OFFICE BUILDING-G

Eastern United States

Technical Report II

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Structural Option

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Executive Summary

Technical Report II is an analysis of the existing floor design compared to three alternative floor systems for Office Building-G. Alternative designs were chosen for analysis based on their ability to maintain the architectural features, serviceability requirements, and constructability. The three alternative systems designed are: precast pre-stressed hollow core plank, composite metal deck on steel girders, and a two-way flat plate slab. These systems were compared to the existing design to determine if the existing floor design is the most logical for Office Building-G. Considerations of floor weight, constructability, architectural impact, cost and feasibility were taken into account when determining the best system.

When all of the systems were compared to each other, it was determined that the existing floor design of a one-way slab spanning between post-tensioned is the best floor system for Office Building-G. The proposed alternatives had certain advantages over the existing design in the categories of cost, ease of construction, and weight. However, each system had either constructability issues, large vibrations or impacted the architecture too greatly to be considered as a reasonable alternative. The composite metal deck proposal would be a very reasonable floor system to use based on the geometry of Office Building-G but, the material of the superstructure eliminated this as an option due to the connection between steel and concrete.

Throughout the analysis and discussion of the systems, it was concluded that in order to effectively compare the chosen alternatives, a redesign of the superstructure should also be considered. Connecting hundreds of steel girders to cast-in-place concrete columns would be very difficult, time consuming, and expensive to construct, effectively eliminating this proposed system as a reasonable alternative. From a similar point of view, erecting precast hollow core plank on cast-in-place columns and beams creates a scheduling issue making a precast system an unlikely choice. In order for these two systems to have an accurate comparison to the existing design, steel frame and precast member designs should also be analyzed.

Introduction

Due to owner restrictions, the building name, location and tenant of Office Building-G cannot be disclosed. Neighboring an existing metro station, this 14 story building will become one the tallest of the modest skyline. Beneath the superstructure is a below grade, 4-story parking garage with space for 662 cars. On the first two floors of the building, a larger floor plan is used to accommodate for rentable space for retail, a restaurant, a bank and a loading dock. Typical floors have a square footage of 25,376 sf with a floor to floor height of 12'-3". The roof of the mechanical penthouse is 195 ft above grade and the gross square footage of the superstructure and garage combined is 649,461 sf.

The southern façade of the building is a curved glass curtain wall, breaking the mold of precast concrete panels the other three sides of the building follow. There is a setback on the first floor of the glass façade, exposing the exterior row of columns. On the first and second floor, the restaurant has a glass façade with concrete pilasters between the panes of glass.

Gravity System

Gravity loads are carried down the building through a combination of interior and exterior concrete columns and a shear wall core. The typical floor system is a cast-in-place concrete one-way slab. Thickness changes based on loading conditions but the typical floor is a 7", 5000 psi normal weight concrete slab. On the first floor, there is a 12" concrete slab designed for fire separation between the parking garage and superstructure. The slab system carries the loads to post-tensioned concrete beams with spans between 41'-5" and 45'-1 1/4".

The post tensioned beams range in width from 18" to 48" and have a maximum depth of 24". In Office Building-G, the typical girder is 18" deep by 48" wide. Forces in the beams are between 162 kips to 675 kips. These beams collect the floor loads from the slab and distribute their reactions to the columns supporting them.

Rectangular and round concrete columns then transfer the loads down the strictly followed grid. Typical floors have columns sizes of 24" x 24", 24" x 30", and 30" diameter. Smaller columns are used in the mechanical penthouse due to the much lower loads they are carrying. On above grade floors, higher strength concrete is placed below columns and shear walls in the slab to accommodate for any possibility of punching shear. In the parking garage, 8" drop panels are used instead of the different concrete strengths. The typical floor plan shown in Figure 1 below highlights the post-tensioned beams in yellow, the reinforced beams in purple, shear walls in green and blue, and the columns in red.

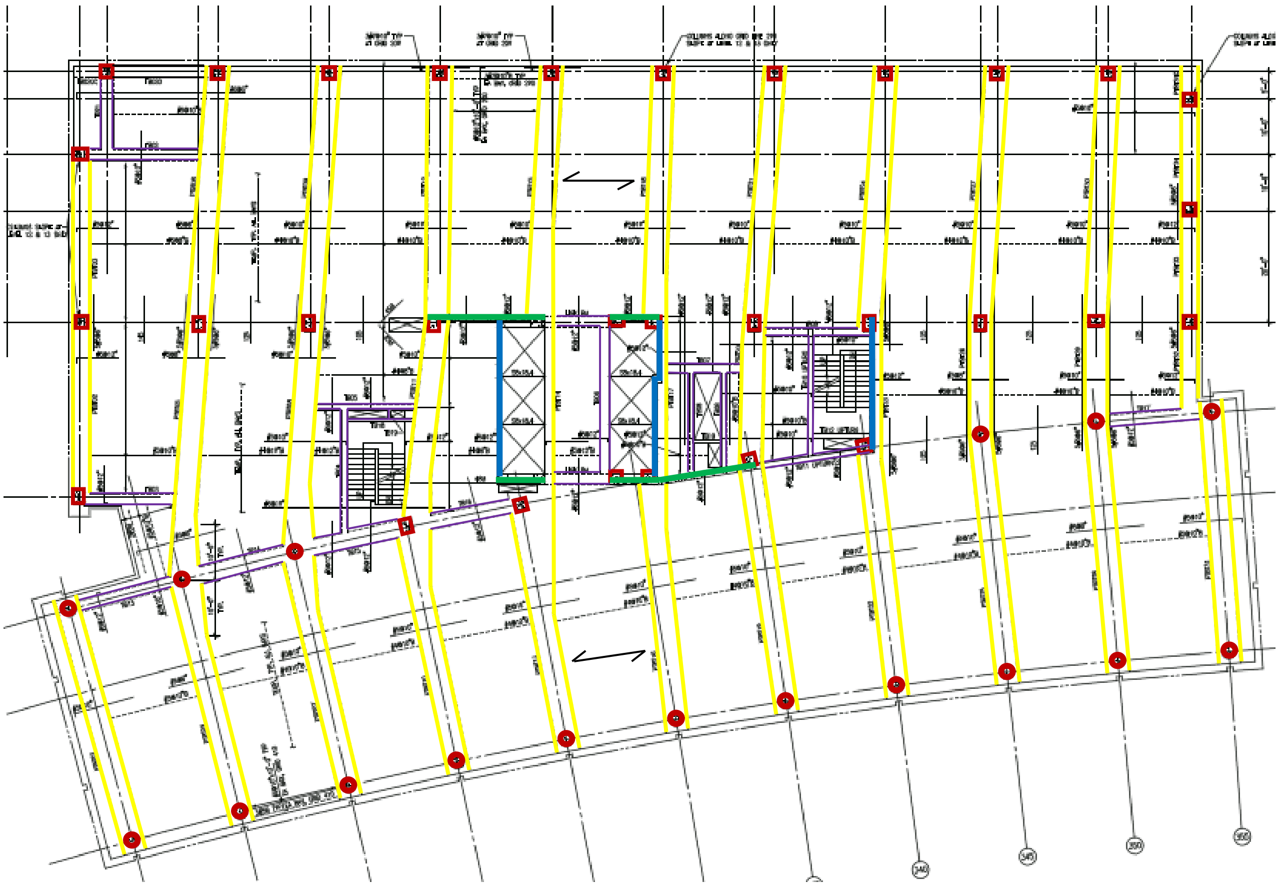


Figure 1

The above floor plan displays the skewed nature of Office Building-G. This condition was ignored during the design of the existing system as well as the alternative proposals. However, when comparing the systems at the end of the report, the advantages and disadvantages of the alternative systems on a skewed grid was considered.

Lateral System:

Wind and seismic forces are resisted by an internal shear wall core. The core is made of reinforced concrete walls which have a consistent floor plan from the bottom floor of the parking garage up to the slab of the roof. Basement shear walls were designed with f'c = 10,000 psi, levels 1-4 use f'c = 8,000 psi, and levels 5-14 use f'c = 5,000 psi. Precast concrete beams attached to concrete columns using precast lateral connections provide the required resistance for the mechanical penthouse and elevator machine room.

Lateral forces are engaged by the shear walls through the use of floor diaphragms. The building façade collects wind forces that are then transferred to the respective floor diaphragm. Forces then travel through the diaphragm until the shear walls are engaged, at which point the forces are distributed based on the relative stiffness of the walls.

Foundation System:

Schnabel engineering performed a geotechnical study for the location of Office Building-G which determined the possible foundation systems as spread footings, caissons or geopiers. The engineers of SK&A decided to use a system of spread footings under the columns, shear walls and along the perimeter concrete bearing wall. Square footage and depth of the footings are based on the load carrying capability of the soil and the vertical load on the column.

Service loads on the columns ranged greatly depending on whether or not the column extended up into the superstructure of the building. Based on the structure above the foundation, the load capacity of soil was determined to support a range of 3,000 psf to 10,000 psf. Loads on the footings varied between 60 kips to 3075 kips, once again depending on which part of Office Building-G they are supporting.

Design Loads and Deflection Limits

Superimposed Dead Loads		
Load Description	Load Location	Design Load
Superimposed	All	5 - Mech/Elec/Ceiling
Curtain Wall	Levels 1-14	25 - Vertical Surface

*Take note that the superimposed dead load has been changed from 15 psf to 5 psf. This change was made based the one-way slab design check that determined 5 psf is most likely the value used by the design engineer.

