The Regent

950 N. Glebe Road
Arlington, VA

Architect: Cooper Carry Architects

Structural Technical Report 2
Pro-Con Structural Study of Alternate Floor Systems

Prepared By:	**Kristin Ruth**
Option:	**Structural**
Date:	**October 31, 2005**
Consultant:	**Mr. Schneider**

Executive Summary

This report provides an overview of the existing structural system, focusing on the existing typical floor framing system and four other alternative floor framing systems for The Regent, which is currently under construction in Arlington, VA. The Regent is a 12-story office building which has retail space on the first level and a 3-story parking garage below grade.

The four alternative systems considered include: hollow-core planks with steel framing system, precast double tees with precast framing system, cast-in-place, one-way, wide module joists with cast-in-place framing system, and finally, a two-way flat slab with drop panels with cast-in-place framing system. Each alternative floor system design is discussed and their advantages and disadvantages are compared among each other and to the existing floor framing system. A schematic floor framing system plan, showing representative members of the floor framing system is provided with each alternative system discussed. The Appendix includes all of the calculations and design aids used to complete the preliminary structural floor designs as well as existing typical structural floor plans for The Regent. A typical structural floor plan and typical bay plan have been included in the body of this report.

After completing the designs and discussing the advantages and disadvantages for each floor system, it is recommended that the hollow-core planks with steel framing, the precast double tees with precast framing, and the one-way joists with cast-in-place framing systems be studied further.

The existing system has proven to be a very efficient system with many advantages and few disadvantages. Some of the advantages include: relatively small member sizes and self weights, smaller floor system depths, and being able to span the longer spans in the bays. Some disadvantages include: more framing members and likelihood that the long span steel system will cause concrete ponding due to deflection.

The two-way flat slab with drop panels should not be studied further as a two-way CIP system with the existing bay sizes. A 16.5” slab is not practical and not easily constructible. Switching to a two-way post-tensioning system may thin out the slab depth making a post-tensioning system a practical option.

The cast-in-place, one-way, wide module joists have both several advantages and disadvantages. The structure, as preliminarily designed, would weigh a lot more than the existing system and would require larger foundations. Also, the amount of labor that needs to be done on site would require a lot of construction time and field labor, which can be expensive. For a spec office building, construction time is very critical and would be very risky for the involved placement of the cast-in-place concrete joist system. However, this system does provide a uniform depth that does not exceed the existing design’s maximum depth. This system also has a good fire rating and can accommodate the longer spans in the larger bay sizes. Considering more columns and

smaller bay sizes may reduce the size of the framing members and the entire structural system may be more efficiently designed as a result.

The hollow-core plank system has several advantages over the existing structure including quicker construction time since the hollow-core planks are precast, the quality control advantage of the planks being precast in a plant, good fire rating, good acoustical value, and less steel beams per bay. Some disadvantages discussed include the labor and cost going into the angle connection to hold the hollow-core planks for a flush floor system, the downtown site being able to accommodate the extra precast deliveries, and the increased beam depths and weights and their effects on the foundations and floor depth.

The precast double tees with precast framing member system is also another possible good alternative. Its advantages over the existing system include: concrete quality control, quick construction time, lighter self weight of the double tees, good fire resistance, and good acoustical value. The disadvantages include heavier beams and columns and the resulting larger foundations, the extra deep depth of the flooring system, and the downtown site being able to accommodate all of the precast deliveries.

All of the alternative systems that have been discussed will be studied further either as a continuation of the preliminary design or a modified design based on what has been learned in from this report.

Codes and Code Load Requirements

The 2000 ICC International Building Code (IBC 2000) was used for the structural design of The Regent. IBC 2000 incorporates many of the design load procedures of ASCE 7-02. ASCE 7-02 was also used for calculating the snow loads and roof live loads. The live loads were taken from Table 1607.1 of IBC 2000. The equations, tables, and procedures used to calculate the design loads listed in this report were taken from ASCE 7-02. LRFD was used for the existing structural design.

Since this report focuses on alternate flooring system designs, only gravity loads were considered in this report. The *Gravity Loads* section summarizes all of the gravity loads considered for the entire building. Furthermore, since the scope of this report includes designing preliminary sizes for representative members for each floor system, worst case typical floor bays were chosen to evaluate each floor system. Since The Regent is primarily an office building, with office space on floors 2-12, the typical bays are found on all of the office use floors. The gravity loads considered for a typical office floor bay are bolded in the *Gravity Loads* section.

Gravity Loads

- **Dead Loads**
  - Roof

3” - 22 Gage Metal Deck	5 PSF
Insulation	3 PSF
Misc. DL	10 PSF
Roofing	20 PSF

  - **Typical Floor**

3 ¼” lt. wt. slab on 3” - 20 gage metal deck **(United Steel Deck design manual p. 40)**	**46 PSF**
Concrete Ponding ***included because of the long** **steel spans and cambers**	**10 PSF**
Misc. DL **(mechanical ducts, sprinklers,** **ceiling, plumbing, etc.)**	**15 PSF**

  - **Construction Loads**

3 ¼” lt. wt. slab on 3” -20 gage metal deck	**46 PSF**
Concrete Ponding	**10 PSF**

Live Loads (IBC 2000 and special loadings)	
Corridors	100 PSF
Stairs	100 PSF
Mechanical Spaces	150 PSF
Offices	**100 PSF**
Retail – 1st Level	100 PSF
Terrace Above 1st Floor Retail	100 PSF
Loading Dock	350 PSF
Parking Garage (Garages having trucks and busses)	50 PSF
Plaza Deck (Fire Truck Loading)	350 PSF
Snow Load	30 PSF
Construction Live Loads (unreducible)	**20 PSF**
Roof Live Loads (as calculated per ASCE 7-02)	30 PSF

Overview of Existing Structural System

The existing structural system was previously described in *Structural Technical Report 1: Structural Concepts/Structural Existing Conditions Report.* Parts of Technical Report 1 are reproduced in this section in order to put the existing structural system into context.

Foundations

The foundations for The Regent consist of square footings ranging in size from 4’ x 4’ to 9’ x 9’ with depths ranging from 24” to 50” respectively. They are located on a 30’ x 30’ square grid. The two allowable bearing pressures for the square footings are 25 ksf and 40 ksf. The southwest quarter of the building has allowable bearing pressures of 25 ksf while the other three quarters of the building have a 40 ksf allowable bearing pressure. The larger square footings are located in the central core of the building below the elevator shafts. There are also continuous 24” wide, 12” deep concrete footings under the 12” thick continuous walls. The slab on grade is 4” thick reinforced with 6 x 6, 10/10 WWF. The concrete strength for all foundations, walls, and slabs on grade is a minimum of 3000 psi.

Concrete Parking Garage Below Grade

There is a 3-level concrete parking garage below grade. The typical bay size for the three levels of below grade parking is 30’ x 30’. The most common column sizes are 16” x 24”and 28” x 36” and the most common beam sizes are 12” x 24”, 12” x 18”, 8” x 18”, and 18” x 30”. All of the columns are of design strength f’c = 5000 psi, although a few are f’c = 7000 psi and the 28-day design strength of the beams is f’c = 4000 psi.

The parking garage slabs are 8” thick with a typical drop panel size of 10’ x 10’ x 5 ½” and a 28-day strength of 4000 psi.

Plaza and 1st Floor Slabs

The Plaza level slab is 12” thick with 10’ x 10’ x 12” drop panels. The design loads for the Plaza level include a 350 PSF live load which accounts for the weight of a fire truck loading during the case of an emergency.

The first floor slab is 9” thick with 10’ x 10’x 5 ½” drop panels. The Plaza and 1st floor slabs are both of strength f’c = 4000 psi.

Steel Framing Above Grade

There are two typical bay sizes for the steel superstructure above grade; 30’ x 30’ and approximately 43’ - 46’ x 30’. From North to South the columns are at a 30’ spacing. From East to West the columns spacings are approximately 46’, 30’ and 43’ respectively. The most common column sizes are W14 x 145, W14 x 99, and W14 x 176.

The most common beam sizes are W18 x 50, W18 x 46, and W16 x 26 with cambers ranging from ¾” to 2” which are designed to 75% dead load. The most common girder sizes are W18 x 65, W24 x 55, W24 x 62, and W24 x 55.

The typical floor slab is 3 ¼” light weight concrete with an f’c = 3000 psi and is reinforced with 6 x 6 10/10 WWF on top of a 3” – 20 gage composite steel deck for a total slab thickness of 6 ¼”. The shear studs are ¾” diameter, 5” headed studs.

The existing typical bay floor construction and member sizes are approximately the same for all office floors 2-12.

There is an elevator core running up the center of the building and through the center of each floor. The elevator core was neglected when exploring alternative structural floor framing systems since the alternative floor system designs are preliminary. The elevator core and its effects on the design of the floor framing will be considered in later reports.

The roof deck construction is 3” x 22 gage, deep rib, type N, painted roof deck. There are a few full moment connections at certain corners of the roof and penthouse roof.

Lateral Load Resisting System

The lateral load resisting system for The Regent consists of five braced frames at the core of the building (See the Typical Floor Plan). There are two braced frames, #4 and #5, that span along the building’s North / South axis, and three braced frames, #1, #2, and #3, that span along the building’s East / West axis. The braced frames are approximately 30’ in width and run the full height of the building from the first floor to the penthouse roof.

Frames #1, #3, and #5 have chevron style bracing and Frames #2 and #4 have single diagonal bracing. The typical diagonal steel members used in the braced frames are HSS 8” x 8”’s, 10” x 10”’s, and 12” x 12”’s with thicknesses ranging from 3/8” to 5/8”. The braced frame columns are all 14” wide flange members ranging in size from W14 x 233’s and W14 x 257’s near the base to W14 x 53’s to W14 x 72’s at the top.

Scope

The scope of this report focuses on alternative typical floor framing systems for the office tower floors. Alternate flooring and framing systems for The Regent’s below-grade parking structure may be considered in later reports.

Typical Existing Floor System Design

Levels 2-12 are intended to be used as rentable office space. The loads considered for the existing floor system design were listed in detail in the *Gravity Loads* section of this report and are summarized below.

Loads:

Dead:		
	3 ¼” lt. wt. slab on 3” - 20 gage metal deck	46 PSF
	Concrete Ponding	10 PSF
	Misc. DL	15 PSF
	Façade	15 PSF
	Construction DL	56 PSF
Live:		
	Office	100 PSF (reducible)
	Construction LL	20 PSF

The existing typical office floor system design consists of a concrete slab on metal deck supported by composite steel beams. The slab is 3 ¼” light weight concrete with an f’c = 3000 psi and is reinforced with 6 x 6 10/10 WWF. The metal deck is 3” – 20 gage composite steel deck for total slab thickness to 6 ¼”. The composite action between the slab on metal deck and the steel beams is provided by ¾” diameter, 5” headed shear studs.

There are three typical bay sizes for the steel superstructure above grade; 30’ x 30’, approximately 46’ x 30’, and approximately 43’ x 30’. From North to South the columns are at a 30’ spacing. From East to West the columns spacings are approximately 46’, 30’ and 43’ respectively.

All of the columns are W14’s.

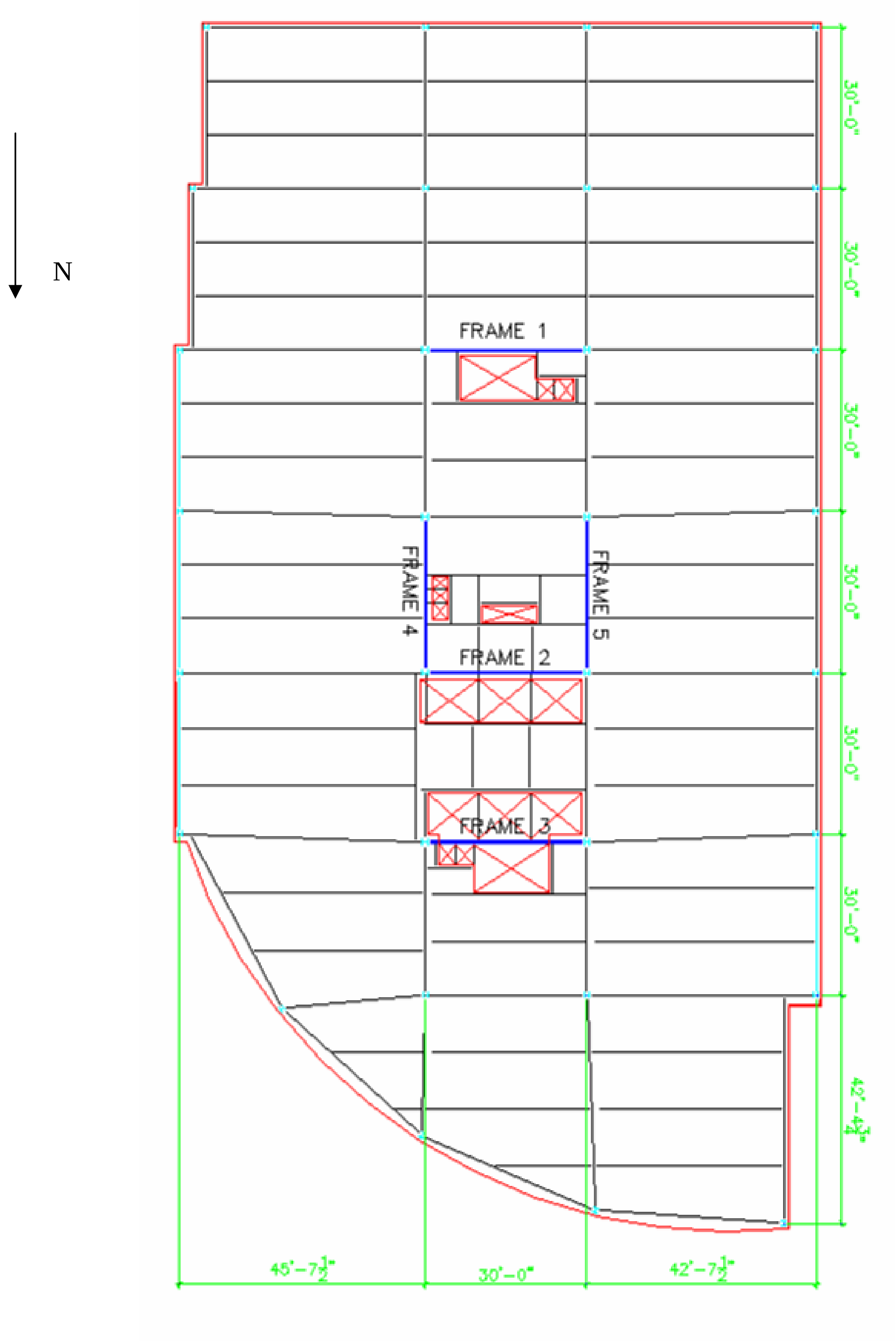
The most common composite beam sizes for the beams spanning in the long direction are W18 x 50 for the 46’ x 30’ bays, W18 x 46 for the 43’ x 30’ bays, and W16 x 26 for the 30’ x 30’ bays with cambers ranging from ¾” to 2”, which are designed to 75% dead load. The most common girder sizes are W18 x 65, W24 x 55, W24 x 62, W24 x 55 and W21 x 44 around the perimeter.

The existing framing members were checked using a simplified RAM model. Some of the members were exactly the same, and some of the RAM-designed members were smaller than the existing members. The results of the RAM analysis can be found in the Appendix of this report. The number of shear studs and camber sizes varied slightly from the existing design. There are several reasons as to why some of the members did not exactly match the existing design. These reasons are summarized below.

- Only gravity loads were considered for this report. Although there are braced frames designed to primarily take the lateral loads, the existing members may be larger as a result of the lateral effects on the floor framing members.
- In the RAM analysis, the typical bay sizes were rounded up to the nearest foot. This should not have had a significant effect on the size of the beams, though. Also, slight column offsets were neglected.
- Openings in the floor system were neglected in the RAM analysis and may have had an impact on the existing member sizes. If higher loadings were anticipated in an area, they were considered for the existing system, but only a uniformed distributed office live and dead loading were considered in the RAM analysis.

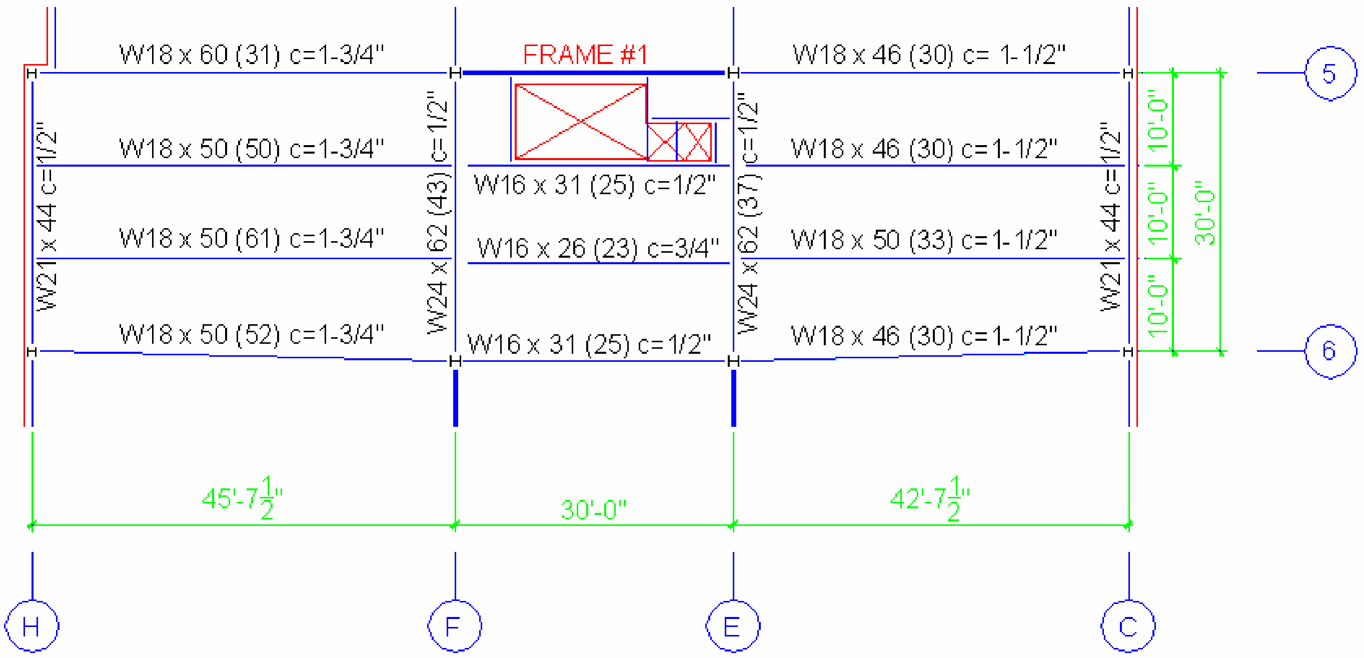
In conclusion, the existing member sizes were the same or slightly larger than the RAM analysis results. Therefore, the applied loads considered are proven to be correct or very close in value to the loads considered for the design of the existing system.

Typical Floor Plan



Floor plans for levels 2-12 have been included in the Appendix of this report.

Typical Bay for the Existing Design of the Office Floors

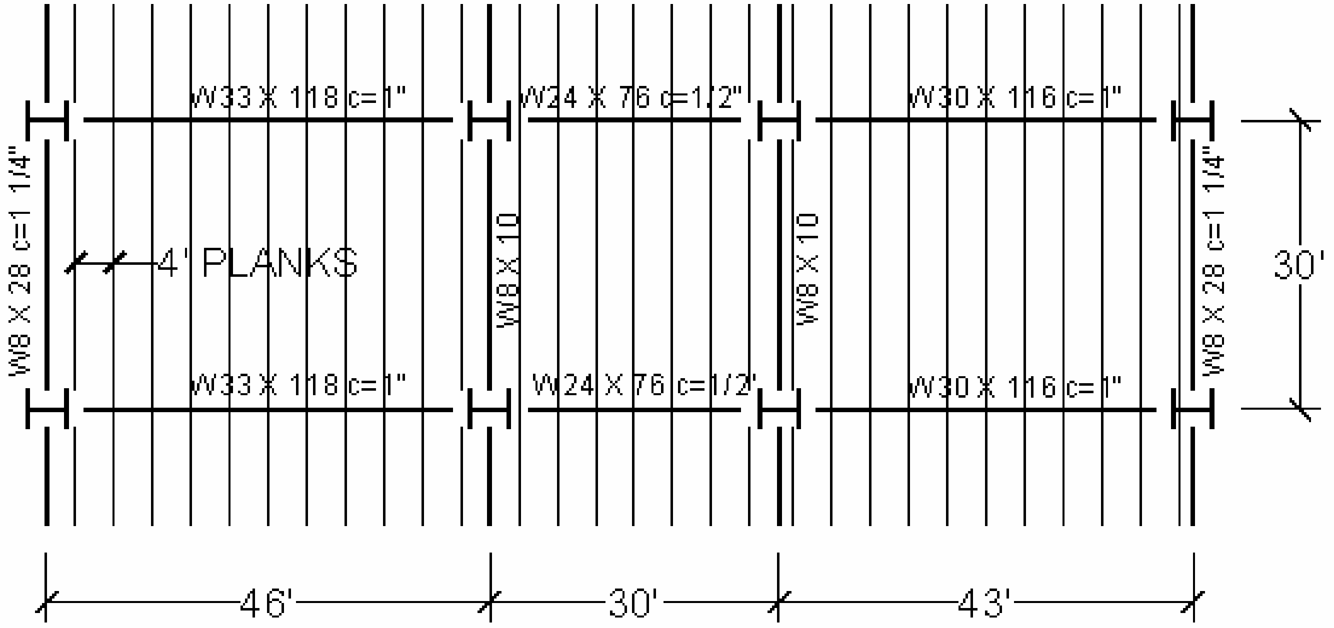


NOTE: ALL COLUMNS ARE W14's

Alternate Floor System Designs

Hollow-Core Planks with Steel Framing System

Typical Floor Framing Plan for Hollow-Core Planks with Steel Framing



ALL COLUMNS W14's MIN.

