floor-to-floor height and the additional costs that may be present, and no further investigation is necessary.

ALTERNATIVE #3: Two-Way Post Tensioned concrete slab

Material properties

Two-Way Post Tension before pour concrete

Concrete: 7" slab (NWC)
f'_c = 5,000 psi
f'_{ci} = 3,000 psi

Rebar: f_y = 60,000 psi

Tendons: unbonded
½" ø – 7 wire strands
A_{pt} = 0.153 in^2
f_{pu} = 270,000 psi

Loadings: Dead (self weight) = 87.5 psf
Live = 40 psf
Superimposed = 25 psf

Description

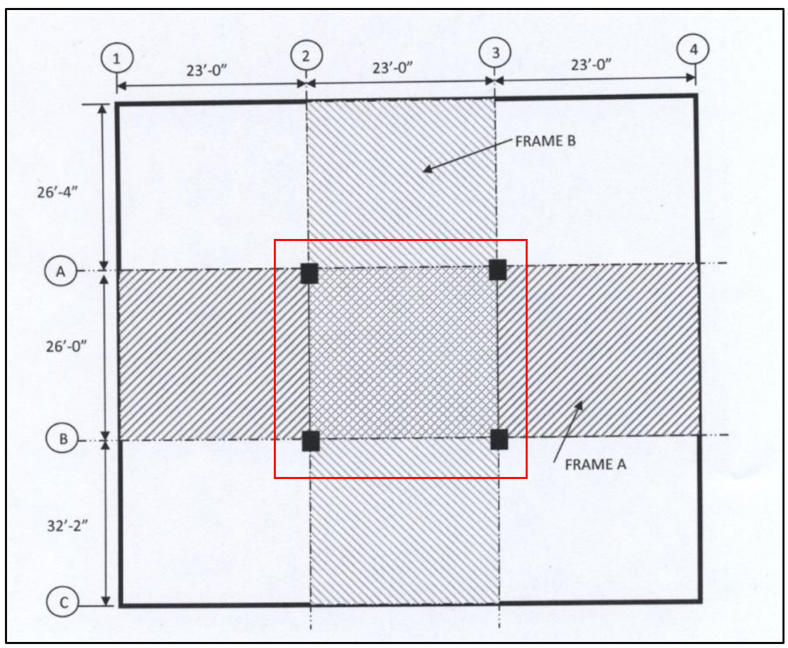


Figure 8.1 – Two-Way Post Tensioned Slab

Through the design of a two-way post tensioned slab, a typical bay size of 26'-0"x23'-0" was used as seen in Figure 8.1. The preliminary slab thickness of 7" was determined by the slab/depth ratio of 45. Conservatively, the slab/depth ratio of 40 would give a slab thickness of 8", but in order to exceed to advantages of the existing floor system, a thinner slab thickness of 7" was designed. Columns were not analyzed in this study with the new alternate floor system, but the design assumptions made in this analysis use the existing column conditions for the building structure. The existing load bearing masonry walls that support the current floor system would not be necessary in the building with this floor system because there would be columns extending from the foundation through all floors.

Assuming the direct design method, 12 uniform tendons were required in the long direction and 13 banded tendons necessary in the short direction with a resistance of 26.6

kips/tendon. The banded tendons in the short direction and uniformly distributed tendons in the long direction works well with this type of construction in regards to the placement of tendons at openings. The only large opening in this bay would be the core elevator and stair shafts located within shear walls. In addition to the unbonded tendons, reinforcement was necessary at the interior and exterior supports and ends of the spans. Supporting calculations can be found in Appendix E.

Advantages

The two-way post tensioned slab has many advantages. The thin floor allows for an increase in floor-to-floor height. The thinner slab reduces the amount of concrete needed and can reduce the overall building weight. In turn, this reduces the foundation load and can be a major factor in areas where the soil can't support a heavy building. The post tensioning allows for longer clear spans while the slab can still carry large live loads. The existing building design consists of load bearing masonry walls and transfer beams that carry the weight to the columns down to the foundation, but post tensioning slabs would neglect the use of the load bearing walls completely and could reduce or neglect the use of transfer beams throughout the structure. The rigidity of the post tension limits the effects of vibration in the structure, while the tendons in the slab reduce floor deflection. The reduced amount of concrete and transfer beams in the structure, would impact the overall cost of the project dramatically.

Disadvantages

The two-way post tensioned slab can be very labor intensive and potentially dangerous. This type of system requires people who have experience with its construction. In the construction of a post-tensioned slab, the tendons require jacking to meet the require strength. If the tendons are jacked improperly or place incorrectly, before the concrete is poured, a tendon could snap and rupture through the concrete. This would put a delay in the construction of the slab, in addition to the curing time required for the concrete. Once the concrete is poured, it is very difficult to cut openings into the system because there is a chance a stressed tendon could be cut, altering the design of the slab reinforcement.

Feasibility

The use of a two-way post tensioned slab is a possibility. The use of this system in the Fairfield Inn and Suites could reduce the overall building weight and could eliminate the use of the load bearing masonry walls and transfer beams. The drastic reduce in cost of the project would ultimately outweigh the concerns in the construction of the system. This design is a realistic alternative system for the building and further investigation may prove it is better suited for the building than the existing floor system.

OVERALL SYSTEM COMPARISON

COMPARISON CRITERIA	PRECAST PLANK ON LOAD BEARING WALLS	PRECAST PLANK ON STEEL FRAMING	NON-COMPOSITE STEEL FRAMING	TWO-WAY POST TENSIONED CONCRETE SLAB
Slab Self Weight	56 psf	74 psf	45 psf	87.5 psf
Slab Depth	8”	8”	4.5”	7”
System Depth	8”	25.7”	25.2”	7”
Deflection	1.15”< 1.3”	0.360”<0.675”	1.19”<1.30”	Further study necessary
Vibration	Further Study	Further Study	Poor	Good
Fire-Rating	2 hour	1.5 – 2 hour	1.5 – 2 hour	2 hour
Fire Protection	None	Spray	Spray	None
Impact on Building Design	Existing	Reduces floor-to-ceiling height	Reduces floor-to-ceiling height	Increases floor-to-ceiling height
Constructability	Easy	Easy	Easy	Hard
System Cost*	$12.80/SF	$23.36/SF	$32.50/SF	$19.85/SF
Feasibility	Yes	Yes	No	Yes

*The system cost is a rough estimate using RS Means Assemblies Cost Data and RS Means Facilities Construction Cost Data

CONCLUSION

In analyzing the existing floor system of the Fairfield Inn and Suites, a better understanding of the design decisions was formed. Designing alternative options for the floor system of the Fairfield Inn and Suites, allowed me to understand why certain design considerations were taken into account when designing the building.

After comparing each alternative floor system with the existing system, it was concluded that the existing floor system is the most efficient in construction time, cost, and physical properties for the Fairfield Inn and Suites. However, some of the alternate systems may be a realistic solution for the building as well. A two-way post tensioned slab offered a thinner floor thickness even though it is a heavier system and it is has a very intense and involved construction process. The hollow-core precast plank on steel offers a design that is consistent with the existing system. It is still a light weight system that is time efficient at a low cost. The down fall is, with the addition of the steel beams, the floor depth increases from 8" to 25" sacrificing the floor-to-floor height. A non-composite steel framing system presented the same increase in floor depth for the system. This system is also the most expensive system to construct and is a much heavier system. Overall, this system is the less likely alternative solution for the Fairfield Inn and Suites.

The most likely alternative system for the Fairfield Inn and Suites, other than its existing system, is the two-way post tensioned slab. This system created a thinner overall floor depth being very effective for the building. This system is very cost effective to save the project money. The tendons throughout the slab help carry additional live load while limiting deflection and reduce vibration in the system. The system would alter the lateral system of the building because it eliminates the use of the load bearing walls by using columns, but this could also eliminate the transfer system throughout the building; reducing the overall use of steel and additional fireproofing that would be necessary for the steel beams.

Concrete systems are common in construction practices for midrise hotels; therefore it is logical that a concrete system would be more applicable and feasible for the Fairfield Inn and Suites comparison. Please refer to the following appendices for detailed calculations and analysis of each floor system designed for the Fairfield Inn and Suites.