Floor Live Loads			
Load Description	Load Location	Design Load (psf)	ASCE 7-10 Load (psf)
Office	Levels 1-14	80 20 - Partitions	80

Live Load deflection limitation will be L/360

Service Load deflection limitation will be L/240

Construction Load deflection limitation will be L/180

Structural Materials

Structural Materials			
Material	**Element**	**Level**	**Strength**
Cast-in-Place Concrete	Spread Footings	Foundation	f'_c = 10,000 psi
			f'_c = 6,000 psi
			f'_c = 3,000 psi
	Foundation Walls	B4	f'_c = 5,000 psi
		B3-B1	f'_c = 4,000 psi
	Shear Walls	B4-B1	f'_c = 10,000 psi
		L1-L4	f'_c = 8,000 psi
		L5-L7	f'_c = 6,000 psi
		L8-L14	f'_c = 5,000 psi
	Columns	B4-B1	f'_c = 10,000 psi
			f'_c = 6,000 psi
		L1-L4	f'_c = 10,000 psi
			f'_c = 8,000 psi
			f'_c = 6,000 psi
		L5-L7	f'_c = 6,000 psi
		L8-Roof	f'_c = 5,000 psi
	Reinforced Beams	ALL	f'_c = 5,000 psi
	Post-Tensioned Beams	ALL	f'_c = 5,000 psi
Tendons	Post-Tensioned Beams	ALL	F_u= 270 ksi
Reinforcing Steel	Concrete	ALL	F_y = 60 ksi
Structural Steel	Elevator Framing - A36	ALL	F_y = 36 ksi
	Bolts - A325	ALL	F_u= 120ksi

Code and Design Requirements

Design Codes:

National Model Code:

Local building code based on the 2006 International Building Code

Sections: 1603.1.1-1603.1.7, 1603.2, 1607.11, 1608.1, 1608.7, 1608.8, 1609.1

Design Codes:

American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete and Commentary

ACI 301, Specifications for Structural Concrete for Buildings

ACI 347, Standard Recommended Practice for Concrete Formwork

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Thesis Codes:

National Model Code:

International Building Code, 2006

Design Codes:

ACI 318-08, Building Code Requirements for Structural Concrete and Commentary

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Structural Standards:

American Standards of Civil Engineers (ASCE), ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

Existing Floor Design: One-Way Slab on Post-Tensioned Girders

The existing floor system was analyzed as a control to compare each of the alternate floor designs against. Figure 2 below is a typical bay for the northern half of the floor plan and was chosen for analysis because it has the longest girder spans of Office Building-G. The bay is 20' x 45' with a 7" normal weight cast-in-place reinforced concrete one-way slab supported by 18" x 48" post-tensioned concrete girders. The girders and slab were poured integrally resulting in a total structural floor thickness of 18". Reference Appendix A for the calculations verifying the current design.

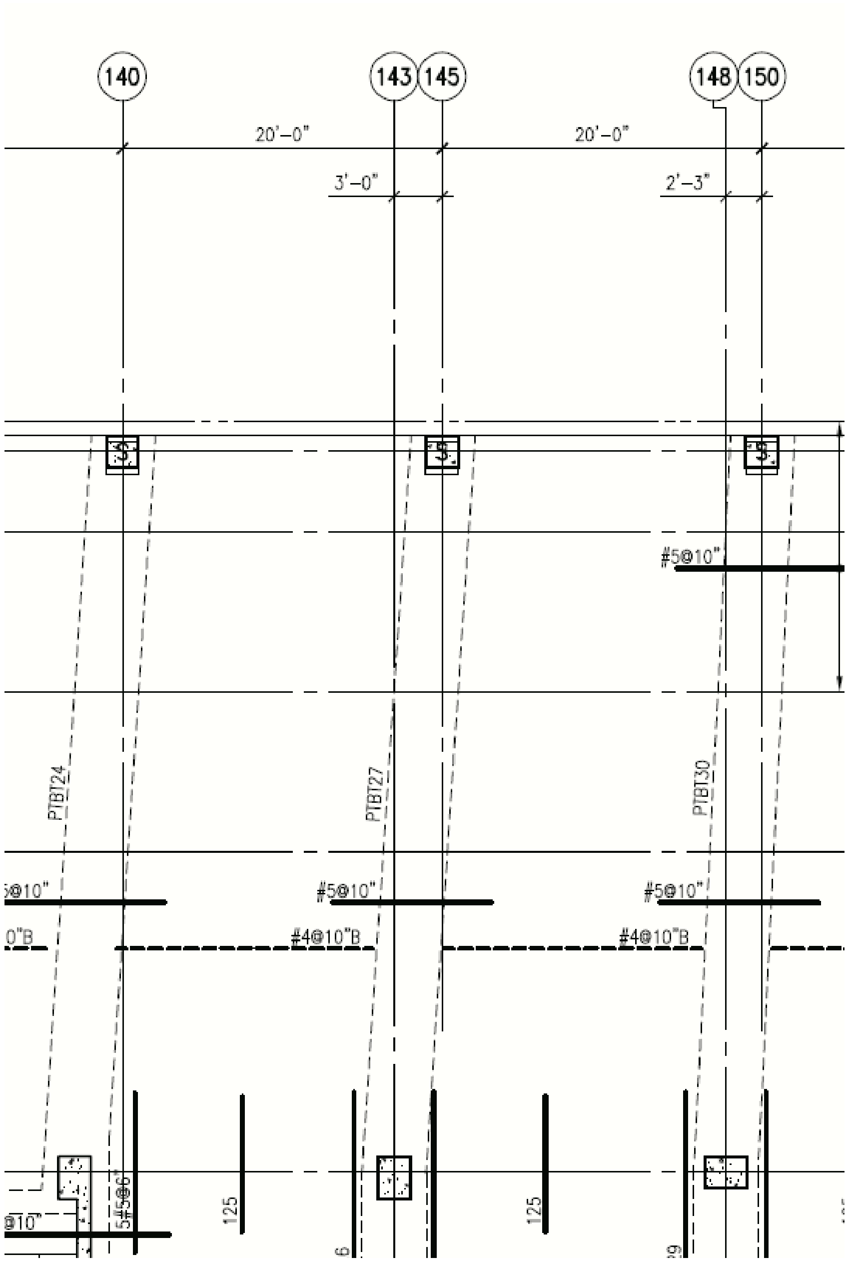


Figure 2

Advantages and Disadvantages

The 45’ x 20’ bays of Office Building-G are very large for a cast-in-place concrete building. Post-tensioning allows for longer clear spans, thinner slabs, fewer beams and a small structural depth. All of these advantages result in less concrete than a regularly reinforced building. The implication of a lighter building can have a significant reduction in seismic loads as well as the foundation loads.

A structural depth of only 18” gives the building a very modest floor-to-floor height of 12’-3”. Not only does this result in less concrete but saving corresponding to a shorter building translate to considerable savings in the mechanical systems and façade costs.

All cast-in-place concrete structures have certain characteristics associated with them. An example of an advantage is they do not require additional fireproofing due to concrete resisting heat so effectively. This is beneficial because additional time and money does not need to be spent on going back through and spraying all of the structural members to reach the required two hour rating. Another advantage of post-tensioning and cast-in-place construction is that beams and slabs can be continuous, allowing beams to run continuously from one end of the building to another. This type of construction is much more efficient that one in which beams go from one column to the next.

A disadvantage of cast-in-place concrete is formwork. Formwork takes time to construct, move throughout a construction site and is a large part of the cost associated with cast-in-place buildings. This system in particular has more complicated formwork than other systems like flat plate slabs, driving up the cost even more.

Post-tensioned members require much more skill during the construction compared to reinforced concrete buildings. The system relies on anchors to keep the tendons in tension and these anchorages can be difficult and time consuming to install. A more specialized and experienced subcontractor would have to be hired and would charge a higher rate based on their skill set.

Alternate Flooring Systems

Precast Pre-Stressed Hollow Core Slab

Hollow core slabs were chosen for an alternative floor system for Office Building-G because of the long spans between framing members. 45 ft is a difficult span to achieve with concrete construction but pre-stressed hollow core slabs are a strong, lightweight option. The specific hollow core system selected is a 16" deep Standard Spancrete, 150 ksi strand with a 2" concrete topping. When analyzing this system it was necessary to add beams on the interior and exterior to support the ends of the panels. See the modified floor plan in Figure 3 below and the design calculations in Appendix B.