Hollow-core plank design: PCI Designation 4LHC8+2
Width = 4'
Depth = 8" + 2" normal weight topping = 10"
f'c = 5,000 psi
f'ci = 3,500 psi
Allowable safe superimposed service load = 125 PSF

Please refer to PCI Design Handbook page 2-27, which can be found in the Appendix, for the hollow-core member cross-section, dimensions, and properties.

System Description

This alternate flooring system consists of steel framing with precast hollow-core planks. The planks are designed in the North / South direction across the 30' typical bays. The column placement and bay sizes are the same as the existing floor system.

System Design

Please refer to the Appendix for detailed calculations, design assumptions, and design aids.

For the initial design of the precast hollow-core planks, the PCI Design Handbook, 5th edition was used. The hollow-core plank selected is able to span the 30' tributary width and carry a safe superimposed load of 125 PSF which exceeds the calculated safe superimposed loading of 115 PSF. The 100 PSF office live load was not able to be reduced since the tributary area for each plank was less than 400 SF. A 2" normal weight topped member was selected in order to help provide extra stability to the flooring system. Several hollow-core plank members of different depths and self weights were considered and the lightest member was selected.

The hollow-core plank selected is 4' wide x 10" deep and has a self weight of 68 PSF.

The steel framing members were designed using RAM to carry the weight of the hollow-core planks instead of the slab on deck as well as the other original superimposed dead loads and live loads.

In order to keep the depth of the flooring system as small as possible, it is proposed that the hollow-core planks sit on angles welded to the web of the supporting steel members so that the top of the flange and the top of the hollow-core plank are flush. This will decrease the total depth at the supporting beams from 43" to 33".

Comparison to the Existing System

Depth

The 10" depth of the hollow-core slab exceeds the depth of the existing slab on metal deck which is only 6.25". The deepest steel member of this system is approximately 33" deep, whereas in the existing system the deepest member is only 24" deep. Although it is proposed that the planks be flush with the top of the beams, this system will still be 33" deep at the supporting members as compared to the existing system which has a maximum depth of 30.25".

Member Sizes

The steel framing members that span East / West are significantly deeper and heavier than the existing design. The increase in size is due to the loss of composite action between the slab and the composite beams and also because the self weight of the hollow-core slabs exceeds the self weight of the slab on deck, including ponding, by 12 PSF. Since the weight of the flooring system has increased, the columns will need to be larger. The existing system uses W14 members.

Impact on the Existing Foundations

Since the steel framing includes heavier members, and since the weight of the hollow-core planks exceeds the weight of the slab on metal deck, the weight of the superstructure is going to increase, resulting in larger foundations. This system is still relatively light compared to the concrete framing options though.

Advantages

Time

The most significant advantage is the elimination of cast-in-place concrete. Hollow-core planks are quicker to erect since they are precast, eliminating pouring and curing time and on-site cast-in-place labor. In combination with the time-savings of steel erection over a concrete system, this system has the potential to be one of the quickest systems to erect.

Depth

Although the depth of the flooring system at the supporting beams is approximately 33", the depth of the planks is only 10" deep spanning the entire bay with no interior bay beams as in the case of the existing system. Therefore, the depth of the hollow-core plank flooring system is relatively shallow throughout the entire bay.

Less Beams

This system allows for the elimination of the beams not directly connecting to the columns. These infill beams are needed in the existing system to participate in the composite action of the flooring system. Since the planks are able to span 30', these extra beams are not needed in this floor system design, reducing the amount of steel needed to be erected and the reduction material and labor costs associated with it. Adding intermediate beams may allow for the reduction in size of the proposed beams and hollow-core planks, and the most efficient solution will need to be designed.

Quality Control

The hollow-core planks are precast in a concrete plant, so there is quality control in the manufacturing of the hollow-core planks over a cast-in-place slab on metal deck system which is constructed on site.

Fire-Rating

Precast systems typically have good fire ratings.

Acoustics

Precast members have good acoustical value. The precast members can help resist noise penetration through the floors, which may be advantageous in an office building.

Weight

Although, this system does not require the additional beams every 10', the weight of some of the supporting members have significantly increased, while some have decreased. Also, the weight of the hollow-core planks is greater than the weight of the slab on deck. The weight of this system could be potentially heavier than the existing system and could result in larger foundations. Although, there is an increase in weight of some of the members and in the additional weight of the hollow-core planks, the overall weight of the structure compared to the weight of a concrete systems is still relatively light.

Disadvantages

Detailing

In order to make this system a good alternative in relation to overall depth, the hollow-core planks need to be flush with the top of the flanges of the steel in order to decrease the depth. In order to do this, steel angles need to be connected to the webs of all the supporting members adding both material and labor costs. If the planks were not to be carried by the angles and were selected to span on top of the supporting flanges, then the depth of the flooring system would be a total of 43” at the supporting members.

Deliveries

Since this building would be all pre-fabricated members, they would all just need to be delivered to site and immediately erected. Since The Regent is on a downtown site, frequent deliveries and staging room could be an issue.

Cost

The material cost of the hollow-core planks may be higher than the cast-in-place slab on metal deck. Also, the precast planks would need to be installed with a crane and may require additional crane costs. There would be additional costs to detail and construct the supporting angle connections.

Other Considerations

Composite Action – Smaller Framing Members

One alternative to making this system more efficient is to make the hollow-core planks composite with the steel beams through shear studs welded to the steel and grouted into pre-drilled holes in the hollow-core planks. Making the hollow-core planks composite with the steel beams would result in smaller supporting steel beams.

Pre-Connected Angles

The steel angles could be pre-attached to the web prior to coming onsite eliminating the need for field connection the angles.

Infill Beams

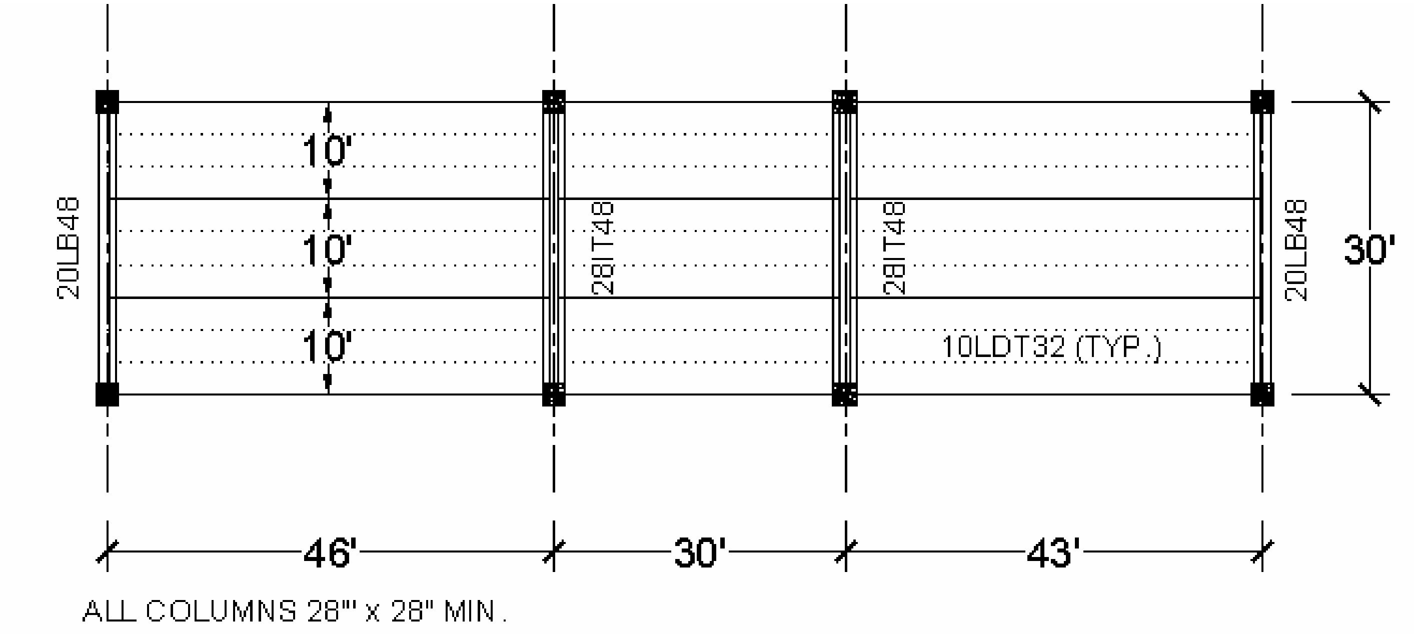
Instead of spanning the planks 30’, a beam could be added at 15’ reducing the span of the hollow-core planks. This would reduce the size of the planks needed, resulting in a smaller plank depth and self-weight and thus reducing the size of the supporting members. More steel framing members would need to be erected as a result.

Lateral Load Resisting System

The lateral load resisting braced frame system can remain, although the braced frame member sizes may need to be increased to handle the heavier dead loads.

Precast Double Tees with Precast Framing System

Typical Floor Framing Plan for Precast Double Tees with Precast Framing System



Double Tee Selection: 10LDT32 120 PSF < 130 PSF ∴ OK

12 strands, 8/16" = 0.5" diameter strands
1 depression point
f'c = 5,000 psi
f_{pu} = 270,000 psi
2.4" estimated camber at erection
2.9" estimated long-time camber

Inverted Tee-Beam Selection: 28IT48 4,560 PLF < 9,741 PLF ∴ OK

22 strands, 8/16" = 0.5" diameter strands
low-lax strands
f'c = 5,000 psi
f_{pu} = 270,000 psi
0.4" estimated camber at erection
0.1" estimated long-time camber

L-Beam Selection: 20LB48 2,760 PLF < 9,231 PLF ∴ OK

21 strands, 8/16" = 0.5" diameter strands
low-lax strands
f'c = 5,000 psi
f_{pu} = 270,000 psi
0.5" estimated camber at erection
0.2" estimated long-time camber

Please refer to PCI Handbook, pages 2-42, 2-44, and 2-16, which can be found in the Appendix, for the member cross-sections, dimensions, properties, and prestressing strand details.

System Description

This flooring system consists of entirely of precast members. The floor system consists of precast double tees spanning 46', 30' and 43' in the East / West direction. They are supported by interior precast inverted tee-beams and exterior precast L-beams which span 30' in the North / South direction. The bay sizes are the same as the existing system.

System Design

Please refer to the Appendix for detailed calculations and design assumptions.

The precast members were oriented as described previously so that the supporting girders would not have to span the 46' and would result in smaller members throughout the floor framing structure. The live load was able to be reduced slightly since the tributary area of each double tee member exceeded 400 SF.

For the preliminary design of the precast members, the PCI Design Handbook 5th edition was used. A 10' wide member was selected so that exactly 3 of them would fit inside of the 30' bay. The worst case span for the double tee was 46' and it needed to carry a safe superimposed service load of 120 PSF to account for a ¾" normal weight topping added on top of the double tees and their supporting members for stability. Several double tee sections were considered, but the lightest section, with a PCI designation of 10LDT32, was selected and is able to carry 130 PSF. It has an overall depth of 32", prior to adding the ¾" topping for stability and a self weight of 49 PSF.

An interior precast inverted tee-beam was selected to carry the double tee members on both sides and an L-beam was selected to carry the double tees on one side at the exterior.

Comparison to the Existing System

Depth

The depth of the double tees will be approximately 33" throughout the bay and 49" at the supporting members. These depths exceed the depths of the existing system, significantly at both the supporting members (49" vs. 30.25") and throughout the bay (33" vs. 6.25").

Member Sizes

The self weight of the double tees is approximately the same as the existing system. The double tee self weight is 49 PSF as compared to the existing system which is 46 PSF not including the 10 PSF used to account for concrete ponding during placement. Because this system is all precast, the precast members are significantly larger in depth, width, and mass than the steel framing members. The self weights of the supporting members are significantly larger than the existing steel framing. The precast columns would have to be at least 28” square in order to support the 28” width of the precast beams.

Impact on the Existing Foundations

Since this system is all precast, the weight of the structure will increase significantly requiring larger foundations.

Advantages

Erection Time

Erection time will be very quick since the members will arrive on site ready to be placed.

Quality Control

Since the precast members are formed and cured in a plant, the precast members have better quality control over cast-in-place members.

Fire-Rating

Precast members typically have good fire ratings

Acoustics

Precast members have good acoustical value. They can more easily resist noise penetration through the members, which may be advantageous in an office building.

Disadvantages

Depth

The depths of this system significantly exceed the depths of the existing system both at the supports and spanning throughout the bay. Since The Regent is built to it maximum height, minimum floor structure depth is critical.

Site Congestion

The Regent is located in downtown Arlington, VA, so the site is rather limiting. It may be difficult to coordinate cranes and precast deliveries on a small downtown site.

Material Costs

Although, this system will have lower construction costs, the cost of an all precast system can get very expensive especially for larger members such as those required in this initial design.

Other Considerations

Shallower Members

Shallower members could be selected to carry the loads. The tops of the beams will need to be filled-in in order to make the tops of the beams flush with the top of the double tees. Although the supporting members can be smaller, they will still be deeper than just the double tees because of the depth of concrete needed to support the double tees. So although, lighter shallower members could carry the loads, detailing of the supporting beams and their depths need to be considered.

Smaller Spans

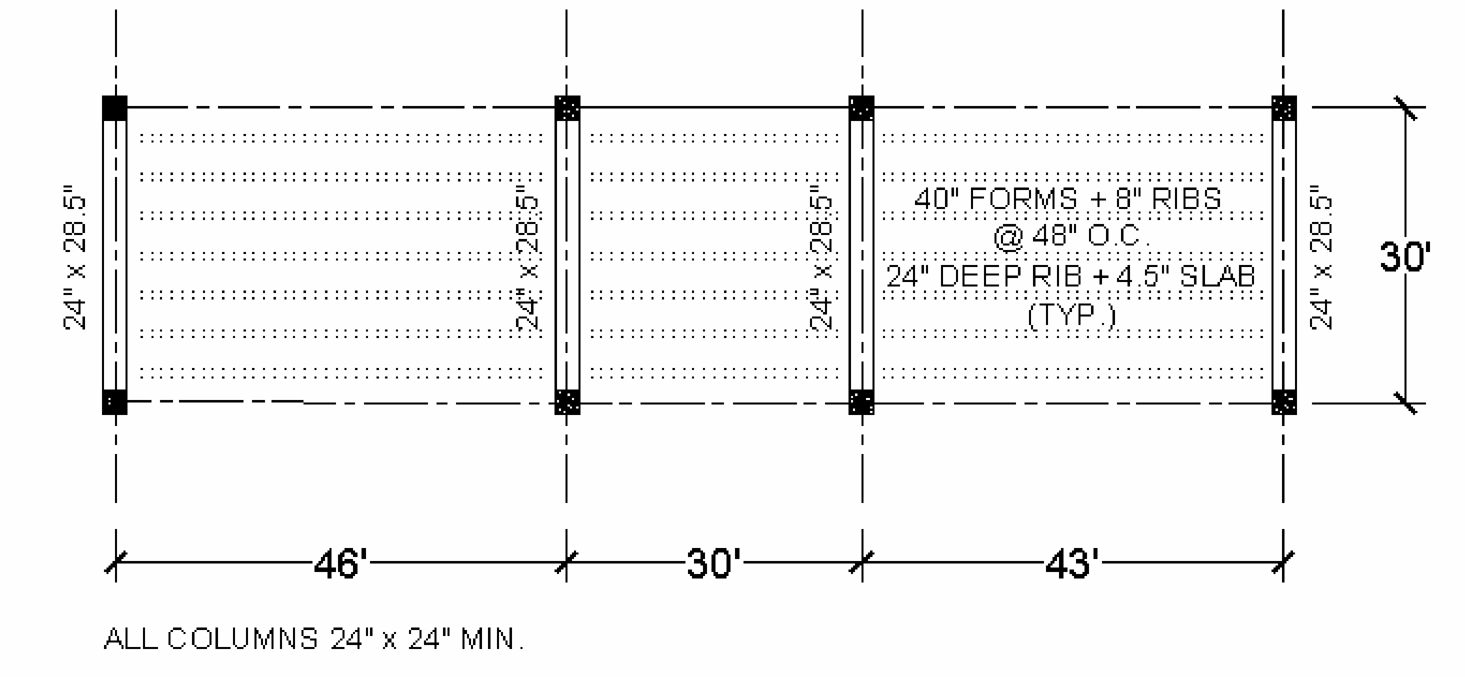
The overall depth and size of the framing members would be reduced if smaller bay sizes were introduced. Smaller bay sizes would require more columns which may be undesirable for an upscale spec office building where an open floor plan is the most profitable and optimum design.

Lateral Load Resisting System

The lateral load resisting system will need to be changed to a concrete system and sized to handle the increased dead load of the building.

One-way Wide Module Joists, Multiple Spans, with Cast-In-Place Framing System

Typical Floor Framing Plan for One-way Wide Module Joists with Cast-In-Place Framing System



Joist Selection: 40” Forms + 8” Ribs @ 48” o.c.
24” Deep Rib + 4.5 “Top Slab = 28.5” Total Depth
f’c = 4,000 psi
fy = 60,000 psi

End Span: 764 PLF < 873 PLF ∴ OK
Top Bars: #7 @ 9”
Bottom Bars: 1 - #10 and 1-#10
Stirrups: #3 @ 13” for 204”

Interior Span: 764 PLF < 926 PLF ∴ OK
Top Bars: #6 @ 7”
Bottom Bars: 1 - #8 and 1-#9
Stirrups: #3 @ 13” for 167”

Interior Beam Selection:
24” x 28.5”
Top: (5) #14
Bottom: (2) #14
Stirrups (Closed): (16) #5, 1@2”, 25@7”
12.5 PLF > 10.83 PLF ∴ **OK**

Exterior Beam Selection:
24” x 28.5”
Top: (4) #14
Bottom: (2) #14
Stirrups (Closed): (23) #5, 1@2”, 22@8”
10.1 PLF > 6.9 PSF ∴ **OK**

Please refer to CRSI, pages 8-67, 12-93, and 12-107, which can be found in the Appendix, for dimensions, reinforcing details, and properties of members.

System Description

This system consists of cast-in-place, one-way wide module joists spanning 46’, 30’, and 43’ in the East / West direction. The joists span into cast-in-place beams that span 30’ along the North / South direction. The column grid of the existing system was used in this design.

System Design

Please refer to the Appendix for detailed calculations and design assumptions.

The 2002 CRSI Design Handbook was used to size the one-way, wide module joists and their supporting interior and exterior beams. The joists and beams are oriented this way so that the beams would not have to span 46’ thus minimizing the beam member sizes. A 4.5” slab is the minimum for having a fire resistance rating for the floor assembly.

Several joist sizes were considered, but the one selected was chosen because it had the lightest self-weight. All of the joists that were able to span 46’ had a rib depth of 24” and a slab depth of 4.5”. The beams were also designed using the 2002 CRSI Handbook and the beams were selected to span 30’ and to have a depth of 28.5” equal to that of the joists.

Comparison to the Existing System

Depth

The maximum depth of the one-way wide module joist system and the beams is 28.5”. This depth at the beam supports is shallower than the slab on deck composite beam system which has an overall depth of 30.25” at the beams. Spanning throughout the bay, the wide module joists have a 4.5” depth, whereas the composite beam system has a 6.25” depth.

