Pro-Con Structural Study of Alternate Floor Systems

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building

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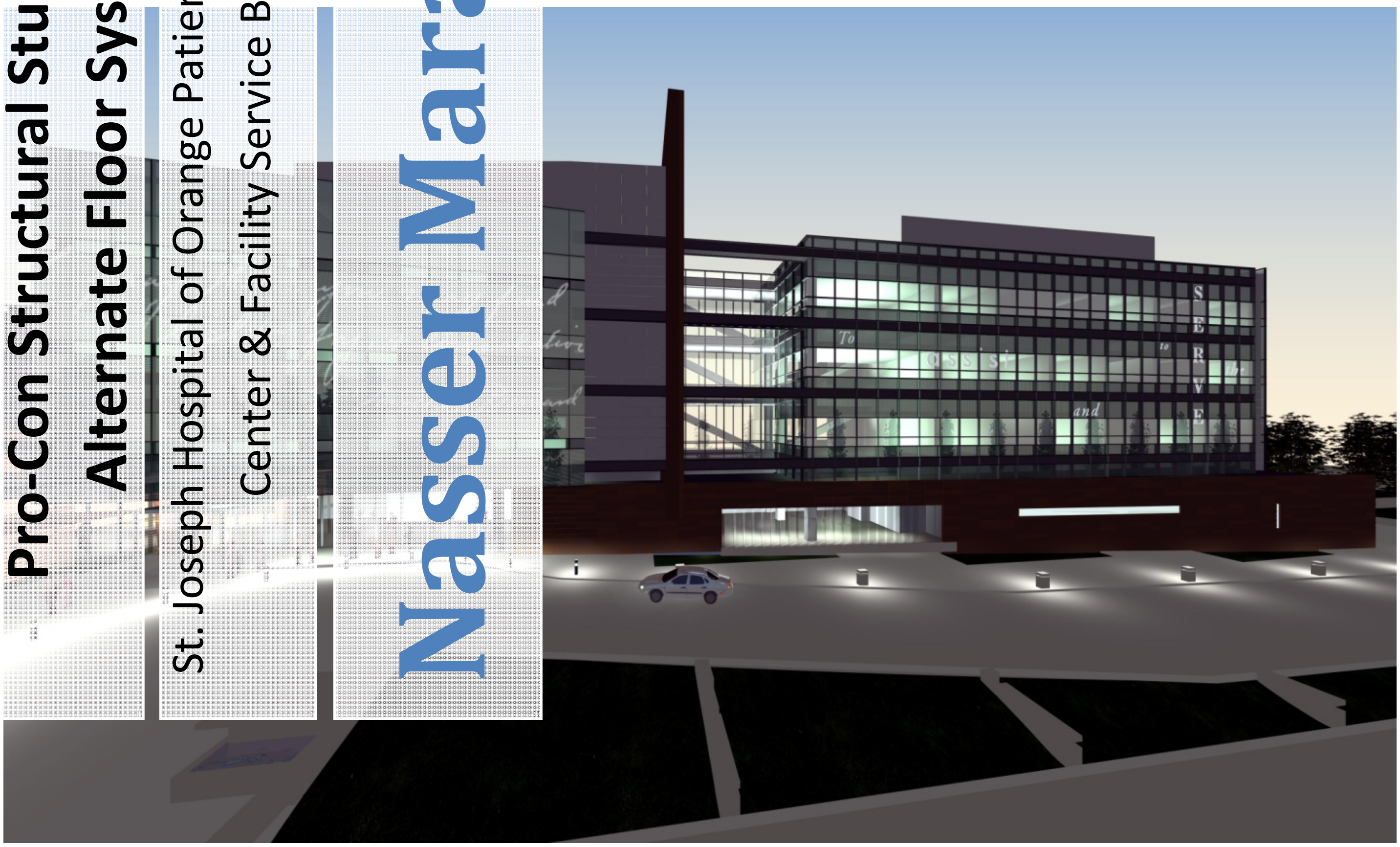

Technical Report 2

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Executive Summary

This report is in an investigation to alternate floor framing systems for Saint Joseph Hospital of Orange Patient Care Center & Facility Service Building. Preliminary designs were performed upon investigation of the different floor systems. Floor systems analyzed are as follows:

1. Composite Concrete on Steel Beams (Existing System)
2. Two Way Post Tensioned Slab
3. Precast Prestressed Hollow Core Slab.
4. Two Way Concrete Waffle Flat Slab System
5. Two Way Concrete Slab with Beams

Comparisons were made upon analysis, factors taken under considerations include; fire rating, cost and rate of construction, floor membrane thickness, building's self weight and effect on architectural and mechanical plans.

The original design of a composite concrete on steel beams floor system seems to be the best solution. Advantages like a lighter overall system with the ability to accommodate future equipment layout alterations with ease, make this system rather unique and best for a hospital building use.

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Introduction

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building is to be built within Saint Joseph Hospital Campus serving the healthcare needs of the Orange county community in Orange, CA. The Patient Care Center is linked to the main hospital through an underground tunnel to further serve the patients' needs. The patient care center consists of two towers joined together with a central courtyard.