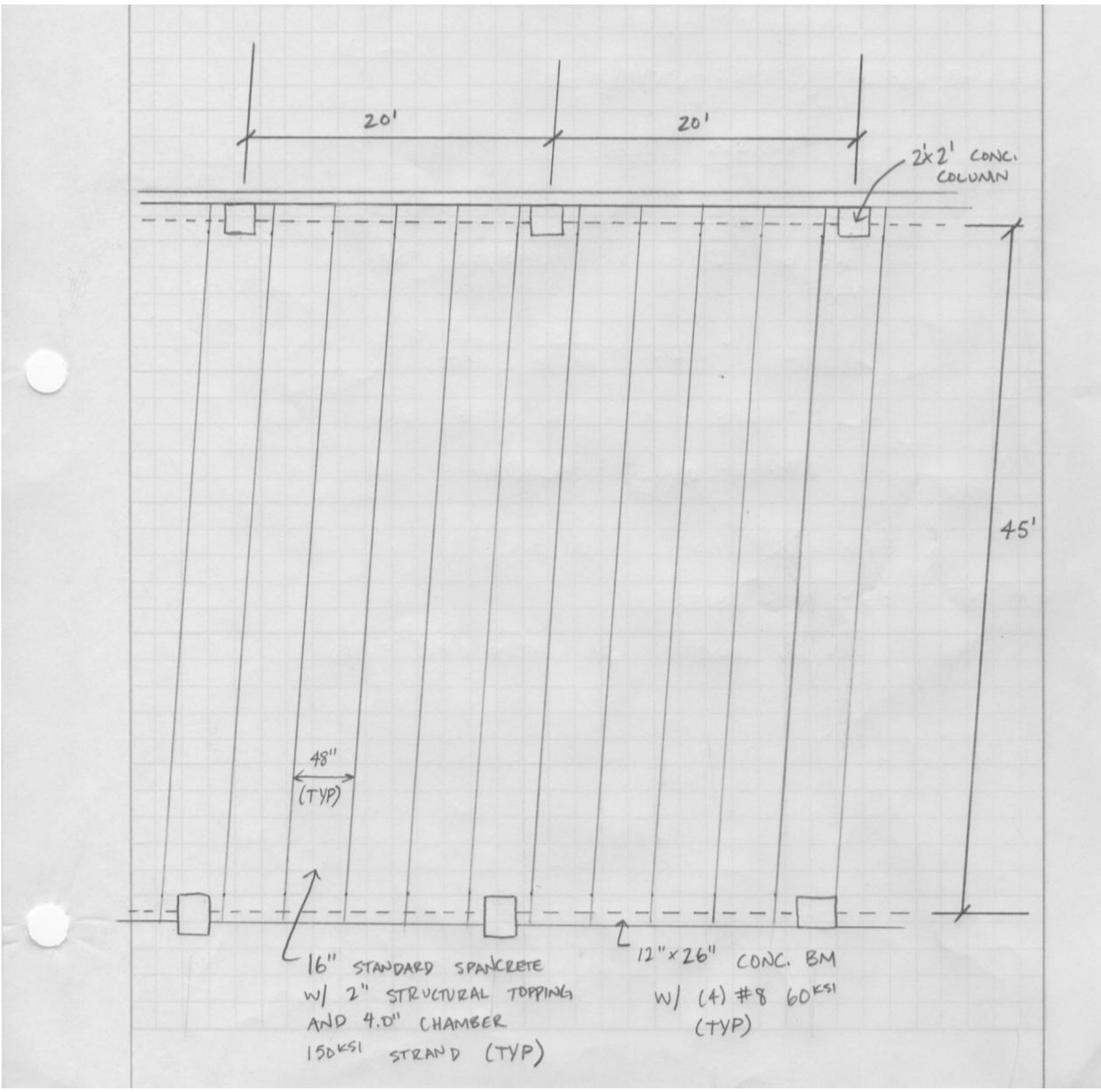


Figure 3

Advantages and Disadvantages:

A 16” depth of the plank allows for a straight forward ceiling cavity design. Sections show a proposed cavity of 3 ft after tenant fit outs and a structural depth of 18”, 16” plus 2” topping, gives the MEP systems plenty of room to provide access to the necessary parts of the building.

Another advantage of the system is the performance of concrete in fires. Hollow core systems have very good retention of material strength and containment of fire. The system was given a fire rating of 2 hours, eliminating the need for additional spray on fireproofing, saving time and material costs. (Note: The manufacturer of the Spancrete gave the system a 4 hour rating but 2 hours is more likely.)

Low construction costs are the greatest advantage hollow core slabs have to offer. In general, these systems can be installed year-round, do not require shoring, and provide a work surface immediately after being set. These advantages can provide a large amount of savings which are expressed in the cost per square foot of the design.

The tendons within the planks cannot be cut due to the large amount of tension already in them. Due to this, precast pre-stressed floor systems can create difficulty around stairs, elevator shafts or any other opening in the slab. A precast system would be particularly difficult to work with on Office Building-G because the skewed structural layout. Also, the southern façade is curved is plan, creating the need for a curved floor plan. Precast hollow core planks are molded with 48” wide ends. This would create an unusual condition on the southern façade. Beams supporting the planks on the southern edge could be designed to be wider to support the entire width of the member. Concrete would then to be poured to fill the void in the floor left by the out of plumb meeting of the precast planks and the beam supporting them.

A disadvantage to this system is the need to chamber the pre-stressed panels to meet the set deflection requirements. A deflection calculation determined the deflection at the midspan of the slab to be 5.89”. Cambering the panels 4.0” does effectively satisfy the deflection requirements but it greatly increases the chances of vibration in the floor. Chambers do not make the member any stiffer so the total possible movement of the floor system remains at 5.89”, a noticeable amount. The possibility of vibration should definitely be considered but it should be mentioned that this deflection was reached using the maximum loads the slab will ever support. The chance of the floor being fully loaded with 100 psf live load is low, creating a smaller chance of having noticeable vibrations.

Composite Metal Deck on Steel Girders

This floor system was analyzed to try to reduce construction time and the weight of the floor system. A reduced floor weight would decrease the dead load Office Building-G is currently designed to withstand. This reduces the size of the vertical members as well as the seismic forces the lateral system would have to resist. The design chosen was is EDC750 long span deck with a 5" topping of lightweight concrete on W21x73 girders spaced at 20' on center. Figure 4 displays this system and the design calculations can be found in Appendix C.

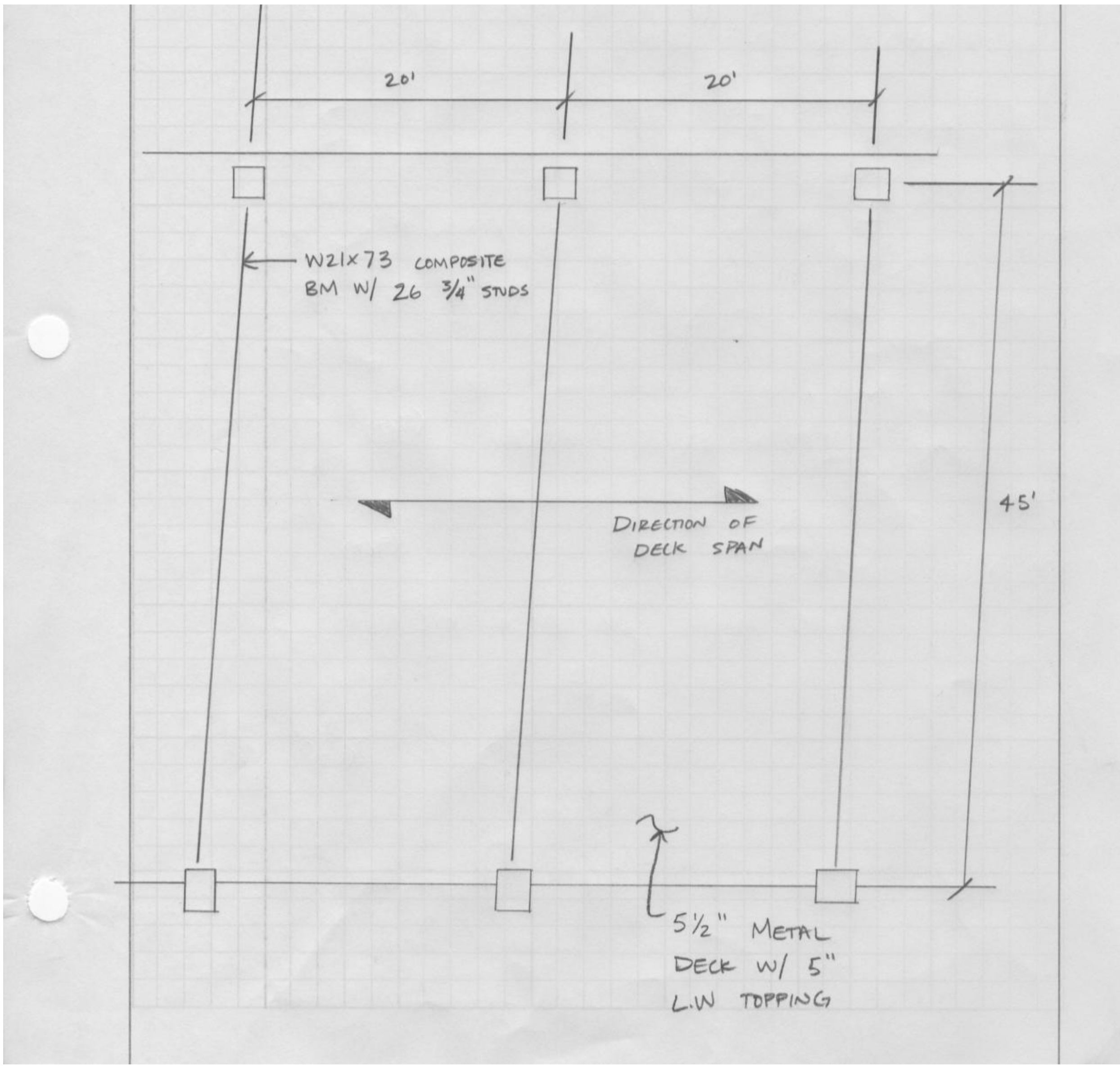


Figure 4

Advantages and Disadvantages:

A major advantage of this system is the short construction time associated with it. Long span *EPIC* metal decks were effectively used so neither formwork nor shoring need to be used during construction, greatly reducing the time and cost of the slab construction. Also, there would not be the additional cost associated with post-tensioning the current girders.