APPENDIX A

Building Layout

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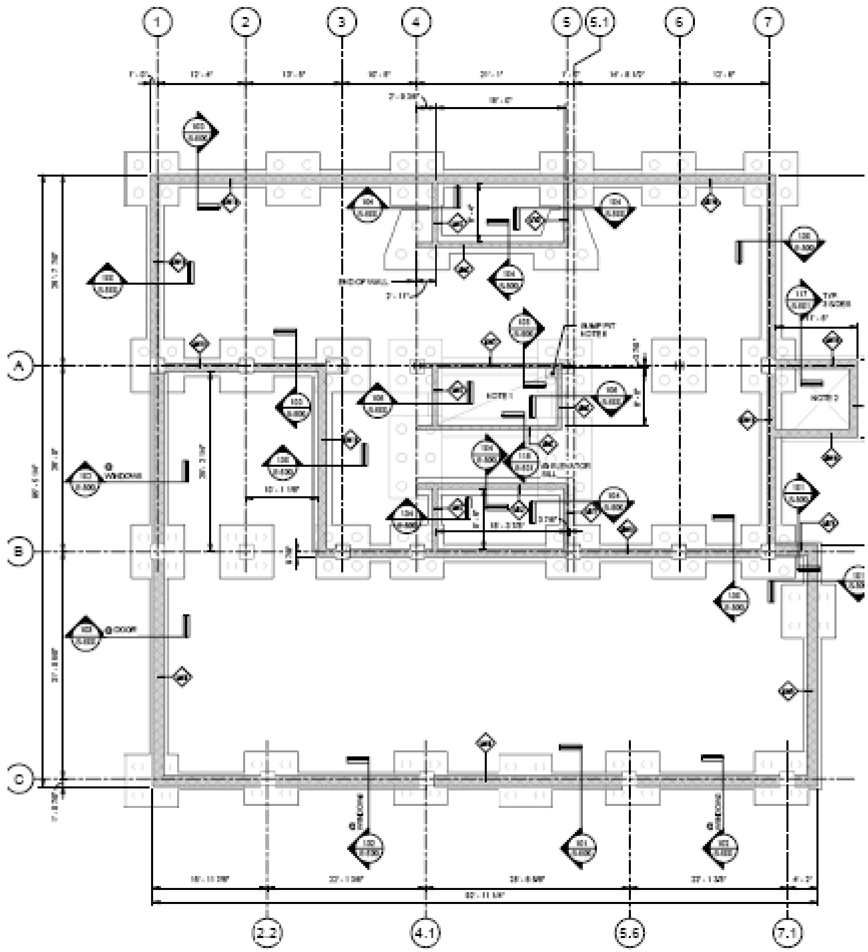
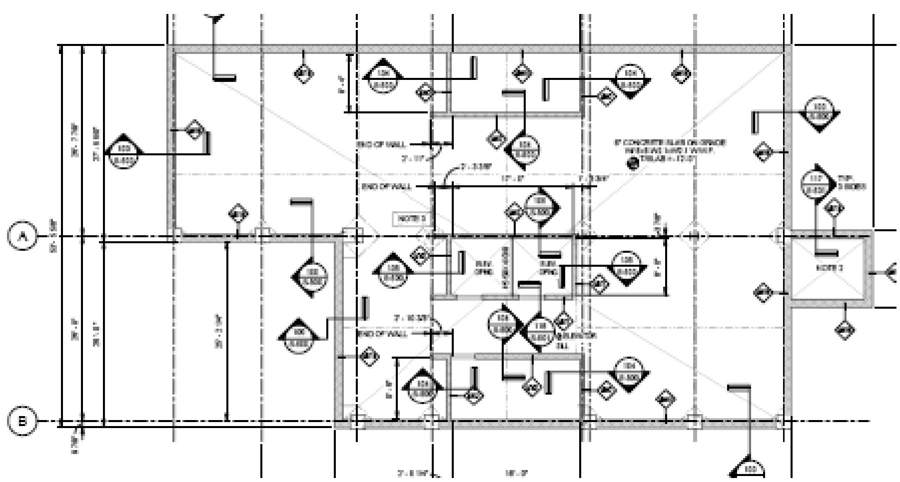
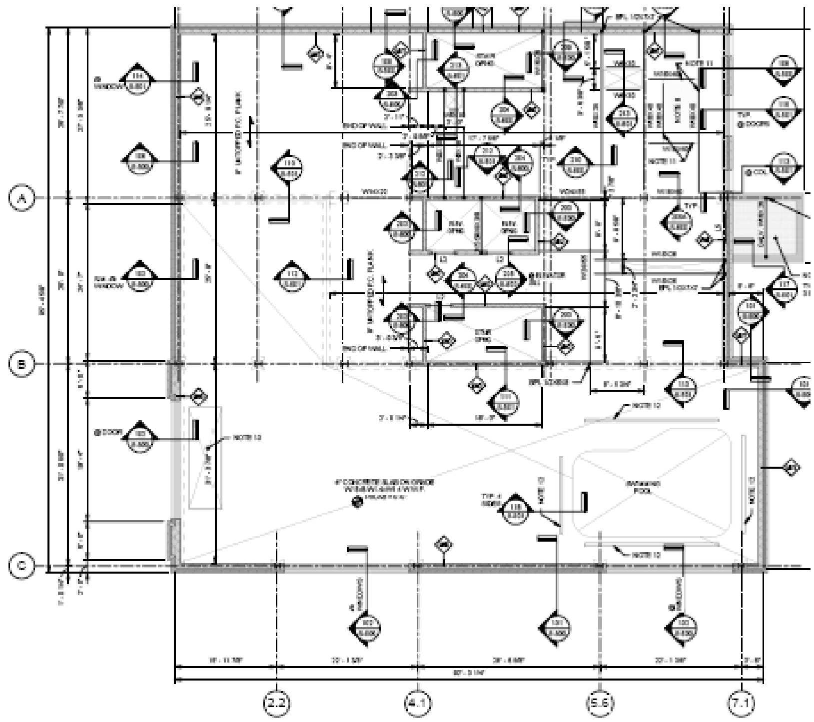