The main entrance to the lobby is connected to the adjacent hospital reception area. The Patient Care Center consists of operating rooms to expand the surgical capacity of the main hospital. Operating rooms are equipped with latest innovative technology and medical equipment. To help further serve the main hospital, the Patient Care Center also has additional room for incoming patients and rooms for patients requiring intensive care.

The Patient Care Center has a central sterile plant located on the basement level with MEP equipment. The first level of the hospital consists of surgical rooms, administrative rooms and the lobby. The upper floors are separated by the central courtyard. The west side consists of patient rooms and the east side consists of intensive care units. The remaining mechanical equipment is located on the roof level.

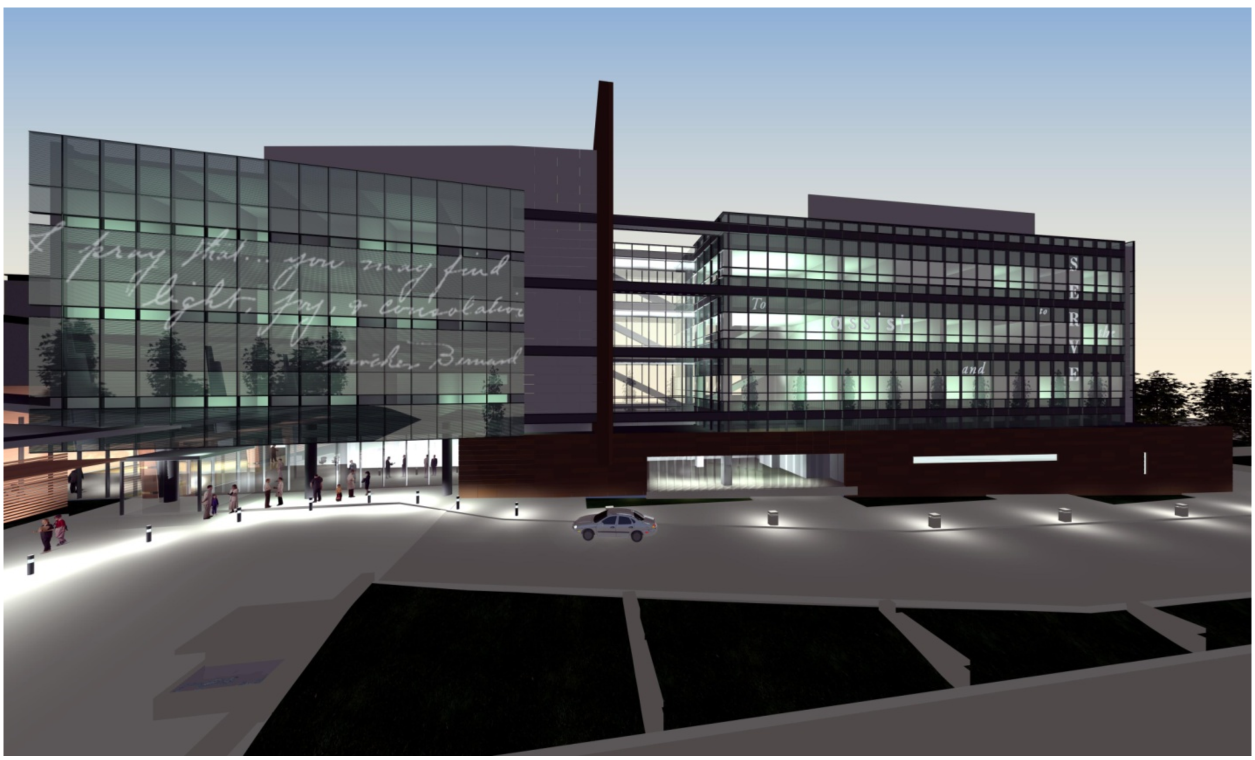


Figure 1. Computer rendering of Patient Care Center's North elevation.

Existing Structural Systems

Floor Framing

There are minor variations to the floor framing through the Patient Care Center. The typical floor system is a composite steel framing using lightweight concrete and a total thickness of 6¼", 3" composite deck is used with 5" long, ¾"diameter shear studs for composite action. The typical infill beam is a W16x31, 30'-0" long spaced at 10'-0" on center, which frame onto a W24x68 30'-0" long. Variations from the typical floor system are based on the use of the space. Light weight concrete was used in the typical steel deck configuration to reduce shear and overturning moment during seismic events.