Another advantage to this system is the lightweight design. The existing one-way slab has a dead load of 87.5 psf. When compared to the 52.3 psf of the metal deck, there is a reduction of over 30 psf of dead load. Steel girders are also much lighter than the post-tensioned beams. When the thickness of the slab is not included, the post-tensioned beams have a weight of 137.5 lb/ft, compared to the 73 lb/ft of the steel girders. This reduces the total dead load on the building and these reductions are noticeable in the seismic loads and the size of vertical members.

A major disadvantage to this floor system is the need for additional fireproofing. As the building is designed now, no fireproofing is needed due to concrete's high resistance to fire. The composite metal deck does have a fire rating of 1 hour but there would have to be an additional spray on coating for the deck as well as the girder to meet the IBC requirements.

Another disadvantage to this floor design is the depth of the steel members. The design of the system resulted in a 31.5" deep composite design spanning 45'. This member is not unusually deep for the span of the building but it is significantly deeper than the 18" existing design. Both systems fit within the 36" ceiling cavity but when MEP equipment is taken into account, there is a possibility of an air duct intersecting with the deep steel members. These conflicts can be resolved by cutting through members at strategic locations but this complicates the construction and adds cost to the project.

It is unusual to pour concrete columns and then attach steel girders to them after they reached the necessary strength. Not only would this greatly slow down the construction of project but each column would have to have an embedded plate or a seat for the connection between the beam and column. Throughout the entire building this would amount to hundreds of embedded plates or seat connections, also adding to the cost of the building. If composite steel is the floor system chosen, switching to steel columns should be strongly considered.

Two-Way Flat Plate Reinforced (New Grid)

A concrete building with 45' spans between columns greatly limits the number of floor alternative floor systems available for comparison. Typical cast-in-place concrete buildings are designed with spans between 20' to 30'. In order to analyze additional concrete floor systems, two additional column lines were inserted, cutting the span between columns roughly in half. With more manageable span lengths, a two-way flat plate system was analyzed for current loads and new span lengths. The design of the system, shown in Figure 5, resulted in a 9" thick slab spanning between a typical bay size of 20' x 25'. The new column layout, calculations and specific reinforcing details can be found in Appendix D.

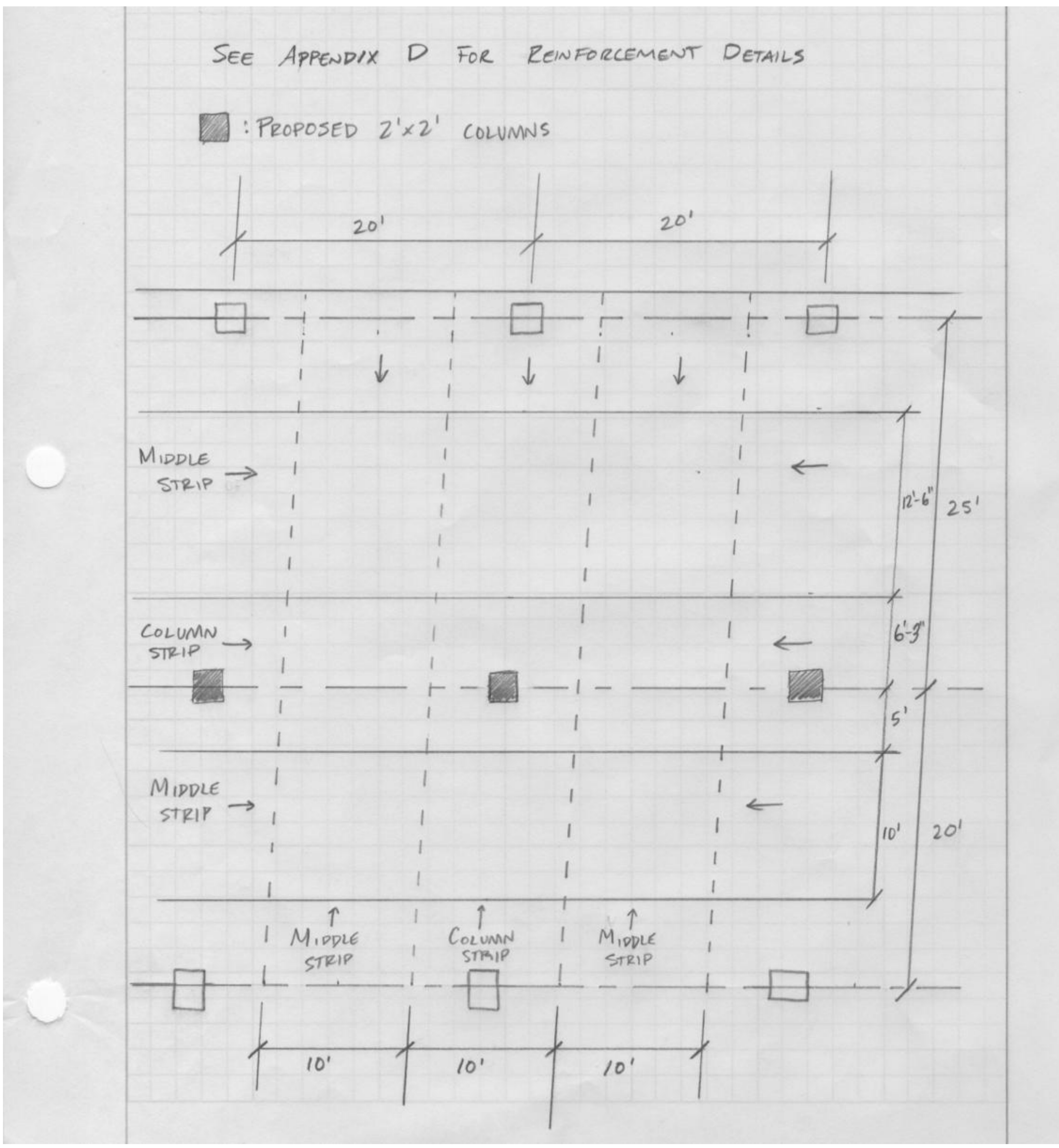


Figure 5

Advantages and Disadvantages:

This system was chosen over other concrete floor designs due to the flexibility of column locations allowed in flat plate systems. Office Building-G has slightly off axis columns, and unique edge conditions and flat plate systems allow for these types of variations. Also, a two-way flat plate system can be designed to eliminate beams between supports and column capitals, greatly simplifying the formwork used.

Another advantage to this system is the 9” depth of the structural system. As mentioned in the hollow core advantages, this gives the other building systems plenty of room in the ceiling cavity to access the entire building. Like the other concrete floor systems, additional fireproofing is not necessary due to the material properties of concrete.

An obvious disadvantage to the proposed flat plate system is the introduction of the new columns. Placing columns in the middle of the existing open floor plan affects the architecture because tenants will have less freedom to use the space as they would with the existing open floor plan. Additional column grids also create more foundations. Without the addition of a transfer girder, the column line will continue through the parking garage to a new set of spread footings. Additional footings will slightly decrease the size of the existing foundation due to the smaller tributary area each element is responsible.

Flat plate slabs are constructed using cast-in-place concrete. Although flat plates require less that other systems, it will require formwork and thus drive up the cost due to the increased labor time associated with this type of construction. Cast-in-place concrete also needs to reach a certain strength before it can be used as a construction surface. The floors above a recently poured slab cannot be worked on because the formwork must be supported by the floor below. This is easily worked around through scheduling but the slab will not be able to be constructed as quickly as other floor systems.

Comparison

The chart below outlines the variety of different considerations in which all of the floor systems were compared to each other. The alternative systems were compared to the existing system to determine their efficiency, impact and feasibility.

Comparison Between Floor Systems				
	Existing	Alternative		
Considerations	One-Way Slab	Composite Metal Deck	Hollow Core	Two-way Flat Plate
Total Structural Depth (in.)	18	31.5	16	9
Constructability	Medium-Difficult	Easy-Medium	Medium	Medium
Foundation Impact	N/A	Reduces Capacity Requirements	Increases Capacity Requirements	Additional footings
Lateral System Impact	N/A	No	Possible	Possible
Floor Weight (psf)	87.5	52.3	130	112.5
Live Load Deflection (in.)	0.49	1.17	5.89	N/A[1]
Chamber (in.)	No	No	4	No
Relative Vibration	Low	Average	Above Average	Low
Fireproofing	No	Yes	No	No
Fire Rating (hrs)	2	2	2	2
Cost ($/ft^2)	13.61[2]	23.30	13.59	13.90
Bay Size	20' x 45'	20' x 45'	20' x 45'	20' x 25'
Architectural Impact	No	No	No	Yes
Feasibility	N/A	Yes	Yes	No
Further Analysis	N/A	Yes	Yes	No

[1] Live load deflection not calculated because minimum slab thickness determined by limitations due to live load deflection requirements

[2] Price does not include cost of post-tensioning beams. PT construction costs are still being researched and will be included in future reports as needed

Constructability

Office Building-G is located in a region of the United States where the majority of high rise buildings are constructed out of concrete due to cheap materials and labor union costs. Due to this fact many of the contractors are familiar with concrete pouring and formwork. Flat plate systems have very simple formwork casts compared to those needed to pour the post-tensioned beams. The existing post-tensioned system will also require a more specialized group to lay out the system effectively. Equipment needed for post-tensioning would have to be stored on site and alterations would have to be made to the construction timeline to account for the concrete to reach sufficient strength for tensioning.

The post-tensioning construction is not the only system which requires special equipment. The steel girders in the composite metal deck system and the hollow core planks would need to be installed with the use of a crane. These floors are predicted to have a much faster construction timeframe when compared to the cast-in-place concrete because neither requires the use formwork or shoring. An additional benefit of the hollow core planks is that once the grout has set, the members can be used as a construction platform. The precast planks allow for fast construction but the square ends of these planks create a unique detail on the curved southern wall.