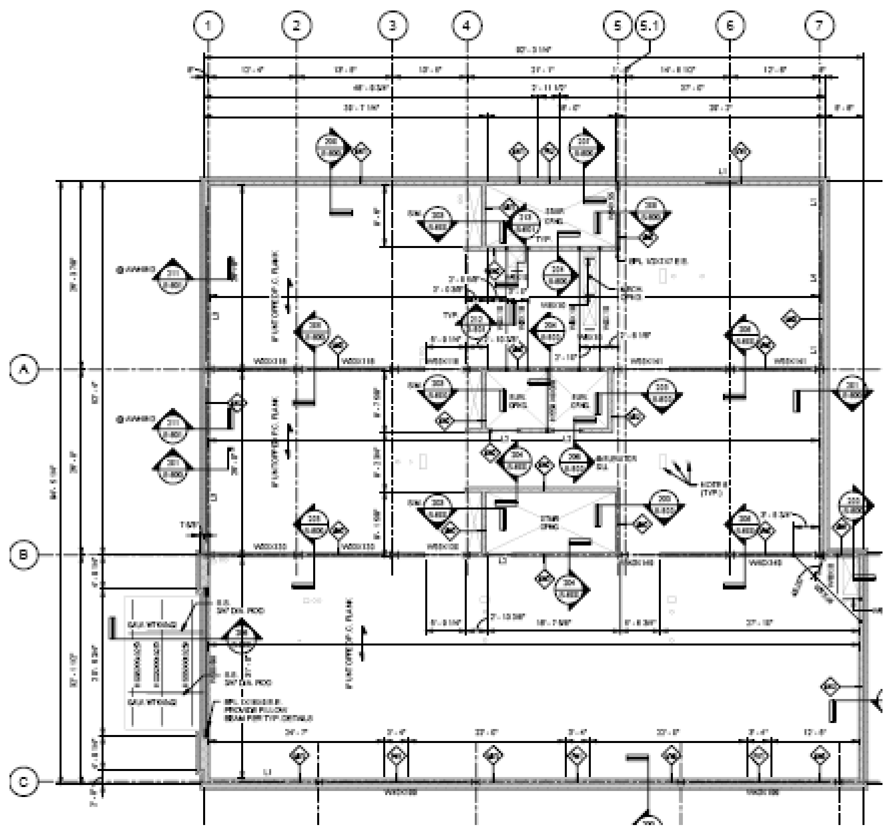
Foundation Plan



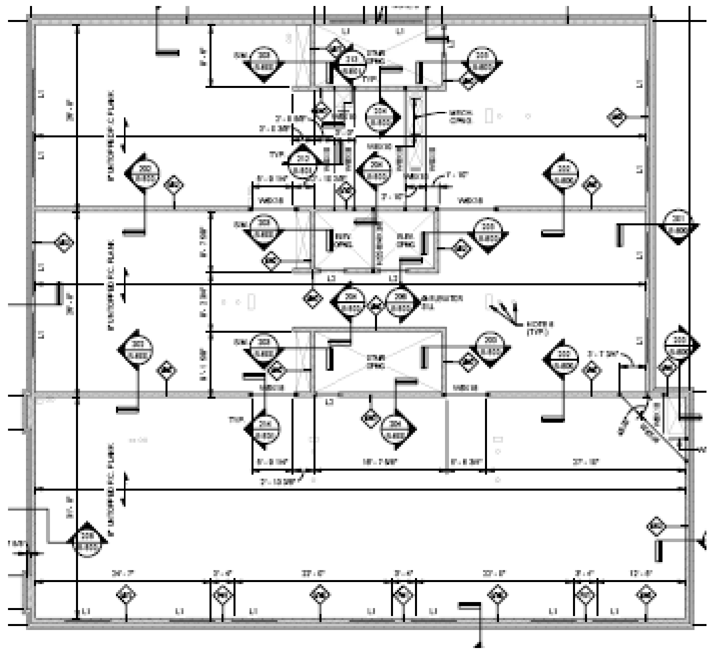
Basement Plan



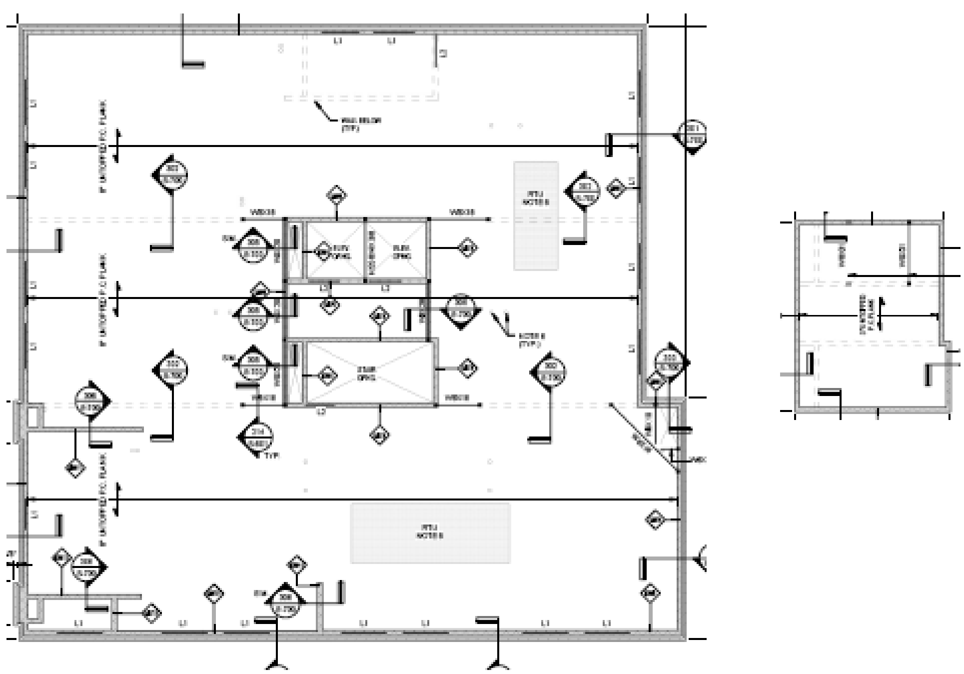
First Floor Framing Plan



Second Floor Framing Plan



Third thru Tenth Floor Framing Plan



Roof/Penthouse Roof Plan

APPENDIX B

Existing Floor System:

Hollow-core precast concrete plank system on load bearing masonry

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Hollow-Core Precast Concrete Plank Floor

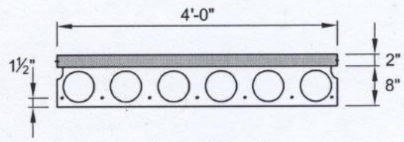
Strand Pattern Designation
76-S
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped		Topped	
A =	215	in.^{2}	311	in.^{2}
I =	1,666	in.^{4}	3,071	in.^{4}
y_b =	4.00	in.	5.29	in.
y_t =	4.00	in.	4.71	in.
S_b =	417	in.^{3}	581	in.^{3}
S_t =	417	in.^{3}	652	in.^{3}
wt =	224	plf	324	plf
DL =	56	psf	81	psf
V/S =	1.92	in.		

4HC8

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.4						
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

4HC8 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

2–32

PCI Design Handbook/Sixth Edition
First Printing/CD-ROM Edition

HOLLOW-CORE PLANKS ON LOAD BEARING MASONRY 1/2

- Loads
  - LL = 40 psf (hotel rms)
  - SDL = 25 psf (MEP, Part, fin)
  - DL = 10 psf (PCI handbook → untopped)

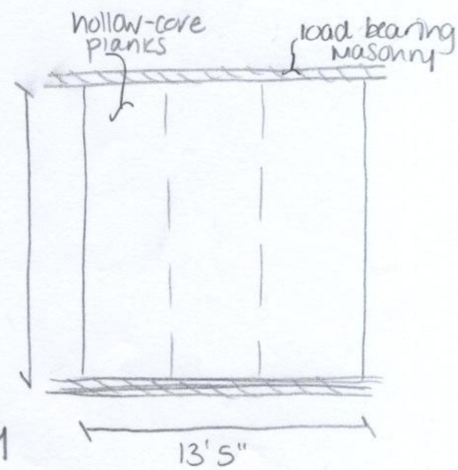
Total Load = 40 + 25 + 10 = 75 psf

$f'_c = 5000$ psi

$f'_{ci} = 3{,}500$ psi

$f_{pu} = 270{,}000$ psi

Span = 26'-0"



- designed using 8" untopped
  - 4'-0" x 8" NWC UNTOPPED (4HC8)
- from table in PCI design handbook:
  - 76-S carrying 95 psf capacity @ 26'-0" span
  - 0.3" camber @ erection
  - 0.3" camber (long term)
  - 7 strands @ 6/16" ϕ – STRAIGHT
  - Self wt. slab = 56 psf
- Load to masonry bearing walls

$W_u = 1.2(25+56) + 1.6(40)$

$W_u = 161.2$ psf

$$M_u = \frac{161.2\text{ psf}(13.5')(26')^2}{8} = 183.9^{k} \rightarrow 184^{k}$$