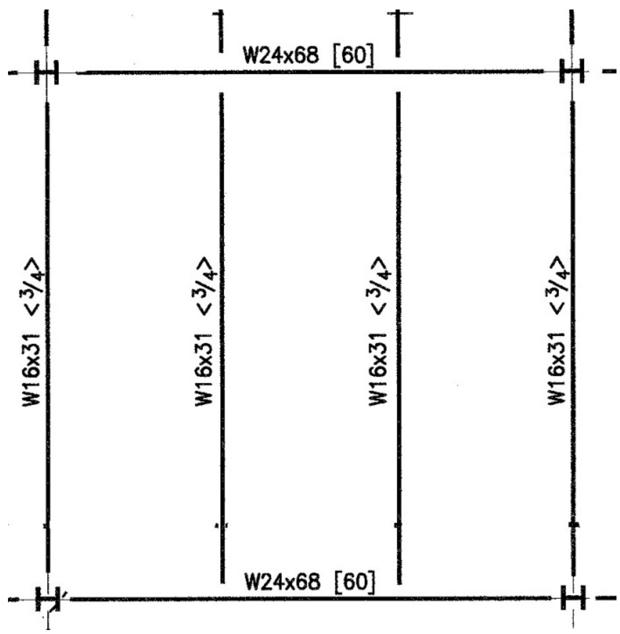


Fig2. Typical 30'-0"x30'-0" bay located on Levels 2, 3 and 4

First floor

The floor framing plan on the first floor differs from the rest due to different loading criterion used. Typical infill members used are W18x35 framing into W24x68 girders. Composite steel framing is used with normal weight concrete and a total thickness of 7½", 3" composite deck with 5" long, ¾" diameter shear studs.

Second floor

There is a central courtyard which is supported by the second floor framing system. Due to the high loading W21x111 infill beams are used which frame into W30x148. A composite steel framing system is also used with normal weight concrete and a total thickness of 9", 3" composite deck with 5" long, ¾" diameter shear studs.

Roof

Due to the location of air handling units on the roof, members with a higher loading capacity are required. Therefore the member sizes change to a W18x40 for beams and W24x84 for girders. A 9" composite steel system exists similar to the second floor courtyard but covered with insulation.

Columns

There are two columns sets per gridline intersection which are usually spliced at 5'-0" from the Level 2. Typical columns sizes are W14x99 on the upper levels (Level 2 to Roof); while the lower columns are W14x145 or W14x132 depending on location and loadings. Columns existing in the brace frame are usually W14x145 except the end columns which are W14x211 on the top and W14x311 at the bottom. These columns have greater strength capacities due to the excess tension and compression they carry from the bracing system induced moment.

Identification of other structural elements

There are several areas in the building that were not discussed in depth in this report. These include the underground tunnel connecting to the adjacent hospital, the canopy at the building's main entrance, and the elevator machine rooms located at the roof. Other structural elements like checking the braced system connections and the foundation system where not discussed in this report but will be analyzed and justified in later reports.

Typical Floor Plans

2nd Floor Plan

The figure below represents the 2nd floor plan occupant use. Loadings here are assumed to be 80 psf where patient rooms and Intensive care units exist. While at the court yard a super imposed dead load is added counting for pavements, planters and trees. The roof on the west side is designed for future planters. The third and fourth floor plans are similar but do not include the central courtyard shaded in green and roof areas shaded in blue. This report only designs the floor system at typical bays located in the patient room and intensive care unit areas.

Fig3. 2nd floor plan showing occupant use.

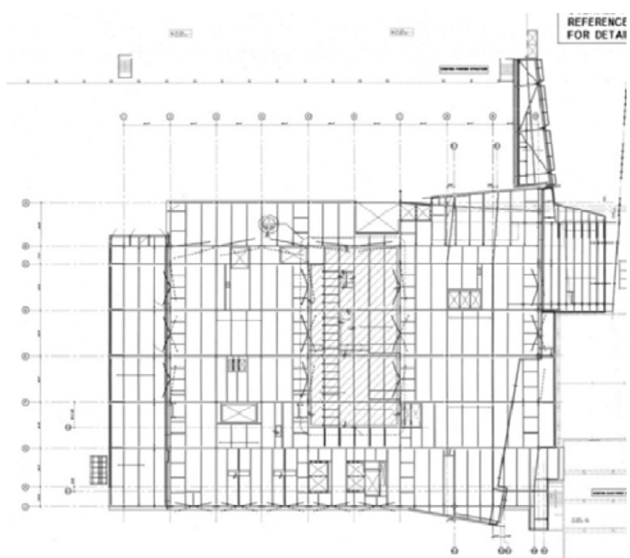


Fig4. 2nd floor plan showing framing plan.

Codes

Codes and Referenced Standards

The following table shows the codes that were adopted in this report and codes that were implemented by the designer.

Codes adopted by this report	Codes adopted by the designer
2007 California Building Code	Title 24, Part 1 2001 California Building Code
ASCE 7-05	1997 Uniform Building Code with California amendments
ACI 318-05 (PCI Handbook) -02(pcaSlab & RAM Concept) -99(CRSI Handbook)	
13th Edition of the AISC Manual of Steel Construction	
6th Edition of the PCI Handbook	

Typical floor bay loadings

Live Loads

Live loads are determined in accordance with ASCE 7-05.

Occupancy	Designer's Uniform Live load (psf)	2007 CBC Uniform Live loads (psf)
Patient Rooms	80[1]	40
Operating Rooms, Laboratories	80[1]	60
Corridors	80[1]	100
Office	80[1]	50

[1] Designer's value used for simplicity reasons.