It would be uncommon to use the composite metal deck and hollow core plank with cast-in-place columns. The construction of the superstructure would be delayed while the concrete columns are reaching an acceptable strength for load to be applied.

Architectural Impacts

A large part in the decision of the alternative systems was trying to stay to the same column grid. Two systems were analyzed which achieve this goal but 45' spans limit the available systems so flat plate slab was analyzed. The system required the long spans to be cut in half, placing a column directly in the middle of the once open floor plan. This is a severe alteration because it limits the freedom the tenants have to fit out the rented space as they choose.

In addition to the floor plans, ceiling plans should also be considered when comparing the floor systems. Post-tensioned girders are a great way to limit the depth of structural members while achieving large spans. As determined in the analysis of the alternative systems, this is a difficult to achieve. Composite girders were designed to have more than a 12 in greater thickness that the post-tension girders. This creates issues with the MEP systems and it would be necessary to cut through the steel girders if the 3 ft ceiling cavity is to be maintained. The hollow core plank has a depth less than the post-tensioned girder but it also has significant deflections. This creates a very good chance of vibrations in the slab and could be interpreted as uncomfortable by the building occupants.

Foundation Impacts

Columns in Office Building-G run from the top level down through the parking garage to the substructure foundations. This is a major reason for the efforts taken to keep the same column layouts as the existing design. The two-way slab requires two additional column lines resulting in either transfer girders to span the new column loads to the existing foundation or additional spread footings.

If a proposed floor has a greater self-weight than the existing system, the foundation will experience higher loads. The composite metal deck effectively reduced the weight of the floor but the hollow core plank and flat plate slab induce a larger load. The difference between the existing design, plank and flat plate is exaggerated in the above table because the weight of the post-tensioned girders is not accounted for.

Lateral System Impacts

Seismic loads are directly related to the weight of a structure. Due to this relationship, switching to the hollow core and flat plate system may cause the current lateral system to be inefficient. The flat plate system is especially concerning because the additional columns are not accounted for in the floor weight recorded above. All of the proposed floor systems will create diaphragms capable of transferring the external lateral forces to the shear wall core. Additional research and a more in-depth analysis of the performance of the alternative floor systems as necessary.

System Cost

RS Means Assemblies Cost Data, 2011 was used to roughly estimate the different slab costs. The most expensive system was found to be the composite metal deck on steel framing. This comparison is not completely accurate because pricing values could not be obtained for the cost associated with the post-tensioned girders. The cost of the one-way slab is shown and with the additional price of the post-tensioning could drive the total cost of the system much closer to the composite metal deck. The most inexpensive alternative is the hollow core plank followed closely by the flat plate slab. However, the additional columns necessary for the flat slab are not included in the price, creating a larger gap between the costs of the two proposals.

Conclusion

Four different floor slab systems were analyzed as possibilities for Office Building-G. These systems were compared on a multitude of criteria in order to determine which is most applicable. After analyzing the results and considering the impact of the design on the other building systems, it was determined that the current, one-way slab on post-tensioned girders, is the best system. It has a very low structural depth and very manageable deflections. The one sacrifice for this functional design is the cost of construction. The proposed systems did have certain advantages but when considering the architectural implications and constructability none of them presented themselves as a legitimate replacement.

Since the open floor plan was such an important architectural feature of Office Building-G, which ever system was chosen for the final proposal had to maintain this condition. Both the hollow core plank and composite steel girder designs were capable of maintaining the open plans. A two-way flat plate slab would require two additional column lines, disrupting the space provided to tenant fit outs thus determining it as an unacceptable solution.

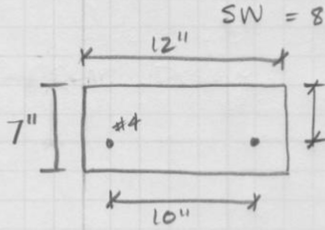
Serviceability requirements, construction difficulty and ceiling cavity limitations were the deciding factors responsible for eliminating hollow core planks and composite steel as possible alternatives. Hollow core plank is capable of spanning up to 60 ft however; the members experience large deflections, creating a possibility of vibrations. Composite steel has very manageable deflections and vibrations but the time and detailing associated with connecting 30 steel girders to concrete columns per floor would be hard to justify. Long span metal deck was effectively used to limit the amount of steel framing. However this resulted in deeper members. This would create issues with ceiling cavity spacing between the other building systems. Additional framing options with beams spanning between the girders will be considered to limit the depth of these members in the future if further analysis of this design is necessary.

Buildings with large spans are limited in the number of floor systems available for consideration. Results of this report determined these options are controlled by the entire structure design, especially the gravity system, not just the distance between columns. When considering the constructability of the proposed systems, two of the possible options had to be discredited due to the odd construction techniques and design details necessary to build them. Cast-in-place concrete must reach a certain strength before the members can be used to support any framing. This being the case, the hollow core planks and steel girders could be placed much quickly than the concrete columns would be able to reach an acceptable strength, creating gaps in the schedule where no work would be done on the superstructure. In order for the hollow core planks or composite steel designs to be accurately compared to the existing floor design, analyzing the effectiveness of precast and steel columns should also be performed.

EXISTING SYSTEM 1/3

SLAB DESIGN:

LOADS · $L_L = 100$ psf
$D_L = 5$ psf (superimposed)
$SW = 87.5$ psf



$d = 7 - 3/4 - .5/2 = 6"$

$f_y = 60^{ksi}$, $f'_c = 5000$ psi, $\beta_1 = .8$

$$A_s/ft = \frac{0.2}{10} \times 12 = 0.24 \text{ in}^2/ft$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.24(60)}{.85(5)(12)} = 0.2824 \qquad c = a/\beta_1 = .2824/.8 = .353$$

$$0.375d = .375(6) = 2.25 \qquad 2.25 > .353 \therefore \text{T.C. } \phi = 0.9$$

$$M_n = A_s f_y (d - a/2)/12$$
$$= .24(60)(6 - .2824/2)/12 = 7.031$$

$$\phi M_n = 0.9(7.031) = 6.33 \text{ ft-k/ft}$$

POSITIVE MOMENT – INTERIOR SPAN

$$M_u = \frac{w_u l^2}{16}$$

NEGATIVE MOMENT – INTERIOR SPAN

$$M_u = \frac{w_u l^2}{11} \leftarrow \text{CONTROLS}$$

$$w_u = 1.2 D_L + 1.6 L_L \qquad l = 20' - 4/2 - 4/2 = 16'$$
$$= 1.2(87.5 + 5) + 1.6(100)$$
$$= .271$$

$$M_u = \frac{.271(16)^2}{11} = 6.31 \text{ ft-k/ft}$$

$\phi M_n = 6.33$ ft-k/ft > 6.31 ft-k/ft $= M_u$ $\therefore$ Slab works

EXISTING SYSTEM 2/3

POST-TENSION BEAM DESIGN

LOADS $L_L = 100$ psf (reducible)

$D_{L_{slab}} = 87.5(20)/1000 = 1.75\ ^{k}/_{ft}$

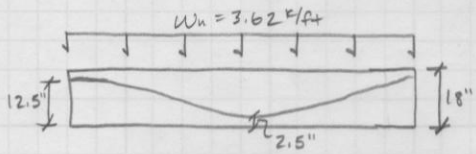
$D_{L_{S.I.}} = 5(20)/1000 = 0.1$

$D_{L_{S.W.}} = (48/12)(11/12)(150)/1000 = .550$

REDUCED $L_L = 61.2$ psf

$L_L = 61.2(20)/1000 = 1.22\ ^{k}/_{ft}$

SERVICE LOAD $= 1.22 + .55 + .1 + 1.75 = 3.62$



$h = L/30 = \frac{45 \times 12}{30} = 18''$

$A = 48 \times 18 = 864\ in^2$

BALANCING 75% OF LOAD ON BEAM

$0.75(3.62) = 2.715\ ^{k}/_{ft}$

$P = \frac{w\alpha L^2}{8} = \frac{2.715(10/12)(45^2)}{8} = 573^{k}$

ASSUME 15^{ksi} LOSE IN STRAND

$f_{se} = 0.75 f_p = 0.75(270{,}000) - 15{,}000 = 187.5^{ksi}$

$P_{eff} = f_{se} A = 187.5(864) = 28.69$

OF TENDONS

$\frac{573}{28.69} = 19.97 \Rightarrow$ TRY 21

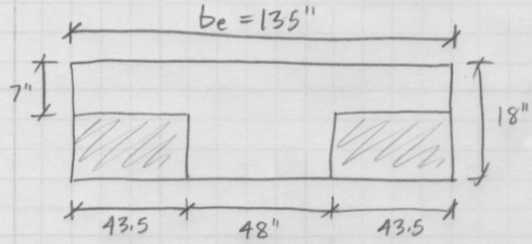
$21(28.69) = 602.49 \cong 574$ AS DESIGNED

EXISTING SYSTEM 3/3

DEFLECTION:

DETERMINE I:

$b_{eff} = \min \begin{cases} 8\times7 = 56'' \times 2 + 48'' = 160'' \\ 16\times12/2 = 96 \leftarrow \text{Won't Control} \\ \dfrac{45\times12}{4} = 135'' \leftarrow \text{Controls} \end{cases}$