A_{ps} = 7 strands @ 6/16" ϕ

$= 7(0.375) = 2.625\text{ in}^2$

$f_{ps} = 270$ ksi

$b = 4 \times 12 = 48$ in

$d_p = 8" - 1\tfrac{1}{2}"\text{ clr} = 6\tfrac{1}{2}"$

HOLLOW-CORE PLANKS (cont). 2/2

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{2.625\,in^2 (270\,ksi)}{0.85(5\,ksi)(48\,in)} = 3.474\,in$$

$$\phi M_n = \phi \left[A_{ps} f_{ps} \left(d_p - a/2 \right) \right]$$
$$= \phi \left[2.625(270)\left(6.5 - \frac{3.475}{2}\right) \right]$$
$$= 0.9(3375.42)$$
$$\phi M_n = 3038\ in\text{-}K = 253\ 'K$$

$$\phi M_n = 253\ 'K > 184\ 'K = M_u$$ ✓ OKAY DESIGN

- DEFLECTION

$$E_c = 57000\sqrt{f'_c} = 57000\sqrt{5000}$$
$$E_c = 4030\ ksi$$
$$I = 1666\ in^4 \rightarrow \text{untopped}$$

$$\Delta_{LL} = l/360 = \frac{26 \times 12}{360} = 0.867''$$

$$\Delta_{LL} = \frac{5(40)(13.5)(26)^4}{384(4030000)(1666)} \times 1728 = 0.827'' < 0.867''$$ ✓ OKAY

$$\Delta_{TL} = l/240 = \frac{26 \times 12}{360} = 1.3''$$

$$\Delta_{TL} = \frac{5(40+25+56)(13.5)(26)^4 \times 1728}{384(4030000)(1666)} = 1.15'' < 1.3''$$ ✓ OKAY

- EXISTING DESIGN EFFICIENT IN CARRYING LOADS

APPENDIX C

Alternative Floor System #1:

Hollow-core precast concrete plank system on steel framing

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Hollow-Core Precast Concrete Planks

Strand Pattern Designation
76-S

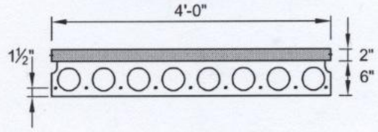
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

	Untopped		Topped	
A =	187	in.^{2}	283	in.^{2}
I =	763	in.^{4}	1,640	in.^{4}
y_b =	3.00	in.	4.14	in.
y_t =	3.00	in.	3.86	in.
S_b =	254	in.^{3}	396	in.^{3}
S_t =	254	in.^{3}	425	in.^{3}
wt =	195	plf	295	plf
DL =	49	psf	74	psf
V/S =	1.73	in.		

4HC6

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

Strand Designation Code	Span, ft 10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.3	−0.5	−0.7	−0.9	−1.2	−1.5	−1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	−0.1	−0.3	−0.4	−0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.7	−0.9	−1.2	−1.6	−2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	−0.1	−0.3	−0.6	−0.9	−1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	−0.3	−0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	−0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft 12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	−0.0	−0.1	−0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2	−1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	−0.1	−0.3	−0.5	−0.7	−1.0	−1.4	−1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	−0.1	−0.3	−0.5	−0.8	−1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	−0.2	−0.5	−0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$ *; see pages 2–7 through 2–10 for explanation.*

PCI Design Handbook/Sixth Edition
First Printing/CD-ROM Edition

2–31

HOLLOW CORE PRECAST ON STEEL

1/1

LOAD = LL = 40 psf
SDL = 25 psf
DL = 15 psf (PCI handbook) → topped member pg. 71

Total Load = 15+25+40 = 80 psf [superimposed service]

$f'_c = 5000$ psi
$f'_{ci} = 3500$ psi
$f_{pu} = 270{,}000$ psi
Span = 26'-0"

4'-0" x 6" NWC w/ 2" NW TOPPING
[keeps with the 8" precast currently existing]

From table on next page:
96-S carrying 82 psf
0.3" camber @ errection
-0.5" camber - long term

9 strands @ 6/16" ϕ STRAIGHT

Self wt. = 74 psf

GIRDERS

Load = 1.2(25+74) + 1.6(40) = 183 psf

$$M_u = \frac{(183\text{ psf})(16.5)(26)^2}{8} = 247.4\ ^{\prime k}$$

use W18x35 (AISC table 3-2)

$\phi M_n = 249\ ^{\prime k} > 247\ ^{\prime k}$ ✓OKAY

$$\Delta_{LL} = \frac{l}{360} = \frac{16.5(12)}{360} = 0.53"$$

$$0.53 = \frac{5(40)(26)(16)^4 \times 1728}{384(29000) I_x (1000)}$$

$I_x = \cancel{99.8\text{ in}^4}$ 510 in⁴ ✓OKAY
W18x35

$$\boxed{\begin{array}{l}\text{4'0" x 6" NWC w/ 2" TOPPING} \\ \text{4HC6-2 87-S on W18x35}\end{array}}$$

∘ using W10x12 beams parallel to panels to add stability

$$\Delta_{TL} = \frac{5(40+25+74)(26)(16)^4 \times 1728}{384(29000)(1000)(510)} = 0.360" < \frac{l}{240} = 0.675" \text{ ✓OKAY}$$

APPENDIX D

Alternative Floor System #2:

Non-composite Steel Framing

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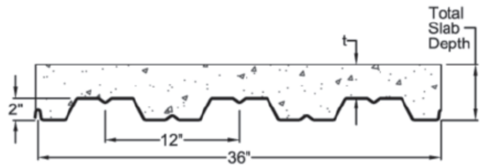
VULCRAFT

NON-COMPOSITE

2 C CONFORM



Interlocking side lap is not drawn to show actual detail.

MAXIMUM CONSTRUCTION CLEAR SPANS (S.D.I. CRITERIA)

Total Slab Depth	DECK	WEIGHT PSF	NW CONCRETE N=9 145 PCF			WEIGHT PSF	LW CONCRETE N=14 110 PCF		
			1 SPAN	2 SPAN	3 SPAN		1 SPAN	2 SPAN	3 SPAN
4.5 (t=2.50)	2C22	44	6-11	9-0	9-4	34	7-8	9-10	10-2
	2C20	45	8-2	10-3	10-7	34	9-0	11-3	11-7
	2C18	45	10-2	12-4	12-4	35	11-2	13-1	13-1
	2C16	46	10-5	12-6	12-11	36	11-7	13-8	13-10
5 (t=3.00)	2C22	50	6-7	8-7	8-11	39	7-4	9-5	9-9
	2C20	51	7-9	9-10	10-2	39	8-7	10-9	11-2
	2C18	51	9-7	11-10	11-11	40	10-9	12-9	12-9
	2C16	52	9-11	12-0	12-4	40	11-0	13-1	13-5
5.5 (t=3.50)	2C22	56	6-4	8-0	8-6	43	7-0	9-1	9-5
	2C20	57	7-5	9-5	9-9	43	8-3	10-4	10-9
	2C18	57	9-2	11-4	11-7	44	10-3	12-5	12-5
	2C16	58	9-5	11-6	11-10	45	10-6	12-7	13-0
6 (t=4.00)	2C22	62	6-1	7-5	8-2	48	6-9	8-9	9-1
	2C20	63	7-1	9-1	9-4	48	7-11	10-0	10-4
	2C18	63	8-10	10-11	11-3	49	9-10	12-0	12-1
	2C16	64	9-1	11-1	11-5	49	10-1	12-2	12-7
6.5 (t=4.50)	2C22	68	5-11	6-11	7-11	52	6-6	8-6	8-9
	2C20	69	6-11	8-9	9-0	53	7-7	9-8	10-0
	2C18	69	8-7	10-6	10-11	53	9-6	11-8	11-10
	2C16	70	8-10	10-8	11-0	54	9-9	11-10	12-2
7 (t=5.00)	2C22	74	5-10	6-6	7-5	57	6-4	8-0	8-6
	2C20	75	6-9	8-6	8-9	57	7-4	9-5	9-8
	2C18	75	8-4	10-2	10-6	58	9-2	11-4	11-7
	2C16	76	8-7	10-4	10-8	59	9-5	11-5	11-10