Member Sizes

The cast-in-place concrete beams are deeper and wider and have more mass over the existing steel framing system. The columns sizes would have to be approximately 24” square or wider in order to support the 24” wide beams.

Impact on the Existing Foundations

The cast-in-place framing system will weigh significantly more than the existing steel framing system. The concrete beams used are very large and will weigh a lot more than the steel framing. The wide module joists have a self weight of 119 PSF which is significantly more than the 56 PSF accounting for the slab on deck and concrete ponding of the existing design. The foundations will need to be sized larger in order to accommodate for the significant increase in weight of the structure.

Advantages

Depth

In considering the overall depth of the floor system at the supporting beams, this system is slightly shallower than the existing system. The 4.5” slab is also less than the 6.25” slab on deck.

Fire Resistance

The 4.5” slab depth ensures a fire resistance rating.

Resistance of Lateral Loads

This one-way wide module joist system is a very sizable and rigid floor framing system and would probably help resist the lateral loads.

Disadvantages

Construction

The one-way wide module joists will require lots of construction time to form, pour and cure. It may also require a significant shoring system which is not currently needed in the existing design.

Weight

Being an all cast-in-place concrete system, the weight of the structure will significantly increase, requiring larger foundations.

Site Limitations

Since this an all cast-in-place concrete system, a concrete batch plant may be necessary on site. The Regent’s downtown site may not be able to accommodate a batch plant if one is necessary for this system.

Labor

This system will involve a large construction labor force in order to form and pour all of the cast-in-place concrete.

Column Size

The larger mass columns may be undesirable in an open floor plan office building.

Cost

The construction and material costs would be significant with this system. There is a lot of concrete material that needs to be formed, poured, and cured for a large office building. The labor costs would be very high.

Other Considerations

Lateral Load Resisting System

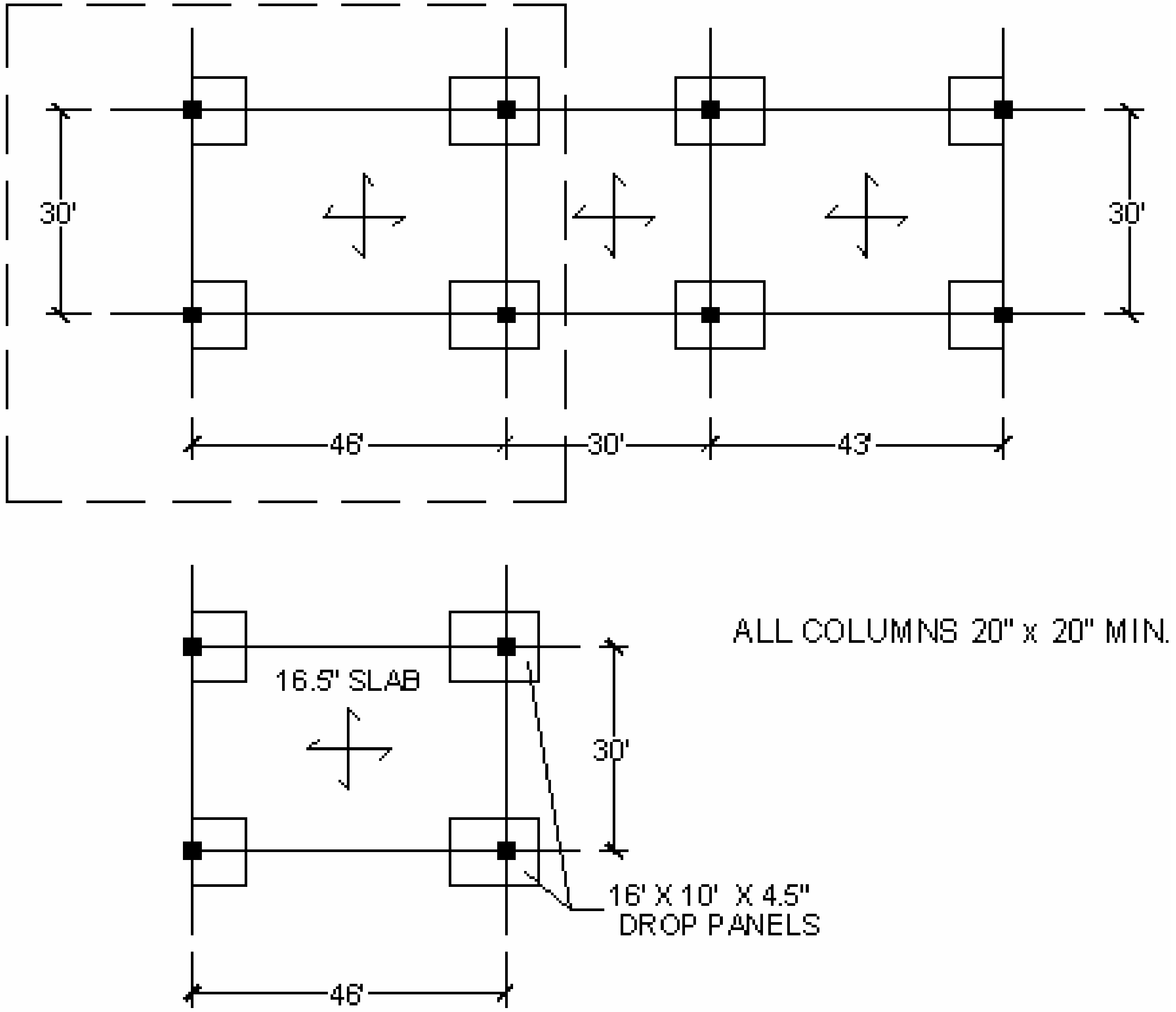
The lateral load resisting system will need to be changed to a concrete system and sized to handle the increased dead load of the building.

Smaller Spans

The overall depth and size of the framing members would be reduced if smaller bay sizes were introduced. Smaller bay sizes would require more columns which may be undesirable for an upscale spec office building where and open floor plan is the most efficient and optimum design.

Two-way Flat Slab with Drop Panels with Cast-In-Place Framing System

Typical Floor Framing Plan for Two-way Flat Slab with Drop Panels with Cast-In-Place Framing System



System Description

This floor system is a two-way, cast-in-place flat slab system with drop panels and cast-in-place framing members designed with the existing system's column grid.

System Design

Please refer to the Appendix for detailed calculations and design assumptions.

The layout and loading of this structure met ACI 318-02 requirements for the use of the Direct Design Method to design this two-way slab. The Direct Design Method was used to design the slab and the drop panels in both the long and the short span directions. 20" x 20" columns were assumed for the initial calculations. In actuality, the columns would need to be significantly larger. The minimum slab thickness for this slab and these spans, according to ACI Table 9.5(c), is 16.5" and 21" at the drop panels. Since the depth of the slab is greater than 12", it is not practical or constuctible.

Comparison to the Existing System

Depth

The maximum depth of this system is 21" at the drop panels and is 16.5" throughout the span of the bay. The maximum depth of the existing system, 30.25" is greater than the maximum depth of this system.

Member Sizes

This system was designed without any interior or exterior beams. The columns will need to be very large in order to handle the very heavy and deep two-way slab.

Impact on the Existing Foundations

This system would be extremely heavy and the foundations would definitely need to be larger.

Advantages

Fire Rating

A 16.5" slab would have a good fire rating.

Disadvantages

Depth

The overall depth of this system, 16”, is desirable compared to some of the deeper systems, but it is not practical since the entire depth is solid concrete.

Constructibility

A 16.5” and 21” solid slab are not practical and are not constructible for the existing bay sizes.

Weight

The weight of a structure with a solid slab 16.5” deep would be very heavy and would impact the foundations greatly.

Cost

The cost of this system would be extremely expensive. Material costs, labor costs, shoring cost, forming costs, and rebar costs.

Other Considerations

Two-Way Post-tensioning

Since a two-way cast-in-place concrete system with drop panels is not practical or constructible with the existing bay sizes, post-tensioning may be a consideration in order to be able to use a thinner, more practical slab thickness.

Smaller Bay Sizes

Smaller bay sizes would reduce the size of the slab and the columns, but would require more columns. More columns may be undesirable in an upscale spec office building.

System Comparison Chart

System	Pros	Cons	Considerations
Existing Composite Slab on Metal Deck with Composite Steel Beams and Steel Framing	• Lighter structure • Quick construction • Smaller foundations • Relatively small depths • Smaller columns sizes • Can efficiently accommodate longer spans	• Concrete ponding over the long spans • Lots of beams	• None at this point
Precast Hollow-Core Planks / Steel Framing	• Quick construction • Relatively smaller foundations • Lighter structure • Smaller column sizes • Quality control • Relatively small depths • Less steel beams needed per bay • Good fire rating • Good acoustical value	• Lots of deliveries to a downtown site • Angle detailing to support the planks • Deeper, heavier steel members • Material costs	• Composite action between the steel beams and the hollow-core planks • Prefabrication of angles to the webs • Adding infill beams to get smaller beam and plank sizes • Untopped planks for a lighter section
Precast Double Tees / Precast Framing	• Quick construction • Quality control • Good fire resistance • Can accommodate longer spans • Less labor intensive • Less labor costs • Good acoustical value • Double tee self weight comparable to slab on deck weight	• Larger foundations • Deep flooring system • Heavy beams and columns • Lots of deliveries to a downtown site • Material costs	• Smaller bay sizes • Shallower supporting members (not flush)
CIP One-way Wide Module Joists / CIP Framing	• Uniform depth • Rigid floor system • Slab and supporting beam depths are less than existing depths • Can accommodate longer spans • Good fire rating	• Larger foundations • Heavy structure • Labor intensive • Longer construction time • More field labor intensive • Larger column sizes • Forming and shoring system required • Labor costs	• Smaller bay sizes, more columns
CIP Two-way Flat Slab with Drop Panels / CIP Framing	• Good fire resistance	• Not practical from a constructability, cost, labor, standpoint for the existing bay sizes • Very heavy structure • Larger foundations • Larger column sizes • Extensive forming and shoring systems required • Material and labor costs	• Two-way post-tensioning • Smaller bay sizes, more columns

Final Summary and Recommendations

After completing the designs and discussing the advantages and disadvantages for each floor system, it is recommended that the hollow-core planks with steel framing, the precast double tees with precast framing, and the one-way joists with cast-in-place framing systems be studied further.

The existing system has proven to be a very efficient system with many advantages and few disadvantages. Some of the advantages include: relatively small member sizes and self weights, smaller floor system depths, and being able to span the longer spans in the bays. Some disadvantages include: more framing members and likelihood that the long span steel system will cause concrete ponding due to deflection.

The two-way flat slab with drop panels should not be studied further as a two-way CIP system with the existing bay sizes. A 16.5" slab is not practical and not easily constructible. Switching to a two-way post-tensioning system may thin out the slab depth making a post-tensioning system a practical option.

The cast-in-place, one-way, wide module joists have both several advantages and disadvantages. The structure, as preliminarily designed, would weigh a lot more than the existing system and would require larger foundations. Also, the amount of labor that needs to be done on site would require a lot of construction time and field labor, which can be expensive. For a spec office building, construction time is very critical and would be very risky for the involved placement of the cast-in-place concrete joist system. However, this system does provide a uniform depth that does not exceed the existing design's maximum depth. This system also has a good fire rating and can accommodate the longer spans in the larger bay sizes. Considering more columns and smaller bay sizes may reduce the size of the framing members and the entire structural system may be more efficiently designed as a result.

The hollow-core plank system has several advantages over the existing structure including quicker construction time since the hollow-core planks are precast, the quality control advantage of the planks being precast in a plant, good fire rating, good acoustical value, and less steel beams per bay. Some disadvantages discussed include the labor and cost going into the angle connection to hold the hollow-core planks for a flush floor system, the downtown site being able to accommodate the extra precast deliveries, and the increased beam depths and weights and their effects on the foundations and floor depth.

The precast double tees with precast framing member system is also another possible good alternative. Its advantages over the existing system include: concrete quality control, quick construction time, lighter self weight of the double tees, good fire resistance, and good acoustical value. The disadvantages include heavier beams and columns and the resulting larger foundations, the extra deep depth of the flooring system, and the downtown site being able to accommodate all of the precast deliveries.

All of the alternative systems that have been discussed will be studied further either as a continuation of the preliminary design or a modified design based on what has been learned in from this report.

Appendix

Existing Structural System Check

Existing Structural Floor System Check

Loads:

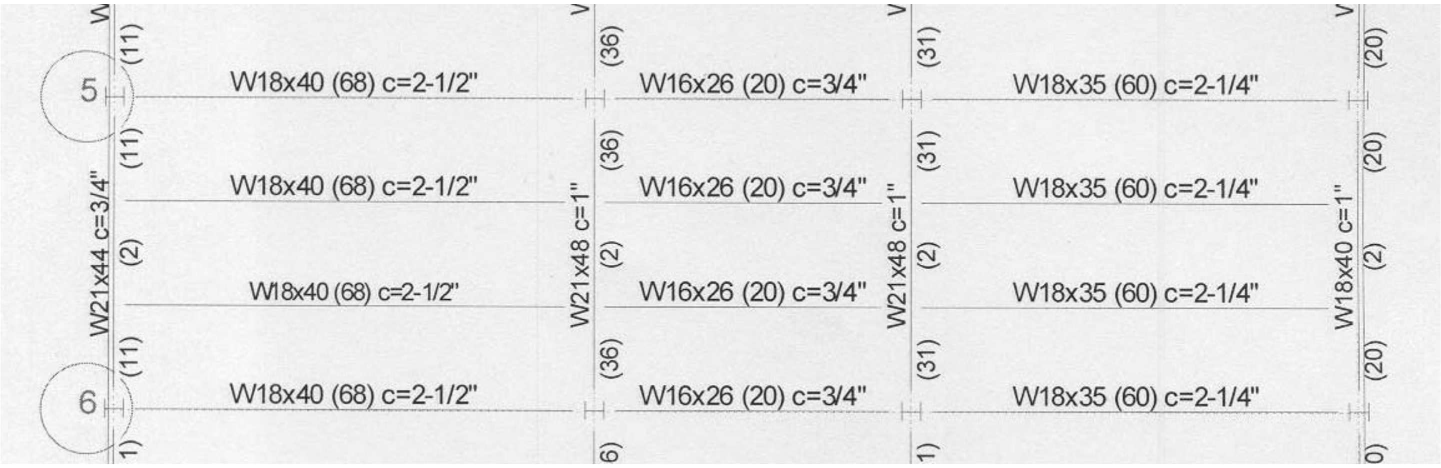
Dead:

3 ¼” lt. wt. slab on 3” - 20 gage metal deck	46 PSF
Concrete Ponding	10 PSF
Misc. DL	15 PSF
Façade	15 PSF
Construction DL	56 PSF

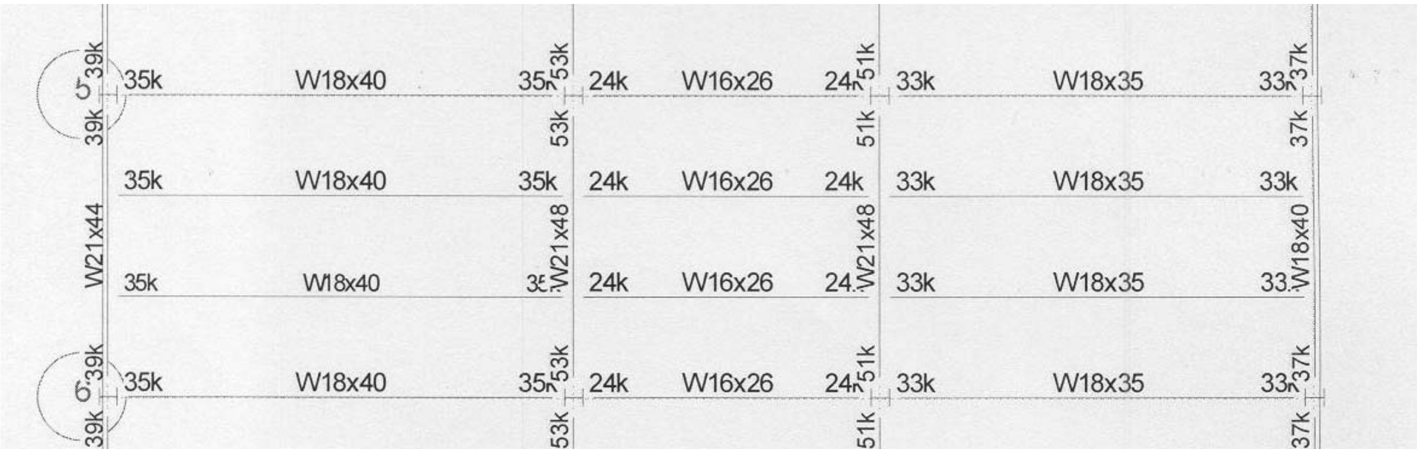
Live:

Office	100 PSF (reducible)
Construction LL	20 PSF

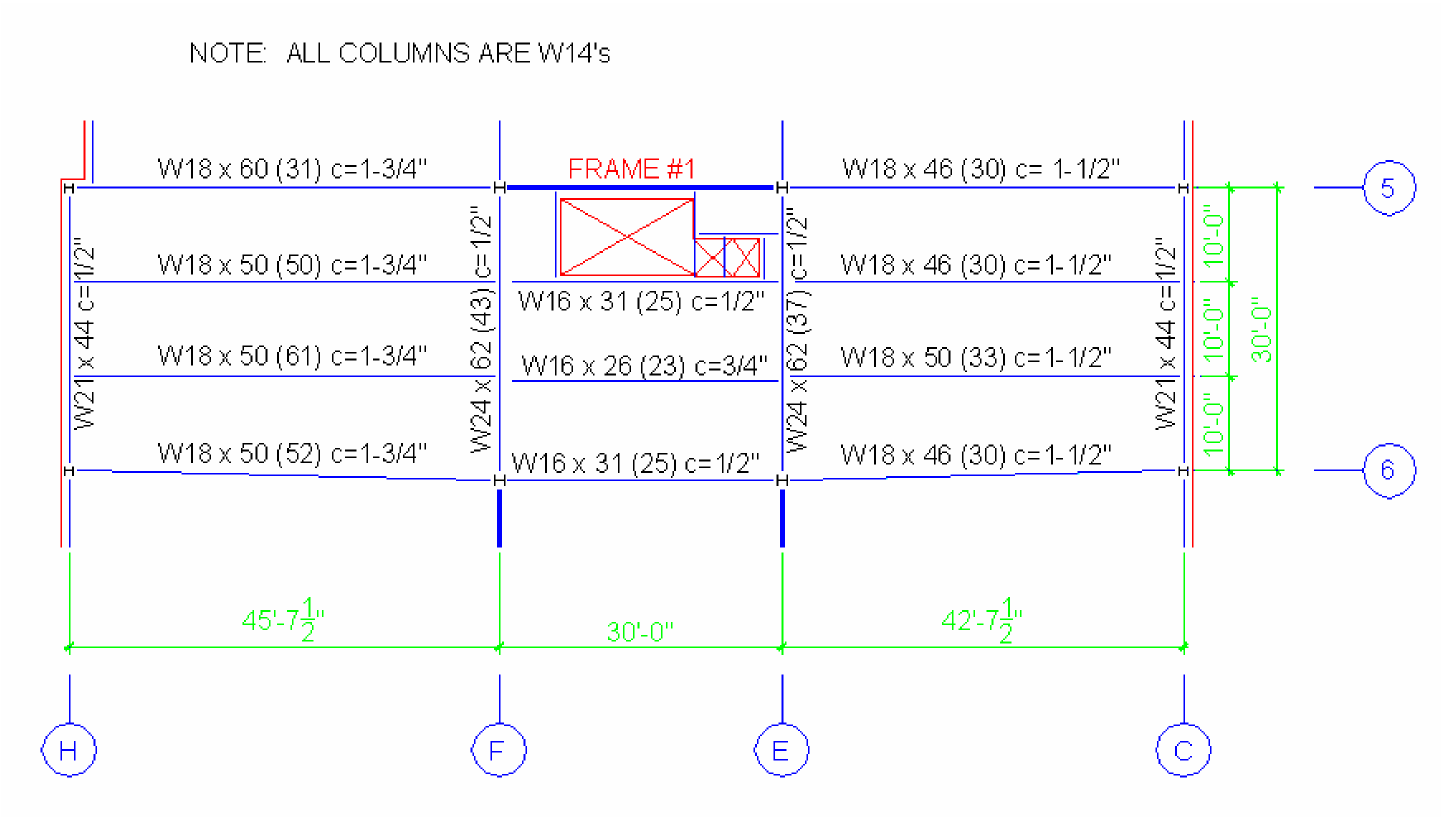
Typical Floor Framing of Existing – RAM Output Member Sizes, Number of Shear Studs, and Cambers



Typical Floor Framing Plan of Existing Composite Steel and Concrete Deck – RAM Output Unfactored Reactions

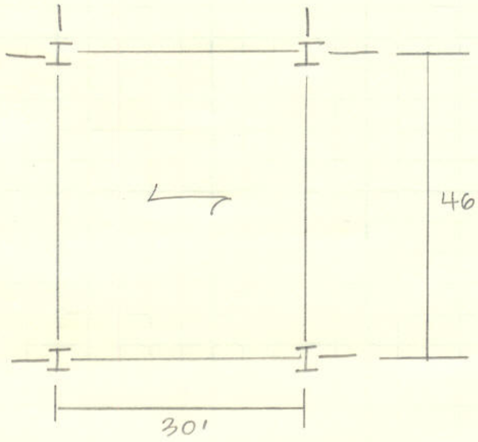


Typical Frame (Existing)



Hollow-Core Planks with Steel Framing System

Hollow-core plank Design



Safe Superimposed Service Load:

Dead:
Misc. DL = 15 PSF

Live:
Office: 100 PSF TA = 30' (4') = 120 Ft^2 < 400 Ft^2
∴ Not-reducible

Total safe-superimposed service Load = 115 PSF

span = 30'

Hollow-Core Precast Plank Calculations and Selection

Service Loads:

Dead
    Misc. DL – 15 PSF

Live
    Office Space – 100 PSF

Total Safe Superimposed Service Load = 115 PSF

Maximum Span: 30 ft

Information Taken From PCI Design Handbook, 5th edition.

