40 Bond

New York, NY

Technical Report 2

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Structural Option

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October 24, 2008

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Executive Summary

The pro-con structural study of alternate floor systems discusses the existing structural floor framing of 40 Bond and three alternate framing systems. Each option is examined using typical bay analysis of a 25'x20' bay. The existing structure is a 9" thick two-way flat plate slab with #4@12 top and bottom and additional reinforcement at the supports due to increased moments at these locations. Using ACI 318-08, the minimum slab thickness was determined and is less than the designed thickness of 9". The Direct Design Method was then used to design the reinforcement in the column strips and middle strips in each direction. Both methods of shear, wide beam action and punching shear, were checked and satisfied, as was the deflection.

The three alternate systems that were analyzed included:

- Non-composite steel framing
- Hollow core precast concrete on steel beams
- Two-way post-tensioned slabs

The non-composite steel framing was designed using the AISC *Steel Construction Manual* and *Vulcraft Steel Roof and Floor Deck Guide.* The preliminary design was composed of 2C20 metal deck with 4.5" slab, W12x19 beams and W18x35 girders. The 4'-0"x6" hollow core precast panels with 2" topping were selected from the PCI *Design Handbook* and the supporting girders were determined to be W18x35 when optimized. The two-way post-tensioned slab was designed to be 8" thick with 12 tendons distributed uniformly over the long span direction and 12 tendons banded at the columns along the short span. Due to the small amount of tendons over 25' and 20' spans it does suggest that the thickness of 8" is conservative and with further analysis may be determined to work at a thinner dimension with an increased number of tendons. A minimum amount of mild steel was designed as well for this slab.

The advantages and disadvantages were discussed for each framing system, and it was determined that the steel framing in both the non-composite and hollow core systems were not feasible. They increased the floor depth to 22.2" and 27.5" which did not even compare to the 9" two-way flat plate and the 8" two-way post-tensioned slab. Overall, the post-tensioned slab was the best choice for further investigation as a possible framing system for 40 Bond. Not only is the slab depth minimal, allowing for greater floor-to-ceiling heights, but the post-tensioning allows for longer spans, can carry a greater load, reduces vibration, limits deflection and requires no additional fireproofing. There are concerns in regards to the construction of the system, however, because it does require trained contractors and special safety procedures to ensure it is done correctly. With this is mind, the two-way post-tensioned slab seems like a feasible option for the structural proposal required by Senior Thesis.

Introduction

The pro-con structural study of alternate floor systems examines the existing floor framing of 40 Bond that was designed by DeSimone Consulting Engineers (DCE) and analyzes three other possible systems. The existing design is a two-way flat plate slab and the alternates that were studied include non-composite steel framing, hollow core precast concrete panels on steel beams, and a two-way post-tensioned slab. Gravity loads determined in Technical Report 1 were used in the design to help determine slab thicknesses, member sizes and necessary reinforcement. Aside from the actual composition of each system, the report aims to compare and contrast the advantages and disadvantages related to constructability, system weight, system depth, fire protection and various other criteria to determine which systems may be possible topics for the structural proposal required by Senior Thesis.

Some information pertinent to understand 40 Bond is that the building is located on a 13,600 ft^2 parcel of land located on Bond Street between Lafayette and Bowery Street in New York City. The footprint of the building is 64'-8" by 134'-4" and the building has an overall building height of 152'-0" from cellar to the top of the penthouse structure. There is a 20'-0" setback at the seventh floor with a roof terrace that occupies this space. Typical spans range from 19'-6" x 25'-0" to 23'-2 ½" x 25'-0" and floor-to-ceiling heights range from 11'-10" to 14'-0". A total of 23 condominium units and 5 townhouses are contained within this building and the plans vary as the type and number of units change throughout. In addition to the building there is also a 140'-0" long, 22'-0" high cast aluminum gate located along Bond Street that was designed to withstand the lateral forces that are present at this site.

Architectural Design Concepts

40 Bond Street was designed by the Swiss firm Herzog & de Meuron with New York based Handel Architects. The idea behind this luxury residential building was to reinvent the cast iron building typology that is prevalent in this lower Manhattan neighborhood. The building consists of one cellar that houses a fitness center, storage space and equipment rooms. The first and second floors are devoted to five through-building, 2-level townhouses. The layout then changes to accommodate four condominium units on each level from the third to the sixth floor. Once again, at the seventh floor the plans change incorporating a 20'-0" setback and reduced number of condominium units including only two per floor from levels 7 to 9. The tenth floor is a full plan condominium with a penthouse structure that rises 20'-0" above the main roof. It is in the penthouse that a direct relation can be made between architectural concepts and structure. A 44'-0" clear span is achieved with two hidden columns and the core shear wall as supports leaving nearly three completely glass walls.


Figure 1 – South Facade

The south face also enforced some strict tolerances in regard to structure. Operable floor-to-ceiling windows are held in place with green glass mullions (Figure 1). This entirely glass façade limits the variation in columns to less than ½". The north façade contains the same windows but the glass mullions are exchanged with pre-patina copper. These mullions then serve as a grid for the perimeter columns along the north and south faces. Small 10"x10" concrete columns are located behind these mullions and space at 6'-3" on center between the second and tenth floors. The variation in layout, fluctuating column dimensions, and necessary setbacks resulted in different transfer locations that required beams to redirect the loads.

With many buildings located in cities such as New York, there is always an awareness of retail value. The more space there is to offer the more expensive the unit may be. The flat plate concrete system allows for tall floor-to-ceiling heights that remain unobstructed because of the limited number of beams and girders dropping into the space. In order to preserve the architectural design, maximize area and create appealing spaces, the concrete structure deviates from what is typical in the design and construction of a residential building to create an aesthetically pleasing and interesting structure. As a result of such specialization, however, this 90,000 sf building had a very high cost in comparison to its size which is attributed to such things as formwork required for transfer beams and many slender columns.

Structural System

Foundation

The geotechnical engineering study was performed by Langan Engineering & Environmental Services on September 10, 2004. In this study it was found that the water level was approximately 42.8' below the existing ground surface. The cellar extends 12'-8" below grade and therefore there was not a concern in regard to increased uplift pressures at this level. Langan noted that the bearing materials were suitable for a shallow foundation and that the recommended allowable bearing pressure would be 5 kips/ft^2. As a result, a 30" reinforced concrete mat foundation was designed with bearing walls and buttresses supported by a strip footing.

The 30" slab is 5 ksi normal weight concrete (NWC) and increases to a thickness of 48" and 84" within the core shear walls where the elevator pit is located. Reinforcement varies throughout this mat slab. Buttresses ranging in size from 14"x29 ½" to 18"x79" are located around the perimeter. Interior columns ranging in size from 12"x22" to 28"x28" have an increased strength of 8 ksi. Located at columns 3B, 3C and 3F (Figure 2), there are also foundation mat shearheads to resist punching shear due to high loads that continue from the roof down to the foundation.

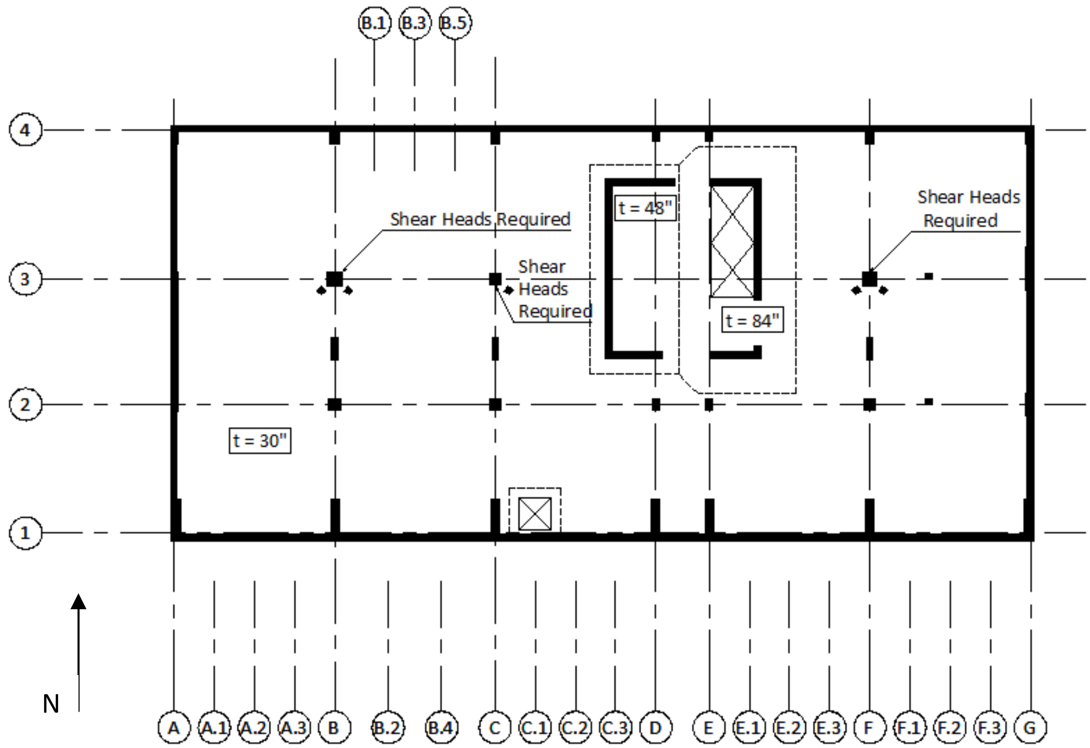

Figure 2 – Foundation Plan with Typical Column Grid and Shearhead Locations Noted

Superstructure

The ground floor is a 9" two-way flat plate slab (NWC) with a compressive strength (f'_c) of 5.95 ksi and typical reinforcement of #4@12 top and bottom with various sizes and spacing of bars at column locations. Located at the south face is a slab step that transitions to a 12" slab for the townhouse entrances. Typical to the floors above, there are also 3" slab depressions at the fireplaces and toilet areas and 14" slabs within the core. Perimeter columns ranging in size from 10"x24" to 16"x58" are located on the north, south and east walls while a 12" thick shear wall runs along the west face. The interior columns dimensions are then 12"x22", 22"x22" and 28"x28". All of the columns from the foundation to those supporting the fourth floor have a concrete strength of 8 ksi. There are beams located around the stair openings in the townhouses and collector beams that tie together the core shear walls which are typical on all floors.

The second and third floors have the same two-way flat plate slab as noted above minus the slab step. Particular to the second floor is the introduction of the 10”x10” concrete columns spaced at 6’-3” on center along the north wall that extend up the remaining height of the building. Because these closely spaced columns need to transition to fewer columns below, 14”x40” transfer beams ($f'_c = 10$ ksi, typical to all transfer beams) run the full length of this wall. The beams around the townhouse stair openings are also present on the second floor. The third floor then has the introduction of the 10”x10” columns spaced at 6’-3” on center along the south face. The transfer beams at this level are 60”x16” and extend the full length of this wall. These columns continue to the seventh floor where they step back 20’-0” due the setback at that level. This thin, wide transfer was implemented to limit the intrusion into the space below. Also, all the 10”x10” columns only have a 7” slab encroachment that has a 1” slab depression around each column (Figure 3).