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

Slab Depth	REINFORCEMENT		Superimposed Uniform Load (psf) – 3 Span Condition Clear Span (ft.-in.)										
	W.W.F.	As	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0	9-6	10-0
4.5 (t=2.50)	6X6-W2.1XW2.1	0.042*	84	69									
	6X6-W2.9XW2.9	0.058	114	94									
	4X4-W2.9XW2.9	0.087	167	138									
5 (t=3.00)	6X6-W2.1XW2.1	0.042*	**153**	**127**	**107**	**91**	**78**						
	6X6-W2.9XW2.9	0.058*	**206**	**170**	**143**	**122**	**105**						
	4X4-W2.9XW2.9	0.087	**305**	**252**	**212**	**180**	**155**						
5.5 (t=3.50)	6X6-W2.9XW2.9	0.058*	**255**	**211**	**177**	**151**	**130**	**113**	**100**				
	4X4-W2.9XW2.9	0.087	**378**	**313**	**263**	**224**	**193**	**168**	**148**				
	4X4-W4.0XW4.0	0.120	**400**	**400**	**351**	**299**	**258**	**224**	**197**				
6 (t=4.00)	6X6-W2.9XW2.9	0.058*	**304**	**251**	**211**	**180**	**155**	**135**	**119**	**105**	**94**		
	4X4-W2.9XW2.9	0.087	**400**	**374**	**314**	**267**	**231**	**201**	**177**	**156**	**140**		
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**359**	**309**	**270**	**237**	**210**	**187**		
6.5 (t=4.50)	6X6-W2.9XW2.9	0.058*	**353**	**292**	**245**	**209**	**180**	**157**	**138**	**122**	**109**	**98**	**88**
	4X4-W2.9XW2.9	0.087*	**400**	**400**	**365**	**311**	**268**	**234**	**205**	**182**	**162**	**146**	**131**
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**400**	**361**	**315**	**277**	**245**	**219**	**196**	**177**
7 (t=5.00)	4X4-W2.9XW2.9	0.087*	**400**	**400**	**400**	**355**	**306**	**266**	**234**	**207**	**185**	**166**	**150**
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**400**	**400**	**360**	**316**	**280**	**250**	**224**	**202**
	4X4-W5.0XW5.0	0.150	**400**	**400**	**400**	**400**	**400**	**400**	**389**	**344**	**307**	**276**	**249**

NOTES:
1. * As does not meet A.C.I. criterion for temperature and shrinkage.
2. Recommended conform types are based upon S.D.I. criteria and normal weight concrete.
3. Superimposed loads are based upon three span conditions and A.C.I. moment coefficients.
4. Load values for single span and double spans are to be reduced.
5. Vulcraft's painted or galvanized form deck can be considered as permanent support in most building applications. See page 23. If uncoated form deck is used, deduct the weight of the slab from the allowable superimposed uniform loads.
6. Superimposed load values shown in bold type require that mesh be draped. See page 23.

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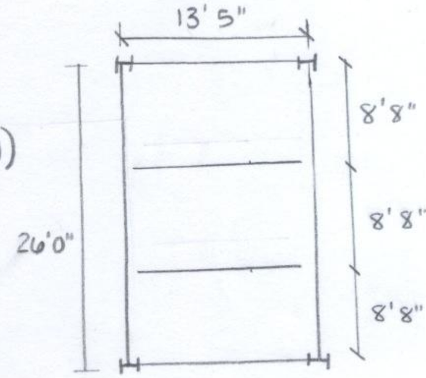
∘ Non-Composite Steel Calculations

layout of floor system designed:



LOADS: live load = 40 psf (hotel rms)
Superimposed = 25 psf (part, MEP, finish)
dead load = 45 psf (selfweight)

* DL found using vulcraft for steel decks
slab depth = 4.5"
topping = 2.5"
NWC. N = 9 145 pcf
3 span = 12'4"
use: 2C18 Deck (2C conform)

$f'_c = 3000$ psi
$f_{y\,rein} = 60{,}000$ psi

Total Load = LL + SDL + DL = 110 psf

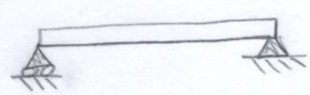
DECK: 2C18 Deck, 3 span (refer to vulcraft page for all design numbers used)
clear span used: 9'0"
18 gauge

3 span: $f_b = 30{,}000$ load = 151 psf > 110 psf ✓ OKAY
defl $\ell/240$ load = 95 psf > 40 psf (LL) ✓ OKAY
defl $\ell/360$ load = 126 psf > 45 psf (wt. conc.) ✓ OKAY

BEAMS:

Load = 1.2 D + 1.6 L
1.2 (25+45) + 1.6(40) = 148 psf → 0.148 ksf

Trib. Length = 8'8" = 8.66'



* ASSUME FULLY BRACED

$W_u = 8.66' \times 0.148\ \text{ksf} = 1.28\ \text{klf}$

$V_u = \dfrac{1.28\ \text{klf}\,(13.42)}{2} = 8.58^{k}$

$M_u = \dfrac{1.28\ \text{klf}\,(13.42)^2}{8} = 28.8^{\prime k}$

Non Composite Steel (cont) 2/3

From AISC Steel Manual: table 3-2:

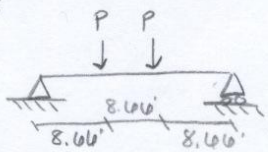
using W 8x10 $\phi M_n = 32.9 > 28.8'^K$ ✓OKAY

$$\Delta_{LL} = l/360 = \frac{(13.42)(12)}{360} = 0.447''$$

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(40psf)(8.66')(13.42)^4(1728)}{384(29000)(30.8)(1000)} = 0.283''$$

$0.447'' > 0.283''$ ✓OKAY

GIRDERS:



$P_u = 17.16$ for interior girder

$V_u = P = 17.16^K$

$M_{max} = Pa = 17.16(8.66') = 148.61'^K$

using W14x26 $\phi M_p = 151$ 'K (from AISC - table 3-2)

$$\Delta_{max} = 0.0857\frac{PL^3}{EI} \qquad I_x = 245 \text{ in}^4$$

$w_u = 40(8.66') = 346.4$ plf $\rightarrow$ 0.346 Klf

$$V_u = \frac{0.346(13.42')}{2} = 2.32^K$$

$$\Delta_u = l/360 = (26'0'')(12)/360 = 0.867''$$

$$\Delta_{max} = \frac{0.0857(2.32)(26')^3 \times 1728}{29000(245)} = 0.849''$$

$0.867'' > 0.849''$ ✓OKAY

Non Composite Steel (cont)

3/3

ALL BEAMS:

$$\Delta_{TL} = l/240 = (13.42 \times 12)/240 = 0.671''$$

$$W_{TL} = (25+45+40)(8.66) = 952.6 \text{ plf} \rightarrow 0.953 \text{ Klf}$$

$$\Delta_{TL} = \frac{5 W_{TL} L^4}{384 EI} = \frac{5(0.953)(13.42)^4(1728)}{384(29000) I_x} = 0.671''$$

$$I_x = 35.74 \text{ in}^4$$

using AISC table 3-3: all interior beams will be $\boxed{W10\times12}$

$I_x = 53.8 \text{ in}^4$

*due to inspection, will pass previous checks

ALL INTERIOR GIRDERS:

$$\Delta_{TL} = l/240 = (26 \times 12)/240 = 1.30''$$

$$W_{TL} = 110 \text{ psf}$$

$$V = \frac{110 \text{ psf } (8.66)(13.42')}{2 \times 1000} = 6.39\ \underset{\text{both sides}}{(2)} = 12.78^K$$

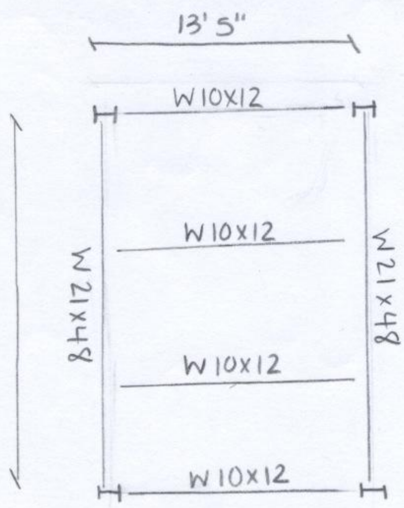
$$\Delta_{TL} = \frac{0.0857 PL^3}{EI} = \frac{0.0857(12.78)(26)^3 \times 1728}{29000 I_x} = 1.30''$$

$$I_x = 882.34 \text{ in}^4$$

using AISC table 3-3: all interior girders will be $\boxed{W21\times48}$

$I_x = 959 \text{ in}^4$

designed floor system:



APPENDIX E

Alternative Floor System #3:

Two-Way Post Tension Concrete Slab

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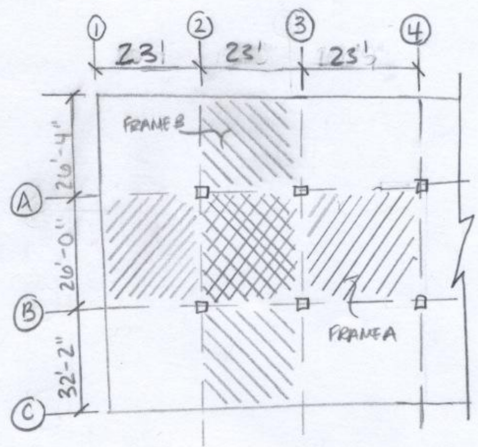
TWO-WAY POST TENSIONED CALCULATIONS

1/12

layout of floor system to design



• banded tendons along short direction (1, 2, 3, 4)

• uniform tendons parallel to long direction (A, B, C)

ASSUMPTIONS:

1. Simplified for ease of calculations all bays (23' x 26')
2. No live load reductions
3. Neglect shear wall effect & column stiffness
4. No pattern loading if $LL/DL < 3/4$ (ACI 13.7.6)
5. slab depth ratio for two-way slab 40-45
6. target balanced load 60-70% (DL)

USE DIRECT DESIGN METHOD

MATERIALS:

Concrete (NWC) = 150 pcf

$f'_c = 5000$ psi

$f'_{ci} = 3000$ psi

Rebar (steel) $f_y = 60{,}000$ psi

Post-Tensioning = unbonded tendons

½"ϕ, 7-WIRE STRANDS, $A_{PT} = 0.153 \text{ in}^2$

$f_{pu} = 270$ KSi

estimated prestress losses = 15 KSi (ACI 18.6)

$f_{se} = 0.7 f_{pu} - \text{losses}$ (ACI 18.5.1)

$= 0.7(270) - 15 = 174$ KSi

$P_{eff} = A_{PT} f_{se} = (0.153 \text{ in}^2)(174 \text{ KSi}) = 26.62 \text{ K/tendon}$

Two-Way Post Tensioned (cont)

2/12

Preliminary Slab thickness:

$\frac{\text{Span}}{\text{depth}} = L/45 = 26' \times 12 / 45 = 6.93''$ $\boxed{\text{use } 7'' \text{ slab}}$

if being conservative $L/40 = \frac{26 \times 12}{40} = 7.8'' = 8''$ Slab

Loading:

Dead Load (selfwt.) $= 7''/12'' \times 150\text{ pcf} = 87.5\text{ psf}$

Superimposed = 25 psf

Live Load (hotel rms) = 40 psf

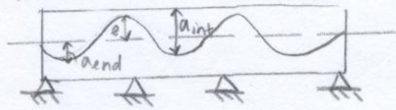
Design Parameters:

- prestressed 2-way slab system shall be designed as U class
  $f_t < 6\sqrt{f'_c}$ (ACI 18.3.3)
- at time of jacking (ACI 18.4.1)
  $f'_{ci} = 3000\text{ psi}$
  compression $= 0.6 f'_{ci} = 0.6(3000\text{ psi}) = 1800\text{ psi}$
  tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164\text{ psi}$
- at time of service loads (ACI 18.4.2 & 18.5.3)
  $f'_c = 5000\text{ psi}$
  compression $= 0.45 f'_c = 0.45(5000\text{ psi}) = 2250\text{ psi}$
  tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424\text{ psi}$
- according to ACI 18.12.4 $P/A > 125\text{ psi}$ (min)
  300 psi (max)
- target load balances
  60-70% of DL → will use avg. of 65%
  $0.65(87.5\text{ psf}) = 56.88\text{ psf}$
- to achieve 2 hr fire rating (assume carbonate aggregate)
  restrained slab (int) = 3/4" bottom
  unrestrained slab (ext) = 1½" bottom
  3/4" top

Two-Way Post Tensioned (cont)

3/12

Tendon Profile:



Tendon Ordinate	Tendon (CG) Location
Ext. support anch.	4"
Int. Support top	7"
Int. Span bottom	1"
end Span bottom	1.75"

*measured from bottom slab

$a_{INT} = 7" - 1" = 6"$

$a_{END} = \frac{(4" + 7")}{2} - 1.75" = 3.75"$

Frame A calculations:

$A = bh = (26 \times 12)(7) = 2184 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(26 \times 12)(7)^2}{6} = 2548 \text{ in}^3$

balanced load $W_b = 56.88 \text{ psf} (26') = 1478.9 \text{ plf} = 1.48 \text{ Klf}$

Force needed to counteract load in end bay:

$P = \frac{W_b L^2}{8 a_{end}} = \frac{1.48 \text{ Klf} (23')^2}{8(3.75/12")} = 313.14^K$

- # tendons to achieve $= \frac{313^K}{26.6^K/\text{tend.}} = 11.77 \rightarrow 12$ tendons
- actual force for bonded tendon

  $P_{act} = (12)(26.6) = 319.2^K$
- balanced load for end span

  $W_b = \left(\frac{319.2}{313.14}\right)(1.48) = 1.51 \text{ K/ft.}$
- actual pre compression stress

  $P_{act}/A = \frac{319.2(1000)}{2184} = 146.15 \text{ psi} > 125 \text{ psi}$ ✓OKAY!
- Check interior span

  $P = \frac{1.51 \text{ Klf} (23')^2}{8(6/12")} = 199.70^K < 319^K$ ✓ OKAY

  $W_b = \frac{(319)(8)(6/12)}{(23)^2} = 2.41 \text{ K/ft}$

Two-Way Post Tensioned (cont)

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$W_b / W_{DL} = 2.41 / 0.1125(26) = 0.824 < 1.0$ OKAY

effective prestress force $P_{eff} = 319^K$

- check slab stresses (moments from DDM)

$M_o = \frac{1}{8}(26')(23' - 24/12)^2 w =$

		DEAD	LIVE	$B_{\Delta L}$ (end)	$B_{\Delta L}$ (INT)
	W	112.5 psf	40 psf	$1510/26 = 58$ psf	$2410/26 = 92.7$ psf
	M_o	$161.24^{\prime K}$	$57.33^{\prime K}$	$83.13^{\prime K}$	$132.9^{\prime K}$
EXT	M^-_{INT}	$0.7(M_o) = 112.9^{\prime K}$	40.13	$58.2^{\prime K}$	
EXT	M^+	$0.5(M_o) - 80.62^{\prime K}$	28.67	$41.57^{\prime K}$	
EXT	M^-_{ext}	$0.3(M_o) = 48.37^{\prime K}$	17.2	$24.9^{\prime K}$	
INT	M^-	$0.65(M_o) = 104.81^{\prime K}$	37.26		$86.4^{\prime K}$
INT	M^+	$0.35(M_o) = 56.43^{\prime K}$	20.1		46.52

- Stress immediately after jacking (DL + $B_{\Delta L}$)

*COMPRESSION MUST BE $< 0.6f'_{ci} = 1800$ psi

*tension $< 3\sqrt{f'_c} = 164$ psi

- midspan · $f_{top} = (-M_{DL} + M_{bal})/S - P/A$

  $f_{bot} = (+M_{DL} - M_{bal})/S - P/A$

- interior $f_{top} = \dfrac{(-56.43 + 46.52)(12)(1000)}{2548} - 146 \text{ psi} = -192.7$ psi COMP* ✓

  $f_{bottom} = \dfrac{(56.43 - 46.52)(12)(1000)}{2548} - 146 \text{ psi} = -142$ psi COMP* ✓

- END $f_{top} = \dfrac{(-80.62 + 41.57)(12)(1000)}{2548} - 146 \text{ psi} = -330$ psi COMP* ✓

  $f_{bott} = \dfrac{(80.62 - 41.57)(12)(1000)}{2548} - 146 \text{ psi} = 37.91$ psi tension* ✓

- SUPPORT $f_{top} = (M_{DL} - M_{bal})/S - P/A$

  $f_{bott} = (M_{DL} + M_{bal})/S - P/A$

  $f_{top} = \dfrac{(112.9 - 58.2) \times 12 \times 1000}{2548} - 146 \text{ psi} = 111.6$ psi tension* ✓