PCI Designation	Width (ft)	Depth (in)	2" Normal Weight Topping	Total Depth (in)	LW vs. Normal Weight	Safe Superimposed Service Load (PSF)	Strand Designation Code	Self Weight (PSF)
4LHC8+2	4	8	YES	10	LW	125	68-S	68*
4HC8+2	4	8	YES	10	NW	138	78-S	81
4HC10+2	4	10	YES	12	NW	128	58-S	93
4LHC12+2	4	12	YES	14	LW	160	58-S	93
4HC12+2	4	12	YES	14	NW	124	76-S	77

*denotes lightest design

Selection: **4LHC8+2**
125 PSF > 115 PSF $\therefore$ OK

Try 4'-0" x 8" lightweight, hollow-core planks with 2" normal weight topping
Self Weight = 68 PSF > 46 PSF + 10 PSF = 56 PSF

Strand Pattern Designation

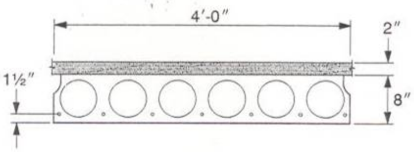
76–S
S = straight
Diameter of strand in 16ths
No. of strand (7)

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. Check availability of lightweight sections.

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

HOLLOW-CORE

4′-0″ x 8″
Lightweight Concrete



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

Section Properties

		Untopped		Topped	
A	=	215	in²	—	
I	=	1,666	in⁴	3,529	in⁴
y_b	=	4.00	in.	5.70	in.
y_t	=	4.00	in.	4.30	in.
S_b	=	416	in³	619	in³
S_t	=	416	in³	821	in³
b_w	=	12.00	in.	12.00	in.
wt	=	184	plf	272	plf
		46	psf	68	psf
V/S	=	1.92	in.		

Key
346 — Safe superimposed service load, psf
0.3 — Estimated camber at erection, in.
0.4 — Estimated long-time camber, in.

4LHC8

Table of safe superimposed service load (psf) and cambers (in.)

No Topping

Strand Designation Code	Span, ft																						
	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
66-S	346	297	257	224	196	172	152	135	120	107	95	85	76	68	61	55	49	44	39	35			
	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2	0.1	0.0			
	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.0	−0.1	−0.3	−0.5	−0.8			
76-S		348	302	263	231	204	181	161	144	129	115	104	93	84	76	68	62	56	50	45	41	36	
		0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	
		0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.3	0.1	−0.1	−0.3	−0.6	−0.9	
58-S		350	325	304	286	265	236	211	189	170	154	139	126	114	104	95	86	79	72	66	60	55	50
		0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.7
		0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	0.9	0.8	0.7	0.5	0.2	0.0
68-S			334	313	292	274	258	243	229	206	187	169	154	140	128	117	107	98	90	83	76	70	64
			0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.5	1.5	1.4
			1.0	1.1	1.2	1.2	1.3	1.4	1.5	1.6	1.6	1.7	1.7	1.7	1.7	1.7	1.7	1.6	1.5	1.4	1.3	1.1	0.9
78-S			343	319	301	283	267	249	237	225	212	197	181	165	151	139	127	117	108	100	92	85	78
			0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.7	1.8	1.9	2.0	2.0	2.1	2.1	2.1	2.2	2.2	2.1	2.1
			1.2	1.3	1.4	1.5	1.6	1.8	1.9	2.0	2.0	2.1	2.2	2.2	2.3	2.3	2.3	2.3	2.2	2.2	2.1	2.0	1.8

4LHC8+2

Table of safe superimposed service load (psf) and cambers (in.)

2″ Normal Weight Topping

Strand Designation Code	Span, ft																						
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
66-S	320	277	242	211	186	163	144	127	113	100	88	78	69	60	53	45							
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2							
	0.4	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2	0.0	−0.1	−0.3	−0.5	−0.7	−1.0							
76-S		327	286	251	222	196	174	155	138	123	109	98	87	77	69	61	52	43					
		0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3					
		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.1	−0.1	−0.3	−0.6	−0.9	−1.2					
58-S				327	290	258	231	206	185	167	150	135	122	110	99	90	81	72	62	53	45		
				0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	1.0	0.9	0.8	0.7		
				0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.9	−1.3		
68-S					323	304	278	250	225	204	184	167	151	138	125	114	103	93	83	73	64	56	48
					1.1	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2
					1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.3	1.3	1.2	1.1	0.9	0.8	0.6	0.3	0.0	−0.3	−0.7	−1.2
78-S					332	313	297	279	263	238	216	197	179	163	149	136	125	113	102	91	81	72	64
					1.3	1.4	1.5	1.6	1.7	1.7	1.8	1.9	2.0	2.0	2.1	2.1	2.1	2.2	2.2	2.2	2.1	2.1	2.0
					1.5	1.6	1.7	1.7	1.8	1.8	1.8	1.8	1.8	1.8	1.7	1.6	1.5	1.3	1.1	0.9	0.6	0.2	−0.1

Strength based on strain compatibility; bottom tension limited to $6\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.

Steel Design For Hollow-core Planks

Loads:

Dead:

Misc. DL = 15 PSF
Planks = 68 PSF
83 PSF

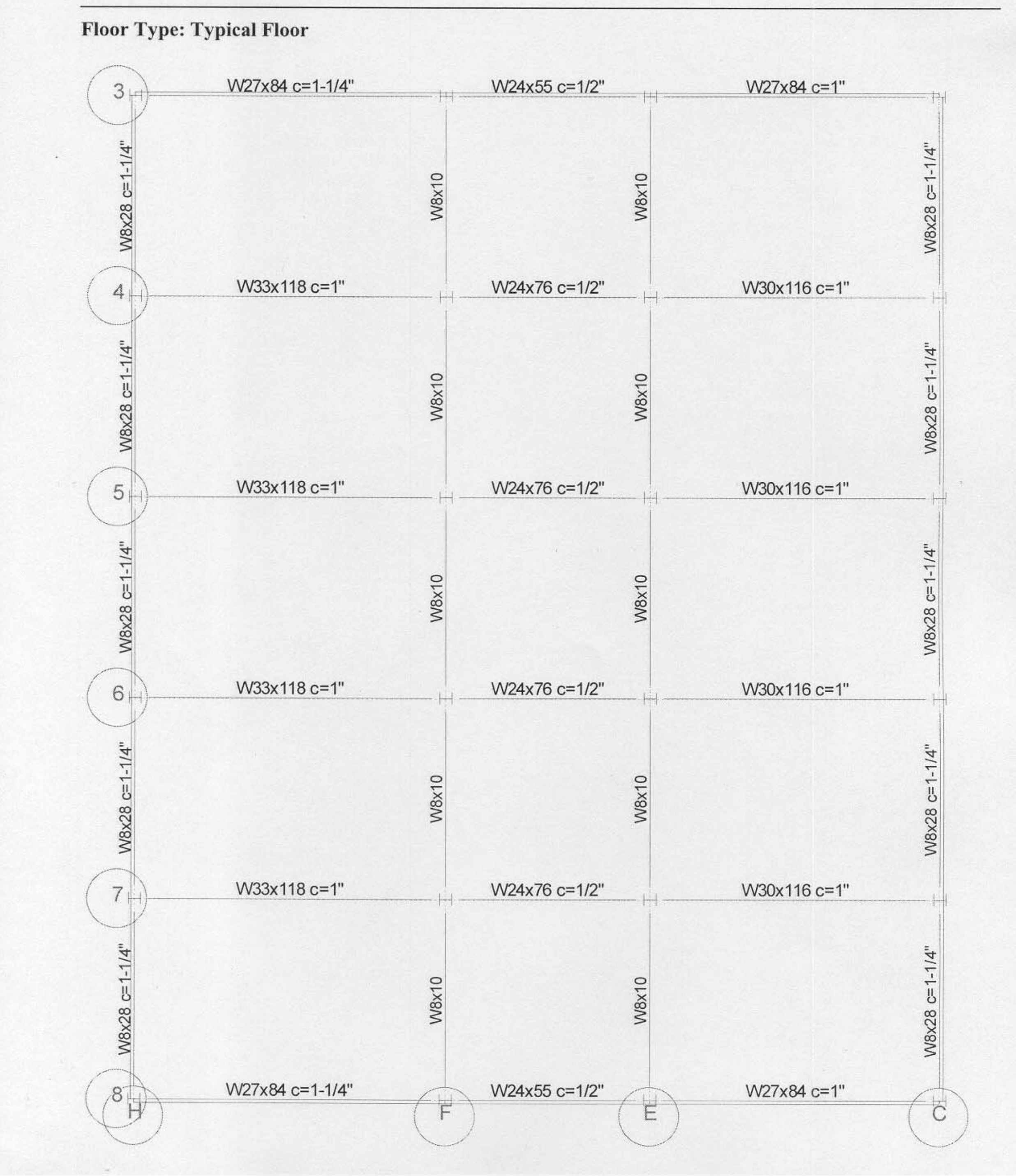
Live:

Office = 100 PSF (reducible)

construction:

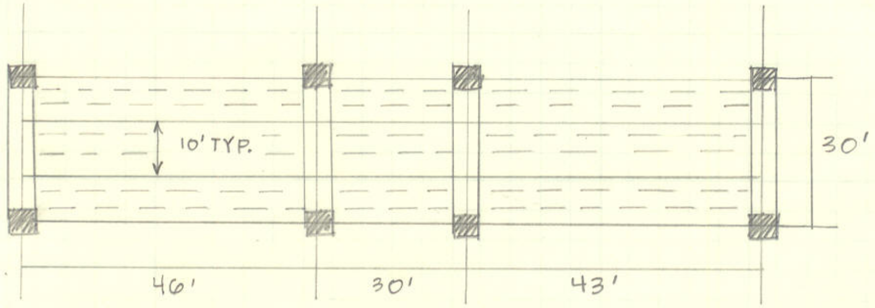
Dead = 68 PSF
Live = 20 PSF

Results of Steel Framing Member Design for Hollow-core Plank Flooring System – RAM Output



Precast Double Tees with Precast Framing System

Precast Double Tees | Precast Framing



Precast Double Tee Calculations and Selection

Loads

Dead:

Misc. DL = 15 PSF (HVAC, sprinklers, plumbing, ceiling, etc.)

Live:

Office = 100 PSF → 95 PSF

Live Load Reduction (ASCE 7-02)

$$L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right)$$

$K_{LL} = 1$ other

$A_T = 46'(10') = 460 \text{ FT}^2$

$L_o = 100$ PSF

$$L = 100 \text{ PSF} \left(0.25 + \frac{15}{\sqrt{1 \cdot 460 \text{ FT}^2}} \right)$$

$L = 95$ PSF

Total safe superimposed service load = 15 + 95 = 110 PSF

Worst-case span = 46'

Try 10' wide double tee : $\frac{30'}{10'}$ = 3 double tees | bay

Precast Double Tee Calculations and Selection

Summary of possible double tee sections taken from PCI Design Handbook, 5th edition.

PCI Designation	Strand Pattern	LW* vs. NW*	Additional 2" Normal Weight Topping?	Self Weight (PSF)	Total Depth (IN)	Width (FT)	Safe Superimposed Service Load (PSF)
Double Tee							
10LDT32	128-D1	LW	No	49	32	10	130
10LDT32+2	108-D1	LW	Yes	74	34	10	150
10DT32	128-D1	NW	No	64	32	10	182
10DT32+2	108-D1	NW	Yes	89	34	10	138

LW = Lightweight Concrete
NW = Normal Weight Concrete

The self weight of the double tees was accounted for in the member capacities.

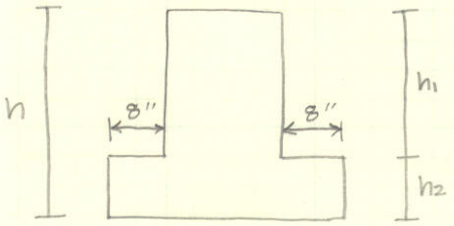
Selection: 10LDT32 110 PSF < 130 PSF $\therefore$ OK

12 strands
8/16" = 0.5" diameter strands
1 depression point
f'c = 5,000 psi
f_{pu} = 270,000 psi
2.4" estimated camber at erection
2.9" estimated long-time camber

This selection was made because this member has the lightest self weight and the smallest depth. Although, a topped section is preferred for stability and the prevention of differential movement between the double tee beams, an untopped section was selected and it is anticipated that a ¾" normal weight topping could be added on top of the double tees and their supporting beams in order to add that stability, yet keep self weight to a minimum. With the added weight of the topping, the new safe superimposed service load increases 110 PSF to 120 PSF, and the member can carry 130 PSF so it is still OK.

120 PSF < 130 PSF $\therefore$ OK

Interior Precast Inverted Tee-Beam calculations and Selection



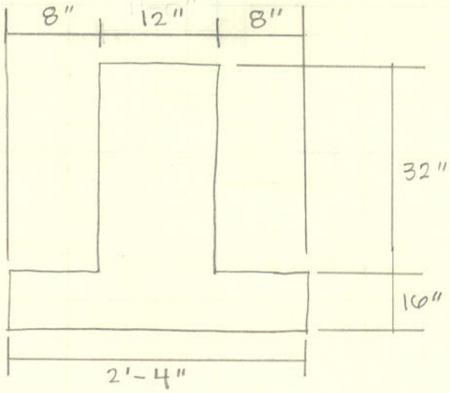
worst case span = 30'

Worst case loading:

$$120 \text{ PSF} \left(\frac{46'+30'}{2}\right) = 4{,}560 \text{ PLF}$$

Try to find a section where $h_1 = 32"$ for a flush top finish between double tee and inverted tee prior to adding 3/4" topping

Selection: 28IT48



<u>Interior Precast Inverted Tee Beam Calculations and Selection</u>

In order to get a flush finish across the top of the double tees and the supporting inverted tee beam prior to adding the additional ¾" topping, h_1 has to be 32".

Summary of possible inverted tee beam sections taken from PCI Design Handbook, 5th edition.

PCI Designation	**h (IN)**	**h_1 (IN)**	**h_2 (IN)**	**Self weight (PLF)**	**Safe Superimposed Service Load Capacity (PLF)**
Members with h_1 = 32"					
28IT48	48	32	16	867	At least 9,741
34IT48	48	32	16	1,167	At least 9,049
40IT48	48	32	16	1,467	At least 9,808
Members selected based on capacity					
28IT32	32	20	12	600	4,698
34IT28	28	16	12	725	5,316
40IT24	12	12	12	800	5,060

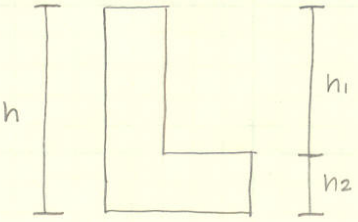
Selection: 28IT48 4,560 PLF < 9,741 PLF ∴ OK

22 strands
8/16" = 0.5" diameter strands
low-lax strands
f'c = 5,000 psi
f_{pu} = 270,000 psi
0.4" estimated camber at erection
0.1" estimated long-time camber

This member was selected because it was the lightest section that had an h_1 = 32" so that the top of the double tees and the top of the inverted tee beam would be flush. The disadvantage is that the inverted tee beam will extend 16" below the bottom of the double tees. This should not be a significant problem since the interior inverted tee beams will span along the perimeter of the central core.

Possible alternative tee beam members of smaller sizes were listed, but the top of the double tee beam will be higher than the top of the inverted tee beam. This difference in height will result in a void that needs to be filled in order to have a continuous flat floor finish. Even if a smaller section was selected, the bottom of the beam would still extend beyond the bottom of the double tee beams because of the depth of the flanges that the double tee beams need to rest on.

Exterior Precast L-Beam calculations and selection



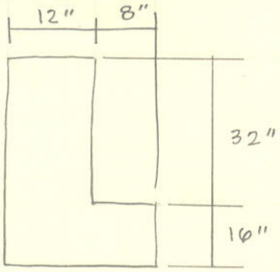
Worst case span = 30'

Wost case loading:

$$120 \text{ PSF}\left(\frac{46'}{2}\right) = 2760 \text{ PLF}$$

Try to find a section where $h_1 = 32''$ for a flush top finish between double tee and L-beam prior to adding 3/4" topping.

Selection: 20LB48



Exterior Precast L-Beam Calculations and Selection

In order to get a flush finish across the top of the double tees and the supporting inverted tee beam prior to adding the additional ¾" topping, h_1 has to be 32".

Summary of possible L-beam sections taken from PCI Design Handbook, 5th edition.

PCI Designation	h (IN)	h_1 (IN)	h_2 (IN)	Self weight (PLF)	Safe Superimposed Service Load Capacity (PLF)
Members with h_1 = 32"					
20LB48	48	32	16	733	At least 9,231
26LB48	48	32	16	1,033	At least 9,590
Members selected based on capactiy					
20LB28	28	16	12	450	3,416
26LB24	24	12	12	550	3,718

Selection: 20LB48 **2,760 PLF < 9,231 PLF ∴ OK**

21 strands
8/16" = 0.5" diameter strands
low-lax strands
f'c = 5,000 psi
f_{pu} = 270,000 psi
0.5" estimated camber at erection
0.2" estimated long-time camber

This member was selected because it was the lightest section that had an h_1 = 32" so that the top of the double tees and the top of the L-beam would be flush. The disadvantage is that the L-beam will extend 16" below the bottom of the double tees. This should not be a significant problem since the interior inverted tee beams will span along the perimeter of the central core.

Possible alternative L-beam members of smaller sizes were listed, but the top of the double tee beam will be higher than the top of the L-beam. This difference in height will result in a void that needs to be filled in order to have a continuous flat floor finish. Even if a smaller section was selected, the bottom of the beam would still extend beyond the bottom of the double tee beams because of the depth of the flange that the double tee beams need to rest on.

Strand Pattern Designation

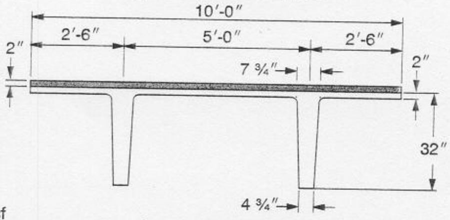
No. of strand (10)
S = straight D = depressed
108-D1
No. of depression points
Diameter of strand in 16ths

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.

Key
130 — Safe superimposed service load, psf
2.4 — Estimated camber at erection, in.
2.9 — Estimated long-time camber, in.

DOUBLE TEE

10′-0″ x 32″
Lightweight Concrete



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

Section Properties

		Untopped		Topped	
A	=	615	in²	—	
I	=	59,720	in⁴	83,019	in⁴
y_b	=	21.98	in.	25.40	in.
y_t	=	10.02	in.	8.60	in.
S_b	=	2,717	in³	3,268	in³
S_t	=	5,960	in³	9,652	in³
wt	=	491	plf	741	plf
		49	psf	74	psf
V/S	=	1.69	in.		

10LDT32

Table of safe superimposed service load (psf) and cambers (in.)