$I_{TOT} = \dfrac{bh^3}{12} = \dfrac{135(18^3)}{12} = 65610 \text{ in}^4$

$I_{void} = 2\times\dfrac{bh^3}{12} = \dfrac{43.5(11^3)}{12}\times2 = 9650 \text{ in}^4$

$I_{SECTION} = 65610 - 9650 = 55960 \text{ in}^4$

LIVE LOAD

$\Delta = \dfrac{5}{384}\cdot\dfrac{wl^4}{EI} = \dfrac{5(1.22)(45^4)(1728)}{384(4074)(55960)} = 0.49''$

$E = 33\times145^{1.5}\sqrt{5000} = 4074 \text{ ksi}$

$L/360 = 45\times12/360 = 1.5''$ ∴ GOOD

SERVICE: $L/240 = 45\times12/240 = 2.25''$

$\Delta = \dfrac{5(3.62)(45^4)(1728)}{384(4074)(55960)} = 1.47''$ ∴ GOOD

CONSTRUCTION: $L/180 = 45\times12/180 = 3.0''$

$\Delta = \dfrac{5(2.4)(45^4)(1728)}{384(4074)(55960)} = 0.97$ ∴ GOOD

Appendix B

PRECAST PRESTRESSED HOLLOW CORE SLAB ½

LOADS: Live Load = 100 psf

Dead Load = 5 psf imposed

Topping & Self Weight included in table values

SPAN = 45' LOADS = 105 psf

∴ Use 12" STANDARD SPANCRETE w/ .75" cover

– MEETS REQUIREMENTS w/ A 1.5" CAMBER

LIVE LOAD

$$\frac{5 w_L l^4}{384 EI} = \frac{5(.100)(20)(45)^4(1728)}{384(4000)(8904)} = 5.18''$$

$$5.18'' - 1.5 = 3.94$$

$$L/360 = \frac{45 \times 12}{360} = 1.5'' < 3.94 \quad \therefore \text{NOT GOOD}$$

∴ TRY 16" STD w/ 1.5" cover w/ 1.5" CAMBER

$$\Delta = \frac{5(.100)(20)(45^4)(1728)}{384(4000)(18403)} = 2.51''$$

$$2.51 - 1.5 = 1.01''$$

∴ Meets Deflection Criteria

SERVICE LOAD DEFLECTION

$$\frac{5(.1+.13+.005)(20)(45^4)(1728)}{384(4000)(18403)} = 5.89$$

$$L/240 = \frac{45 \times 12}{240} = 2.25 \qquad 5.89 - 2.25 = 3.64''$$

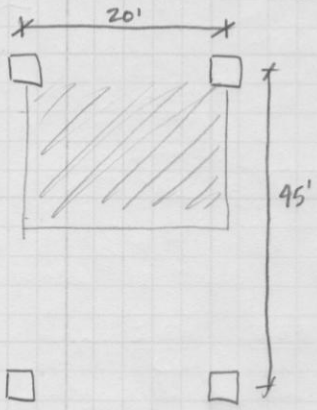
∴ USE 4.0" chamber

Use 16" Standard Spancrete w/ 1.5" strand cover

2" Structural topping and 4.0" chamber

HOLLOW CORE SLAB 2/2

TRANSFER BEAM DESIGN



TRIB ON BEAM $= 20(45/2) = 450$

$L_{LR} = 100\left[.25 + \frac{15}{\sqrt{2 \cdot 450}}\right] = 75$ psf

$D_L = 130$ psf WEIGHT OF HOLLOW CORE

$= 5$ psf SUPERIMPOSED

$w_u = [1.6(75) + 1.2(135)]\,45/2 = 6.2$ k/ft

$M_u = \frac{wl^2}{8} = \frac{6.2(20)^2}{8} = 310$ ft-K

$bd^2 \geq 20M_u$

$d = 2.0b$

$4b^3 \doteq 20(310) = 11.57$ $b = 12''$ $d = 24''$

$h = 26.0''$

$A_s \geq \frac{M_u}{4d} = \frac{310}{4(24)} = 3.2\ in^2$

$\therefore$ Use (4) #8 bars $\rightarrow A_s = 3.16\ in^2$

$M_n = 3.16(60)(24 - 3.72/2)/12 = 349.8$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{3.16(60)}{.85(5)(12)} = 3.72$ $c = 3.72/.8 = 4.65$

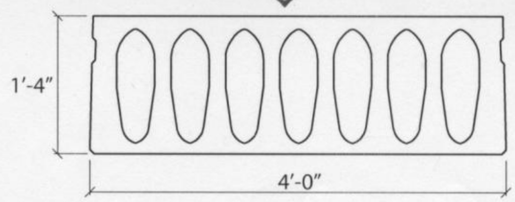
$0.375d = 9 > 4.65$ $\therefore$ T.C. $\phi = 0.9$

$\phi M_n = 314.8 > 310 = M_u$ $\therefore$ Beam Works

SPANCRETE

16″ STANDARD SPANCRETE
1.50″ Strand Cover
With Structural Topping

2 INCH MINIMUM AT MIDSPAN
Dead Load Weight of Slab = 130 psf



Section Properties					
A=518 in.^{2} I=18,403 in.^{4}	Yt=8.61 in. Yb=9.39 in.		b=16.0 in. wt=130 psf		
ØMn ft-k/ft	62.8	82.29	101.36	119.82	137.8
Series	1.5G-16708T	1.5G-16808T	1.5G-16810T	1.5G-16812T	1.5G-16814T
Span in Feet	Allowable Superimposed Load in Pounds Per Square Foot				
31	229				
32	209				
33	191				
34	174				
35		238			
36		220			
37		203			
38		187			
39		173	236		
40		160	219		
41			204	236	
42			190	226	243
43			175	213	232
44			160	197	220
45			147	182	208
46				168	198
47				155	186
48				143	172
49				132	160
50				121	148
51					137
52					127
53					117
54					108

Fire Rating (IBC)

Unrestrained 1 1/2 hours
Restrained 4 hours

Camber

1"-1 1/2"

≥ 1 1/2 "

Load Tables are presented as guidelines only. Design requirements must be reviewed by the engineer of record for each specific project.
Spancrete | P.O. Box 828 | Waukesha, WI 53187 | 414-290-9000 | www.spancrete.com

MKLT112-0109

Appendix C

COMPOSITE METAL DECK 1/3

LOADS : $L_L = 100$ psf (reducible)
$D_L = 5$ psf (superimposed)
SW: Only have N.W. concrete tables

$$\frac{145}{69} = \frac{116}{x} \Rightarrow x = 52.3 \text{ psf} \qquad \frac{120}{145} = \frac{x}{69}$$

$$L_{Le} = 100\left[0.25 + \frac{10}{\sqrt{2 \cdot 45 \cdot 20}}\right] = 60.4$$

LOAD ON DECK = 60.4 + 5 + 52.3 psf = 117.7 psf

DECKING : EDC 750, 16 Gage
- Clear Span w/ out shoring = 22'-1"
- Slab depth = 10½", 5" Topping w/ 5½" deck

DESIGN MOMENT : $M_u = \frac{w l^2}{8}$ $\qquad w_u = 20[1.6(60.4) + 1.2(52.3+5)]/1000$

$w_u = 3.31$

$$M_u = \frac{3.31(45^2)}{8} = 838 \text{ ft-k}$$

∴ TRY W18×60

DETERMINE FULL COMPOSITE STRENGTH

$$b_{eff} = \min \begin{cases} \frac{\text{span}}{8} = \frac{45 \times 12}{8} = 67.5'' \leftarrow \text{Controls} \\ \frac{1}{2} W = \frac{1}{2}(20 \times 12) = 120 \end{cases}$$

$Y2 = 10.5 - \frac{a}{2} \geq 7.0$ calcs are conservative

W18×60 $\quad \phi M_n = 925$ ft-k , $\Sigma Q_n = 619$

$$a = \frac{\Sigma Q_n}{.85(f'_c)(b_{eff})} = \frac{619}{.85(4)(67.5)} = 2.70$$

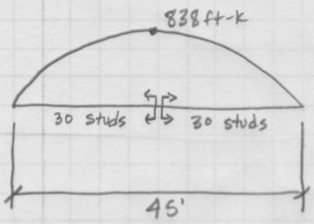
$Y2 = 10.5 - \frac{2.7}{2} = 9.15$ ∴ Values are Conservative ✓

COMPOSITE METAL DECK 2/3

NUMBER OF STUDS

- Use 3/4" bolts, $Q_n = 21.2$

$$n = \frac{\Sigma Q_n}{Q_n} = \frac{619}{21.2} = 29.2 \rightarrow 30 \text{ STUDS}$$



60 STUDS over 45' ⇒ 1 stud every 9"

STRENGTH & DEFLECTION CHECKS

UNSHORED STRENGTH - GOOD BASE ON DESIGN TABLES

LIVE LOAD DEFLECTION:

$$w_{LL} = 60.4(20) = 1.21 \text{ klf}$$

$$\Delta = \frac{5 w_{LL} \ell^4}{384 E I} = \frac{5(1.21)(45^4)(1728)}{384(29000)(3280)} = 1.17$$

$I_{LB} = 3280$ (table 3-20)

$L/360 = 1.5" > 1.17"$ ∴ GOOD

CONSTRUCTION DEFLECTION

$$w_{CL} = (\underbrace{20}_{LL} + 52.3)20 + \underbrace{60}_{R.W.} = 1.506$$

$$\Delta = \frac{5(1.506)(45^4)(1728)}{384(29000)(984)} = 4.87" \rightarrow$$ Does not meet $L/180 = 3.0"$ requirement

$$I_{need} = \frac{5(1.506)(45^4)(1728)}{384(29000)(3.0)} = 1597 \text{ in}^4 \text{ min}$$

∴ W21×73, chosen over W24×68 to limit structural depth

COMPOSITE METAL DECK 3/3

SERVICE LOAD DEFLECTION

$w_n = (60.4 + 52.3)(20) + 73 = 2.3\ k/ft$

$\Delta = \frac{5wl^4}{384EI} = \frac{5(2.3)(45^4)(1728)}{384(29000)(4930)} = 1.48''$

$l/240 = {}^{45\times12}/_{240} = 2.25'' > 1.48''$ ∴ GOOD

NUMBER OF STUDS

$n = \frac{\Sigma Q_n}{Q_n} = \frac{269}{21.1} = 13 \rightarrow 26$ STUDS, 1 STUD every 18"

∴ USE EDC750, 16 gage deck on W21×73
w/ 26 studs at 18"

CONSIDER LWT
CONC. TABLES NOT
INCLUDED IN
MANUAL.