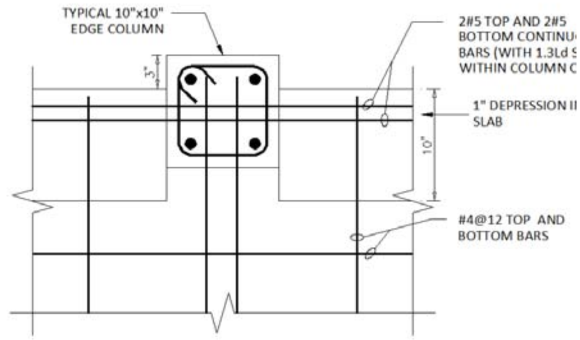

Figure 3 – Typical Perimeter Column Detail

All floors between level 4 to the penthouse level use a 9” two-way flat plate slab with #4@12 top and bottom plus various reinforcement at columns and a reduced compressive strength of $f'_c = 5$ ksi. Similar slab depressions and increased slab thickness at the core are present. The columns supporting the fifth floor and above also have a reduced compressive strength of $f'_c = 5$ ksi. The columns along the north and south façade remain 10”x10” while those located on the east and west walls and within the interior vary between 12”x22” to 28”x28”. There is also the introduction of 22” diameter (Ø) circular columns that are used on some floors dependent on the tenant’s request in their condominium. In addition to the beams within the shear wall core, there are also spandrel beams along the east and west faces.

At the fourth floor a transfer beam is present along the east wall (Figure 4). This 12”x50” transfer was designed after construction began due to the presence of an adjacent chimney encroachment on site. Then at the seventh floor the setback takes place. It is here that loads increase due to the roof terrace provided by this setback. A 20”x24” transfer beam along line 2 is needed due to the introduction of the 10”x10”

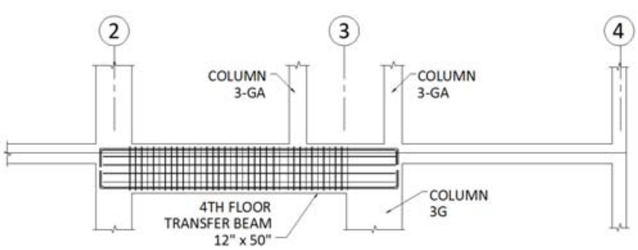

Figure 4 – Transfer Beam at Fourth Floor

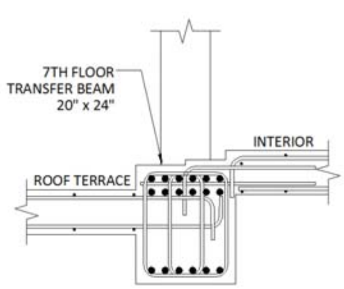

Figure 5 – Transfer Beam at Seventh Floor

columns along this line (Figure 5).

The penthouse level and its roof are a great example of what can be achieved when using concrete. The dimensions of the penthouse are 23'-4" x 44'-6" and it has a thickened 19" slab with #4@12 top bar reinforcement and #5@8 bottom bar reinforcement. A 44'-0" clear span is achieved with the support of the concrete shear walls to the east and two 28"x16" columns to the west. The loads from the two columns need to be transferred and a 32"x24" beam is used to direct these loads to nearby columns, one of which is only 10"x14". The roof above this long span structure is a combination of upturned beams, inclined piers, and two separate 8" slabs with #5@12 top and bottom spanning between its two supports (Figure 6). Located on the other side of the core is an enclosed elevated mechanical room. Additional loads due to the equipment and its surrounding 8" CMU walls will be applied at this level.

Figure 6 –Penthouse Roof Structure

Lateral System

The lateral system is a combination of 12" ordinary reinforced concrete shear walls (Figure 7). Within the core shear walls there are the stair, elevator and mechanical shafts. The typical horizontal reinforcement in these walls is #4@12 while the vertical reinforcement ranges from #4@12 to #8@6 depending on the level they are located on and which portion of the shear wall is being examined. The west shear wall is reinforced with #4@12 as the horizontal reinforcement and a range of vertical reinforcement from #4@12 to #7@12. All shear walls supporting the ground floor to those supporting the fourth floor have concrete with $f'_c = 8$ ksi while those supporting the rest of the building have an $f'_c = 5$ ksi.

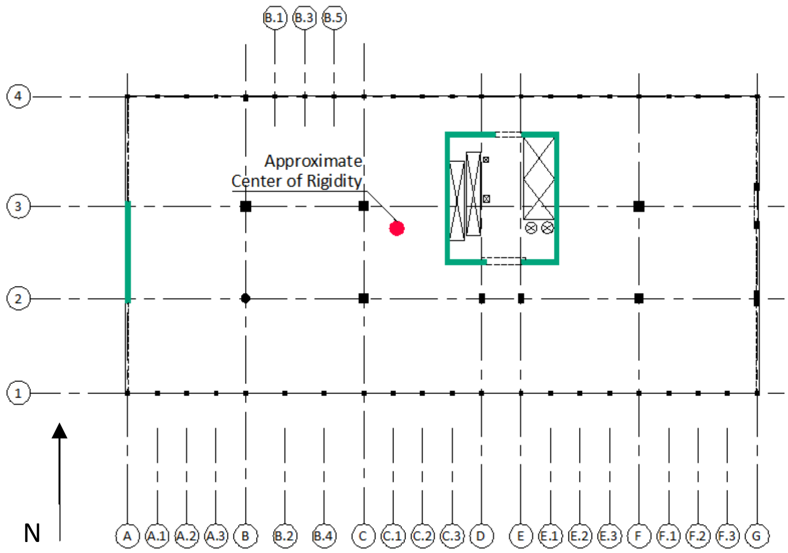

Figure 7 – Typical Plan with Lateral System Highlighted

The presence of the west shear wall allows for the center of rigidity to move closer towards the middle of the plan. Because the core shear walls are not centralized within the building they draw the rigidity to the east. When the center of rigidity is not in line with the resultant lateral force there is eccentricity and moments due to torsion become a factor. These wind and seismic loads travel through the rigid diaphragm (flat plate slab) to the shear walls and then down into the foundation. This load path is governed by the concept of relative stiffness.

Loads

Gravity Loads

The determination of gravity loads by DCE was done using the New York City Building Code (NYCBC 2003), while American Society of Civil Engineers (ASCE) 7-05 was the main reference for this report. A different standard was used to comply with the requirements of AE Senior Thesis; ASCE 7-05 was the logical reference. Another note is that DCE chooses not to use live load reductions in their design. In order to keep the loading consistent, the reductions will be not be factored into the live loads determined by code. The loads that were determined from each reference as well as the design loads are noted in Table 1.

Table 1 - Gravity Loads

Description	NYCBC (2003)	ASCE 7-05	DCE Value	Design Value
DEAD (DL)				
Concrete	150 pcf	150 pcf	150 pcf	150 pcf
LIVE (LL)				
Condominiums & Townhouses	40 psf	40 psf	40 psf	40 psf
Corridor (first floor, main lobby)	100 psf	100 psf	100 psf	100 psf
Corridor (serving independent units)	40 psf	40 psf	40 psf	40 psf
*Exterior Balconies	60 psf	100 psf	60 psf	100 psf
SUPERIMPOSED (SDL)				
Finishes, MEP, Partitions	20-25 psf	20-25 psf	20 psf	25 psf
**Concrete Pavers	40 psf	40 psf	40 psf	40 psf
SNOW (S)				
***Snow	30 psf	21 psf	30 psf	30 psf

* In NYCBC, exterior balcony LL is 150% of adjacent areas. Therefore (40psf)x(1.5)=60psf.

** Superimposed load on 7th Floor and Penthouse terraces will be replaced as 40 psf over area.

*** Snow load calculations are located in appendix. Due to greater live load required on roof terraces, the roof live load on these areas will be 100 psf.

Floor Systems

Two-Way Reinforced Flat Plate - Existing

Material Properties			**Loading:**	
Concrete:	9" slab (NWC)		Dead (self weight):	112.5 psf
	22"x22" columns		Live:	40 psf
	f'_c =	5000 psi	Superimposed:	25 psf
Reinforcement:	f_y =	60,000 psi		

Description

This two-way reinforced flat plate system designed by DCE includes a 9" NWC slab that contains #4@12 top and bottom. Additional reinforcement is placed at supports, to resist increased moments at these locations and range from #4 to #6 bars at varying spacing, depending on the magnitude of the moments.

A typical interior bay analysis, done at the 6th floor, was completed using the Direct Design Method reviewed in *Design of Concrete Structures* by Nilson, Darwin, and Dolan with the loads determined by ASCE 7-05. The bay was split into two frames, Frame A and Frame B noted in Figure 8, which were checked for minimum slab thickness and reinforcement design. The slab thickness of 9" exceeded the minimum of 8.42" and the reinforcement was found to be the same as that designed by DCE, or resulted in a fewer number of bars, due to the absence of lateral loads in this analysis. The typical bay was taken as 25'x20' to simplify calculations rather than looking at the 25'x20' bay, 25'x19.5' bay and 25'x23.25' bay separately. There are calculations reviewing the wide beam action (one-way) and punching shear (two-way) within the slab which did not prove to be an issue and did not require any additional shear reinforcement. Deflections were also computed and found to be within the limits of l/480 for long-term deflection. All supporting calculations for this analysis can be found in Appendix A.

Advantages

This particular floor system was a likely choice for 40 Bond. In New York City, midrise residential buildings are most often concrete structures and the use of a flat plate system is both economical and advantageous. The smooth slab makes it possible to have an exposed ceiling because there are a limited number of beams penetrating into the area. The smooth slab also allows for larger floor-to-ceiling heights than those that would be provided by a steel frame. Large spans could not be done with this system, but 40 Bond has moderate spans reaching a maximum of 25', which is within the limits of flat plate design. There is also no additional fireproofing needed.

Disadvantages

As with all flat plate systems, shear is often a concern. There is a transfer of moments from the slab to the columns, which increases shear stresses at these connections. 40 Bond, however, does not have any shear related issues in the elevated slabs due to the 9" slab thickness required for the 25' spans. The only locations within the building that were designed with additional shear reinforcement were at columns B3, C3 and F3 at the basement level. Another issue that was unable to be resolved due to the use of the two-way flat plate slab was the presence of transfer beams. These beams are needed at building setbacks, transitions from many slender columns to fewer, larger columns, and at the penthouse structure to allow for the long clear span. Other systems may help to eliminate the large transfer beams which will not only be aesthetically pleasing as fewer beams drop into the space, but will also reduce the labor and cost associated with preparing the formwork.