No Topping

Strand Pattern	e_e, in. e_c, in.	Span, ft																						
		54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90	92	94	96	98
128-D1	12.81 18.73	130 2.4 2.9	118 2.5 3.0	108 2.6 3.0	98 2.7 3.0	89 2.7 3.0	82 2.7 2.9	74 2.8 2.9	68 2.8 2.8	62 2.8 2.7	56 2.7 2.5	51 2.7 2.3	47 2.6 2.1	42 2.5 1.8	38 2.4 1.5	35 2.2 1.1	31 2.0 0.6							
148-D1	10.48 18.48	153 2.7 3.3	139 2.8 3.4	127 2.9 3.5	116 3.0 3.5	107 3.1 3.6	98 3.2 3.6	89 3.3 3.6	82 3.3 3.5	75 3.4 3.4	69 3.3 3.2	63 3.3 3.0	58 3.3 2.9	53 3.2 2.6	49 3.1 2.3	44 3.0 2.0	40 2.8 1.6	37 2.6 1.2	33 2.4 0.7					
168-D1	8.98 18.23	175 2.9 3.7	160 3.1 3.8	147 3.2 3.9	135 3.3 4.0	124 3.5 4.1	114 3.6 4.1	105 3.7 4.2	96 3.8 4.2	89 3.8 4.1	82 3.9 4.0	75 3.9 3.9	69 4.0 3.8	64 3.9 3.5	59 3.9 3.2	54 3.8 2.9	50 3.6 2.6	46 3.5 2.2	42 3.3 1.7	38 3.1 1.2	35 2.8 0.7			
188-D1	7.59 17.98							119 4.0 4.7	110 4.1 4.7	101 4.2 4.7	94 4.3 4.7	87 4.4 4.6	80 4.4 4.5	74 4.4 4.3	69 4.4 4.1	63 4.4 3.8	59 4.4 3.5	54 4.3 3.1	50 4.1 2.6	46 3.9 2.1	42 3.7 1.6	39 3.4 1.0	36 3.1 0.4	
208-D1	6.48 17.73													84 4.9 5.0	78 4.9 4.8	72 4.9 4.6	67 4.9 4.3	62 4.9 4.0	58 4.8 3.6	53 4.7 3.1	48 4.5 2.6	44 4.3 2.0	41 4.0 1.3	38 3.7 0.6
228-D1	5.57 17.48																	70 5.4 4.8	64 5.4 4.5	59 5.3 4.1	55 5.2 3.6	50 5.0 3.0	46 4.8 2.4	42 4.6 1.6

10LDT32+2

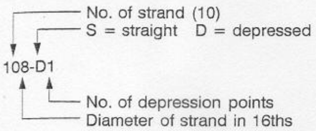
Table of safe superimposed service load (psf) and cambers (in.)

2″ Normal Weight Topping

Strand Pattern	e_e, in. e_c, in.	Span, ft																				
		42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82
108-D1	16.08 18.98	192 1.5 1.5	169 1.6 1.5	150 1.7 1.6	133 1.8 1.6	118 1.9 1.6	105 2.0 1.6	93 2.1 1.5	82 2.1 1.4	73 2.2 1.3	64 2.2 1.2	56 2.2 1.0	49 2.3 0.8	43 2.3 0.5								
128-D1	12.81 18.73			188 2.0 1.9	168 2.1 2.0	150 2.1 1.9	135 2.3 2.0	121 2.4 2.0	108 2.5 1.9	97 2.6 1.8	87 2.7 1.7	77 2.7 1.5	69 2.8 1.3	61 2.8 1.1	55 2.8 0.8	48 2.8 0.5						
148-D1	10.48 18.48				199 2.3 2.2	178 2.4 2.3	161 2.5 2.3	145 2.7 2.3	130 2.8 2.3	118 2.9 2.3	106 3.0 2.2	96 3.1 2.1	86 3.2 1.9	77 3.3 1.7	70 3.3 1.4	62 3.4 1.1	56 3.3 0.7	50 3.3 0.3				
168-D1	8.98 18.23							168 2.9 2.7	152 3.1 2.7	138 3.2 2.7	125 3.3 2.6	113 3.5 2.5	103 3.6 2.4	93 3.7 2.2	84 3.8 2.0	76 3.8 1.8	69 3.9 1.5	62 3.9 1.1	56 4.0 0.7			
188-D1	7.59 17.98													108 4.0 2.7	99 4.1 2.5	90 4.2 2.3	82 4.3 2.0	74 4.4 1.7	67 4.4 1.3	61 4.4 0.8	55 4.4 0.3	
208-D1	6.48 17.43																		78 4.8 1.9	71 4.9 1.5	65 4.9 1.0	58 4.9 0.4

Strength based on strain compatibility; bottom tension limited to 12$\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.
Shaded values require release strengths higher than 3500 psi.

Strand Pattern Designation

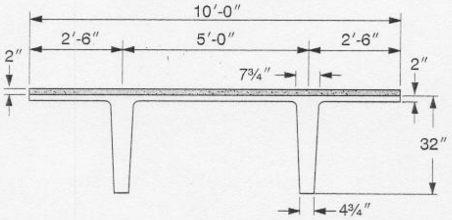


Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Key
182 — Safe superimposed service load, psf
1.2 — Estimated camber at erection, in.
1.6 — Estimated long-time camber, in.

DOUBLE TEE

10′-0″ x 32″
Normal Weight Concrete



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

Section Properties

		Untopped		Topped	
A	=	615	in²	—	
I	=	59,720	in⁴	77,131	in⁴
y_b	=	21.98	in.	24.54	in.
y_t	=	10.02	in.	9.46	in.
S_b	=	2,717	in³	3,142	in³
S_t	=	5,960	in³	8,149	in³
wt	=	641	plf	891	plf
		64	psf	89	psf
V/S	=	1.69	in.		

10DT32

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

Strand Pattern	e_e, in. e_c, in.	Span, ft																				
		46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88
128-D1	12.81 18.73	182 1.2 1.6	163 1.3 1.6	146 1.3 1.6	131 1.4 1.7	118 1.4 1.7	106 1.4 1.7	95 1.5 1.7	86 1.5 1.6	77 1.4 1.6	69 1.4 1.5	62 1.4 1.4	55 1.3 1.2	49 1.2 1.0	44 1.1 0.8	39 1.0 0.5							
148-D1	10.48 18.48		191 1.4 1.8	172 1.5 1.9	155 1.6 1.9	140 1.6 2.0	127 1.7 2.0	115 1.7 2.0	104 1.7 2.0	94 1.8 1.9	85 1.7 1.9	77 1.7 1.8	70 1.7 1.7	63 1.6 1.5	57 1.5 1.4	51 1.4 1.2	46 1.3 0.9	41 1.2 0.5					
168-D1	8.98 18.23			199 1.6 2.1	180 1.7 2.2	163 1.8 2.2	148 1.9 2.3	134 1.9 2.3	122 2.0 2.3	111 2.0 2.3	101 2.0 2.3	92 2.1 2.2	84 2.1 2.1	76 2.0 2.0	69 2.0 1.9	63 1.9 1.7	57 1.8 1.5	52 1.7 1.2	46 1.5 0.9	42 1.3 0.5			
188-D1	7.59 17.98								140 2.2 2.6	127 2.2 2.6	117 2.3 2.6	107 2.3 2.6	97 2.3 2.5	89 2.3 2.4	81 2.3 2.3	74 2.3 2.1	68 2.2 2.0	62 2.1 1.8	56 2.0 1.5	51 1.8 1.2	46 1.7 0.8		
208-D1	6.48 17.73												110 2.5 2.9	101 2.6 2.8	93 2.6 2.7	85 2.6 2.6	78 2.5 2.4	72 2.5 2.2	66 2.4 2.0	60 2.3 1.7	55 2.2 1.4	50 2.0 1.0	
228-D1	5.57 17.48																88 2.8 2.9	81 2.8 2.7	75 2.7 2.5	69 2.6 2.2	63 2.5 1.9	58 2.4 1.5	53 2.2 1.2

10DT32+2

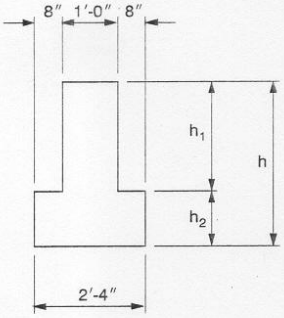
Table of safe superimposed service load (psf) and cambers (in.) **2″ Normal Weight Topping**

Strand Pattern	e_e, in. e_c, in.	Span, ft																
		42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76
108-D1	16.08 18.98	179 0.9 0.9	157 1.0 1.0	138 1.1 1.0	121 1.1 0.9	106 1.1 0.9	92 1.2 0.9	81 1.2 0.8	70 1.2 0.7	60 1.2 0.5	52 1.2 0.4								
128-D1	12.81 18.73		199 1.1 1.1	176 1.2 1.2	156 1.3 1.2	138 1.3 1.2	122 1.4 1.1	108 1.4 1.1	96 1.4 1.0	85 1.5 0.9	74 1.5 0.7	65 1.4 0.6	57 1.4 0.4						
148-D1	10.48 18.48				186 1.4 1.4	166 1.5 1.4	148 1.6 1.4	132 1.6 1.3	118 1.7 1.3	1.5 1.7 1.2	94 1.7 1.1	83 1.8 0.9	74 1.7 0.8	65 1.7 0.5	57 1.7 0.3				
168-D1	8.98 18.23					194 1.6 1.6	174 1.7 1.6	156 1.8 1.6	140 1.9 1.5	126 1.9 1.5	113 2.0 1.4	101 2.0 1.3	91 2.0 1.1	81 2.1 0.9	72 2.1 0.7	64 2.0 0.5			
188-D1	7.59 17.98									145 2.1 1.7	131 2.2 1.7	118 2.2 1.6	107 2.3 1.4	96 2.3 1.3	86 2.3 1.1	77 2.3 0.8	69 2.3 0.5	62 2.3 0.2	
208-D1	6.48 17.73														100 2.5 1.4	90 2.6 1.2	82 2.6 0.9	73 2.6 0.6	66 2.5 0.2

Strength based on strain compatibility; bottom tension limited to $12\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.
Shaded values require release strengths higher than 3500 psi.

INVERTED TEE BEAMS

Normal Weight Concrete



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

½ in. diameter
low-relaxation strand

Key
6,929 — Safe superimposed service load, plf
0.3 — Estimated camber at erection, in.
0.1 — Estimated long-time camber, in.

Section Properties

Designation	h in.	h_1/h_2 in.	A in^2	I in^4	y_b in.	S_b in^3	S_t in^3	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

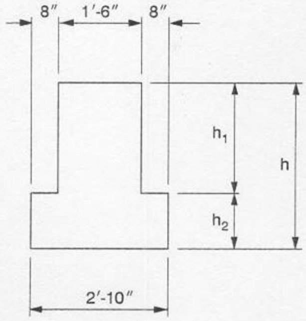
3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Desig-nation	No. Strand	e	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	9	5.82	6929	5402	4310	3502	2887	2409	2029	1723	1473	1265	1091							
			0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7	0.8	0.8							
			0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	−0.1	−0.1							
28IT24	11	6.77	9714	7580	6054	4925	4066	3398	2868	2440	2090	1799	1556	1351	1175	1024				
			0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8				
			0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	−0.1	−0.2				
28IT28	13	8.44			8505	6951	5768	4848	4118	3529	3047	2648	2313	2030	1788	1579	1399	1242	1103	981
					0.3	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1
					0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.0	−0.1
28IT32	15	9.17				9202	7646	6435	5474	4698	4064	3538	3097	2724	2406	2132	1894	1687	1505	1345
						0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9
						0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	−0.1
28IT36	16	10.81						8485	7236	6227	5402	4718	4145	3660	3246	2890	2581	2311	2075	1866
								0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9
								0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	−0.1
28IT40	19	11.28							8615	7415	6433	5620	4938	4361	3868	3444	3077	2756	2475	2226
									0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9
									0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
28IT44	20	12.89								9308	8092	7083	6239	5524	4913	4388	3932	3535	3186	2879
										0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8
										0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
28IT48	22	14.16									9741	8539	7532	6680	5952	5326	4783	4310	3894	3528
											0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8
											0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT52	24	15.44											8935	7934	7080	6345	5707	5151	4664	4233
													0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8
													0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT56	26	16.74												9284	8294	7442	6703	6059	5493	4994
														0.5	0.6	0.6	0.7	0.7	0.8	0.8
														0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT60	28	18.04													9590	8613	7766	7027	6379	5807
															0.6	0.6	0.6	0.7	0.7	0.8
															0.1	0.2	0.2	0.2	0.2	0.2

INVERTED TEE BEAMS

Normal Weight Concrete



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

½ in. diameter
low-relaxation strand

Key
8,164 — Safe superimposed service load, plf
0.4 — Estimated camber at erection, in.
0.1 — Estimated long-time camber, in.

Section Properties								
Designation	**h in.**	**h_1/h_2 in.**	**A in^2**	**I in^4**	**y_b in.**	**S_b in^3**	**S_t in^3**	**wt plf**
34IT20	20	12/8	488	16,082	8.43	1,908	1,390	508
34IT24	24	12/12	624	27,825	10.15	2,741	2,009	650
34IT28	28	16/12	696	44,130	11.79	3,743	2,722	725
34IT32	32	20/12	768	65,856	13.50	4,878	3,560	800
34IT36	36	24/12	840	93,616	15.26	6,136	4,513	875
34IT40	40	24/16	976	128,656	16.85	7,635	5,558	1,017
34IT44	44	28/16	1,048	171,157	18.58	9,212	6,733	1,092
34IT48	48	32/16	1,120	221,906	20.34	10,910	8,023	1,167
34IT52	52	36/16	1,192	281,504	22.13	12,721	9,424	1,242
34IT56	56	40/16	1,264	350,546	23.95	14,637	10,938	1,317
34IT60	60	44/16	1,336	429,623	25.78	16,662	12,556	1,392

1. Check local area for availability of other sizes.

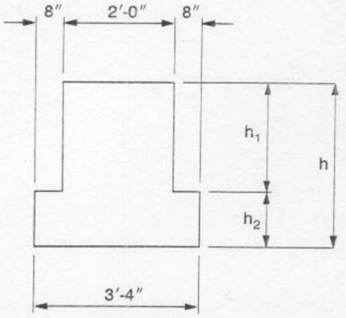
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Desig- nation	No. Strand	e	Span, ft																
			18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
34IT20	14	9.48	8164	6525	5313	4391	3674	3104	2645	2269	1958	1697	1476	1287	1125	984			
			0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.3			
			0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0			
34IT24	17	7.32		9171	7478	6190	5187	4392	3750	3225	2790	2425	2116	1853	1626	1429	1258	1107	974
				0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2
				0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0	−0.1
34IT28	20	8.63				8675	7295	6200	5316	4593	3994	3492	3067	2704	2392	2121	1885	1678	1495
						0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3
						0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1
34IT32	23	10.00					9743	8301	7138	6186	5397	4736	4177	3699	3288	2932	2621	2348	2107
							0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
							0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
34IT36	24	12.32							9477	8239	7207	6340	5605	4978	4439	3971	3563	3205	2892
									0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.3	1.3	1.4
									0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
34IT40	30	12.30								9771	8550	7527	6662	5923	5287	4735	4254	3832	3460
										0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2
										0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4
34IT44	30	14.04										9478	8406	7490	6702	6019	5423	4900	4439
												0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.1
												0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
34IT48	33	15.42												9049	8110	7295	6585	5962	5412
														0.8	0.8	0.9	0.9	1.0	1.1
														0.2	0.2	0.2	0.3	0.3	0.3
34IT52	36	16.81													9637	8681	7848	7116	6470
															0.8	0.9	0.9	1.0	1.0
															0.3	0.3	0.3	0.3	0.3
34IT56	39	18.21															9208	8360	7611
																	0.9	0.9	1.0
																	0.3	0.3	0.3
34IT60	42	19.59																9689	8831
																		0.8	0.9
																		0.2	0.2

INVERTED TEE BEAMS



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

½ in. diameter
low-relaxation strand

Key
8,741 — Safe superimposed service load, plf
0.5 — Estimated camber at erection, in.
0.2 — Estimated long-time camber, in.

Normal Weight Concrete

Section Properties								
Designation	h in.	h_1/h_2 in.	A in^2	I in^4	y_b in.	S_b in^3	S_t in^3	wt plf
40IT20	20	12/8	608	20,321	8.74	2,325	1,805	633
40IT24	24	12/12	768	35,136	10.50	3,346	2,603	800
40IT28	28	16/12	864	55,765	12.22	4,563	3,534	900
40IT32	32	20/12	960	83,200	14.00	5,943	4,622	1,000
40IT36	36	24/12	1,056	118,237	15.82	7,474	5,859	1,100
40IT40	40	24/16	1,216	162,564	17.47	9,305	7,215	1,267
40IT44	44	28/16	1,312	216,215	19.27	11,220	8,743	1,367
40IT48	48	32/16	1,408	280,266	21.09	13,289	10,415	1,467
40IT52	52	36/16	1,504	355,503	22.94	15,497	12,233	1,567

1. Check local area for availability of other sizes.

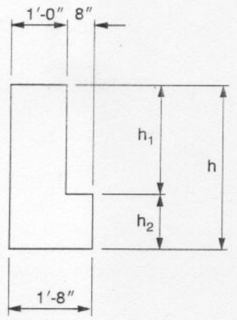
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Designation	No. Strand	e	Span, ft															
			20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
40IT20	18	6.65	8741 0.5 0.2	7124 0.6 0.2	5895 0.7 0.2	4938 0.8 0.3	4179 0.9 0.3	3567 1.0 0.3	3066 1.1 0.3	2650 1.2 0.3	2302 1.3 0.3	2008 1.4 0.3	1756 1.4 0.2	1538 1.5 0.2	1349 1.5 0.1	1184 1.5 0.0	1039 1.5 0.1	
400IT24	22	7.67			8313 0.6 0.2	6976 0.7 0.2	5916 0.8 0.3	5060 0.9 0.3	4360 1.0 0.3	3780 1.0 0.3	3293 1.1 0.3	2882 1.2 0.3	2530 1.3 0.3	2228 1.3 0.3	1966 1.4 0.2	1737 1.4 0.2	1536 1.4 0.1	1359 1.4 0.0
40IT28	26	9.06				9787 0.6 0.2	8327 0.7 0.3	7149 0.8 0.3	6185 0.9 0.3	5386 1.0 0.3	4716 1.0 0.3	4149 1.1 0.3	3666 1.2 0.3	3249 1.3 0.3	2888 1.3 0.3	2573 1.4 0.3	2297 1.5 0.3	2053 1.5 0.2
40IT32	30	10.50						9577 0.7 0.3	8308 0.8 0.3	7256 0.9 0.3	6375 1.0 0.3	5629 1.1 0.4	4992 1.1 0.4	4444 1.2 0.4	3969 1.3 0.4	3555 1.4 0.4	3191 1.4 0.4	2870 1.5 0.3
40IT36	32	12.32								9610 0.8 0.3	8453 0.9 0.3	7474 1.0 0.3	6638 1.0 0.3	5918 1.1 0.3	5295 1.2 0.3	4751 1.3 0.3	4276 1.3 0.3	3860 1.4 0.3
40IT40	38	12.92										8963 0.9 0.3	7977 1.0 0.3	7129 1.0 0.4	6394 1.1 0.4	5753 1.2 0.4	5190 1.2 0.4	4694 1.3 0.4
40IT44	40	14.73												9016 1.0 0.3	8106 1.0 0.3	7311 1.1 0.3	6614 1.2 0.3	5999 1.2 0.3
40IT48	44	16.17													9808 1.0 0.3	8861 1.0 0.3	8030 1.1 0.3	7296 1.2 0.4
40IT52	48	17.62															9537 1.0 0.3	8666 1.1 0.3

L-BEAMS



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

½ in. diameter
low-relaxation strand

Key
6,471 —Safe superimposed service load, plf
0.3 —Estimated camber at erection, in.
0.1 —Estimated long-time camber, in.