Dead Loads

Refer to Appendix in Tech. 1 for dead load calculations. Material weights are taken from the ASCE 7-05 Chapter C3.

	LVL2	LVL3	LVL4
Concrete Topping*	44	44	44
Steel Deck (18 Gage)*	3	3	3
Super Imposed	12	12	12
Partitions	20	20	20
Total Dead Load (psf)	**79**	**79**	**79**

* Dead Load refer to existing condition

Composite Concrete with Steel Beams Framing System (Existing System)

System Effectiveness

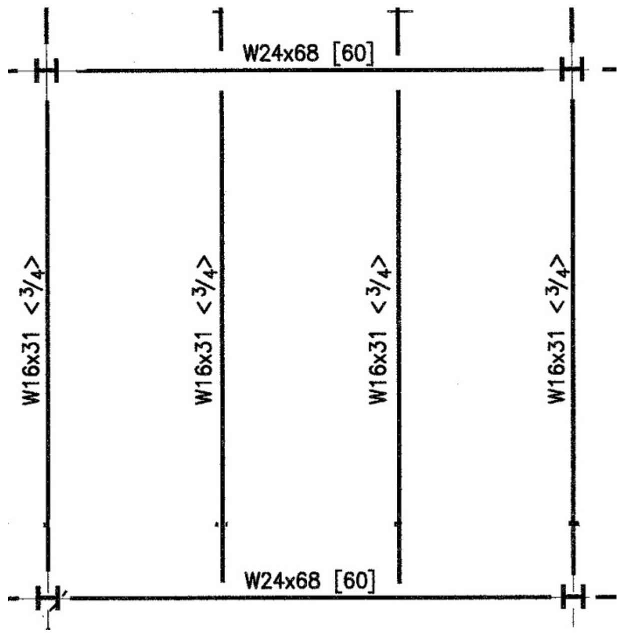


Fig5. Typical 30'-0"x30'-0" bay located on Levels 2, 3 and 4

Material Properties
f'c = 4000 psi
fy = 60,000 psi
Lightweight concrete (110 pcf)
Total slab thickness = 6¼"
3" 18 gage steel deck
¾" diameter shear studs

Design Summary
$I_{LBgirder}$ = 4680 in^3
I_{LBbeam} = 1200 in^3
Floor Dead Weight = 55 psf
$\Delta_{midspan}$ = .54"
Floor Self weight = 54 psf

Structural Impact

The use of composite floor system is by making the concrete slab double counting in its existence in the floor system. The concrete slabs works in compression while the steel beams work mainly tension. This enforces both items to work best to their advantage as suppose to a girder slab where the structural steel is doing the majority of the work. This system ensures smaller member sizes to be used, greater stiffness, and does not have major cost differences with a girder slabs system.

This system is capable of handling heavy loads at long column spans. The system is fairly light in comparison with concrete systems but has a fairly large total floor thickness at about 23" in mid span.

Lateral Resisting System

The lateral system used in conjunction with this floor system is braced frames. The floor system self weights is a key issue regarding the total building self weight which would have minimal effect on the lateral resisting system.

Architectural Impact

The column sizes are fairly small in comparison with concrete floor systems, therefore would not have as much effect to architectural drawings. The floor thickness is larger than a concrete system and would therefore reduce the amount of floor height. The use of braced frames was accentuated architecturally therefore this floor system would have a contribution to architectural aesthetics.