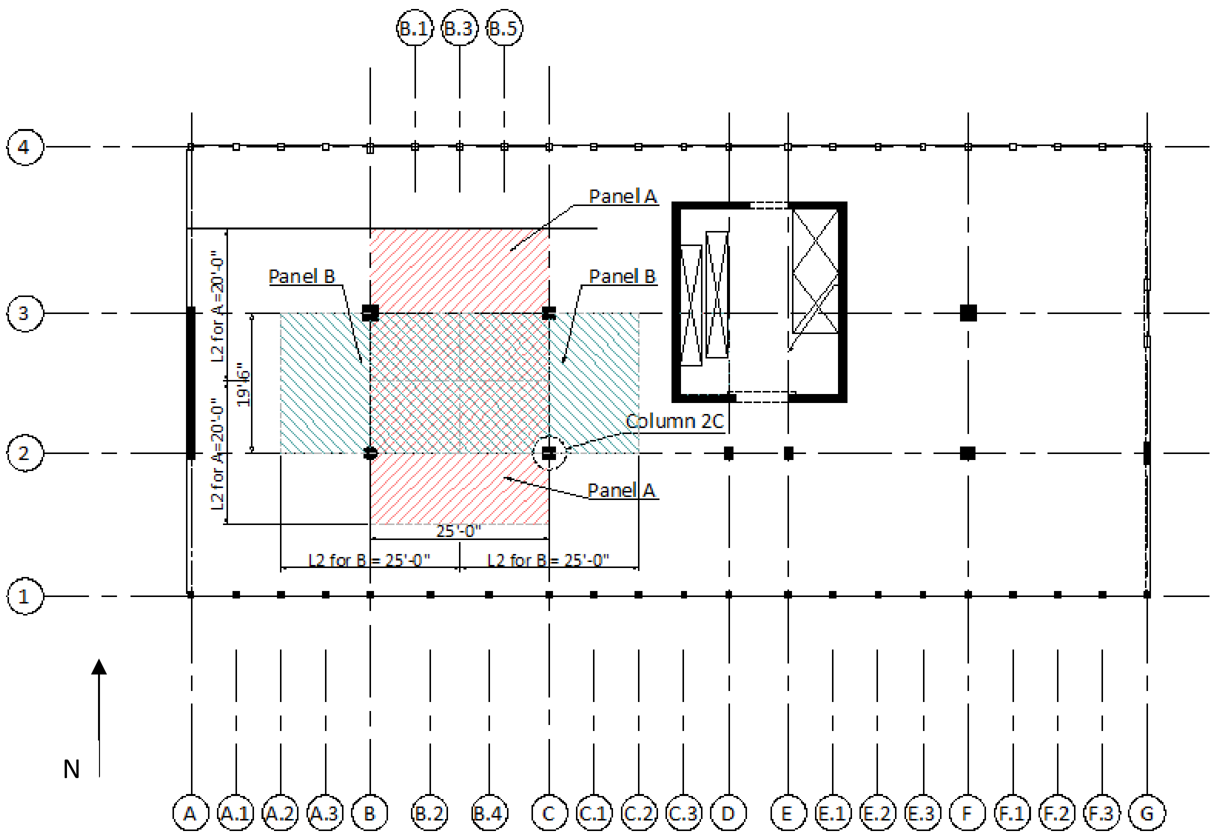

Figure 8 – Slab and Column Spot Check Locations

Non-Composite Steel – Option #1

Material Properties

Concrete:	4.5" slab (2.5" topping)	
	f'_c =	3,000 psi
Steel:	f_y =	50,000 psi
Reinforcement:	f_y =	60,000 psi
Metal Deck:	2C20 - 3 span	

Loading

Dead (self weight):	45 psf
Live:	40 psf
Superimposed:	25 psf

Description

This non-composite steel system was designed using a typical bay of 25'x20' with intermediate beams spaced equally at 8'-4". The Vulcraft 2C20 non-composite deck is able to span 10'-7" unshored given a 3-span condition, which is greater than the 8'-4" spacing proposed for this layout. The 2C20 system is accompanied with a total slab depth of 4.5" satisfying the load and deflection limits of this system.

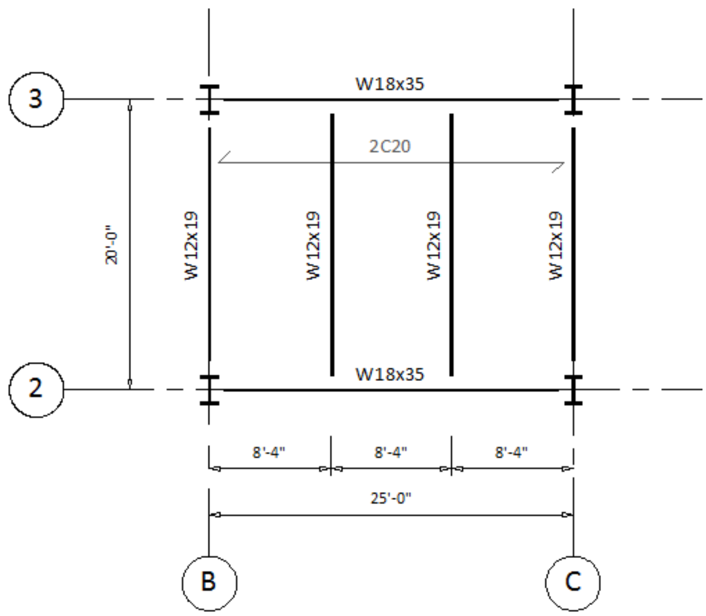

Figure 9 –Non-Composite Steel Layout

Calculations were completed using the AISC *Steel Construction Manual* to size the beams and girders at this interior bay. Controlled by deflection, the sizes determined can be seen in Figure 9**.** At this stage of preliminary consideration, columns have not yet been designed. Supporting calculations for the slab and framing can be found in Appendix B.

Advantages

This system has several advantages including the speed in erection and lower cost due to the absence of shear studs. It is a fairly lightweight system in comparison to concrete within the same bay as well. There is no need for formwork which reduces labor cost related to preparing the forms and because the decking is able to span 10'-7" during unshored construction there is also no need for shoring. Additionally, there is flexibility in laying out other building systems due to the service plenum that will be produced by the drop ceiling.

Disadvantages

In New York City it is most common to construct midrise residential buildings with concrete. A reason for not using steel is the deeper floor system that results from its implementation. In addition to the 4.5" slab depth, the W18x35 girders add another 17.7", bringing the total system depth to 22.2", which is much greater than that achieved using concrete flat plate slabs. This reduces the floor-to-ceiling height and in some instances where building height limits are enforced, could lead to fewer floors in the building. In the case of 40 Bond, reducing the floor-to-ceiling height would negatively impact the architectural concept as well.

Other disadvantages to this system include possible floor vibrations and the need for additional fire protection to obtain a 2 hour fire rating. Lead time is another issue because fabrication, detail and transport are all required for the steel. Lastly, there is the issue with the existing lateral force system of ordinary reinforced concrete shear walls. If these shear walls are to remain, special connections need to be considered as the two materials frame together. Unions in New York City also require that no trade is above the steel erectors, so to use this system with the existing shear walls, a plan must be formulated to organize how the concrete and steel will be constructed. Otherwise, a steel lateral system may have to be designed.

Feasibility

In regards to 40 Bond, the disadvantages outweigh the advantages when looking into non-composite steel floor framing. After this comparison, it is suggested that no further investigation be done on this system.

Hollow Core Precast Panels on Steel – Option #2

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

Loading

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

Description

Particular to the hollow core precast concrete plank system is the slight adjustment to all bays within the building. Because these precast panels come in 4'-0" increments it seems most logical to have the bays as 25'x20' and 25'x24' rather than the actual 25'x20', 25'x19'-6" and 25'x23'-2 ½" bays. In regards to this analysis, an interior 25'x20' bay was used and is shown in Figure 10. At this point, column design has not been completed.

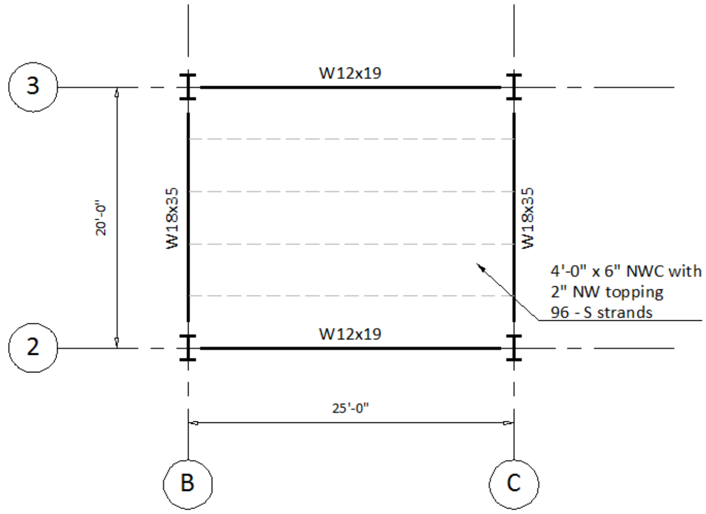

Figure 10 –Hollow Core Precast Planks on Steel Layout

A 6" thick plank with 2" topping was selected using the *PCI Design Handbook.* The span of 25'-0" was achieved using 96-S strands within the hollow core panel. This designation relates to the number of strands (9), the diameter of strands in 16ths (6) and that the strands are to be straight (S). This assembly is capable of holding a service load of 87 psf which exceeds the value of 80 psf calculated using the live load, superimposed load and an additional 15 psf for the 2" concrete topping.

The beams that the precast will frame into were determined with the AISC *Steel Construction Manual* and are sized as W18x35. Supporting calculations may be found in Appendix C.

Advantages

The hollow core precast system has numerous benefits. The construction can be completed quickly which allows for earlier occupancy in the building and the possibility to fast-track the project. This product is durable, low maintenance and it is easy to construct year-round because no curing time is needed. It also attenuates noise and is recognized as a LEED rated system.

Disadvantages

There are also many disadvantages associated with the use of this system. The bay sizes would have to be adjusted to accommodate the width of the precast panels and in turn result in an increase in building size that may or may not be acceptable. At this time, the vibration associated with this system is unknown.

As with all steel systems, there is also a deeper floor system, 25.7” in this study that reduces the floor-to-ceiling height within the space. This can become an issue in instances were building height limits are present and within 40 Bond it would sacrifice open space provided by the high ceilings. Although the lead time is relatively short for the panels, it is longer for the steel to account for fabrication, detailing and transportation. The steel also requires spray fireproofing to obtain the appropriate fire rating. There is a concern with the connections required at the concrete shear walls and its impact on design and cost. In addition, effort must be put forth to determine the scheduling that would be required for the interaction of trades in New York City.

Feasibility

This system does not seem like a likely candidate for further investigation. Its benefits are outweighed by the many disadvantages of its use within 40 Bond.

Two-Way Post-Tensioned– Option #3

Material Properties

Concrete:	8" slab (NWC)		
	f'_c	=	5,000 psi
	f'_{ci}	=	3,000 psi
Tendons:	Unbonded Tendons		
	1/2" diameter - 7 wire strand		
	$A_{pt} = 0.153\ in^2$		
	f_{pu}	=	270,000 psi
Reinforcement:	f_y	=	60,000 psi

Loading

Dead (self weight):	100 psf
Live:	40 psf
Superimposed:	25 psf

Description

A two-way post-tensioned slab was designed for a typical 25'x20' bay shown in Figure 11. Conservatively, the span/depth ratio was taken to be 40 which resulted in a preliminary slab thickness of 8". After following an example provided by the Portland Cement Association, it was determined that 12 tendons, providing 26.6k resistance/tendon, were needed in both the long and short directions. The number of tendons implies that the slab depth may successfully work at a thinner dimension if further study is done and as a result a larger number of tendons would be required.

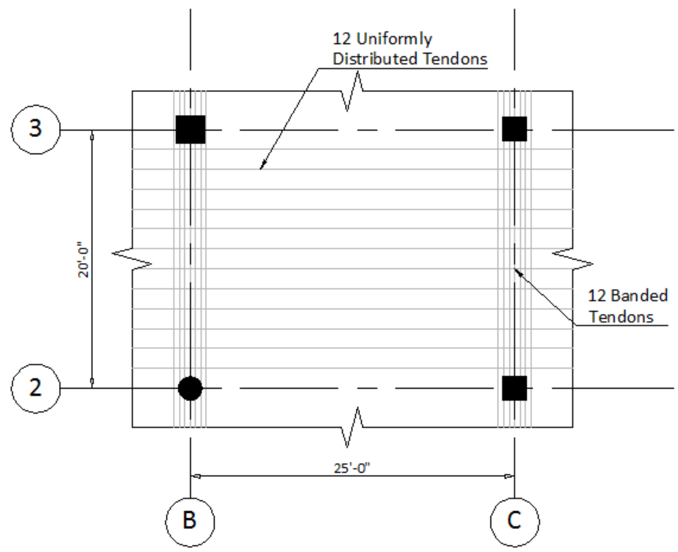

Figure 11 –Two-Way Post-Tensioned Layout

These tendons are banded in the short span direction and uniformly distributed in the long span direction. This is a typical layout for this type of construction and works particularly well in regards to placement of tendons at openings. The only large opening is the core located within two shear walls and its long dimension is perpendicular to the uniformly distributed tendons. Dead end anchors may then be placed on either side of the opening. In addition to the tendons required, there is also a limited amount of mild reinforcing that is needed. Additionally, at this point in design, column sizes were taken to be the same as those used in the existing flat plate system. Calculations supporting the design can be found in Appendix D.