Normal Weight Concrete

Section Properties								
Designation	**h in.**	**h_1/h_2 in.**	**A in^2**	**I in^4**	**y_b in.**	**S_b in^3**	**S_t in^3**	**wt plf**
20LB20	20	12/8	304	10,160	8.74	1,163	902	317
20LB24	24	12/12	384	17,568	10.50	1,673	1,301	400
20LB28	28	16/12	432	27,883	12.22	2,282	1,767	450
20LB32	32	20/12	480	41,600	14.00	2,971	2,311	500
20LB36	36	24/12	528	59,119	15.82	3,737	2,930	550
20LB40	40	24/16	608	81,282	17.47	4,653	3,608	633
20LB44	44	28/16	656	108,107	19.27	5,610	4,372	683
20LB48	48	32/16	704	140,133	21.09	6,645	5,208	733
20LB52	52	36/16	752	177,752	22.94	7,749	6,117	783
20LB56	56	40/16	800	221,355	24.80	8,926	7,095	833
20LB60	60	44/16	848	271,332	26.68	10,170	8,143	883

1. Check local area for availability of other sizes.

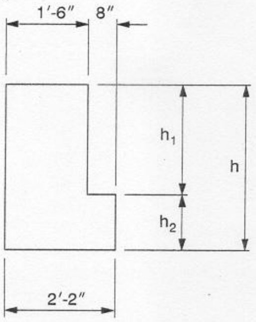
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Table of safe superimposed service load (plf) and cambers

Desig-nation	**No. Strand**	**e**	**Span, ft**																	
			16	**18**	**20**	**22**	**24**	**26**	**28**	**30**	**32**	**34**	**36**	**38**	**40**	**42**	**44**	**46**	**48**	**50**
20LB20	**9**	**6.00**	6471 0.3 0.1	5053 0.4 0.1	4038 0.5 0.1	3288 0.5 0.2	2717 0.6 0.2	2273 0.7 0.2	1920 0.8 0.2	1636 0.9 0.2	1403 1.0 0.2	1210 1.0 0.2	1049 1.1 0.1							
20LB24	**10**	**7.37**	9518 0.3 0.1	7444 0.3 0.1	5961 0.4 0.1	4864 0.4 0.1	4029 0.5 0.1	3380 0.6 0.1	2865 0.7 0.1	2449 0.7 0.1	2108 0.8 0.1	1826 0.9 0.1	1590 0.9 0.1	1390 1.0 0.1	1219 1.0 0.0	1072 1.1 0.0				
20LB28	**12**	**8.56**			8193 0.3 0.1	6701 0.4 0.1	5566 0.5 0.1	4682 0.5 0.1	3981 0.6 0.2	3416 0.7 0.2	2953 0.8 0.2	2569 0.8 0.2	2248 0.9 0.2	1976 1.0 0.1	1744 1.0 0.1	1544 1.1 0.1	1370 1.1 0.1	1219 1.1 0.0	1087 1.2 0.0	970 1.2 −0.1
20LB32	**14**	**9.80**				8820 0.4 0.1	7339 0.4 0.1	6187 0.5 0.2	5272 0.6 0.2	4534 0.6 0.2	3931 0.7 0.2	3430 0.8 0.2	3011 0.8 0.2	2656 0.9 0.2	2353 1.0 0.2	2092 1.0 0.2	1866 1.1 0.2	1669 1.1 0.1	1496 1.2 0.1	1343 1.2 0.1
20LB36	**16**	**11.05**					9335 0.4 0.1	7881 0.5 0.2	6727 0.5 0.2	5796 0.6 0.2	5034 0.7 0.2	4402 0.7 0.2	3873 0.8 0.2	3425 0.9 0.2	3043 0.9 0.2	2714 1.0 0.2	2428 1.1 0.2	2180 1.1 0.2	1961 1.2 0.2	1768 1.2 0.2
20LB40	**18**	**11.99**						9663 0.4 0.1	8253 0.5 0.2	7116 0.5 0.2	6185 0.6 0.2	5413 0.6 0.2	4767 0.7 0.2	4220 0.8 0.2	3752 0.8 0.2	3350 0.9 0.2	3002 1.0 0.2	2698 1.0 0.2	2431 1.1 0.2	2196 1.1 0.2
20LB44	**19**	**13.61**								8866 0.5 0.1	7718 0.5 0.2	6766 0.6 0.2	5969 0.7 0.2	5294 0.7 0.2	4717 0.8 0.2	4221 0.8 0.2	3791 0.9 0.2	3416 0.9 0.2	3087 1.0 0.2	2797 1.0 0.2
20LB48	**21**	**14.86**									9231 0.5 0.2	8101 0.6 0.2	7155 0.6 0.2	6353 0.7 0.2	5669 0.7 0.2	5081 0.8 0.2	4570 0.9 0.2	4125 0.9 0.2	3735 1.0 0.2	3390 1.0 0.2
20LB52	**23**	**16.12**										9545 0.5 0.2	8438 0.6 0.2	7500 0.6 0.2	6700 0.7 0.2	6011 0.8 0.2	5415 0.8 0.2	4894 0.9 0.2	4437 0.9 0.2	4033 1.0 0.2
20LB56	**25**	**17.37**											9817 0.6 0.2	8733 0.6 0.2	7808 0.7 0.2	7012 0.7 0.2	6323 0.8 0.2	5721 0.8 0.2	5192 0.9 0.3	4726 0.9 0.3
20LB60	**27**	**18.63**													8996 0.6 0.2	8086 0.7 0.2	7296 0.7 0.2	6608 0.8 0.3	6004 0.9 0.3	5470 0.9 0.3

L-BEAMS



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

½ in. diameter
low-relaxation strand

Key
9,737 —Safe superimposed service load, plf
0.4 —Estimated camber at erection, in.
0.2 —Estimated long-time camber, in.

Normal Weight Concrete

Section Properties								
Designation	**h in.**	**h_1/h_2 in.**	**A in^2**	**I in.^{4}**	**y_b in.**	**S_b in^3**	**S_t in^3**	**wt plf**
26LB20	20	12/8	424	14,298	9.09	1,573	1,311	442
26LB24	24	12/12	528	24,716	10.91	2,265	1,888	550
26LB28	28	16/12	600	39,241	12.72	3,085	2,568	625
26LB32	32	20/12	672	58,533	14.57	4,017	3,358	700
26LB36	36	24/12	744	83,176	16.45	5,056	4,255	775
26LB40	40	24/16	848	114,381	18.19	6,288	5,244	883
26LB44	44	28/16	920	152,104	20.05	7,586	6,351	958
26LB48	48	32/16	992	197,159	21.94	8,986	7,566	1,033
26LB52	52	36/16	1,064	250,126	23.83	10,496	8,879	1,108
26LB56	56	40/16	1,136	311,586	25.75	12,100	10,300	1,183
26LB60	60	44/16	1,208	382,118	27.67	13,810	11,819	1,258

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

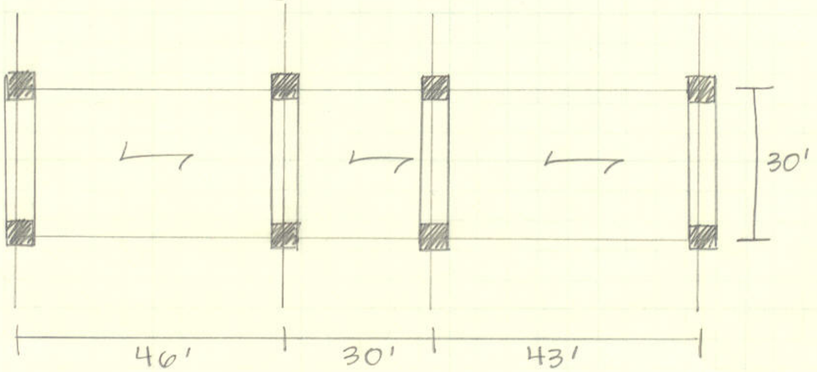
Table of safe superimposed service load (plf) and cambers

Desig-nation	No. Strand	e	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
26LB20	15	6.35	9737 0.4 0.2	7609 0.5 0.2	6088 0.6 0.3	4962 0.7 0.3	4106 0.8 0.4	3439 0.9 0.4	2911 1.0 0.5	2484 1.1 0.5	2135 1.3 0.5	1846 1.4 0.6	1603 1.5 0.6	1398 1.6 0.6	1223 1.7 0.6	1072 1.7 0.6				
26LB24	15	7.78			8987 0.4 0.1	7341 0.5 0.2	6089 0.6 0.2	5115 0.7 0.2	4342 0.8 0.2	3718 0.9 0.2	3208 0.9 0.2	2785 1.0 0.2	2430 1.1 0.2	2130 1.2 0.2	1874 1.2 0.2	1654 1.3 0.1	1463 1.3 0.1	1296 1.4 0.0	1150 1.4 0.0	1020 1.4 −0.1
26LB28	18	9.06					8394 0.5 0.2	7069 0.6 0.2	6017 0.7 0.2	5169 0.8 0.2	4474 0.9 0.2	3899 0.9 0.3	3417 1.0 0.3	3009 1.1 0.3	2660 1.2 0.3	2361 1.3 0.2	2101 1.3 0.2	1874 1.4 0.2	1675 0.4 0.2	1499 1.5 0.1
26LB32	21	10.37						9325 0.6 0.2	7953 0.6 0.2	6847 0.7 0.2	5941 0.8 0.3	5191 0.9 0.3	4562 1.0 0.3	4029 1.0 0.3	3575 1.1 0.3	3184 1.2 0.3	2845 0.3 0.3	2549 1.3 0.3	2289 1.4 0.3	2060 1.4 0.2
26LB36	24	11.68								8739 0.6 0.2	7596 0.7 0.3	6648 0.8 0.3	5855 0.9 0.3	5183 1.0 0.3	4609 1.0 0.3	4116 1.1 0.3	3688 1.2 0.3	3314 1.3 0.3	2987 1.3 0.3	2698 1.4 0.3
26LB40	27	12.71									9338 0.7 0.2	8180 0.7 0.3	7210 0.8 0.3	6390 0.9 0.3	5689 0.9 0.3	5086 1.0 0.3	4563 1.1 0.3	4107 1.2 0.3	3707 1.2 0.3	3354 1.3 0.3
26LB44	28	14.39											9013 0.7 0.2	8001 0.8 0.2	7136 0.9 0.2	6392 0.9 0.3	5747 1.0 0.3	5185 1.1 0.3	4684 1.1 0.3	4244 1.2 0.3
26LB48	32	15.71												9590 0.8 0.3	8564 0.8 0.3	7681 0.9 0.3	6916 1.0 0.3	6248 1.0 0.3	5662 1.1 0.3	5145 1.2 0.3
26LB52	35	17.01														9077 0.9 0.3	8182 0.9 0.3	7401 1.0 0.3	6715 1.1 0.4	6110 1.1 0.4
26LB56	37	18.32															9544 0.9 0.3	8641 0.9 0.3	7849 1.0 0.3	7150 1.0 0.3
26LB60	38	19.62																9972 0.8 0.2	9066 0.9 0.3	8266 1.0 0.3

Wide Module One-Way Joists, Multiple Spans with CIP Framing System

Wide Module One-way Joists, Multiple Spans

Joists Spanning Long Direction



Worst case span = 46'

Loads:

Dead Load

Misc. DL = 15 PSF

Live Load

Office = 100 PSF

$A_T = 4'(46') = 184\ \text{FT}^2$
$< 400\ \text{FT}^2$
$\therefore$ Not Reducible

Factored Superimposed Load

1.4 (15 PSF) + 1.7 (100 PSF) = 191 PSF

Use 1.4D + 1.7L in order to use CRSI charts

191 PSF (4') = 764 PLF

Wide Module One-Way Joists Spanning the Long Direction

Possible Joist Systems Take from CRSI

Option	Form Widths (IN)	Rib Widths (IN)	C-C Width (IN)	Rib Depth (IN)	Slab Depth (IN)	End Span Capacity (PLF)	Interior Span Capacity (PLF)	Self Weight (PLF)
1	40	8	48	24	4.5	873	926	475
2	40	9	49	24	4.5	987	1066	505
3	40	10	50	24	4.5	791	844	534
4	53	8	61	24	4.5	794	845	536
5	53	9	62	24	4.5	908	985	566
6	53	10	63	24	4.5	883	1110	595
7	66	9	75	24	4.5	827	903	627

Selection: 40” Forms + 8” Ribs @ 48” o.c.
24” Deep Rib + 4.5 “Top Slab = 28.5” Total Depth
f’c = 4,000 psi
fy = 60,000 psi

End Span: **764 PLF < 873 PLF ∴ OK**

Top Bars: #7 @ 9”
Bottom Bars: 1 - #10 and 1-#10
Stirrups: #3 @ 13” for 204”

Interior Span: **764 PLF < 926 PLF ∴ OK**

Top Bars: #6 @ 7”
Bottom Bars: 1 - #8 and 1-#9
Stirrups: #3 @ 13” for 167”

This wide-module one-way joist system was selected because it was the lightest design and because it had a modular width of exactly 4’. All of the possible systems had the same total depth.

Interior Beam Selection:
24” x 28.5”
Top: (5) #14
Bottom: (2) #14
Stirrups (Closed): (16) #5, 1@2”, 25@7”
12.5 PLF > 10.83 PLF ∴ **OK**

Exterior Beam Selection:
24” x 28.5”
Top: (4) #14
Bottom: (2) #14
Stirrups (Closed): (23) #5, 1@2”, 22@8”
10.1 PLF > 6.9 PSF ∴ **OK**

Interior Joist Band Beams

max. span = 30'

Loads:

Dead:

Misc. DL = 15 PSF (
Joists = 119 PSF

Live:

Office = 100 PSF → 57 PSF

Live Load Reduction

$A_T = 38'(30') = 1140\ FT^2$
$K_{LL} = 2$
$L_o = 100$ PSF

$$L = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 1140}}\right)$$

L = 57 PSF

$W_u = 1.4(15 + 119) + 1.7(57)$
$= 285$ PSF

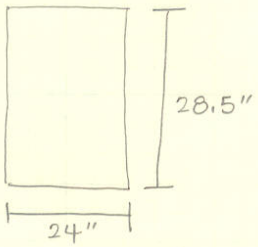
$$W_u = 285\ PSF\left(\frac{46' + 30'}{2}\right)$$

= 10,830 PLF
= 10.83 KLF

d = 28.5' to match total joist depth

Selection:

f'c = 4000 psi
fy = 60,000 psi



span = 30'

Bottom Bars:

$l_n + 12'' =$ (2) #14
$0.875 l_n =$ (1) #14

Top Bars: (5) #14

load capacity = 12.5 K/FT > 10.83 K/FT ∴ OK

Stirrups: 265 E closed stirrups
26 - #5 : 1 @ 2"
25 @ 7"

Exterior Joist Band Beams

max. span = 30'

Loads:

Dead:

Misc. DL = 15 PSF

Joists = 119 PSF

Live:

Office = 100 PSF → 66 PSF

Live Load Reduction

$A_T = 23'(30') = 690 \text{ FT}^2$

$K_{LL} = 2$

$L_o = 100 \text{ PSF}$

$$L = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 690}}\right)$$

L = 66 PSF

$W_u = 1.4(15+119) + 1.7(66)$

$= 300 \text{ PSF}$

$W_u = 300 \text{ PSF}\left(\frac{46'}{2}\right)$

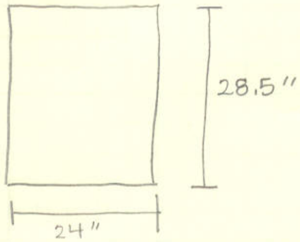
$= 6900 \text{ PLF}$

$= 6.9 \text{ KLF}$

Selection

$f'c = 4000$ psi

$fy = 60{,}000$ psi



Span =

Bottom Bars:

$\ell n + 12" = 2$ #14

$0.875 \ell n = 2$ #14

Stirrups: 235 F

- closed stirrups
- 23 #5: 1 @ 2"

22 @ 8"

Top Bars: (4) #14

Load capacity: 10.1 K/FT > 6.9 K/FT

WIDE MODULE (1) ONE-WAY JOISTS MULTIPLE SPANS

40" Forms + 8" Ribs @ 48" c.-c.

24" Deep Rib + 4.5" Top Slab = 28.5" Total Depth

FACTORED USABLE SUPERIMPOSED LOAD (PLF)

$f_c' = 4,000$ psi
$f_y = 60,000$ psi

TOP BARS NO	# 4	# 5	# 6	# 6	# 7	End Span Defl. Coeff. (2)	# 5	# 6	# 6	# 7	# 8	Int. Span Defl. Coeff. (2)
AT	9.0	8.0	10.0	8.0	9.0		10.0	8.0	7.0	7.5	8.5	
BOTTOM BARS NO	2# 5	1# 8	1# 8	1# 7	1#10		2# 5	1# 8	1# 8	1# 7	1#10	
BARS NO	1# 5	1# 8	1# 9	2# 8	1#10		1# 5	1# 8	1# 9	2# 8	1#10	
STEEL (PSF)	.92	1.64	1.85	2.19	2.70		.89	.89	.89	.89	.89	
CLEAR SPAN	END SPAN						INTERIOR SPAN					
37'-0" (3)	226	832	1026	1393	1713	7.149	631	1512	1795	2329	2794	4.399
STIR	#3- 59	#3-139	#3-153	#3-153	#4-185		#3- 87	#3-143	#3-129	#4-167	#5-174	
38'-0"	180	754	938	1286	1589	7.953	564	1399	1667	2173	2614	4.894
STIR	#3- 55	#3-139	#3-154	#3-162	#4-188		#3- 86	#3-144	#3-136	#4-169	#4-178	
39'-0"	137	682	857	1187	1475	8.824	502	1295	1549	2030	2448	5.430
STIR	#3- 50	#3-140	#3-155	#3-170	#3-190		#3- 84	#3-146	#3-143	#4-172	#4-181	
40'-0"	97	616	782	1096	1370	9.765	444	1198	1440	1897	2295	6.009
STIR	#3- 46	#3-140	#3-156	#3-177	#3-193		#3- 83	#3-147	#3-150	#4-175	#4-184	
41'-0"	61	554	712	1011	1272	10.778	391	1108	1338	1773	2152	6.633
STIR	#3- 41	#3-140	#3-157	#3-179	#3-196		#3- 81	#3-148	#3-158	#4-177	#4-187	
42'-0"		497	647	932	1180	11.869	341	1025	1244	1658	2019	7.304
STIR		#3-140	#3-157	#3-181	#3-199		#3- 78	#3-149	#3-161	#3-180	#4-190	
43'-0"		443	587	859	1096	13.040	295	947	1156	1552	1896	8.025
STIR		#3-139	#3-158	#3-183	#3-179		#3- 76	#3-150	#3-163	#3-182	#4-193	
44'-0"		393	531	790	1016	14.296	252	875	1074	1452	1781	8.798
STIR		#3-139	#3-158	#3-184	#3-187		#3- 73	#3-151	#3-164	#3-185	#4-196	
45'-0"		347	478	726	942	15.641	211	807	998	1359	1673	9.625
STIR		#3-138	#3-158	#3-186	#3-196		#3- 71	#3-152	#3-166	#3-187	#4-198	
46'-0"		303	429	666	873	17.078	174	744	926	1272	1573	10.510
STIR		#3-137	#3-158	#3-187	#3-204		#3- 67	#3-153	#3-167	#3-163	#4-201	
47'-0"		262	383	610	809	18.613	138	684	859	1190	1479	11.454
STIR		#3-136	#3-158	#3-188	#3-207		#3- 64	#3-153	#3-168	#3-170	#4-204	
48'-0"		224	340	558	748	20.248	105	629	796	1114	1390	12.460
STIR		#3-135	#3-158	#3-189	#3-209		#3- 61	#3-154	#3-169	#3-177	#4-206	
49'-0"		188	299	508	691	21.989	74	576	737	1042	1307	13.532
STIR		#3-133	#3-157	#3-190	#3-210		#3- 57	#3-154	#3-170	#3-184	#4-208	
PROPERTIES FOR DESIGN (CONCRETE .79 CF/SF)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	1.07	1.86	2.11	2.64	3.20		1.49	2.64	3.02	3.84	4.46	
ACTUAL STEEL %	.390	.682	.776	.970	1.180		.545	.970	1.109	1.415	1.649	
EFF. DEPTH, IN.	26.75	26.69	26.63	26.63	26.56		26.69	26.63	26.63	26.56	26.50	
– ICR/IGR	.126	.196	.215	.255	.292		.164	.255	.281	.333	.367	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.93	1.58	1.79	2.18	2.54		.93	1.58	1.79	2.18	2.54	
ACTUAL STEEL %	.331	.566	.641	.780	.913		.331	.566	.641	.780	.913	
EFF. DEPTH, IN.	26.69	26.50	26.46	26.52	26.36		26.69	26.50	26.46	26.52	26.36	
+ICR/IGR	.135	.217	.242	.291	.329		.135	.217	.242	.291	.329	
SINGLE LEG STIRRUP AT 13 IN. CONSTANT SPACING-DISTANCE (IN.)												

(1) For gross section properties, see Table 8-3.

(2) Computation of deflection is not required above horizontal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).

(3) Single leg stirrup size space at X in. c.-c. Distance over which stirrups must extend from face of support at each end (in.).

WIDE MODULE (1) ONE-WAY JOISTS MULTIPLE SPANS

40" Forms + 9" Ribs @ 49" c.-c.