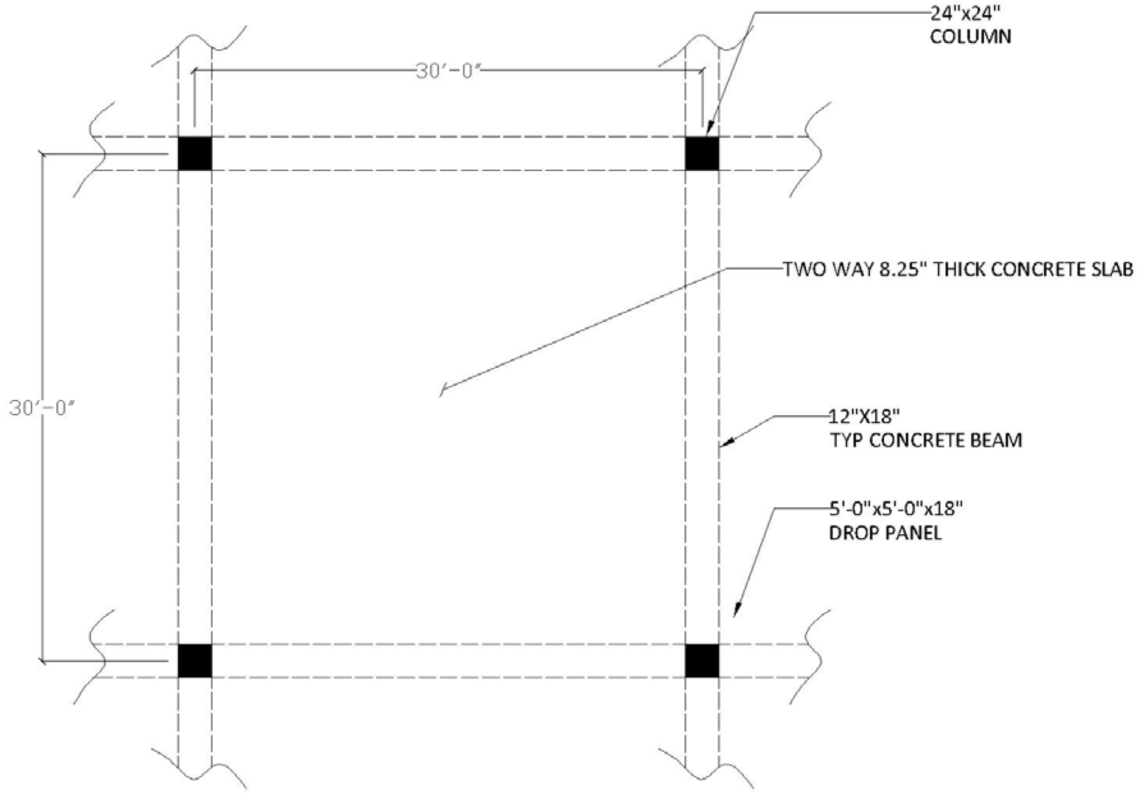
Constructions Impact

This is a cost effective solution, and has a fairly quick method of construction. Shoring may not be required during curing since the steel beams are sufficient to carry the dead weight of the concrete slab.

Summary

Advantages	Disadvantages
Fast construction	Thick floor membrane
Smaller columns	Heavy Sections for floor vibrations
Cost Effective	Structural Steel requires fire proofing
Fast Construction Time	
Large Spans	
Light structure	

Two Way Post Tensioned Slab



Material Properties
f'_c = 5,000 psi
f_{pu} = 270,000 psi
f'_y = 60,000 psi
Normal weight concrete (150 pcf)
Total slab thickness = 9"
Shear Capital: 3'-0"x3'-0"x10"

Reinforcement
(22) Banded tendons running E-W @ gridlines
(3) Strand tendons running N-S @ 3'-4"
See appendix for mild steel reinforcements.

Design Summary
Δmidspan = .35" @ Interior Bays

Floor Self weight = 100 psf

System Effectiveness

Structural Impact

Due to the large span a complex concrete floor system would be required to minimize the change in the architectural floor plans. A post tension floor slab allows the use of large spans, with relative small deflections and vibration control. A post tension system limits cracking due to the prestressed compression therefore water tightness is an advantage. The system is considered very light in comparison with other concrete floor systems. Punching shear controls at every column therefore drop panels or column capitals would be required or addition of stud rails shear reinf.

Lateral Resisting System

Shear walls or moment frames would be considered as the lateral system and would replace all braced frames. Since shear walls are already being used on the basement floor and are connected to the continuous footings, adding more shear walls on higher levels would not be a major issue. Moment frames could be used to replace the braced frames located next to the exterior glazing. One thing that would have to be looked at is the amount of openings in the interior walls. Due to the light weight structure the amount of shear walls frames would not have to increase as much. Further analysis would have to be done to determine the amount of bays required as shear walls and moment frames.

Foundation Effects

Due to the increase in self weight from the original design, larger foundations would be required therefore further foundation design and analysis would be required.

Architectural Impact

Columns sizes would need to get bigger in this floor system in comparison to using steel columns. Since most interior walls exist at grid lines their impact of column sizes would be minimal. One major advantage of using this system is the 9" structural floor thickness at mid span. This would give flexibility to the mechanical equipments running, and allow for a lower floor to floor height reducing the building total height which has the potential to have seismic advantages. Drop panels or column capitals might have the potential to become a problem with the floor plans, but since interior walls are located at most of the gridlines, they can be used to hide the drop panels or capitals. Floor penetrations after the building has been built will become a problem since any tendon that exists in that area will lose structural strength along its entire length.

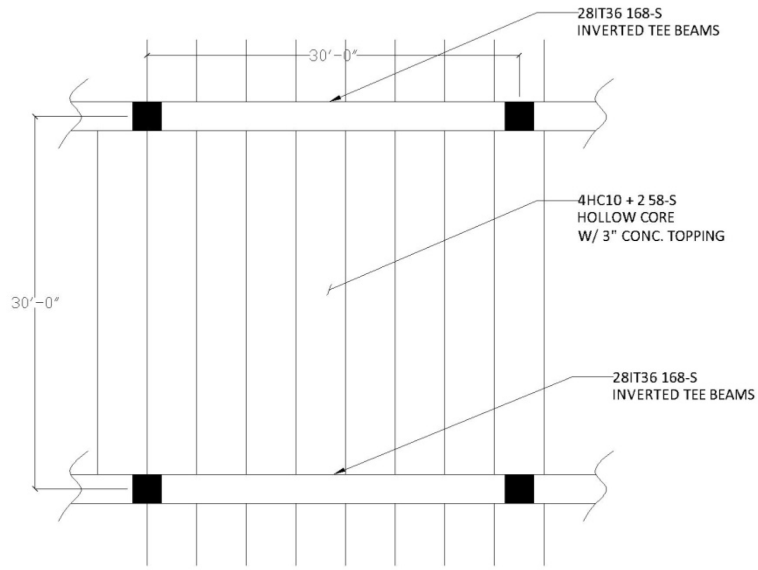
Constructions Impact

Further research would have to be done to determine if contracting companies in the area are familiar with this kind of system. The more common the system is in the area the more economical it would be to use such system in the design.