Advantages

This system appears to be the most advantageous of the three proposed systems reviewed in this report. Post-tensioned slabs allow for a thin floor which immediately challenges any system containing steel framing. The 8" slab thickness determined in this analysis is actually thinner than the 9" two-way flat plate slab used in 40 Bond and it has potential to be even thinner. Similar to the existing design, there would be a clean concrete surface at the ceiling as well. The rigidity and denseness of post-tensioning limit the effect of vibrations and the balanced load provided by the tendons also reduces deflection. This system could be used in 40 Bond without adjusting the layout and would actually increase the floor-to-ceiling height.

40 Bond employs the use of several transfer beams throughout the building. The implementation of post-tensioning may alleviate the need for or at least reduce the size of the current beams, which would save both time and money associated with construction. These slabs have the ability to carry large live loads and can permit larger spans as well. This function may reduce the number of columns due to larger bay sizes. Finally, a short lead time is associated with this construction.

Disadvantages

Although there are many benefits of using this system, there are also some disadvantages and concerns related to it. Most of these negative aspects are related to construction. This system is very labor intensive and has the potential to be dangerous. There are extra safety procedures required and experienced contractors are needed to successfully construct this system. In New York City, there are very few people experienced with post-tensioning so that is definitely a concern for suggesting its use in 40 Bond.

General construction issues also include the need for formwork and shoring. It is also very difficult to cut openings after the concrete is poured in fear of cutting the stressed tendons located throughout the slab. These concerns should be understood and analyzed if this system is to be considered.

Feasibility

The advantages of implementing a two-way post-tensioned slab are very promising when considering an alternate framing system for 40 Bond. The concern is present in regards to construction and whether at this time post-tensioning a building in Manhattan is reasonable. Two other post-tensioned buildings, The Opal in Queens and 140 West 42nd Street in Manhattan, have been designed, which suggests that it is possible to use this system and therefore can be a consideration for further investigation. The ability to reduce the number of transfer beams and columns will be very beneficial when considering redesign. The formwork and installation of formwork required for those members is definitely costly so limiting the need will save a reasonable amount of money.

System Comparison

Comparison Criteria	Existing Two-Way Reinforced Flat Plate	Option #1 Non-Composite Steel Frame	Option #2 Hollow Core Precast Panel on Steel	Option #3 Two-Way Post-Tensioned Flat Plate
Slab Self Weight	112.5 psf	45 psf	74 psf	100 psf
Slab Depth	9"	4.5"	8"	8"
System Depth	9"	22.2"	25.7"	8"
Deflection	0.307" < 0.625"	1.19" < 1.25"	0.846" < 1.0"	Further study needed
Vibration Control	Great	Poor	Further study needed	Great
Fire Rating	2 hour	1.5-2 hour	1.5-2 hour	2 hour
Fire Protection	None	Spray	Spray	None
Architectural Impact	Existing	Negative - Reduces floor-to-ceiling height	Negative - Reduces floor-to-ceiling height	Benefit - Increases floor-to-ceiling height
Constructability	Medium	Easy	Easy	Hard
Formwork	Yes	No	No	Yes
Lead Time	Short	Long	Long	Short
System Cost*	$19.96/SF	$24.85/SF	$32.50/SF	$20.36/SF
Feasibility	Yes	No	No	Yes (**Investigate**)

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

Conclusion

After reviewing the comparison between each of the four systems is seems that the two-way post-tensioned system is the most feasible alternate floor framing system and is comparable to the existing two-way flat plate slab that was designed by DCE. There is concern in regards to the post-tensioned construction because of the intensity of the labor practices and the risks associated with them. Experienced contractors are needed to be able to successfully construct a post-tensioned project because the method and understanding of how it works is crucial for the project to be safe as well.

The benefits that would be gained from this particular system are the ability to increase the floor-to-ceiling height with a slab thinner than the existing 9" two-way flat plate and the opportunity to employ longer spans. The tendons within the slab help to carry additional live load, reduce vibrations and limit deflection because of the balanced load that is produced by these stressed members. There is no additional fireproofing required and the layout of the building does not need to change. A post-tensioned slab also has the possibility to eliminate or significantly reduce the transfer system of 40 Bond. In doing this there would be less beam intrusion into spaces and a reduced cost due to fewer forms needed.

The other two systems, non-composite steel framing and precast hollow core concrete planks on steel, had benefits to their implementation, but these advantages were outweighed by the disadvantages in both cases. The most severe factor in both was the deep system depth. It would reduce the floor-to-ceiling heights and require drop ceilings to enclose the structure. There were also issues with the interaction of the steel systems with the existing concrete shear wall lateral system. Finally, in common practice, concrete construction is the typical means for midrise residential buildings in Manhattan so it is logical that a concrete system would seem more applicable and feasible in comparison.

At the completion of this technical assignment, it has been decided that due to the information discovered through this set of analyses that the two-way post-tensioned slab system deserves further investigation as a possible proposal topic for AE Senior Thesis.

Appendix A

Two-Way Flat Plate – Existing

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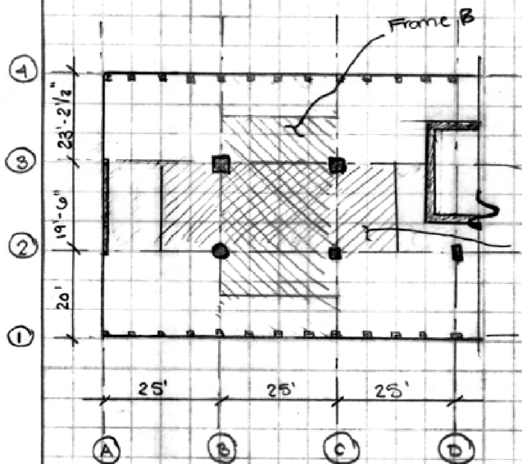
TECH 2 | 40 BOND | ORIGINAL SYSTEM - 2 WAY FLAT PLATE | 1/8

Flat Plate: As designed by DCE
- 9" slab
- $f'_c = 5000$ psi (NWC), $f_y = 60{,}000$ psi
- #4 @ 12" top + bottom w/ additional noted
- Two-way slab, no int. beam
- DL = 112.5 psf; LL = 40 psf, SDL = 20 psf



DIRECT DESIGN METHOD (DDM)
ACI 318-08 Ch. 13

* Assuming 25'-20' bay to simplify calculation.

MS = Middle Strip
CS = Column Strip

Factored Loads:
$(1.2)(137.5) + (1.6)(40) = 0.229$ ksf

* All references made to ACI 318-08 unless otherwise noted

PANEL A
22" × 22" and 22" Ø columns (B2 + C2)
§ 13.2.1: CS is a width on each side of a column

centerline = $0.25\ell_2$ or $0.25\ell_1$ whichever is less
$(0.25)(20) = 5'$ → 10' column strip
$(0.25)(25) = 6.25'$

$M_o = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(20')(25 - 22/12)^2(0.229) = 307^{k}$
$\alpha = 0$ since $I_b = 0$ for no beams

Min thickness of slab w/o beams [Table 9.5c]
Int. panel w/o drop panels → $\ell_n / 33 = \frac{(25 - 22/12)(12")}{33} = 8.42 <$ design 9" OKAY!

Frame A

$M^- = 0.65 M_o = 199.6^{k}$
$M^+ = 0.35 M_o = 107.5^{k}$

$\ell_2/\ell_1 = 20/25 = 0.8$, $\alpha \ell_2/\ell_1 = 0$

§ 13.6.4.1

ℓ_2/ℓ_1	0.5	0.8	1.0
$\alpha \ell_2/\ell_1 = 0$	75	$\boxed{75}$	75

TECH 2 | 40 BOND | ORIGINAL SYSTEM · 2 WAY FLAT PLATE | 2/8

75% of M^- to CS = 149.7^{1k}
25% of M^- to MS = 49.9^{1k}

§ 13.6.4.4

ℓ_2/ℓ_1	0.5	0.8	1.0
$\alpha \ell_2/\ell_1 = 0$	60	$\boxed{60}$	60

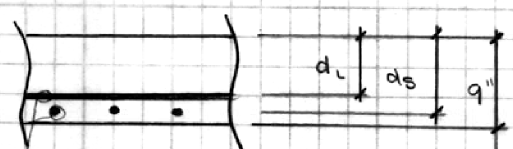
60% of M^+ to CS = 64.5^{1k}
40% of M^+ to MS = 43^{1k}

	M^-	M^+	M^-
Total Moment	-199.6	107.5	-199.6
CS	-149.7	64.5	-149.7
MS	-49.7	43	-49.9

Panel A · Total width = 20'
CS = 10' = 120"
MS = 5' on either side = 10' = 120"

- Design of Slab Reinf.

- Max spacing = 2t = 18"
- Min Steel = Temp + Shrinkage Reinf
$A_{s,min} = 0.0018bt$ for $f_y = 60$ ksi



* 3/4" clear cover

4 → $d_s = 9 - 3/4 - 1/2(0.5) = 8"$
$d_L = 8 - 0.5 = 7.5"$
5 → $d_s = 7.94"$
$d_L = 7.32"$
6 → $d_s = 7.88"$
$d_L = 7.13"$

#4@12, #5@8, #6@8 (CS)
mid left right
#4@12 (MS)

- Design Slab Reinf. in CS — Interior Span

Item	Description	Left	Mid	Right
1	M_u (1K)	-149.7	64.5	-149.7
2	CS width b (in)	120	120	120
3	Effective depth d	7.32	7.5	7.13
4	$M_n = M_u/0.9$	-166.3	71.7	-166.3

* Check $d_{min} = \sqrt{\dfrac{M_n(1000)(12")}{\rho f_y b(1 - 0.59\rho f_y/f'_c)}}$

w/ $\rho = 0.0243$ for $f'_c = 5$ ksi, $f_y = 60$ ksi

$d_{min} = \sqrt{\dfrac{166.3(1000)(12")}{(0.0243)(60000)(120)(1-0.59(0.0243)(60/5))}} = 3.71" < 7.5, 7.32, 7.13$ OKAY!

Item	Description	Left	Mid	Right
5	$R_u = M_n(12000)/bd^2$	310	127.5	327
6	ρ from Table A.5a (NDS)	0.0053	0.0022	0.0056
7	$A_s = \rho bd$	$\boxed{4.66}$	$\boxed{1.98}$	$\boxed{4.79}$
8	$A_{s,min} = 0.0018bt$	1.94	1.94	1.94

TECH 2 | 40 BOND | ORIGINAL SYSTEM - 2 WAY FLAT PLATE | 3/8

		M^-_L	M^+_M	M^-_R
9	N = larger of 7 & 8 / Area	15.03 → $\boxed{16}$	9.9 → $\boxed{10}$	10.8 → $\boxed{11}$
		$A_L = 0.31 in^2$	$A_{mid} = 0.20 in^2$	$A_R = 0.44 in^2$
10	N_{min} = width of slab / 2t	6.66 → 7	7	7

Calculated	DCE
Int: $M_L^- = 16\#5$	Int: $M_L^- = 15\#5$
$M_M^+ = 10\#4$	$M^+_M = 10\#4$
$M_R^- = 11\#6$	$M^-_R = 15\#6$

The right reinf for CS-A is less than that used by DCE. A possible reason for this difference may be that this portion of the CS is close to the shear wall which is drawing more moment. My calculation does not include lateral loading at this moment.

The reinf at the left and midspan are slightly larger which may be due to the increased SDL of 25 psf vs. DCE's value of 20psf.