24" Deep Rib + 4.5" Top Slab = 28.5" Total Depth

FACTORED USABLE SUPERIMPOSED LOAD (PLF)

$f_c' = 4,000$ psi
$f_y = 60,000$ psi

TOP BARS NO	# 5	# 6	# 6	# 7	# 8	End Span Defl. Coeff. (2)	# 5	# 6	# 7	# 8	# 9	Int. Span Defl. Coeff. (2)
AT	12.0	11.0	9.0	10.0	11.0		9.5	8.0	9.0	9.5	10.0	
BOTTOM BARS NO	2# 4	1# 6	2# 7	2# 8	1# 8		2# 4	1# 6	2# 7	2# 8	1# 8	
BARS NO	2# 5	2# 7	1# 8	1# 8	2# 9		2# 5	2# 7	1# 8	1# 8	2# 9	
STEEL (PSF)	1.02	1.61	2.01	2.37	2.78		.91	.91	.91	.91	.91	
CLEAR SPAN	END SPAN						INTERIOR SPAN					
37'-0" (3)	271	852	1177	1527	1911	7.149	716	1560	2033	2543	3101	4.399
STIR	#3- 59	#3-133	#3-154	#3-155	#4-185		#3- 87	#3-138	#3-154	#4-166	#5-175	
38'-0"	221	771	1079	1411	1775	7.953	642	1442	1891	2374	2903	4.894
STIR	#3- 55	#3-133	#3-155	#3-163	#4-188		#3- 86	#3-139	#3-133	#4-169	#5-178	
39'-0"	174	696	988	1304	1649	8.824	574	1333	1759	2218	2720	5.430
STIR	#3- 51	#3-133	#3-157	#3-172	#4-190		#3- 84	#3-141	#3-140	#4-171	#5-182	
40'-0"	130	627	905	1205	1533	9.765	511	1233	1637	2074	2551	6.009
STIR	#3- 46	#3-133	#3-158	#3-177	#4-194		#3- 82	#3-142	#3-148	#4-174	#5-185	
41'-0"	90	562	827	1113	1425	10.778	452	1139	1524	1940	2394	6.633
STIR	#3- 42	#3-132	#3-158	#3-178	#3-196		#3- 80	#3-143	#3-155	#4-176	#4-187	
42'-0"	52	503	755	1027	1325	11.869	397	1052	1419	1815	2248	7.304
STIR	#3- 37	#3-132	#3-159	#3-180	#3-199		#3- 78	#3-144	#3-163	#4-179	#4-191	
43'-0"		447	688	947	1231	13.040	347	971	1322	1699	2112	8.025
STIR		#3-131	#3-160	#3-182	#3-178		#3- 76	#3-144	#3-164	#4-181	#4-193	
44'-0"		395	625	873	1144	14.296	299	896	1230	1591	1986	8.798
STIR		#3-130	#3-160	#3-183	#3-186		#3- 73	#3-145	#3-166	#3-184	#4-196	
45'-0"		347	566	803	1063	15.641	255	826	1145	1490	1867	9.625
STIR		#3-129	#3-160	#3-184	#3-195		#3- 70	#3-145	#3-167	#3-186	#4-199	
46'-0"		301	512	738	987	17.078	214	760	1066	1396	1757	10.510
STIR		#3-128	#3-160	#3-186	#3-203		#3- 67	#3-146	#3-169	#3-164	#4-202	
47'-0"		259	460	678	915	18.613	175	698	991	1307	1653	11.454
STIR		#3-126	#3-160	#3-187	#3-207		#3- 64	#3-146	#3-170	#3-171	#4-204	
48'-0"		219	412	621	849	20.248	139	640	921	1224	1556	12.460
STIR		#3-124	#3-160	#3-188	#3-209		#3- 61	#3-146	#3-171	#3-178	#4-207	
49'-0"		182	367	567	786	21.989	104	586	855	1146	1464	13.532
STIR		#3-122	#3-160	#3-188	#3-211		#3- 57	#3-146	#3-172	#3-186	#4-209	
PROPERTIES FOR DESIGN (CONCRETE .82 CF/SF)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	1.27	1.96	2.40	2.94	3.52		1.60	2.69	3.27	4.07	4.90	
ACTUAL STEEL %	.423	.656	.802	.987	1.185		.534	.902	1.097	1.372	1.654	
EFF. DEPTH, IN.	26.69	26.63	26.63	26.56	26.50		26.69	26.63	26.56	26.50	26.44	
– ICR/IGR	.136	.193	.226	.262	.297		.165	.247	.283	.330	.374	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	1.02	1.64	1.99	2.37	2.79		1.02	1.64	1.99	2.37	2.79	
ACTUAL STEEL %	.331	.535	.650	.775	.913		.331	.535	.650	.775	.913	
EFF. DEPTH, IN.	26.71	26.58	26.54	26.5	26.45		26.71	26.58	26.54	26.50	26.45	
+ICR/IGR	.137	.209	.249	.290	.334		.137	.209	.249	.290	.334	
SINGLE LEG STIRRUP AT 13 IN. CONSTANT SPACING-DISTANCE (IN.)												

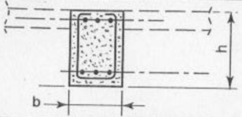
(1) For gross section properties, see Table 8-3.

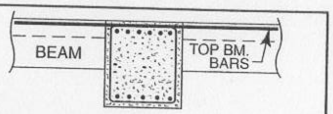
(2) Computation of deflection is not required above horizontal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).

(3) Single leg stirrup size space at X in. c.-c. Distance over which stirrups must extend from face of support at each end (in.).

$f'_c = 4{,}000$ psi
$f_y = 60{,}000$ psi

JOIST-BAND BEAMS, INTERIOR SPANS





STEM		BARS(1)				TOTAL CAPACITY $U = 1.4D + 1.7L$(3)																				+ϕM_n −ϕM_n	DEFL (C)
h	b	BOTTOM		Layers	TOP	SPAN, ℓ_n = 30 ft					SPAN, ℓ_n = 32 ft					SPAN, ℓ_n = 34 ft					SPAN, ℓ_n = 36 ft					(6)	(7)
in.	in.	ℓ_n + 12 in.	0.875 ℓ_n	(2)		LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	ft-kip	$\times 10^{-9}$ in.
28.5	24	2#10	1#10	1	4#10	6.7	113J	23	-	814	5.9	123J	23	-	870	5.2	123J	23	-	917	4.7	123J	23	-	963	419	122
				1			175I	93	2.2	1331		195H	92	2.2	1451		215H	91	2.2	1571		225H	91	2.1	1655	548	
		2#11	1#11	1	4#11	8.1	123J	23	-	990	7.1	133J	23	-	1057	6.3	133J	23	-	1115	5.6	133J	23	-	1172	507	121
				1			235F	93	2.2	1718		245F	92	2.2	1812		195I	91	2.1	1686		205I	91	2.1	1780	659	
		2#14	1#14	1	5#14	12.5	145J	23	-	1825	11.0	144J	23	-	1765	9.7	154J	23	-	1876	8.7	164J	23	-	1988	703	96
				1			265E	93	2.2	2365		285E	92	2.2	2532		265F	91	2.1	2551		275F	91	2.1	2681	1089	
		2#14	2#14	1	6#14	15.4	145J	23	-	2150	13.5	155J	23	-	2295	12.0	165J	23	-	2439	10.7	165J	23	-	2554	904	83
				1			305D	93	2.2	2837		325D	92	2.2	3025		295E	91	2.1	3030		315E	91	2.1	3218	1256	
	36	2#10	2#10	1	5#11	10.0	113J	43	-	1109	8.8	123J	42	-	1185	7.8	123J	42	-	1251	6.9	123J	42	-	1316	562	83
				1			265E	171	3.2	2072		195H	169	3.2	1847		215H	167	3.2	1995		723A	166	3.2	2158	838	
		2#11	2#11	1	5#14	12.1	124J	43	-	1578	10.6	133J	42	-	1560	9.4	133J	42	-	1648	8.4	133J	42	-	1737	681	78
				1			265E	171	3.2	2414		285E	169	3.2	2585		295E	167	3.2	2715		315E	166	3.1	2886	1159	
		2#14	2#14	1	7#14	16.9	135J	43	-	2310	14.8	145J	42	-	2467	13.1	155J	42	-	2624	11.7	154J	41	-	2571	949	67
				1			305D	171	3.2	3136		325D	169	3.2	3344		345D	167	3.1	3552		315E	166	3.1	3552	1544	
		3#14	2#14	1	8#14	20.6	155GeJ	43	-	2765	18.1	165GdJ	42	-	2947	16.0	165GcJ	42	-	3098	14.3	165J	41	-	3248	1159	60
				1			365C	171	3.2	3775		385C	169	3.2	4009		415C	167	3.1	4284		365D	166	3.1	4227	1720	
	48	3#10	3#10	1	6#11	12.4	114J	64	-	1573	10.9	114J	63	-	1661	9.6	114J	63	-	1748	8.6	124J	62	-	1858	838	58
				1			364C	256	4.3	2381		**	253	4.5	1416		**	251	4.4	1504		**	249	4.3	1591	1014	
		3#11	3#11	1	6#14	17.2	125J	64	-	2192	15.1	134J	63	-	2177	13.4	134J	63	-	2294	11.9	144J	62	-	2433	1014	60
				1			455B	256	4.3	3859		285E	253	4.2	3187		295E	251	4.2	3351		315E	249	4.1	3561	1407	
		3#14	3#14	1	9#14	24.4	165FfJ	64	-	3167	21.5	155GeJ	63	-	3304	19.0	165GdJ	63	-	3511	17.0	165GcJ	62	-	3683	1407	49
				1			455B	256	4.3	4693		485B	253	4.2	5005		515B	251	4.2	5316		545B	249	4.1	5628	1998	
		4#14	3#14	1	11#14	28.7	205DIJ	64	-	3850	25.2	185EiJ	63	-	3988	22.3	185FhJ	63	-	4196	19.9	185FgJ	62	-	4404	1612	44
				1			455B	256	4.3	5237		485B	253	4.2	5584		515B	251	4.2	5931		545B	249	4.1	6278	2350	

(1) See "Recommended Bar Details", Fig. 12-1. For girders, use tabulated beam depth — 2 inches (b — 2").

(2) In "Layers" column, first line is number of layers for bottom bars, second line is for number of layers for top bars.

(3) For superimposed factored load capacity, deduct 1.4 x stem weight.

(4) Total capacities tabulated causing deflection in excess of $\ell_n/360$ are designated thus: * —$\ell_n/360 <$ deflection $< \ell_n/240$
X —$\ell_n/240 <$ deflection $< \ell_n/180$
Y — deflection $> \ell_n/180$

(5) For each beam design, first line is for open stirrups, secondline is for closed ties. See Fig. 12-4. At free ends, use stirrups tabulated for "Interior Spans". For b > 24 in., provide 4 legs (two stirrups) of size and spacing tabulated. For stirrup nomenclature, see page 12-13.

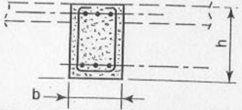
Other notation:
N/A — STIRRUPS ARE NOT REQUIRED
** — MAXIMUM SPACING IS LESS THAN 3 INCHES. NOT RECOMMENDED
*** — SHEAR STRESS IS GREATER THAN $10\sqrt{f'_c}$
**** — TORSION STRESS EXCEEDS ALLOWABLE

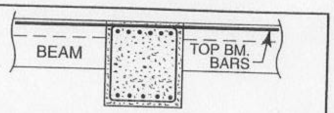
(6) +ϕM_n and —ϕM_n are design moment strength capacities for rectangular section b × h.

(7) Midspan elastic deflection (in.) = C x (w/1.6) x ℓ_n^4, where w = tabulated load (k/ft.), ℓ_n in ft.
"Average service load" is taken as w/1.6.

$f_c' = 4{,}000$ psi
$f_y = 60{,}000$ psi

JOIST-BAND BEAMS, END SPANS





STEM		BARS[1]				TOTAL CAPACITY $U = 1.4D + 1.7L$[3]																				$+\phi M_n$ $-\phi M_n$	DEFL (C)
h	b	BOTTOM		Layers	TOP	SPAN, ℓ_n = 30 ft					SPAN, ℓ_n = 32 ft					SPAN, ℓ_n = 34 ft					SPAN, ℓ_n = 36 ft						
in.	in.	ℓ_n + 12 in.	0.875 ℓ_n	(2)		LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	Aℓ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	Aℓ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	Aℓ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	Aℓ sq. in.	STEEL WGT lb.	(6) ft-kip	(7) × 10^{-9} in.
28.5	24	2#10	1#10	1	4#10	5.1	123J	23	-	845	4.5	123J	23	-	890	4.0	133J	22	-	945	3.6	133J	22	-	989	419	213
				1			185H	91	2.1	1390		195H	90	2.1	1471		215H	90	2.1	1589		225H	89	2.1	1671	548	
		2#11	1#11	1	4#11	6.2	133J	23	-	1030	5.4	143J	23	-	1095	4.8	143J	22	-	1150	4.3	143J	22	-	1206	507	204
				1			175I	91	2.1	1528		185I	90	2.1	1620		215H	90	2.1	1786		225H	89	2.1	1878	659	
		2#14	2#14	1	4#14	10.1	154J	23	-	1797	8.8	164J	23	-	1909	7.8	174J	22	-	2021	7.0	174J	22	-	2114	904	155
				1			235F	91	2.1	2362		245F	90	2.1	2492		265F	90	2.1	2659		275F	89	2.1	2788	904	
		3#14	2#14	1	5#14	12.1	155J	23	-	2358	10.6	165J	23	-	2505	9.4	175J	22	-	2652	8.4	185J	22	-	2799	1089	134
				1			305D	91	2.1	3015		285E	90	2.1	3059		295E	90	2.1	3213		315E	89	2.1	3404	1089	
	36	2#11	2#11	1	5#10	7.7	123J	42	-	1176	6.8	123J	41	-	1241	6.0	133J	41	-	1317	5.4	133J	41	-	1381	681	147
				1			185H	167	3.1	1796		643A	166	3.1	1975		683A	164	3.1	2094		723A	163	3.1	2213	694	
		2#14	2#14	1	5#11	9.3	133J	42	-	1563	8.2	143J	41	-	1662	7.3	143J	41	-	1751	6.5	143J	41	-	1839	949	134
				1			265E	167	3.1	2505		285E	165	3.1	2677		295E	164	3.1	2807		225H	163	3.1	2604	838	
		3#14	3#14	1	6#14	15.1	155J	42	-	2763	13.3	165J	41	-	2935	11.7	175J	41	-	3107	10.5	175J	41	-	3247	1357	103
				1			305D	167	3.1	3525		325D	165	3.1	3747		345D	164	3.1	3970		315E	163	3.1	3985	1357	
		4#14	3#14	1	8#14	18.9	185FgJ	42	-	3415	16.6	185GfJ	41	-	3588	14.7	195GeJ	41	-	3793	13.1*	195GdJ	41	-	3965	1544	89
				1			365C	167	3.1	4329		385C	165	3.1	4584		415C	164	3.1	4882		435C	163	3.1	5138	1720	
	48	3#10	3#10	1	6#11	10.3	124J	63	-	1662	9.0	124J	62	-	1748	8.0	134J	62	-	1856	7.1	134J	61	-	1941	838	109
				1			364C	250	4.2	2448		**	248	4.3	1481		**	246	4.3	1566		**	245	4.2	1652	1014	
		3#11	3#11	1	6#14	12.4	134J	63	-	2162	10.9	144J	62	-	2298	9.6	144J	62	-	2411	8.6	144J	61	-	2525	1014	100
				1			265E	250	4.2	3080		384C	248	4.2	3098		414C	246	4.1	3299		434C	245	4.1	3471	1407	
		4#14	3#14	1	8#14	19.7	165GeJ	63	-	3395	17.3	175GdJ	62	-	3603	15.3	175J	62	-	3775	13.7	175J	61	-	3948	1612	81
				1			455B	250	4.2	4921		485B	248	4.1	5233		515B	246	4.1	5545		545B	244	4.1	5857	1809	
		5#14	4#14	1	10#14	24.2	225DmJ	63	-	4365	21.3	205EjJ	62	-	4514	18.8	205FhJ	62	-	4733	16.8*	205GgJ	61	-	4952	1998	69
				1			455B	250	4.2	5681		485B	248	4.1	6039		515B	246	4.1	6398		545B	244	4.1	6756	2178	

(1) See "Recommended Bar Details", Fig. 12-1. For girders, use tabulated beam depth — 2 inches (b — 2").

(2) In "Layers" column, first line is number of layers for bottom bars, second line is for number of layers for top bars.

(3) For superimposed factored load capacity, deduct 1.4 x stem weight.

(4) Total capacities tabulated causing deflection in excess of $\ell_n/360$ are designated thus: * —$\ell_n/360$ < deflection < $\ell_n/240$
X —$\ell_n/240$ < deflection < $\ell_n/180$
Y — deflection > $\ell_n/180$

(5) For each beam design, first line is for open stirrups, secondline is for closed ties. See Fig. 12-4. At free ends, use stirrups tabulated for "Interior Spans". For b > 24 in., provide 4 legs (two stirrups) of size and spacing tabulated. For stirrup nomenclature, see page 12-13.

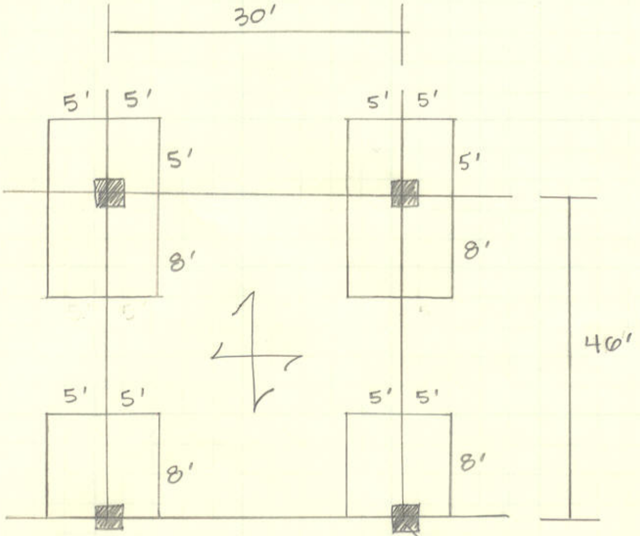
Other notation:
N/A — STIRRUPS ARE NOT REQUIRED
** — MAXIMUM SPACING IS LESS THAN 3 INCHES. NOT RECOMMENDED
*** — SHEAR STRESS IS GREATER THAN $10\sqrt{f_c'}$
**** — TORSION STRESS EXCEEDS ALLOWABLE

(6) $+\phi M_n$ and $-\phi M_n$ are design moment strength capacities for rectangular section b × h.

(7) Midspan elastic deflection (in.) = C x (w/1.6) x ℓ_n^4, where w = tabulated load (k/ft.), ℓ_n in ft.
"Average service load" is taken as w/1.6.

Two-Way Flat Slab with Drop Panels with CIP Framing System

Two-way Flat Slab with Drop Panels with CIP Framing System



assume 20" square columns

- F'c = 4000 psi
- No interior beams or edge beams
- Panel size requirements

$$\frac{l}{6} = \frac{30'}{6} = 5'$$

$$\frac{l}{6} = \frac{46'}{6} = 8'$$

- Direct Design Method Requirements (ACI 318-02, 13.6)
  - $\frac{46}{30} = 1.53 < 2:1$ ✓
  - min of 3 consecutive bays in each direction ✓
  - successive span lengths don't differ by more than 1/3
    $46(1/3) + 30 \approx 46'$ ✓
  - $LL/DL < 2$ ✓

∴ Direct Design Method can be used.