CHNICAL TABLES

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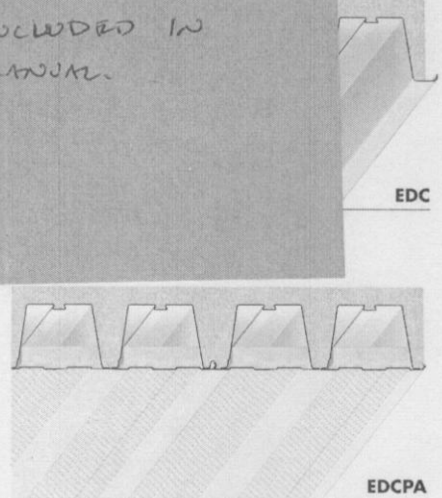


U.L. Fire Ratings**

Fire Rating Type—Hours	U.L. Design Number	Concrete Cover (in.)	Type & Weight of Concrete (pcf)	Bottom Protection	Type of Units Covered
RAR — 2 HR.	D501	2	LT. WT. (110)	⅝" GB	EDC
		2½	REG. WT. (147)		
UAR — 1½ HR.	D501	2	LT. WT. (110)	⅝" GB	EDC
		2½	REG. WT. (147)		
RAR — 1 HR.	D903	3	REG. WT. (147)	NONE	EDC*
		2¼	LT. WT. (110)		
		2¾	REG. WT. (147)	NONE	EDCP
		2½	LT. WT. (110)		
RAR — 2 HR.	D903	4¼	REG. WT. (147)	NONE	EDC*
		3½	LT. WT. (110)		
		4	REG. WT. (147)	NONE	EDCP
		3	LT. WT. (110)		
RAR — 3 HR.	D903	4	LT. WT. (110)	NONE	EDC*
		3¾	LT. WT. (110)	NONE	EDCP

RAR – Restrained Assembly Rating
UAR – Unrestrained Assembly Rating
* Excludes EDC324
NOTE: Add ¼" to concrete cover for EDCP324 units.
** Consult the latest U.L. Fire Resistance Directory for the specific system assembly requirements to achieve the above hourly fire ratings.

Regular Weight Concrete (145 pcf) — Concrete Strength 4 ksi*

Slab Depth Weight (psf*)	Concrete Volume (ft³/ft²)	Wideck Type	Gage*	Maximum Spans Without Shoring (ft.-in.)		Allowable Superimposed Loads (psf) Spans (ft.)															
				Simple	Double	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
6 (52)	.32	EDC324	18	13-4	13-5	200	193	168	147	**129**	**114**	**100**									
			16	14-4	15-9	200	200	178	157	138	**122**	**108**									
		EDCP324	18/18	14-8	13-9	200	200	184	161	142	**125**	**110**									
			16/18	15-5	14-7	200	200	191	168	148	131	**117**									
7½ (58)	.35	EDC450	18	16-3	16-10				196	173	153	136	**121**	**104**	**88**	**74**					
			16	17-4	19-7				200	188	167	149	133	**112**	**95**	**81**					
		EDCP450	18/18	17-9	17-0				200	194	172	153	136	**121**	**107**	**95**					
			16/18	18-7	18-2				200	200	182	163	145	130	**116**	**104**					
9 (63)	.39	EDC600	18	18-8	19-7							132	118	106	**95**	**85**	**76**	**68**	**60**	**54**	**47**
			16	19-11	22-10							152	137	124	112	101	**92**	**83**	**72**	**61**	**52**
		EDCP600	18/18	19-9	19-8							149	134	120	107	**96**	**86**	**77**	**68**	**61**	**54**
			16/18	21-2	20-10							166	150	135	122	111	100	**91**	**82**	**74**	**67**
10½ (69)	.42	EDC750	18	17-9	18-1								146	**132**	**119**	**108**	**98**	**88**	**80**	**72**	**65**
			16	22-1	25-8								171	155	142	129	118	108	**98**	**88**	**76**
		EDCP750	18/18	17-6	17-10								166	150	135	122	111	100	**90**	**82**	**73**
			16/18	23-5	22-9								187	170	155	141	129	118	108	**98**	**90**

No Shoring ■ Shoring Required in Shaded Areas

COMPOSITE SLAB DESIGN NOTES:

1. *Other concrete strengths, slab depths, and deck gages are available. Contact EPIC Metals Corporation.
2. Slab weight shown includes weight of heaviest deck gage.
3. All loads are assumed to be uniformly and statically applied. If loads greater than 200 psf are required, contact EPIC Metals Corporation.
4. Superimposed loads for spans in shaded areas and with bold face type assume deck is shored.
5. Composite slab design is based on simple span analysis.
6. Deflection limit of the composite slab is L/360 under the superimposed load.
7. Load tables are in accordance with SDI recommendations.

DECK DESIGN AS A FORM NOTES:

1. Maximum clear spans without shoring are based on the Steel Deck Institute recommendations for sequential loading and load resistance factor design. The table is based on 40 ksi steel yield stress and deflection limits of L/180 or ¾", whichever is less.
   Loading includes slab weight plus either a 30 psf uniform construction load or a 250-pound concentrated construction load on a 1'-0" width section.
   If heavier construction loads or less form deflection are required, spans must be reduced. Consult EPIC Metals Corporation for recommendations.
2. Runways and planking must be used for all concrete placement.
3. Minimum bearing is 4" at end supports and 6" at interior support bearing lengths.

CONTACT EPIC METALS CORPORATION FOR SPECIFICATION LANGUAGE.

EPIC METALS CORPORATION

Appendix D

Proposed columns for the flat plate slab design are outlined in blue and the existing columns are outlined in red.

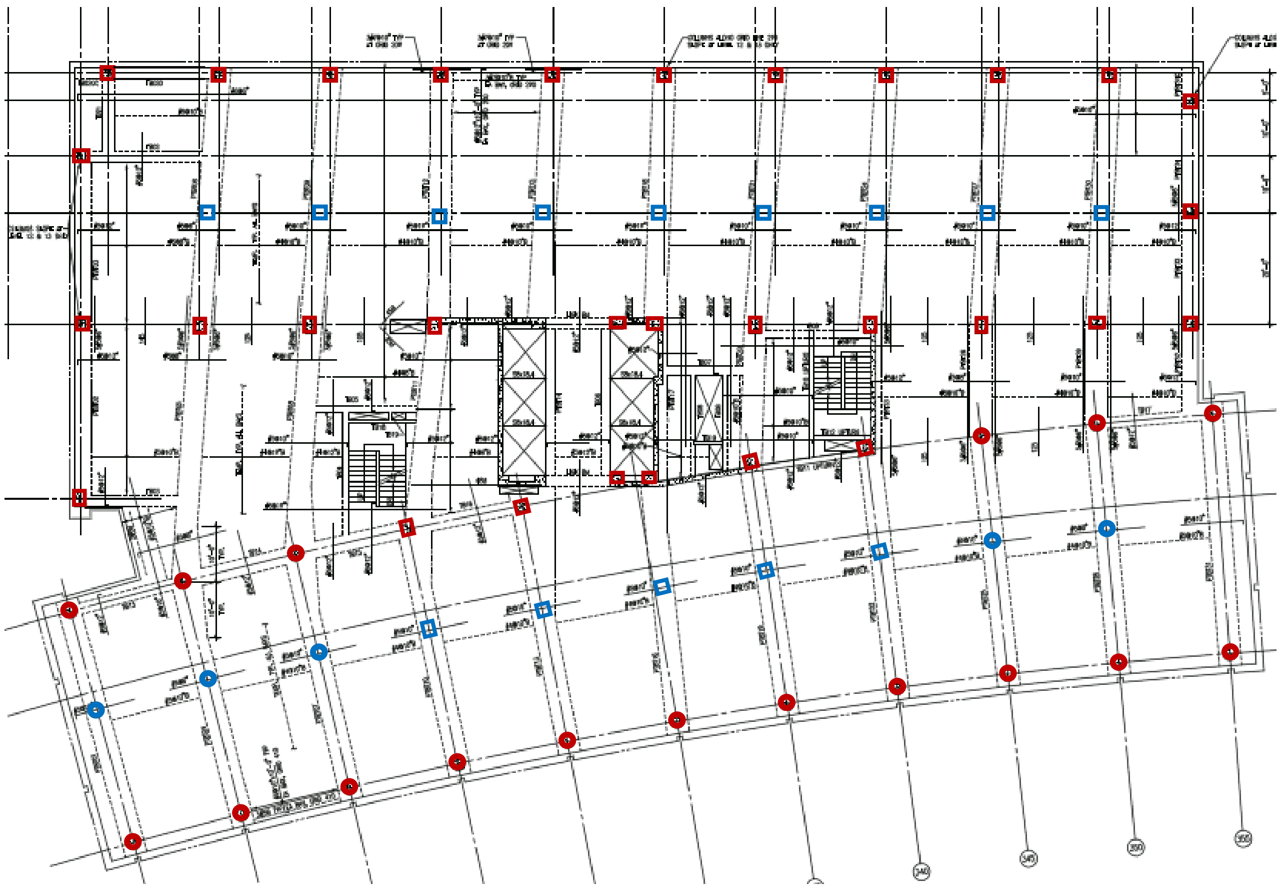
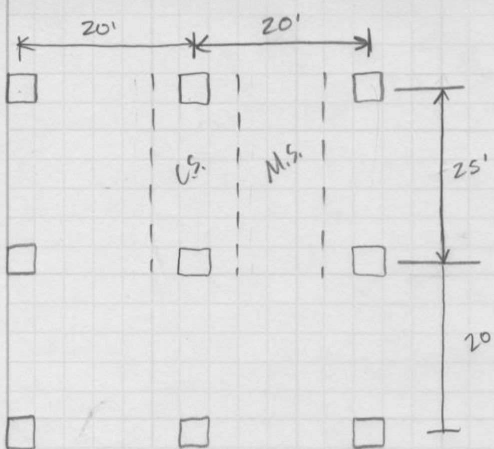