Design of Slab Reinf in MS

		Int. Span	
Item	Description	M^-	M^+
1	M_u (1k)	-49.9	43
2	MS width b (in)	120	120
3	Effective depth (in)	7.5	7.5
4	$M_n = M_u / 0.9$	-55.4	47.8
	* check d_{min} = 2.14 < 7.5' okay		
5	$R_u = M_n(12000)/bd^2$	-98.5	85
6	ρ from Table A.5a (NDD)	0.00165	0.00145
7	$A_s = \rho bd$	1.49	1.31
8	$A_{smin} = 0.0018bt$	$\boxed{1.94}$	$\boxed{1.94}$
9	N = larger of 7 & 8 / 0.20	9.7 → $\boxed{10}$	$\boxed{10}$
10.	N_{min} = width of MS / 2t	6.66 → 7	7

Calculated	DCE
Calculated - 10#4	DCE - 10#4

My calculated value matches that determined by DCE for MS-A.

TECH 2 | 40 BOND | ORIGINAL SYSTEM - 2 way FLAT PLATE | 4/8

PANEL B

22"×22" columns (C2 & C3)
CS = 120" (§ 13.2.1)
MS = 15' = 180"

$M_0 = \frac{1}{8} w \ell_2 \ell_n = \frac{1}{8}(25)(0.229)(20 - 22/12)^2 = 236.2^k$
$\alpha = 0$ since $F_b = 0$ (no beams)

Min thickness of slab w/o int beams [Table 9.5c]
$\frac{\ell_n}{33} = \frac{(20 - 22/12)(12")}{33} = 6.66" < 9"$ OKAY!

Frame B

$M^- = 0.65 M_0 = 153.5^k$
$M^+ = 0.35 M_0 = 82.7^k$
$\alpha \ell_2/\ell_1 = 0$, $\ell_2/\ell_1 = 25/20 = 1.25$

§ 13.6.4.1

ℓ_2/ℓ_1	1.0	1.25	2.0
$\alpha \ell_2/\ell_1 = 0$	75	$\boxed{75}$	75

75% of M^- to CS = -115.1^k
25% of M^- to MS = -38.4^k

§ 13.6.4.4

ℓ_2/ℓ_1	1.0	1.25	2.0
$\alpha \ell_2/\ell_1 = 0$	60	$\boxed{60}$	60

60% of M^+ to CS = 49.6^k
40% of M^+ to MS = 33.1^k

	M^-	M^+
Total Moment	-153.5	82.7
CS	-115.1	49.6
MS	-38.4	33.1

Panel B: total width = 25'
CS = 10' = 120"
MS = 15' = 180"
#6 @ 8" LTR, #4 @ 12 Mid (CS)
#4 @ 12 (MS)

#4 → $d_s = 8"$, $d_L = 7.5"$
#6 → $d_s = 7.88"$, $d_L = 7.13"$

Design of Slab Reinf in CS

Item	Description	M^-_{LTR} Int	M^+
1	M_u (1k)	-115.1	49.6
2	CS width b (in)	120	120
3	Effective depth d (in)	7.88"	7.5"
4	$M_n = M_u/0.9$	-127.9	55.1
*check d_{min} = 3.26" < 7.88", 7.5" OKAY!			
5	$R = M_n(12000)/bd^2$	-206	98
6	ρ from Table A.5a (NDS)	0.0035	0.00165

TECH 2 | 40 BOND | ORIGINAL SYSTEM · 2-WAY FLAT PLATE | 5/8

Item.	Description	M^-	M^+
7.	$A_s = \rho bd$	$\boxed{3.31}$	1.49
8.	$A_{s,min} = 0.0018bt$	1.94	$\boxed{1.94}$
9.	$N = \frac{\text{larger of 7 or 8}}{0.44 \text{ or } 0.20}$	$7.8 \rightarrow \boxed{8}$	$9.7 \rightarrow \boxed{10}$
10	$N_{min} = \frac{\text{width of CS}}{2t}$	$6.66 \rightarrow 7$	7

Calculated:	DCE
Int : $M^- = 8\#6$	Int : $M^- = 12\#6$
$M^+ = 10\#4$	$M^+ = 10\#4$

The reinf at this column strip is 4 bars less for the M^- in comparison to DCE. A reason for this may be its close proximity to the shearwall, which due to relative stiffness, draws more load. With this report lateral loads are not taken into account, but once they are I would expect my reinf to be closer to what designed by DCE.

M^+ reinf is the same as what designed by DCE

- Design of slab reinf. in MS Int.

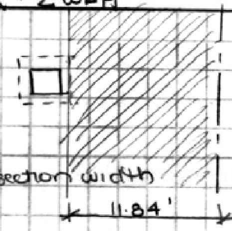
Item	Description	M^-	M^+
1	M_u (k)	-38.4	33.1
2	MS width b (in)	180	180
3	Effective d (in)	7.5	8.0
4	$M_n = M_u/0.9$	-42.7	36.8
	$\times d_{min} = 1.88'' < 7.5''$, 8" okay!		
5	$R = M_n(12000)/bd^2$	-50.6	38
6	ρ from Table A.5a (NDD)	0.0008	0.0006
7	$A_s = \rho bd$	1.08	0.88
8	$A_{smin} = 0.0018bt$	$\boxed{2.92}$	$\boxed{2.92}$
9	$N = \frac{\text{larger of 7 or 8}}{0.2}$	$14.6 \rightarrow \boxed{15}$	$\boxed{15}$
10.	$N_{min} = \frac{\text{width of MS}}{2t}$	10	10

Calculated	DCE
15#4	15#4

My calculated value matches what determined by DCE for MS-B

TECH 2 | 40 BOND | ORIGINAL SYSTEM · 2 WAY | 6/8

SHEAR CHECKS ·



• WIDE BEAM ACTION

$\frac{25'}{2} - \left(\frac{7.875''}{12''}\right) = 11.84'$ critical section width

$w_u = (1.2)(112.5+25) + (1.6)(40) = 229$

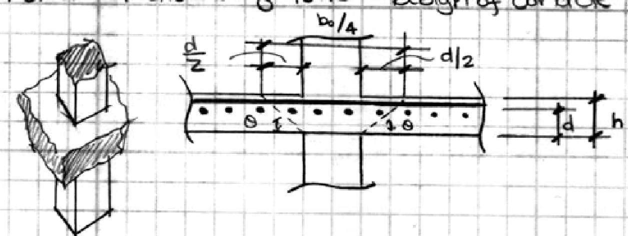
$V_u = w_u \text{ Area} = (0.229 \text{ ksf})(11.84')(20') = 54.2^k$

$V_n = 2\sqrt{f'_c}\, bd = 2\sqrt{5000}\,(20')(7.875'')(12'') = 267^k$

$\phi V_n = 0.75(267) = 200^k$

$\phi V_n = 200^k > V_u = 54.2^k$ GOOD!

• PUNCHING SHEAR § 13.10 · Design of Concrete Structures



Checking column 2C & 9" slab
• 22×22
• #6 @ 8" in both directions
• f'_c = 5ksi

$V_c = 4\sqrt{f'_c}\, b_o d$ [Eq. 13.11a]

$d = 9'' - 3/4'' - 1/2\,(0.75) = 7.875''$

↖ ϕ of #6

$\frac{b_o}{4} = 2(d/2) + 22'' = 2\left(\frac{7.875}{2}\right) + 22'' = 29.875'' \longrightarrow b_o = 119.5''$

$V_c = 4\sqrt{5000}\,(119.5'')(7.875'') = 266.2^k$

Must be $\leq V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$

$\alpha_s = 40$ for int. columns

$= \left(\frac{40(7.875)}{119.5} + 2\right)\sqrt{5000}\,(119.5)(7.875) = 308.5^k$

Must be $\leq V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1.0}\right)\sqrt{5000}\,(119.5)(7.875) = 399.3^k$

$\beta_c = 1.0$ for square columns

$V_u = (0.229 \text{ ksf})(494 \text{ sf} - (22/12)^2) = 112.4^k$

$\phi V_c = 0.75(266.2) = 199.7^k > V_u = 112.4^k$ GOOD!

No additional shear reinforcement req'd!

TECH 2 | 40 BOND | ORIGINAL SYSTEM - 2WAY FLAT PLATE | 7/8

DEFLECTION

• Elastic Δ due to selfwt.

$\Delta_{f,ref} = \dfrac{w\ell^4}{384 E I_{frame}}$

$E = 57000\sqrt{5000} = 4030509$ psi

$I_{20} = \dfrac{(25')(12'')(9'')^3}{12} = 18225 \text{ in}^4$

$I_{25} = \dfrac{(20')(12'')(9'')^3}{12} = 14580 \text{ in}^4$

$w = \left(\frac{9}{12}\right) 150\text{pcf} = 112.5 \text{ pcf}$

$\Delta_{20,ref}$ shortspan $= \dfrac{112.5\text{psf}\ (25')(20')^4\ 1728}{384\ (4030509)\ (18225)} = 0.0276''$

$\Delta_{25,ref}$ longspan $= \dfrac{112.5\text{psf}\ (20')(25')^4\ 1728}{384\ (4030509)\ (14580)} = 0.0673'$

$\Delta_{f,col} = \Delta_{f,ref} \dfrac{M_{col}}{M_{frame}} \dfrac{E_c I_{frame}}{E_c I_{col}}$; $\Delta_{f,mid} = \Delta_{f,ref} \dfrac{M_{mid}}{M_{frame}} \dfrac{E_c I_{frame}}{E_c I_{mid}}$

FRAME A:

$I_{CS} = \dfrac{120''(9'')^3}{12} = 7290 \text{ in}^4 = I_{MS}$

FRAME B:

$I_{CS} = 7290 \text{ in}^4$; $I_{MS} = \dfrac{(180'')(9'')^3}{12} = 10935 \text{ in}^4$

68% to CS (M^+ & M^-avg)
32% to MS (M^+ & M^-avg)

SHORT SPAN (B)

• $0.0276(0.68)\left(\dfrac{18225}{7290}\right)$

$= \Delta_{f,col} = 0.0469$

• $0.0276(0.32)\left(\dfrac{18225}{10935}\right)$

$\Delta_{f,mid} = 0.0147$

LONG SPAN (A)

• $0.0673(0.68)\left(\dfrac{14580}{7290}\right)$

$= \Delta_{f,col} = 0.0915''$

• $0.0673(0.32)\left(\dfrac{14580}{7290}\right)$

$\Delta_{f,mid} = 0.0431$

* SHORT TERM Δ *

Longterm = Δ(3.0)

$\Delta_{f,col} = 0.1407''$
$\Delta_{f,mid} = 0.0441''$

$\Delta_{f,col} = 0.2745''$
$\Delta_{f,mid} = 0.1293''$

TECH 2 | 40 BOND | ORIGINAL SYSTEM · 2WAY FLAT PLATE | 8/8

Shortspan (B)

Long span (A)

* Live load = Δ(40/112.5) by direct proportion *

Short span (B):

$\Delta_{f,col}$ = 0.0167"
$\Delta_{f,mid}$ = 0.0052"

Long span (A):

$\Delta_{f,col}$ = 0.0325
$\Delta_{f,mid}$ = 0.0153

ACI Limit = ℓ/480 > Δ_{max} = Δ_{LL} + 3Δ_{DL}

Short span (B):

- $\Delta_{max,col}$ = 0.0167 + 0.1407
  = 0.1574 < 0.5" GOOD
- $\Delta_{max,mid}$ = 0.0052 + 0.0441
  = 0.0493" < 0.5" GOOD!

Long span (A):

- $\Delta_{max,col}$ = 0.0325 + 0.2745
  = 0.307 < 0.625"
- $\Delta_{max,mid}$ = 0.0153 + 0.1293
  = 0.1446 < 0.625
  GOOD!