Summary

Advantages	Disadvantages
Thin floor membrane	High Cost of Construction
No additional fire proofing	Complex construction
Handles large loads	Shear Capital or Stud Rails
Crack control	Floor penetration hard to deal with
Deflection and vibration control	

Precast Prestressed Hallow Core Slab.



Material Properties
f'_c = 5000 psi
f_{pu} = 270,000 psi
Normal Weight Concrete (150 pcf)
Total slab thickness = 10" Hollow Core + 3" Topping

Design Summary
Interior Girders: 28IT36
Hollow Core 4HC10 + 3 58-S
Δmidspan = .274" @ Hollow Core
Δmidspan = .159" @ Girder
Floor Self weight = 149 psf

System Effectiveness

Structural Impact

Precast prestressed hollow core panels are sufficient for spanning long distances with large loads being applied. The 4'-0" hollow planks do not line up evenly with the 30'-0" bays therefore custom fabricated planks would be required.

Lateral Resisting System

Special reinforced concrete shear walls would be considered to be the main lateral resisting system; the response factor for intermediate the shear walls is 6 (R=6 for concentrically braced frames), an increase of the buildings self weight, will lead to more lateral resisting frames than the initial design. A 3" topping is required by code since the building is considered to be seismic region; the cover would provide a rigid diaphragm.

Foundation Effects

Due to the increase in self weight from the original design, larger foundations would be required therefore further foundation design and analysis would be required.

Architectural Impact

Even though the floor thickness is 13", the floor thickness at the beams is 36", which will make this system not gain any floor height due to the fact that mechanical equipment would have to run underneath the beams. Floor penetrations would be hard; a similar problem to the post tensioned slab, where any cut tendon in the plank would lose structural integrity. Therefore floor penetrations would have to be addressed locally by the structural engineer.

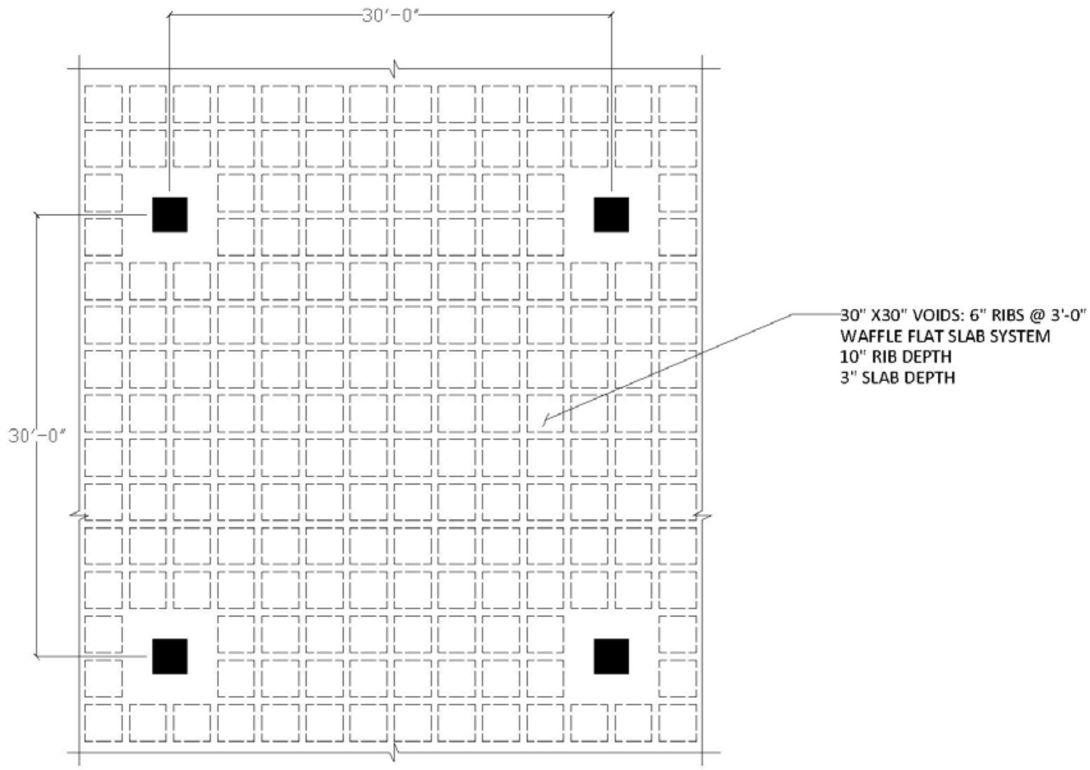
Constructions Impact

The main advantage of this floor system will be the effective time of construction. Since the precast panels are shipped in, and no form work is required the whole structure would be erected in a short matter of time. Connections are simple unlike composite steel and require minimal labor time.