2

Long Span Direction Slab Design

$l_1 = 46'$
$l_2 = 30'$
$l_n = 46' - \frac{20''}{12} = 44.33'$

$t_{\min_{slab}} = \frac{l_n}{33}$ ACI 318-02 Table 9.5 (c)
-drop panels
-without edge beam

$= \frac{44.33'}{33}$
$= 1.34' \times 12''/FT$
$= 16.12'' \rightarrow 16.5''$ slab

Panel Thickness

$2'' = 2''$
$t_{slab}/4 = 16.5/4 = 4.125'' \rightarrow 4.5''$ *

Panel weight

$8'(5')(4 \text{ panels})(4.5''/12)(150 \text{ PCF}) = 9000\#$

$\frac{9000\#}{(46')(30')} = 6.52$ PSF

Loads:

Dead:
SDL = 15 PSF
Slab = $\frac{16.5''}{12} \times 150$ PCF = 206.25 PSF
Panels = 6.52 PSF

Live:
Office = 100 PSF → 66 PSF Live Load Reduction

$A_T = 46'(30') = 1380 \text{ FT}^2$
$K_{LL} = 1$

$L = 100\left(0.25 + \frac{15}{\sqrt{1 \cdot 1380}}\right)$
L = 66 PSF

$W_u = 1.2(15 + 206.25 + 6.52) + 1.6(66)$
$W_u = 379$ PSF

3

$$M_o = \frac{W_u l_2 l_n^2}{8}$$

$$M_o = \frac{379 \text{ PSF} (30')(44.33')^2}{8}$$

$$M_o = 2793 \text{ K}\cdot\text{FT}$$

		Total Mu (FT·K)	Totalwidth (FT)	Mom./FT (FT·K/FT)	Req'd Reif.
Int. supp. (−) 70% Mo 1955	CS 75	1467	15'	97.8	#8@6"
	MS 25	489	15'	32.6	#8@12"
Midspan (+) 52% Mo 1452	CS 60	871	15'	58.1	#8@8"
	MS 40	581	15'	38.8	#8@12"
Ext. supp. (−) 30% Mo 838	CS 100	838	15'	55.9	#8@8"
	MS 0	0	15'	0	—

$$d@\text{panels} = 21 - \underbrace{0.75}_{\text{cover}} - \underbrace{1''}_{d\#8} - \underbrace{1''/2}_{1/2 d\#8} = 18.75''$$

$$d = 16.5 - 0.75 - 1'' - 1''/2 = 14.25''$$

$$A_{s\,min} = 0.0018 A_g = 0.0018(16.5'')(12) = 0.36 \text{ in}^2/\text{ft} < 0.79 \text{ in}^2$$

$$0.0018(21'')(12) = 0.45 \text{ in}^2/\text{ft} < 0.79 \text{ in}^2$$

Try #8 @ 12"

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.79(60)}{0.85(4)(12'')} = 1.16''$$

$$\phi M_n = 0.9(0.79)(60)(14.25 - 1.16/2) = 48.6 \text{ FT}\cdot\text{K/FT}$$

Good For: Int. sup. MS.
Mid. MS

Try #8 @ 6"

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.79(2)(60)}{0.85(4)(12)} = 2.32''$$

$$\phi M_n = 0.9(2)(0.79)(60)(18.75 - 2.32/2) = 125.1 \text{ FT}\cdot\text{K/FT}$$

Good For Int. supp. CS

Try #8@8"

$a = \frac{0.79(1.5)(60)}{0.85(4)(12)} = 1.74''$

$\phi Mn = 0.9(1.5)(0.79)(60)(14.25 - 1.74/2) = 71.35$ FT·K/FT

Good For Mid CS

Ext. supp. CS

Short span Direction Slab Design

$l_1 = 30'$

$l_2 = 46'$

$l_n = 30 - \frac{20''}{12} = 28.33'$

$t_{min, slab} = \frac{l_n}{33}$ ACI Table 9.5(c)

-drop panels

-without edge beam

$= \frac{28.33'}{33}$

$= 0.859 \times 12$

$= 10.3'' < 16.5'' \therefore$ use 16.5"

Loads: same as long span

$W_u = 379$ PSF

$M_o = \frac{W_u l_2 l_n^2}{8}$

$M_o = \frac{379 \text{ PSF}(46')(28.33')^2}{8}$

$M_o = 1749$ FT·K

		Total Mu (FT·K)	Total width (FT)	Mom./FT (FT·K/FT)	Req'd Reinf.
Int. supp. (−) 65% Mo 1137	CS 75	853	15'	57	#7@8"
	MS 25	284	31'	9.2	#7@12"
Midspan (+) 35% Mo 613	CS 60	367	15'	25	#7@12"
	MS 40	245	31'	7.9	#7@12"

$A_{min} = 0.0018(16.5)(12) = 0.36$
$\phantom{A_{min} =} 0.0018(21)(12) = 0.45$

Try #7 @ 12"

$$a = \frac{0.6(60)}{0.85(4)(12)} = 0.88''$$

$$\phi M_n = 0.9(0.6)(60)(14.25 - 0.88/2) = 37 \text{ FT}\cdot\text{K}/\text{FT}$$

Good For Int. supp. MS
Mid. CS & MS

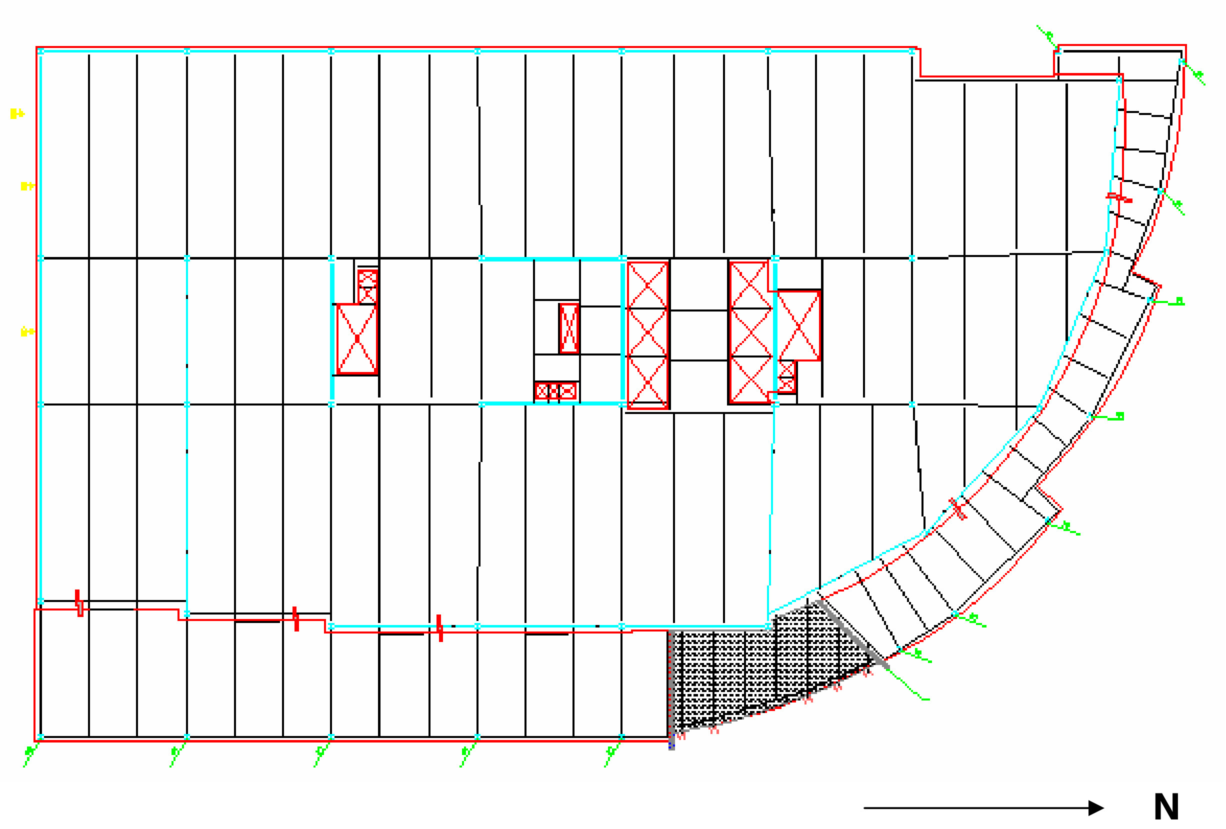
Try #7 @ 8"

$$a = \frac{0.6(60)(1.5'')}{0.85(4)(12)} = 1.32''$$

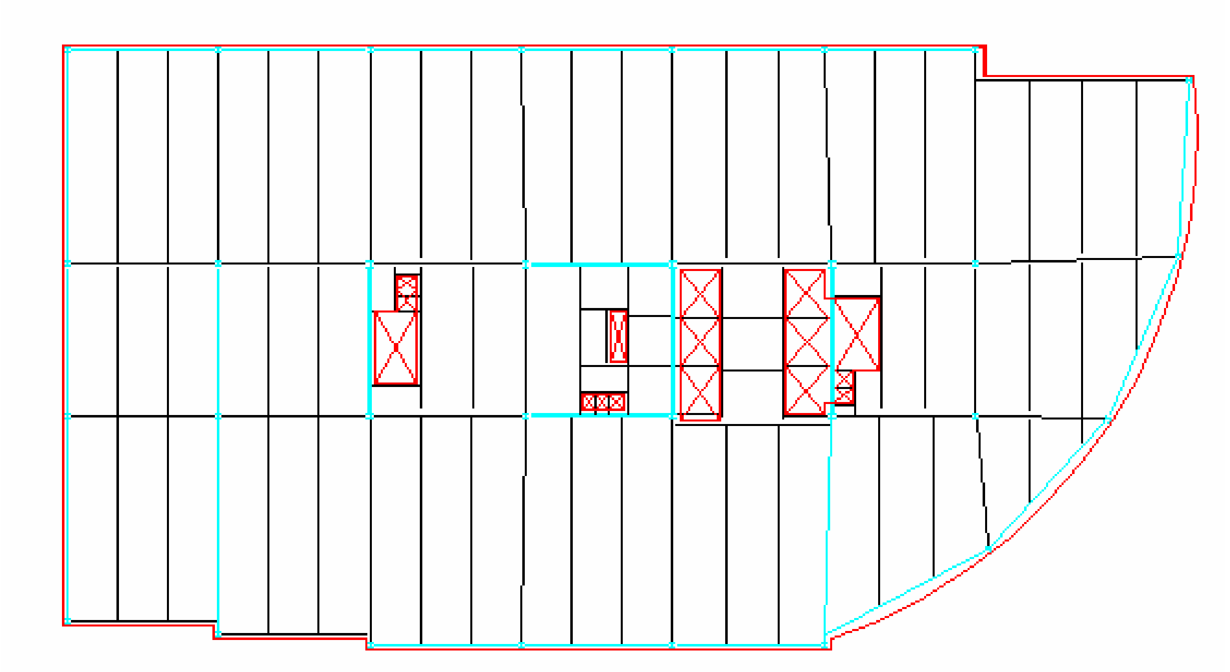
$$\phi M_n = 0.9(0.6)(1.5)(60)(18.75 - 1.32/2) = 73 \text{ FT}\cdot\text{K}/\text{FT}$$

Good For Int. supp. CS

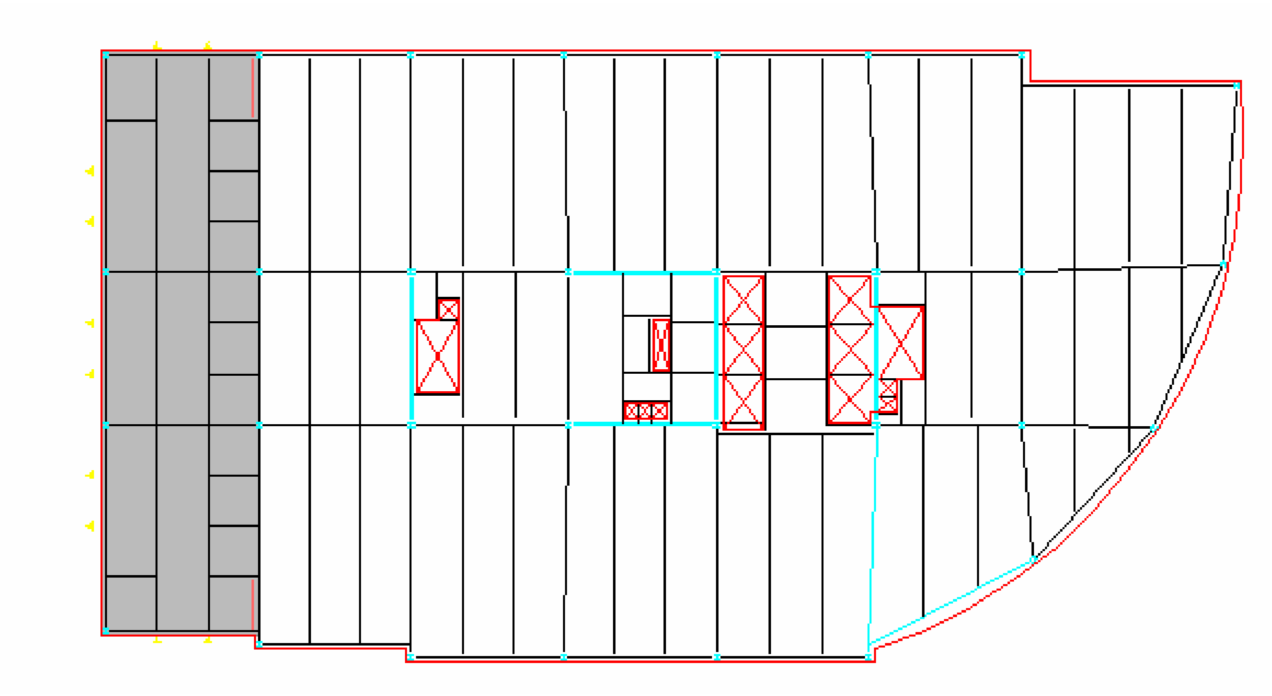
2nd Floor Faming Plan



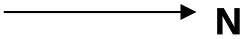
3rd – 5th Floor Framing Plan



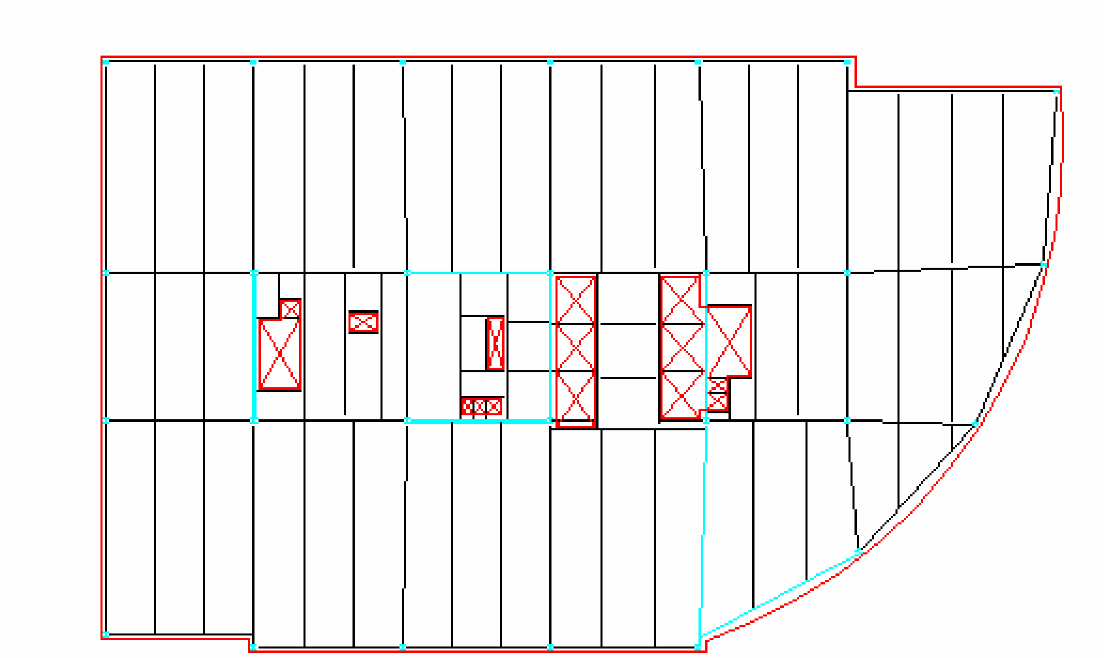
6th Floor Framing Plan



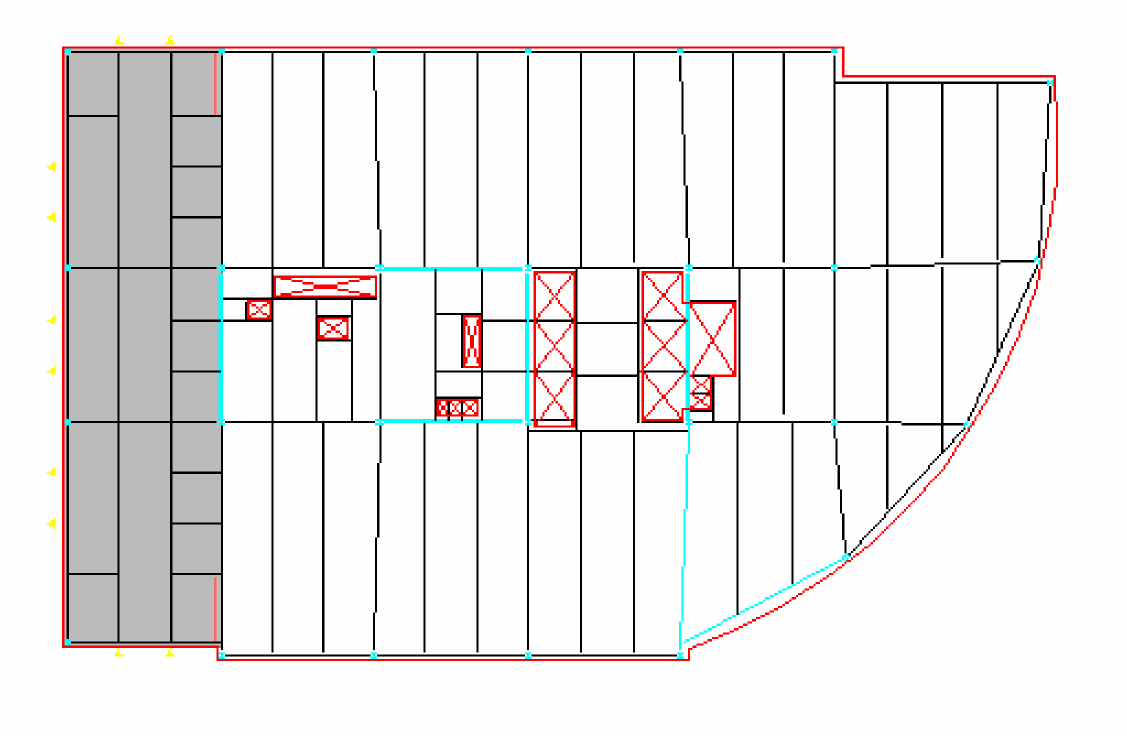
Note: Shaded area is roof construction



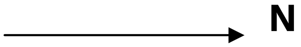
7-9th Floor Framing Plan



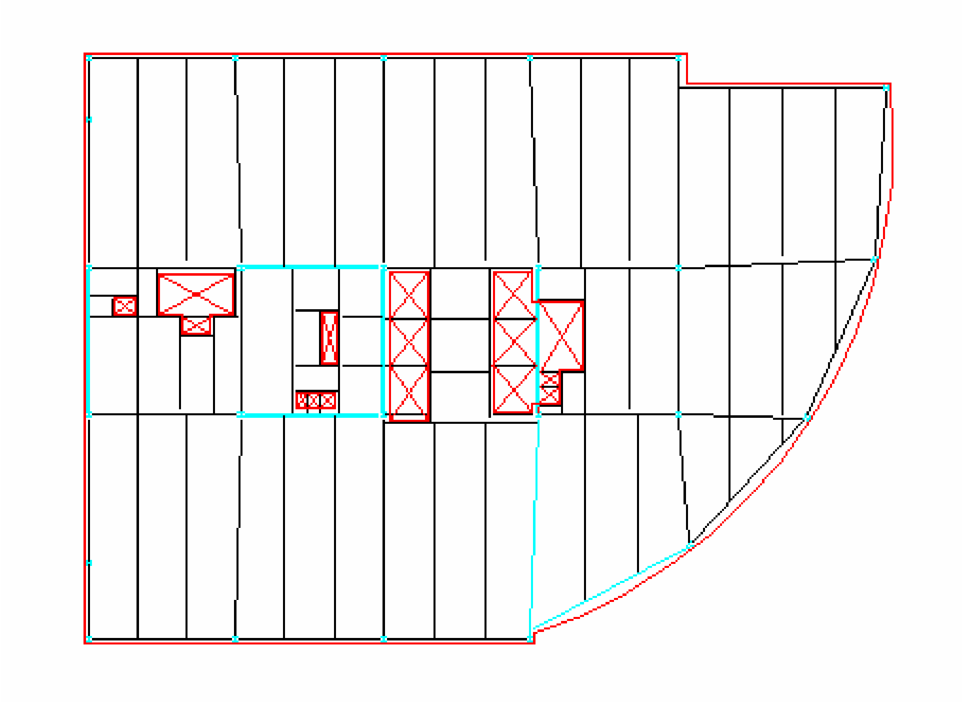
10th Floor Framing Plan



Note: Shaded area is roof construction



11th and 12th Floor Framing Plan



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