* Deflection w/in ACI Limit in both directions

Appendix B

Non-Composite Steel Framing – Option #1

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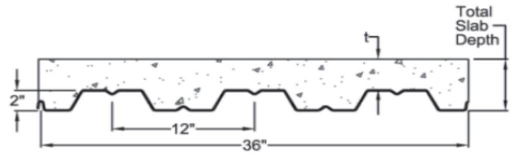
VULCRAFT

2 C CONFORM



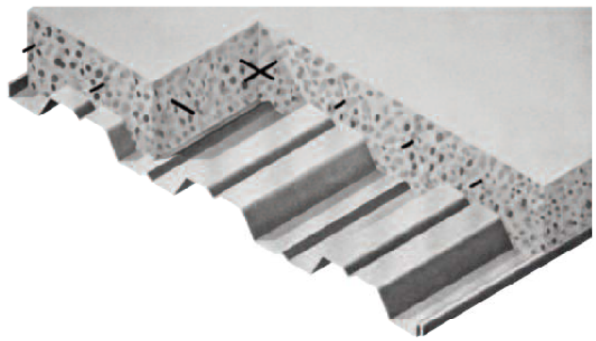
Interlocking side lap is not drawn to show actual detail.

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	[illegible]	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

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd³ / 100 ft²	ft³ / ft²	Recommended Welded Wire Fabric
4	0.93	0.250	6x6 - W1.4xW1.4
4 1/2	1.08	0.292	6x6 - W1.4xW1.4
5	1.23	0.333	6x6 - W1.4xW1.4
5 1/4	1.31	0.354	6x6 - W1.4xW1.4
5 1/2	1.39	0.375	6x6 - W2.1xW2.1
6	1.54	0.417	6x6 - W2.1xW2.1
6 1/4	1.62	0.438	6x6 - W2.1xW2.1
6 1/2	1.70	0.458	6x6 - W2.1xW2.1



SECTION PROPERTIES

Deck Type	Design Thickness in.	Deck Weight psf	Section Properties I_p in⁴/ft	I_n in⁴/ft	S_p in³/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
2C22	0.0295	1.62	0.324	0.321	0.263	0.266	1832	50
2C20	0.0358	1.97	0.409	0.406	0.341	0.346	2698	50

ALLOWABLE UNIFORM LOAD (PSF)

TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0	9-6	10-0	10-6	11-0
2C22	1	Fb = 30,000	210	174	146	124	107	93	82	73	65	58	52	48	43
		Defl. = l/240	170	128	98	77	62	50	42	35	29	25	21	18	16
		Defl. = l/180	227	170	131	103	83	67	55	46	39	33	28	25	21
	2	Fb = 30,000	200	167	141	121	105	92	81	72	64	58	52	47	43
		Defl. = l/240	408	306	236	186	149	121	100	83	70	59	51	44	38
		Defl. = l/180	544	409	315	248	198	161	133	111	93	79	68	59	51
	3	Fb = 30,000	243	204	173	149	129	113	100	89	80	72	65	59	54
		Defl. = l/240	319	240	185	145	116	95	78	65	55	47	40	34	30
		Defl. = l/180	426	320	246	194	155	126	104	87	73	62	53	46	40
2C20	1	Fb = 30,000	272	225	189	161	139	121	106	94	84	75	68	62	56
		Defl. = l/240	215	161	124	98	78	64	52	44	37	31	27	23	20
		Defl. = l/180	286	215	166	130	104	85	70	58	49	42	36	31	27
	2	Fb = 30,000	263	219	185	159	137	120	106	94	84	75	68	62	56
		Defl. = l/240	515	387	298	235	188	153	126	105	88	75	64	56	48
		Defl. = l/180	687	516	398	313	250	204	168	[illegible]	118	100	86	74	65
	3	Fb = 30,000	322	269	228	196	170	149	131	117	104	94	85	77	70
		Defl. = l/240	403	303	233	184	147	119	98	82	69	59	50	44	38
		Defl. = l/180	538	404	311	245	196	159	131	109	92	78	67	58	50

From *Vulcraft Steel Roof and Floor Deck Guide*

TECH2 | 40 BOND | OPTION #1 - NON COMPOSITE STEEL | 1/2

NON COMPOSITE STEEL
- use of Vulcraft website

- LOADS: LL = 40psf
  SDL = 25psf
  DL = 45 psf
  ↳ 4.5 slab depth (t = 2.5) N=9
  NWC, 3 span = 10'-7", 145pcf
  f'_c = 3000 psi
  F_y, reinf = 60,000psi

$\boxed{\text{2C20 Deck (2C conform)}}$

Total load = 40 + 25 + 45 = 110 psf

- 2C20: 3span, 8'-6" span
  F_b = 30,000, 20GA, load = 117 psf > 110 psf OKAY!
  $l/240$, 20 GA, load = 82psf > 40 psf OKAY!
  $l/180$, 20GA, load = 109 psf > 45 psf
  compared to load of wt. concrete

For beams
Load = 1.2 (25+45) + 1.6 (40) = 148 psf = 0.148 ksf
Trib = 8'-4" = 8.33'
ω_u = 8.33' (0.148 ksf) = 1.23 klf
$V_u = \frac{1.23\text{klf}(20')}{2} = 12.3^K$

$M_u = \frac{(1.23\text{klf})(20')^2}{8} = 61.5^{1K}$

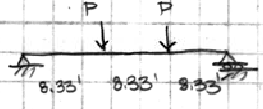
*assume fully braced

BEAM - TABLE 3-2 (AISC Steel Manual)
W12x14 $\phi M_n = 65.2 > 61.5^{1K}$ OKAY
$\Delta_{LL} = l/360 = \frac{(20')(12")}{360} = 0.667$

$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(40psf)(8.33')(20')^4(1728)}{384(29000)(88.6)(1000)} = 0.467"$

0.467 < 0.667 OKAY

GIRDER



P = 24.6 for interior girder

$V_U = P = 24.6^K$
$M_{max} = Pa = (24.6)(8.33) = 205^{1K}$

TECH 2 | 40 BOND | OPTION #1 : NON COMPOSITE STEEL | 2/2

W14x34 : $\phi M_p = 205^{1k} \geq 205^{1k}$

$\Delta_{max} = 0.0357 \frac{PL^3}{EI}$ w/ $I_{14x34} = 340 \text{ in}^4$

$w_u = 40 \text{ psf } (8.33') = 0.333 \text{ klf}$

$V = \frac{0.333 \text{ klf } (20')}{2} = 6.66^k$

$\Delta_u = l/360 = (25'\text{-}0'')(12'')/360 = 0.833''$

$\Delta = \frac{0.0357(6.66)(25')^3(1728)}{29000(340)} = 0.651'' < 0.833''$ OKAY

* For all beams *

$\Delta_{TL} = l/240 = (20'\text{-}0'')(12'')/240 = 1.0''$

$w_{TL} = (25 + 45 + 40)(8.33') = 0.9163 \text{ klf}$

$\Delta_{TL} = 1.0 = \frac{5(0.9163)(20')^4(1728)}{384(29000) I_x}$

$I_x = 114 \text{ in}^4$ → Table 3-3

All int. beams will be $\boxed{W12 \times 19}$ $I_x = 130 \text{ in}^4$

* due to inspection, it will pass previous checks.

* For all interior girders *

$\Delta_{TL} = l/240 = (25)(12)/240 = 1.25''$

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

$V = \frac{110 \text{ psf } (8.33')(20')}{1000(2)} = 9.2^k (2) = 18.3^k$ framing from both sides

$\Delta_{TL} = \frac{0.0357(18.3^k)(25')^3(1728)}{29000 \; I_x}$

$I_x = 487 \text{ in}^4$ → $\boxed{W18 \times 35}$ w/ $I_x = 510 \text{ in}^4$ for all int. girders

* because larger girder size, passes previous checks by inspection

Appendix C

Hollow Core Precast Panel on Steel – Option #2

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Strand Pattern Designation

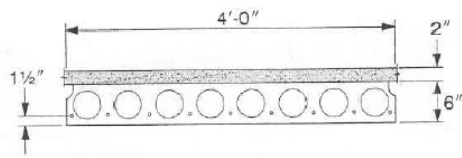
76–S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

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.

HOLLOW-CORE

4′-0″ x 6″
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f'_{ci} = 3{,}500$ psi

Section Properties

		Untopped	Topped
A	=	187 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
b_w	=	16.00 in.	16.00 in.
wt	=	195 plf	295 plf
		49 psf	74 psf
V/S	=	1.73 in.	

4HC6+2

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

Strand Designation Code	Span, ft																
	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	305	258	220	188	162	139	119	97	78	62	47	35					
	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	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9					
76-S	358	304	260	224	194	168	146	122	101	82	66	52	39				
	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				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9				
96-S		390	336	291	253	221	194	170	146	123	104	87	72	58	46	35	
		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	
		0.4	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	
87-S			398	346	302	265	234	206	182	158	136	117	100	85	71	59	47
			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	0.6	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				382	335	294	260	231	205	181	157	137	119	102	88	75	63
				0.7	0.8	0.8	0.9	0.9	0.9	1.0	1.0	0.9	0.9	0.9	0.8	0.8	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 based on strain compatibility; bottom tension limited to $6\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.

From *PCI Design Handbook*

TECH 2 | 40 BOND | OPTION #2 : HOLLOW CORE SLAB ON STEEL | 1/1

HOLLOW CORE PRECAST ON STEEL

LOAD : LL = 40 psf
SDL = 25 psf
DL = 15 psf (from PCI Handbook for topped members)

15 psf + 25 psf + 40 psf = 80 psf super imposed service

$f'c = 5000$ psi
$f'ci = 3500$ psi
Span : 25'-0"

$f_{pu} = 270,000$ psi

4'-0" x 6" NWC w/ 2" NW Topping
96-S carrying 87 psf, 0.4" camber @ erection
-0.3" camber (long period)
9 strands @ 6/16" ϕ - straight

Self wt = 74 psf

Girders :
Load = 1.2 (25+74) + 1.6 (40) = 183 psf
$M_u = \frac{(183 \text{ psf})(25)(20')^2}{8} = 229'^k$

ϕM_n = W 18x35 = 249'k > 229'k OKAY!

$\Delta_{LL} = l/360 = 0.667 = \frac{5(40 \text{ psf})(25')(20')^4(1728)}{384(29000)(1000) I_x}$

$I_x = 186 \text{ in}^4 < 510 \text{ in}^4$ OKAY!