Summary

Advantages	Disadvantages
Fast Erection Time	Thick floor membrane at beams
Stiff members	3" Diaphragm requirement adds cost
	Heavy floor system
	Floor penetrations hard to deal with

Two Way Concrete Waffle Flat Slab System



Material Properties
f'c = 4000 psi
fy = 60,000 psi
Normal Weight Concrete (150 pcf)
Total slab thickness = 10" Ribs + 3" Slab Depth
30"x30" Voids: 6" Ribs @ 36"

Reinforcement
Middle Strip (15')
Bottom Bars: #5 Long Bar per Rib
Top Bars: #5 @ 18" O.C.
Column Strip (15')
Bottom Bars: (2) #7 per Rib
Top Bars: #6 @ 8" O.C.

Design Summary
Δmidspan = .259"
Floor Self weight = 94 psf

System Effectiveness

Structural Impact

This system is highly effective in this case due to the high loading and large span. The load applied to the floor system is being carried in both directions. Concrete is only poured in areas where the reinforcement exists; waffle planks void out places where concrete is not required. This layout creates ribs closely spanned running in both directions. One major advantage of this system is that a lot of concrete dead weight is reduced from the voids, and additional beams are not required in the column strips.

Lateral Resisting System

Shear Walls or moment frames would be considered as the main lateral resisting system. Due to the reduction of concrete being used in the slab, this reduces the total buildings dead weight which would reduce the amount of moment resisting elements required compared to the other floor systems. Moment frames would be an effective system in the floor system, with an R value of 8, and would still maintain the architectural aesthetics of the building.

Foundation Effects

Due to the increase in self weight from the original design, larger foundations would be required therefore further foundation design and analysis would be required.

Architectural Impact

This system has a thin floor thickness therefore in comparison with the composite steel system this would add additional floor height. Although slab openings would become a problem and due to the excessive openings due to mechanical equipment, the structural engineer would have to address this issue from the start.

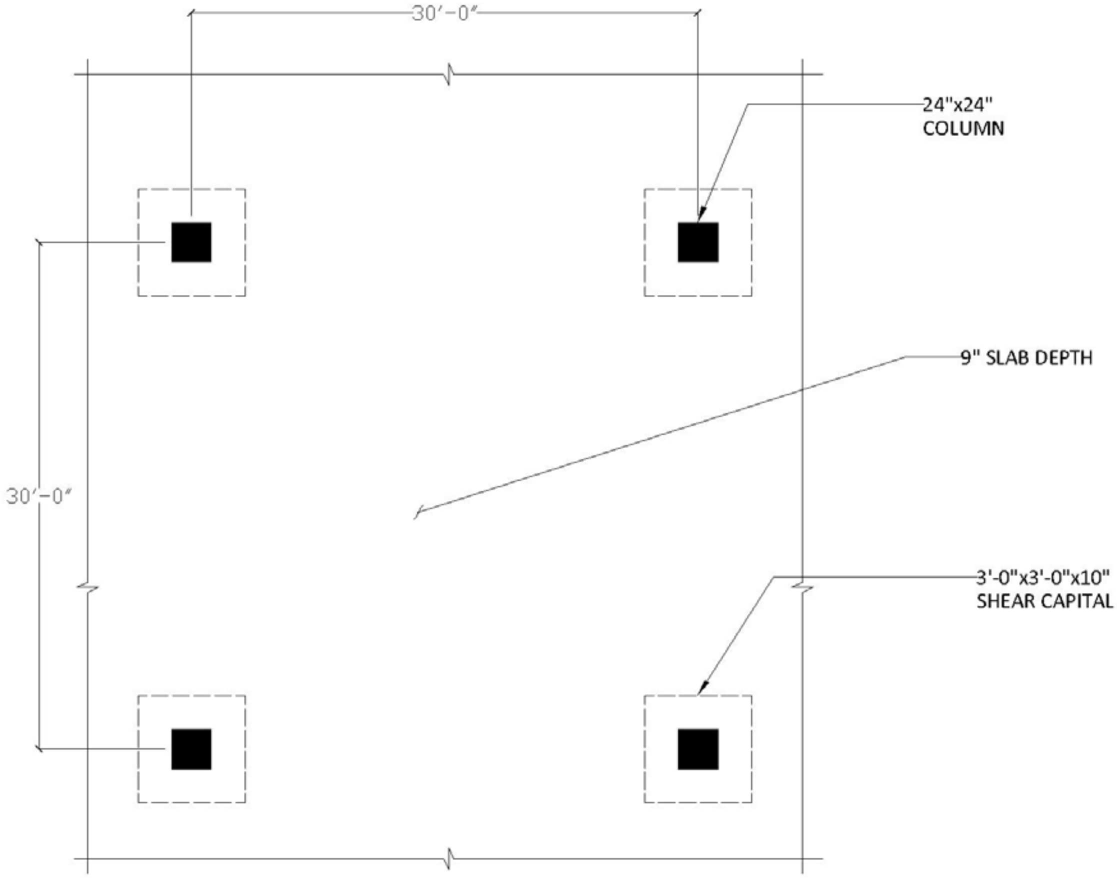
Constructions Impact

Waffle slabs use expensive formwork; therefore the cost of purchasing the waffle plank would drive up construction cost. Although setting up formwork would not be considered time consuming since the same formwork shape is used over and over again.