Figure 6

2 WAY FLAT PLATE DESIGN – DIRECT DESIGN METHOD 1/5



2' SQUARE COLUMNS

$\ell_n = 23'$
$\ell_2 = 20'$

Minimum Thickness $= \ell_n/33$

$t = \frac{23 \times 12}{33} = 8.36''$ ∴ Try 9"

SLAB WEIGHT $= (9/12)(150) = 112.5$ psf

LIVE LOAD = 100 psf

DEAD LOAD = 5 psf

TRIB AREA:

$(25/2 + 20/2)(20) = 450$ sf

$q_o = 1.6(100) + 1.2(5 + 112.5)$

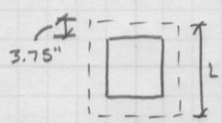
$q_o = .301$ k/ft²

PUNCHING SHEAR CHECK: $\phi V_c > V_u$

$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d$

$\phi V_c = 0.75(4)\sqrt{5000}\,(132)(7.5)/1000$

$\phi V_c = 210.0^k$



$d = 7.5$
$d/2 = 3.75$

$L = 24 + 3.75 \times 2$
$L = 33''$

$b_o = 4 \times 33$
$b_o = 132$ in

$V_u = 450(.301) = 135.45$

$\phi V_c = 210^k > 135^k = V_u$

∴ Punching Shear OK

2-WAY FLAT PLATE 2/5

DESIGN MOMENTS

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{0.301(20)(23^2)}{8} = 398 \text{ k-ft}$$

INTERIOR SPAN: 13.6.3.2

NEG FACTORED MOMENT $= 0.65M_o = 0.65(398) = 257$ ft-k

POS FACTORED MOMENT $= 0.35M_o = 0.35(398) = 139$ ft-k

$l_2/l_1 = 23/18 = 1.28$

$\alpha_{f1} = \frac{E_{cb} I_b}{E_{cs} I_s}$, $I_b = 0$ $\alpha_{f1} = 0$

13.6.4.1 : COLUMN STRIPS

$\alpha_{f1} = 0 \rightarrow 75\%$

MIDDLE STRIP $\rightarrow 25\%$

C.S. NEG MOMENT $= .75(257) = 193$ ft-k

M.S. NEG MOMENT $= .25(257) = 64$ ft-k

C.S. POS MOMENT $= .75(139) = 104$ ft-k

M.S. POS MOMENT $= .25(139) = 35$ ft-k

NEEDED REINFORCING: C.S. TOP

$d = h - 1.5 = 7.5''$

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{193 \times 12}{0.9 \times 60 \times .95 \times 7.5} = 6.02 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6.02(60)}{.85(5)(10 \times 12)} = 0.708$$

$$c = a/\beta_1 = .708/0.8 = 0.885$$

$$0.375d = 0.375(7.5) = 2.8125$$

$0.885 < 2.8125$

$\therefore$ T.C. $\phi = 0.9$

$$jd = d - \frac{a}{2} = 7.5 - \frac{.708}{2} = 7.146$$

$$jd = 0.95 \times 7.5 = 7.125$$

$\therefore$ jd estimate conservative

2-WAY FLAT PLATE 3/5

$A_{s,min} = 0.0018(10 \times 12) \times 9 = 1.944$ ∴ Meet $A_{s,min}$ Requirements

$120/18 = 6.66$ = Minimum # of bar spaces

7 = minimum # of bars

Try #6 Bars:

$A_s = 6.02$
$A_{s6} = .44$

$6.02/.44 = 14.0$ bars

∴ Use (14) #6 bars @ 8" For CS. TOP REINF.

NEEDED REINFORCING: CS BOTTOM

$$A_s = \frac{104 \times 12}{0.9(60)(0.95 \times 7.5)} = 3.24$$

$b = 120"$

∴ Check for T.C. & jd yield same result

$A_{s,min} = 1.944 < 3.24$ ∴ Good

7 bars minimum ∴ Try #5 Bars

$$\frac{3.24}{.31} = 10.5 \Rightarrow 11 \text{ bars}$$

$120/11 = 11"$ ∴ Use (11) #5 BARS @ 11" FOR C.S. BOT. REINF

NEEDED REINF.: MS TOP

$$A_s = \frac{64 \times 12}{0.9(60)(.95 \times 7.5)} = 2.0$$

$b = 120"$

∴ Checks for T.C. & jd good

$$\frac{2.0}{.31} = 6.45$$, 7 bar minimum

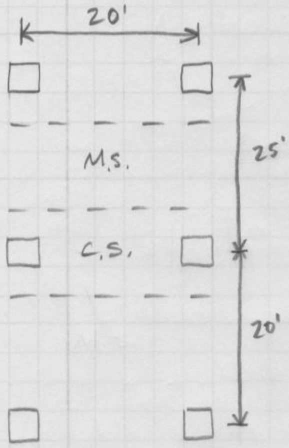
$120/7 = 17.14$ ∴ Use (7) #5 bars @ 18" FOR M.S. TOP REINF

2-WAY FLAT PLATE 4/5

NEEDED REINFORCING: M.S. BOTTOM

$A_s = \frac{35 \times 12}{0.9(60)(.95 \times 7.5)} = 1.09 \text{ in}^2$ ∴ Use $A_{s,min} = 1.944$

∴ Use (7) #5 bars @ 18"
FOR M.S. BOTTOM REINF



$\ell_n = 18'$
$\ell_2 = 25'$

∴ ASSUMING LARGER BAY IS TYPICAL TO BE CONSERVATIVE

$M_o = \frac{q_u \ell_2 \ell_n^2}{8} = \frac{.301(25)(18^2)}{8} = 305$ ft-k

EVALUATING AN INTERIOR SPAN

$-$MOMENT $= 0.65(M_o) = 198$ ft-k

$+$MOMENT $= 0.35 M_o = 107$ ft-k

$\alpha f_1 = 0 \rightarrow 75\%$ TO COLUMN STRIP

C.S. NEG MOMENT $= 0.75(198)$
$= 149$ ft-k

M.S. NEG MOMENT $= 0.25(198)$
$= 50^k$

C.S. POS MOMENT $= 0.75(107)$
$= 80$ ft-k

M.S. POS MOMENT $= 0.25(107)$
$= 27^k$

REINFORCEMENT: C.S. TOP REINF

$A_s = \frac{149 \times 12}{0.9(60)(.95)(7.5)} = 4.64 \text{ in}^2$

$a = \frac{4.64(60)}{.85(5)(12.5 \times 12)} = 0.437$ $c = .55$

$jd = 7.5 - \frac{.437}{2} = 7.282$

$0.375d = 2.81 > .55$ ∴ TC. $\phi = 0.9$

$jd = 0.95(7.5) = 7.125$ ∴ jd is conservative

$A_{s,min} = .0018(12.5 \times 12)9 = 2.43$ ∴ Good

$150/18 = 8.33$ spaces $\rightarrow$ 9 bars minimum

$4.64/.44 = 10.5$

∴ Use (12) #6 bars @ 12"
FOR CS TOP REINF.

2-WAY FLAT PLATE 5/5

REINFORCEMENT : C.S. BOTTOM

$$A_s = \frac{80 \times 12}{0.9(60)(.95)(7.5)} = 2.50 \text{ in}^2$$

$2.5/.2 = 12.5$ bars

∴ Use (13) #4 bars @ 11"
For C.S. Bottom Reinf

REINFORCEMENT : M.S. TOP

$$A_s = \frac{50 \times 12}{0.9(60)(.95)(7.5)} = 1.55 < 2.43 \text{ in}^2 = A_{s\,min} \quad \therefore \text{Use } A_{s\,min}$$

$2.43/.2 = 12.15$

∴ Use (13) # 4 bars @ 11"
For M.S. TOP

REINFORCEMENT : M.S. BOT

$$A_s = \frac{27 \times 12}{0.9(60)(.95)(7.5)} = 0.84 < A_{s\,min} \quad \therefore \text{Use } A_{s\,min}$$

∴ Use (13) #4 bars @ 11"
For M.S. Bot