Other beams : no direct load

$\boxed{\text{4'-0" x 6" NWC w/ 2" Topping} \quad \text{4HC6+2, 96-S on W 18x35}}$

* W12x19 parallel to precast panels.
No acting to resist load, there to add stability

$\Delta_{TL} = \frac{5(40+25+74)(25')(20')^4(1728)}{384(29000)(1000)(510)} = 0.846" < l/240 = 1.0"$

Appendix D

Two-Way Post-Tensioned – Option #3

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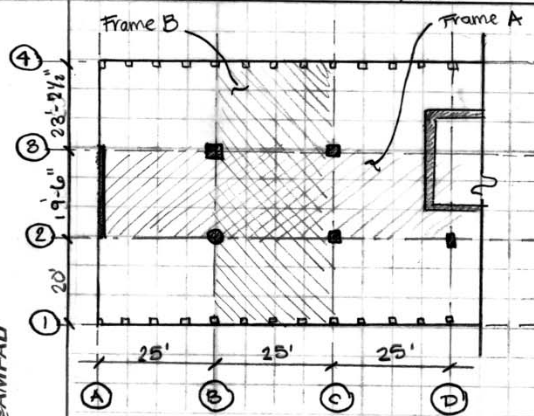
TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 1/2



- BANDED TENDONS along short direction (A, B, C, D, etc.)
- Uniform tendons parallel to long direction (1, 2, 3, 4)

ASSUMPTIONS

- All bays 25' x 20'
  - simplified for ease of calc. @ preliminary stage
- Neglecting shear wall effect.
- No live load reductions considered
- Ignoring Column stiffness
- Use of DDM
- No pattern loading since $LL/DL < 3/4$ (ACI 13.7.6)
- Slab depth ratio for 2-way slab 40-45 (Holbert Apple)
- Target balanced load 60-70% (DL) (Holbert Apple)

MATERIALS

Concrete - 150 pcf (NWC)
- f'_c = 5000 psi
- f'_{ci} = 3000 psi

Rebar - f_y = 60,000 psi

Post tensioning - Unbonded tendons
- 1/2" ø, 7 wire strands, A_{ps} = 0.153 in²
- f_{pu} = 270 ksi
- Estimated prestress losses = 15 ksi (ACI 18.6)
- $f_{se} = 0.7 f_{pu} - \text{losses} = 0.7(270) - 15 = 174$ ksi (ACI 18.5.1)
- $P_{eff} = A_{ps} \cdot f_{se} = (0.153\,in^2)(174\,ksi) = 26.6^k$/ tendon.

PRELIMINARY SLAB THICKNESS

To be conservative $l/h = 40 \rightarrow (25')(12'')/40 = 7.5'' \approx 8''$ thickness

LOADING

DL = Self wt = $(8/12)(150\,pcf) = 100\,psf$
LL = 40 psf (residential)
SDL = 25 psf

DESIGN PARAMETERS

*Prestressed 2-way slab system shall be designed as class U w/ $f_t < 6\sqrt{f'_c}$ (ACI 18.3.3)

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 2/12

At time of jacking (ACI 18.4.1)

$f'_{ci} = 3000$ psi
Comp. $= 0.60 f'_{ci} = 0.6(3000) = 1{,}800$ psi
Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

At service loads (ACI 18.4.2 + 18.5.3)

$f'_c = 5000$ psi
Comp $= 0.45 f'_c = 0.45(5000\text{psi}) = 2{,}250$ psi
Tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

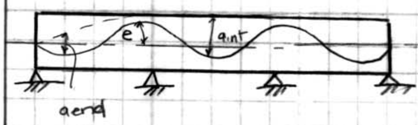
Average precompression limits
$P/A = 125$ psi min (ACI 18.12.4)
$= 300$ psi max

Target load balances:
60-70% of DL → will use avg. of 65%

$0.65(100\text{psf}) = 65$ psf

Cover requirements (2 hr. rating, assume carbonate aggregate)

Restrained slab (int.) = 3/4" bottom
Unstrained slab (ext) = 1 1/2" bottom
= 3/4" top



Tendon ordinate	Tendon CG Loc.*
Ext support - anchor	4.0"
Int. support - top	7.0"
Int. span - bottom	1.0"
End span - bottom	1.75"

*measured from bottom of slab

$a_{int} = 7 - 1 = 6.0''$
$a_{end} = \frac{(4+7)}{2} - 1.75 = 3.75''$

FRAME A CALCULATIONS

$A = bh = (20')(12'')(8'') = 1920\ in^2$
$S = \frac{bh^2}{6} = \frac{(20)(12'')(8'')^2}{6} = 2560\ in^3$

Balanced load $w_b = (65\text{psf})(20') = 1300$ plf $= 1.3$ k/ft

Force needed to counteract load in end bay: $P = \frac{wL^2}{8a_{end}} = \frac{(1.3)(25')^2}{8\left(\frac{3.75''}{12''}\right)} = 325^k$

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 3/12

- # of tendons to achieve $= \frac{325^k}{26.6^k/\text{tendon}} = 12.2 \rightarrow 12$ tendons

- Actual force for bonded tendon

$$P_{actual} = (12)(26.6^k) = 319.2 = 320^k$$

- Balanced load for end span

$$w_b = \left(\frac{320}{325}\right)(1.3\ k/ft) = 1.28\ k/ft$$

- Actual precompression stress

$$P_{actual}/A = \frac{(320^k)(1000)}{1920 in^2} = 166\ psi \quad > 125\ psi\ min \quad < 300\ psi\ max$$

okay!

- Check interior span.

$$P = \frac{(1.3\ k/ft)(25')^2}{8(6''/12'')} = 208^k < 320^k$$

$$w_b = \frac{(320)(8)(6''/12'')}{(25)^2} = 2.048\ k/ft$$

$$w_b/w_{DL} = 2.048 \Big/ (0.125)(20) = 0.82 < 1.0 \quad \text{okay!}$$

Effective prestress force, $\boxed{P_{eff} = 320^k}$

- Check slab stresses (moments from DDM)

$$M_o = \frac{1}{8}(20')(25 - 22/12)^2\, w$$

		DEAD	LIVE	BAL (end)	BAL (int)
	w	125 psf	40 psf	1280/20 = -64 psf	2050/20 = -102 psf
	M_o	$167.7^{'k}$	$53.7^{'k}$	$85.9^{'k}$	136
End span	M^-_{int}	$(0.70)M_o = 117.4^{'k}$	$37.6^{'k}$	$60^{'k}$	
	M^+	$(0.50)M_o = 83.9^{'k}$	$26.9^{'k}$	$43^{'k}$	
	M^-_{ext}	$(0.30)M_o = 50.3^{'k}$	$16.1^{'k}$	$25.8^{'k}$	
Int span	M^-	$(0.65)M_o = 109^{'k}$	$34.9^{'k}$		$88^{'k}$
	M_+	$(0.35)M_o = 58.7^{'k}$	$18.8^{'k}$		$47.6^{'k}$

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 4/12

- Stress Immediately after jacking (DL + BAL)
  - Midspan: $f_{top} = (-M_{DL} + M_{bal})/S - P/A$
    $f_{bot} = (+M_{DL} - M_{bal})/S - P/A$
    - Interior: $f_{top} = \frac{(-58.7 + 47.6)(12)(1000)}{2560 in^3} - 166psi = -218$ psi comp $< 0.6f'_{ci} = 1800psi$ OKAY!
      $f_{bottom} = \frac{(58.7 - 47.6)(12)(1000)}{2560} - 166 = -114$ psi comp $< 1800psi$ OK!
    - End: $f_{top} = \frac{(-88.9 + 43)(12)(1000)}{2560} - 166 = -358$ psi comp. $< 1800psi$ okay!
      $f_{bot} = \frac{(88.9 - 43)(12)(1000)}{2560} - 166 = 26psi$ tension $< 3\sqrt{f'_c} = 164psi$ OK!
  - Support: $f_{top} = (M_{DL} - M_{bal})/S - P/A$
    $f_{bot} = (-M_{DL} + M_{bal})/S - P/A$
    $f_{top} = \left[\frac{(117.4 - 60)(12)(1000)}{2560}\right] - 166 = 103$ psi tension $< 164psi$ OKAY!
    $f_{bot} = \left[\frac{(-117.4 + 60)(12)(1000)}{2560}\right] - 166 = -435$ psi comp $< 1800psi$ OKAY!

- 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: $f_{top} = \frac{(-58.7 - 18.8 + 47.6)(12)(1000)}{2560} - 166 = -306$ psi (C) $< 2250psi$ OK! $0.45f'_c$
      $f_{bot} = \frac{(58.7 + 18.8 - 47.6)(12)(1000)}{2560} - 166 = -26$ psi (C) $< 2250psi$ OK!
    - End Span: $f_{top} = \frac{(-88.9 - 26.9 + 43)(12)(1000)}{2560} - 166 = -483psi$ (C) $< 2250psi$ OKAY!
      $f_{bot} = \frac{(88.9 + 26.9 - 43)(12)(1000)}{2560} - 166 = 152psi$ (T) < 424 psi OKAY

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 5/12

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

$$f_{top} = \frac{(117.4 + 37.6 - 60)(12)(1000)}{2560} - 166 = 279 \text{ psi (T)} < 424 \text{ psi OKAY!}$$

$$f_{bot} = \frac{(-117.4 - 37.6 + 60)(12)(1000)}{2560} - 166 = -611 \text{ psi (C)} < 2250$$

* All stresses are within the permissible code limits *

ULTIMATE STRENGTH

- Determine factored moments
  The primary post tensioning moments, M_1, vary along the length of the span.

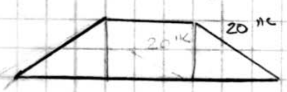
  $M_1 = P(e)$

  $e = 0"$ @ exterior support
  $e = 3.0"$ @ interior support (NA to center of tendon)

  $$M_1 = \frac{(320^k)(3.0")}{12"} = 80^{'k}$$

  The secondary post tensioning moments, M_{sec}, vary linearly between supports

  $M_{sec} = M_{bal} - M_2$
  $= 60 - 80 = -20^{'k}$



  The typical load combo for ultimate strength design
  $M_U = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

  At midspan: $M_U = 1.2(83.9) + 1.6(26.9) + 1.0(-10) = 134^{'k}$
  At support: $M_U = 1.2(-117.4) + 1.6(-37.6) + (1.0)(-20) = -221^{'k}$

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

  Positive moment region
  Interior span: $f_t = -26 < 2\sqrt{f'_c} = 2\sqrt{5000} = 141$ psi

  End span: $f_t = 152 > 2\sqrt{f'_c} = 2\sqrt{5000} = 141$ psi

TECH 2 | 40 BOND | OPTION # 3 - POST-TENSIONED | 6/12

Minimum positive moment reinf. req (ACI 18.9.3.2)

$y = f_t / (f_t + f_c) h$

$= \left[\frac{152}{(152 + 483)}\right] 8'' = 1.91 \text{ in}$

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

$= \frac{(83.9 + 26.9)}{2560} (0.5)(1.91'')(20')(12'')(42'') = 119^k$

$A_{s,min} = N_c / 0.5 f_y = 119^k / (0.5)(60 \text{ ksi}) = 3.97 \text{ in}^2$

$= 3.97 \text{ in}^2 / 20' = 0.1985 \text{ in}^2/\text{ft}$

Use #4 @ 12" o.c. bottom = 0.20 in²/ft
Min. length shall be 1/3 clear span & centered in positive moment region (ACI 18.9.4.1)

- Neg. moment region:

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

- Interior Supports

$A_{cf} = \max. (8'')[(25, 20](12)$

$A_{s,min} = 0.00075 (2400) = 1.8 \text{ in}^2$

10 #4 top (2.0 in²)

- Exterior Supports

$A_{cf} = \max (8)[(25, 20)](12'')$

$= 0.0075 (2400) = 1.8 \text{ in}^2$

10 #4 top (2.0 in²)

- Must span a minimum of 1/6 the clear span on each side of support (ACI 18.9.4)
- At least 4 bars req'd in each direction (ACI 18.9.3.3)
- Place top bars w/in 1.5h away from the face of support on each side (ACI 18.9.3.3) = 1.5(8") = 12"
- Max. bar spacing is 12" (ACI 18.9.3.3)

- Check min. reinf. if it is sufficient for ultimate strength

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

d = effective depth

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

$f_{ps} = f_{se} + 10{,}000 + (f'_c bd) / 300 (A_{ps})$ for slabs w/ $l/h > 35$ (ACI 18.7.2)

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 7/12

$f_{ps} = 174{,}000\text{psi} + 10{,}000 + \left[\frac{(5000)(20)(12)d}{300\,(1.836)} \right]$

$= 184{,}000 + 2179\,d$

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

At supports

$d = 8'' - 3/4''\text{cc} - \frac{1}{2}(1/2'') = 7''$

$f_{ps} = 184{,}000 + (2179)(7) = 199{,}253\ \text{psi}$

$a = \frac{(2.0\,\text{in}^2)(60\text{ksi}) + 1.836\,\text{in}^2\,(199\text{ksi})}{0.85(5)(20')(12'')} = 0.476$

$\phi M_n = 0.9\left[2.0\,\text{in}^2\,(60\text{ksi}) + (1.836)(199)\right]\left(7 - \frac{0.476}{2}\right)/12''$

$= 252^{'k} > 221^{'k}$

Minimum reinf. OKAY! 10 #4 top @ int. & ext. support

when reinforcement is provided to meet ultimate strength req'ts, the minimum lengths must also conform to the provision of ACI 318-08

At midspan (end)

$d = 8'' - 1\tfrac{1}{2}'' - (1/2)(1/2'') = 6\tfrac{1}{4}''$

$f_{ps} = 184{,}000\ \text{psi} + 2179(6.25) = 197619$

$a = \frac{[(3.97\,\text{in}^2)(60\text{ksi}) + (1.836\,\text{in}^2)(197.6)]}{(0.85)(5)(20)(12)} = 0.590$

$\phi M_n = 0.9\left[3.97(60) + (1.836)(197.6)\right]\left(6.25 - \frac{0.590}{2}\right)/12$

$= 298^{'k} > 134^{'k}$ Minimum reinf OKAY!