  $f_{bott} = \dfrac{(-112.9 + 58.2) \times 12 \times 1000}{2548} - 146 \text{ psi} = -404$ psi COMP* ✓

Two-Way Post Tensioned (cont)

$*(T) < 424$ psi
$*(C) < 2250$ psi

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- Stress at service load (DL + LL + BAL)
  - Midspan $f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

    $f_{bot} = (M_{DL} + M_{LL} - M_{bal})/S - P/A$

    - INTERIOR $f_{top} = \dfrac{(-56.43 - 20.1 + 46.52)}{2548} - 146 = -287$ psi $(C)^*$ ✓

      $f_{bot} = \dfrac{(56.43 + 20.1 - 46.52)}{2548} - 146 = -4.67$ psi $(C)^*$ ✓

    - END $f_{top} = \dfrac{(-80.62 - 28.67 + 41.57)}{2548} - 146 = -463$ psi $(C)^*$ ✓

      $f_{bot} = \dfrac{(80.62 + 28.67 - 41.57)}{2548} - 146 = 173$ psi $(T)^*$ ✓

  - SUPPORT $f_{top} = (M_{DL} + M_{LL} - M_{bal})/S - P/A$

    $f_{bot} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

    $f_{top} = \dfrac{(112.9 + 40.13 - 58.2)}{2548} = 300$ psi $(T)^*$ ✓

    $f_{bot} = \dfrac{(-112.9 - 40.13 + 58.2)}{2548} = -593$ psi $(C)^*$ ✓

  [ALL STRESSES ARE WITHIN ALLOWABLE LIMITS]

- ULTIMATE STRENGTH
  - The primary post-tension moments, M_1, vary along length of line span

    $M_1 = P(e)$

    $e = 0"$ @ ext. support

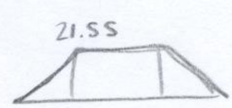
    $e = 3"$ @ int support

    $M_1 = \dfrac{319\,(3.0")}{12"} = 79.8$ 'K

  - the secondary post-tension moments, M_{sec}, vary linearly between supports

    $M_{sec} = M_{bal} - M_2$

    $M_{sec} = 58.2 - 79.8 = -21.55$ 'K



Two-Way Post Tensioned (cont) 6/12

- the typical load combo for ultimate strength design

$M_u = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{sec}$

@ midspan = $M_u = 1.2(80.62) + 1.6(28.67) + (-10.8) = 131.816^{\text{lk}}$

@ support = $M_u = 1.2(-48.37) + 1.6(-17.2) + (-21.55) = -107.2^{\text{lk}}$

- determine min. bonded reinf. to see if acceptable for ultimate design strength

Positive Moment region

Interior Span = $f_t = -4.67 < 2\sqrt{f'c} = 2\sqrt{5000} = 141$ psi

Exterior Span = $f_t = 173 > 2\sqrt{f'c} = 2\sqrt{5000} = 141$ psi

- Minimum positive reinforment required (ACI 18.9.3.2)

$y = f_t/(f_t+f_c)h = \left[\frac{173}{(173+463)}\right]7" = 1.90$ in

$N_c = \frac{M_{DL}+M_{LL}}{S}(0.5)(y)(l_2)$

$= \frac{(80.62+28.67)}{2548}(0.5)(1.90)(26' \times 12) = 152.6^{K}$

$A_{s,min} = N_c/0.5f_y = \frac{152.6^{k}}{0.5(60)} = 5.09 \text{ in}^2$

$= 5.09 \text{ in}^2/26' = 0.1956 \text{ in}^2/\text{ft}$

[USE #4 @ 12" OC bottom = 0.20 in²/ft]

Negative Moment region

$A_{s,min} = 0.00075A_{cf}$ (ACI 18.9.3.3)

- Interior Supports $A_{cf} = \max.\begin{Bmatrix}(7")(26)(12)\\(7")(23')(12)\end{Bmatrix} = 2184 \text{ in}^2$

$A_{s,min} = 0.00075(2184) = 1.64 \text{ in}^2$

[(9) #4 bars top (1.80 in²)]

- Exterior Supports $A_{s,min} = 0.00075(2184) = 1.64 \text{ in}^2$

[(9) #4 bars top (1.80 in²)]

- Max bar Spacing 10.5" (ACI 18.9.3.3)

↳ top bars w/in 1.5h away from face of support on each side $(1.5)(7) = 10.5"$

Two-Way Post Tensioned (cont) 7/12

- check minimum reinforcement for ultimate strength

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

d = effective depth

$A_{ps} = 0.153\ in^2\ (12\ \text{tendons}) = 1.836\ in^2$

$f_{ps} = f_{se} + 10{,}000 + \left(\frac{f'_c bd}{300(A_{ps})}\right)$

$= 174{,}000 + 10{,}000 + \left(\frac{5000(26)(12)d}{300(1.836)}\right) = 184{,}000 + 2832.2d$

$a = \frac{(A_s f_y + A_{ps} f_{ps})}{0.85 f'_c b}$

@ SUPPORTS

$d = 7" - \frac{3}{4}"\ \text{cc} - \frac{1}{2}(\frac{1}{2}") = 6"$

$f_{ps} = 184{,}000 + 2832.2(6) = 200993\ psi$

$a = \frac{(1.80)(60) + 1.64(201)}{0.85(5)(26')(12)} = 0.330$

$\phi M_n = 0.9\,(1.8(60) + 1.64(201))\left(6 - \frac{0.330}{2}\right)/12 = 192^{'k} > 107.2^{'k}$ ✓ OKAY

$\boxed{(9)\ \#4\ \text{top}\ (@\ \text{int-ext supports})}$

@ midspan (end)

$d = 7 - (1\frac{1}{2}) - \frac{1}{2}(\frac{1}{2}) = 5\frac{1}{4}"$

$f_{ps} = 198869\ psi$

$a = \frac{(5.09(60)) + 1.64(199)}{0.85(5)(26)(12)} = 0.476$

$\phi M_n = 0.9(5.09(60) + 1.64(199))\left(5\frac{1}{4} - \frac{0.476}{2}\right)/12 = 237.5^{'k} > 131.8^{'k}$ ✓ OKAY

$\boxed{\#4\ @\ 12"\ OC\ \text{bottom}\ @\ \text{end Spans}}$

Two-Way Post Tensioned (cont)

8/12

Frame B Calculations:

$A = bh = (23')(12)(7) = 1932\ in^2$

$S = \frac{bh^2}{6} = \frac{(23\times12)(7)^2}{6} = 2254\ in^2$

balanced load $W_b = 56.88(23') = 1308.24\ plf \rightarrow 1.31\ K/ft$

Force needed to counteract load in end bay:

- $P = \frac{W_b L^2}{8 a_{end}} = \frac{1.31(26')^2}{8(3.75/12)} = 354^K$
- \# tendons $= \frac{354^k}{26.6^k/tendon} = 13.31 \rightarrow 13$ tendons
- actual force for tendon

$P_{act} = (13)(26.6) = 345.8^K$

- balanced load for end span

$W_b = \left(\frac{345.8}{354}\right)(1.31) = 1.28\ K/ft$

- actual pre-comp. stress

$P_{act}/A = \frac{345.8(1000)}{1932} = 179\ psi > 125\ psi$ ✓OKAY

- check interior span

$P = \frac{1.28(26')^2}{8(6/12)} = 216^K < 346^K$ ✓ OKAY

$W_b = \frac{(346)(8)(6/12)}{26^2} = 2.05\ K/ft$

$W_b/W_{DL} = \frac{2.05}{(0.1125)(23)} = 0.792 < 1.0$ OKAY $\rightarrow P_{eff} = 346^K$

- check slab stresses (DDM Moments)