#4 @ 12" o.c. bottom at end spans.

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 8/12

FRAME B CALCULATIONS

$A = bh = (25')(12'')(8'') = 2400\ in^2$

$S = \frac{bh^2}{6} = \frac{(25')(12'')(8'')^2}{6} = 3200\ in^2$

Balanced load $w_b = (65\ psf)(25') = 1625\ plf = 1.63\ k/ft$

Force needed to counteract load in end bay $P = \frac{w_b L^2}{8 a_{end}} = \frac{(1.63^k/ft)(20')^2}{8\left(\frac{3.75}{12}\right)} = 261^k$

• # of tendons $= \frac{261^k}{26.6/tendons} = 9.81 \rightarrow 10$ tendons

• Actual force $P_{act} = (10)(26.6) = 266^k$

• Balance load for end span
$w_b = \left(\frac{266}{261}\right)(1.63) = 1.66\ k/ft$

• Actual precompression stress
$P_{act}/A = \frac{(266^k)(1000)}{2400} = 111\ psi < 125\ psi$ min

$\therefore P_{act}/A = 125\ psi$

$125\ psi\,(2400\ in^2) = 300{,}000 = 300^k \rightarrow 11.27 \rightarrow 12$ tendons

$(12\ tendons)(26.6^k) = 320^k$

• check interior span
$P = \frac{(1.66\ k/ft)(20')^2}{8(6/12)} = 166^k < 320^k$

$w_b = \frac{(320)(8)(6/12)}{(20)^2} = 3.2\ k/ft$

$w_b = 3.2/(0.125)(25) = 3.2/3.125 = 102\%$ (w/in 5% okay)

Effective prestress force, $P_{eff} = 320^k$

- Check slab stresses (moments from DDM)
$M_0 = \frac{1}{8}(28')(20 - 22/12)^2 w$

			DEAD	LIVE	BAL (END)	BAL (int)
	w		125 psf	40 psf	1630/25 = 66 psf	3200/25 = 128
	M_0		129^k	41^k	68^k	132^k
End	M^-_{ext}	$0.7(M_0)$	90.3	28.7	47.6	
	M^+	$0.5(M_0)$	64.5	20.5	34	
	M^-_{int}	$0.3(M_0)$	38.7	12.3	20.4	
int	M^-	$0.65(M_0)$	83.9	26.7		85.8
	M^+	$0.35(M_0)$	45.2	14.4		46.2

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Stress immediately after Jacking (DL + BAL)

- Midspan

- Int: $f_{top} = \frac{(-45.2 + 46.2)(12)(1000)}{3200} - 125\,psi = -121.25\,psi\ (C) < 1800\,psi$ OKAY!

$f_{bottom} = \frac{(45.2 - 46.2)(12)(1000)}{3200} - 125 = -128.8\,psi\ (C) < 1800\,psi$ OKAY!

- End: $f_{top} = \frac{(-64.5 + 34)(12)(1000)}{3200} - 125 = -239\,psi\ (C) < 1800\,psi$ OKAY!

$f_{bot} = \frac{(64.5 - 34)(12)(1000)}{3200} - 125 = -11\,psi\ (C) < 1800\,psi$ OKAY!

- Support - $f_{top} = \frac{(90.3 - 47.6)(12)(1000)}{3200} - 125 = 35\,psi\ (T) < 164\,psi$ OKAY!

$f_{bottom} = \frac{(-90.3 + 47.6)(12)(1000)}{3200} - 125 = -285\,psi\ (C) < 1800\,psi$ OKAY

Stress @ service load (DL + LL + BAL)

- Midspan - Interior - $f_{top} = \frac{(-45.2 + 14.4 + 46.2)(12)(1000)}{3200} - 125 = -175\,psi\ (C) < 2250\,psi$ OKAY

$f_{bot} = \frac{(45.2 + 14.4 - 46.2)(12)(1000)}{3200} - 125 = -75\,psi\ (C) < 2250\,psi$ OKAY!

- Ext - $f_{top} = \frac{(-64.5 - 20.5 + 34)(12)(1000)}{3200} - 125 = -816\,psi\ (C) < 2250$ OKAY!

$f_{bot} = \frac{(64.5 + 20.5 - 34)(12)(1000)}{3200} - 125 = 66\,psi\ (T) < 424\,psi$ OKAY

- Support - $f_{top} = \frac{(90.3 + 28.7 - 47.6)(12)(1000)}{3200} - 125 = 143\,psi\ (T) < 424\,psi$ OK!

$f_{bot} = \frac{(-90.3 - 28.7 + 47.6)(12)(1000)}{3200} - 125 = -393\,psi\ (C) < 2250\,psi$

* All stresses are w/in the permissible code limits *

ULTIMATE STRENGTH

- Determine factored moments

The primary post tensioning moments, M_1, vary along the length of the span.

TECH 2 | 40 BOND | OPTION #3 - POSTTENSIONED | 10/12

$M_1 = P(e)$
$e = 0''$ @ ext. support
$e = 3.0''$ @ int. support (NA to center of tendon)

$M_1 = \frac{(320)(3.0)}{12} = 80^{1K}$

$M_{sec} = M_{bal} - M_1$
$= 47.6 - 80 = -32.4^{1K}$

The typical load combo for ultimate strength design
$M_U = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{sec}$

At midspan $= 1.2(64.5) + 1.6(20.5) + 1.0(-32.4/2) = 94^{1K}$
At support $= 1.2(-90.3) + 1.6(-28.7) + 1.0(-32.4/2) = -171^{1K}$

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

Positive moment region
- Int. span $= f_t = -75 < 2\sqrt{5000} = 141$ psi

- End span $= f_t = 143 > 141$ psi

- Minimum positive moment reinf. req'd (ACI 18.9.3.2)
$y = f_t / (f_t + f_c)h$

$= \left[\frac{143}{143+316}\right] 8'' = 2.49$ in

$N_c = \left(\frac{64.5+20.5}{3200}\right)(0.5)(2.49)(25')(12'')(12'') = 119^K$

$A_{s,min} = (119) / 0.5(60) = 3.97 in^2$

$= 3.97 in^2 / 25 = 0.153$

#4 @ 12 → $0.20\ in^2/ft$
Min length shall be 1/3 clear span & centered in positive moment region (ACI 18.9.4.1)

- Neg moment region

$A_{s,min} = 0.00075 A_{cf}$ (ACI 18.9.3.3)
- Int. supports | Ext supports
$A_{cf} = \max(8'')(25')(12'') = 2400$
$A_{s,min} = 0.00075(2400) = 1.8$
10 #4 top (2.0)

TECH 2 | 40 BOND | OPTION # 3 : POST-TENSIONED | 11/12

- Must span a minimum of 1/6 the clear span on each side of support (ACI 18.9.4)
- At least 4 bars req'd in each direction (ACI 18.9.3.3)
- Place top bars w/in 1.5h away from the face of the support on each side (ACI 18.9.3.3) = 1.5(8") = 12"
- Max. bar spacing is 12" (ACI 18.9.3.3)

- check min reinf, if it is sufficient for ultimate strength

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

d = effective depth

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

$f_{ps} = f_{se} + 10000 + (f'_c b d)/300(A_{ps})$ for slabs w/ $l/h > 35$ (ACI 18.7.2)

$= 174{,}000 + 10000 + \left[\frac{5000(25)(12)d}{300(1.836)}\right]$

$= 184{,}000 + 2723d$

$a = A_s f_y + A_{ps} f_{ps} / 0.85 f'_c b$

At supports

d = 7"

$f_{ps} = 184{,}000 + 2723(7) = 203061$

$a = \frac{(2.0 in^2)(60) + 1.836(203)}{0.85(5)(25)(12)} = 0.386$

$\phi M_n = 0.9[2(60) + (1.836)(203)](7 - \frac{0.386}{2}) / 12" = 252^{1K} > 171^{1K}$

$\boxed{\text{Min reinf okay! 10\#4 top @ int. \& ext supports}}$

- when reinf is provided to meet ultimate strength req'ts the min lengths must also conform to ACI 318-08

At midspan

$d = 8 - 1\frac{1}{2} - \frac{1}{2}(\frac{1}{2}) = 6.25"$

$f_{ps} = 184{,}000 + 2723(6.25) = 201019$

$a = \left[\frac{(3.97)(60) + (1.836)(201)}{0.85(5)(25)(12)}\right] = 0.476$

$\phi M_n = 0.9[3.97(60) + 1.836(201)]\left[6.25 - \frac{0.476}{2}\right] / 12 = 273^{1K} > 94^{1K}$

$\boxed{\text{Min reinf okay; \# 4 @ 12 o.c. bottom @ end spans}}$

12 tendons banded at columns in short direction (frame B)
12 tendons uniformly distributed in long direction (frame A)
Min reinf (mild steel as noted above: 10#4 top @ int & ext supports, #4 @ 12 bottom at end spans)

TECH 2 | 40 BOND | OPTION #3 - POST-TENSIONED | 12/12

SHEAR • Column 2C + 8"

ACI 318-08 § 11.11.2.2
At columns of two way prestressed slabs + footings that meet req'ts of 18.9.3

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

w/ β_p = smaller of 3.5

$$\left(\frac{\alpha_s d}{b_o} + 1.5\right) = \left(\frac{40(7.0)}{119.4} + 1.5\right) = 3.85$$

$= \boxed{3.5}$

$f_{pc} = (166 + 125)/2 = 146$ psi
$b_o = 119.5$
$d = 8" - 3/4"$ cc $- \frac{1}{2}(\frac{1}{2}\phi$ tendon$) = 7.0"$
V_p = vertical component of all effective stress
= 0 to be conservative as per R11.11.2.2

$$V_c = (3.5(1.0)\sqrt{5000} + 0.3(146\text{ psi}))(119.5)(7.0")$$
$$= 244^k$$

$$\phi V_c = 0.75(244^k) = 183^k$$

$$V_u = (0.214\text{ ksf})\left(494 - \left(\frac{22}{12}\right)^2\right) = 105^k$$

$$\phi V_c = 183^k > 105^k = V_u \qquad \text{GOOD!}$$

* No additional shear reinf. needed *

DEFLECTION

Due to the complex nature of this calculation, normally done w/ computer analysis, a value for deflection for this post-tensioned system will not be evaluated.

Based on staying within the preliminary slab/depth ratio of 40-45 deflection should not be an issue, especially with the help of the balanced load from the post tension tendons.

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