$M_o = \frac{1}{8}(23')(26' - 24/12)^2 W$

		DEAD	LIVE	B_{AL}(end)	B_{AL}(INT)
	W	112.5 psf	40 psf	55.65	89.13
	M_o	186.5	66.24	92.16	147.6
EXT	M^-_{INT} $0.7M_o$	130.6	46.4	64.51	
EXT	M^+ $0.5M_o$	93.25	33.12	46.1	
EXT	M^-_{EXT} $0.3M_o$	56	19.87	27.65	
INT	M^- $0.65M_o$	121.2	43.1		95.9
INT	M^+ $0.35M_o$	65.3	23.2		51.7

Two-Way Post Tensioned (cont) 9/12

- stresses immediately after jacking (DL+BBL)

MIDSPAN - $f_{top} = (-M_{DL} + M_{bal})/S - P/A$ *(C) < 1800 psi

$f_{bot} = (+M_{DL} - M_{bal})/S - P/A$ *(T) < 164 psi

INTERIOR $f_{top} = (-65.3 + 51.7)/2254 - 179\ psi = -251\ psi$ (C)* ✓

$f_{bot} = \frac{(65.3 - 51.7)}{2254} - 179\ psi = -106.6\ psi$ (C)* ✓

END $f_{top} = \frac{(-93.25 + 46.1)}{2254} - 179\ psi = -430\ psi$ (C)* ✓

$f_{bot} = \frac{(93.25 - 46.1)}{2254} - 179\ psi = 72.02$ (T)* ✓

SUPPORT $f_{top} = \frac{(130.6 - 64.51)}{2254} - 179\ psi = 173$ (T)* X

$f_{bott} = \frac{(-130.6 + 64.51)}{2254} - 179\ psi = -531$ (C)* ✓

- Stress at service load (DL+LL+BDL)

MIDSPAN - $f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$ *(C) < 2250 psi

$f_{bot} = (M_{DL} + M_{LL} - M_{bal})/S - P/A$ *(T) < 464 psi

INTERIOR $f_{top} = \frac{(-65.3 - 23.2 + 51.7)}{2254} - 179\ psi = -375$ (C)* ✓

$f_{bot} = \frac{(65.3 + 23.2 - 51.7)}{2254} - 179\ psi = 16.92$ (T)* ✓

END $f_{top} = \frac{(93.25 - 33.12 + 46.1)}{2254} - 179\ psi = -606$ (C)* ✓

$f_{bot} = \frac{(93.25 + 33.12 - 46.1)}{2254} - 179\ psi = -179$ (C)* ✓

SUPPORT $f_{top} = \frac{(+130.6 + 46.4 - 64.51)}{2254} - 179\ psi = 420$ (T)* ✓

$f_{bot} = \frac{(-130.6 - 46.4 + 64.51)}{2254} - 179\ psi = -778$ (C)* ✓

[ALL STRESSES ARE WITHIN ALLOWABLE LIMITS]

Two-Way Post Tension (cont) 10/12

- Primary Post-tension Moment, M_1

$M_1 = P(e)$ $e = 0"$ @ ext. support
$e = 3"$ @ int. support

$M_1 = \frac{346(3)}{12} = 86.5'k$

- Secondary post-tension moment, M_{sec}

$M_{sec} = M_{bal} - M_2$

$M_{sec} = 64.51 - 86.5 = -22\ 'k$

- typical load combo for ultimate design strength

$M_u = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$

@ MIDSPAN = $M_u = 1.2(93.25) + 1.6(33.12) + (-11) = 153.9\ 'k$

@ SUPPORT = $M_u = 1.2(-56) + 1.6(-19.87) + (-22) = -121\ 'k$

- determine min bonded reinf. to see if acceptable for ultimate strength

POSITIVE MOMENT REGION

Interior Span = $f_+ = 16.92 < 2\sqrt{f'c} = 141$ psi

Exterior Span = $f_+ = 179 > 2\sqrt{f'c} = 141$ psi

- Minimum positive reinforcement required

$y = f_+/(f_+ + f_c)\,h = \left[\frac{179}{179+666}\right] 7" = 1.592$ in

$N_c = \frac{M_{DL}+M_{LL}}{S}(0.5)\,y(l_2) = \frac{(93.25+33.12)}{2254}(0.5)(1.592)(23\times12) = 147.8^k$

$A_{s,min} = N_c/0.5f_y = \frac{147.8}{0.5(60)} = 4.93$ in² $\quad \frac{4.93\text{ in}^2}{23'} = 0.214$ in²/ft

[USE #4 @ 10" OC bottom = 0.24 in²/ft]

- Negative Moment region

$A_{s,min} = 0.00075\,A_{cf}$ (ACI 18.9.3.3)

- Interior supports $A_{cf} = \max\begin{cases} 7(26)(12) \\ 7(23)(12) \end{cases} = 2184$ in²

$A_{s,min} = 0.00075(2184) = 1.64$ in²

[(9) #4 bars top (1.80 in²)]

- Exterior Supports $A_{s,min} = 0.00075(2184) = 1.64$ in²

[(9) #4 bars top (1.80 in²)]

Two-Way Post Tensioned (cont) 11/12

- check minimum reinforcement for ultimate strength

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

d = effective length

$A_{ps} = 0.153 in^2 (13) = 1.989 in^2$

$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c bd}{300 A_{ps}} = 174{,}000 + 10{,}000 + \left(\frac{5000(23)(12)d}{300(1.989)}\right)$

$= 184{,}000 + 2313d$

$a = \frac{(A_s f_y + A_{ps} f_{ps})}{0.85 f'_c b}$

@ SUPPORTS

$d = 7'' - \frac{3}{4}c - \frac{1}{2}(\frac{1}{2}'') = 6''$

$f_{ps} = 184{,}000 + 2313(6) = 197876$ psi

$a = \frac{(1.80)(60) + 1.64(198)}{0.85(5)(23')12} = 0.369$

$\phi M_n = 0.9(1.8(60) + 1.64(198))(6 - \frac{0.369}{2}) = 188.74^{'k} > 121^{'k}$ ✓

@ MIDSPAN (end)

$d = 7'' - (1\frac{1}{2}'') - \frac{1}{2}(\frac{1}{2}'') = 5\frac{1}{4}''$

$f_{ps} = 196143$ psi

$a = \frac{4.93(60) + 1.64(196)}{0.85(5)(23)(12)} = 0.526$

$\phi M_n = 0.9((4.93)(60) + 1.64(196))(5\frac{1}{4} - \frac{0.526}{2}) = 230.86^{'k} > 154^{'k}$

#4 @ 10" OC bottom @ end spans
(9) #4 top (@ int-ext supports

Two-Way Post Tensioned (cont) 12/12

Shear:

(ACI 11.11.2.2) @ columns of two way prestressed slabs & footing ~~that~~

$$V_c = (\beta_P \lambda \sqrt{f'c} + 0.3 f_{pc}) b_o d + V_p$$

w/ β_P = Smaller of 3.5

$$\left(\frac{\alpha_s d}{b_o} + 1.5\right) = \left(\frac{40(4)}{138} + 1.5\right) = 3.24 < 3.5$$

$\boxed{\text{USE } 3.24}$

$b_o = 138''$

$\beta_P = 3.24$

$d = 7'' - \frac{3}{4} cc - \frac{1}{2}(\frac{1}{2}'') = 6''$

$f_{pc} = \frac{(179 + 146)}{2} = 162.5 \text{ psi}$

$V_P = 0$ to be conservative

$$V_c = (3.24(1.0)\sqrt{5000} + 0.3(162.5))(138)(6) + 0)$$

$V_c = 230.1^k$

$\phi V_c = 0.75(230.1) = 172.54^k$

$V_u = q_u A$

$q_u = 1.2 D_L + 1.6 LL = 1.2(87.5 + 25) + 1.6(40) = 199 \text{ psf} \rightarrow 0.199 \text{ ksf}$

$A_{TRIB} = (23')(26') - \left(\frac{24}{12}\right)^2 = 594 \text{ ft}^2$

$V_u = 0.199(594) = 118.2^k$

$\phi V_c = 172.5^k > 118.2^k = V_u$ ✓OKAY

[NO ADDITIONAL REINFORCEMENT NEEDED]

- Deflection should not be an issue, especially with the help of balanced loads from the post-tension tendons