Summary

Advantages	Disadvantages
Light structure in comparison with other concrete structures	High construction cost
Stiff system	Difficult floor penetrations
Thin floor membrane	

Two Way Concrete Slab with Beams



Material Properties
f'c = 5000 psi
fy = 60,000 psi
Normal Weight Concrete (150 pcf)
Total slab thickness = 8.25"

Reinforcement
Column Strip:
Top: #5 @ 12" O.C. (11'-0")
Bottom: #5 @ 16" O.C. Continuous
Middle Strip:
Top: #5 @ 9" O.C. (11'-0")
Bottom: #5 @ 16" O.C. Continuous
Beams:
Top: (5) #8 (11'-0)

Bottom: (2) #7
Stirrup: (15) #4 @ 4” , (12) #4 @ 8”
Refer to Appendix for calculation and reinforcement details Top reinforcement is place in the middle of the column with the total length specified Stirrup configurations start 4” from the face of the column at each side.

Design Summary
$\Delta_{midspan}$ = .205” @ Column Strip
$\Delta_{midspan}$ = .176” @ Middle Strip
Floor Self weight = 139 psf

System Effectiveness

Structural Impact

Due to the large span, beams are required to run along the column grid lines.

Lateral Resisting System

Shear walls or moment frames would be considered as the lateral resisting system, since the floor self weight is relatively high; more lateral resisting bays would need to be considered in the final design.

Foundation Effects

Due to the increase in self weight from the original design, larger foundations would be required therefore further foundation analysis and design would be required.

Architectural Impact

Due to the impact of more shear walls, there would be the possibility of altering the floor plans interior and exterior wall openings unless moment frames were considered. Added floor height would be an advantage using this system; although the presence of beams at column gridlines would disturb the mechanical ducts path. Floor penetrations are relatively easy compared to the other systems, and could be solved easily by adding more reinforcement.

Constructions Impact

Cost of construction would become high due to the excessive form work and its slow rate of construction.

Summary

Advantages	**Disadvantages**
Thin slab	Drop Panels Required
Easy floor penetrations	Hard formwork with beams and drop panels
	Heavy floor system
	Slow construction + high cost

Summary Comparison Chart

Criteria	Composite Steel	Post Tensioned	Hollow Core	Waffle Slab	Two Way Slab w/ Beams
Thickness @ Mid Bay	24.25"	8"	13"	13"	8.25"
Thickness @ Grid	30.25"	10"	36"	13"	18"
Cost	$18.95 /SF	$20.23 /SF	$23.25 /SF	$21 /SF	$21.75 /SF
Weight	54 psf	100 psf	149 psf	94 psf	139 psf
Δ_{max}	.54"	.3"	.27"	.26"	.21"
Floor Vibrations	-	Not an issue	Not an issue	Not an issue	Not an issue
Fire Rating	Fire Proofing Spray	Concrete Cover	Concrete Cover+ Spray	Concrete Cover	Concrete Cover
Floor Penetrations	Moderate	Hard	Hard	Moderate	Easy
Construction Time	Fast	Moderate	Fast	Moderate	Slow
Feasibility		Possible but requires further analysis	Few advantages	Possible but requires further analysis	Few Advantages

Membranes thickness are taken both at mid span were there slab might only exist, and at the grid line where the beam and slab exists.

Cost was computed using the 2006 RS Means Square Foot Cost and 2007 RS Means concrete and Masonry Cost Data, 125 psf Super Imposed Dead Load was assumed with a 30'x30' bay super structure. Post Tensioned cost was computed using flat plate floor slab with drop panels and then a stressing tendons cost was applied from 2007 RS Means concrete and Masonry Cost Data.

Please note for comparison purposes all concrete design was done for normal weight concrete, but since the building is located in seismic region, the final floor system design will probably be addressed with light weight concrete to reduce lateral loads on the lateral resisting system. Applying a factor of 110(lightweight)/150(normal weight) = .73, will give you a good comparison with the composite steel system if the concrete floor system was to be considered using light weight concrete. This estimation tells us that the composite steel deck would be the lightest system.

Floor vibration calculations were not taken into consideration in this report; nevertheless floor vibrations should still be looked at when the final design is complete. Therefore since deflection is inversely proportional to stiffness, and the stiffer the member the less floor vibrations would become an issue. We are able to conclude that the less the floor system deflects compared to the original design the less floor vibrations would become an issue.

Fire Rating is achieved by the following methods, fire proofing spray is required by some, while all the concrete system could achieve the first rating with an extra concrete cover. Please note since the hollow core system comes prefabricated, additional fireproofing would be required by spray or custom prefabricated planks can be ordered with custom concrete cover.

Refer to System Effectiveness sections in the floor system to justify floor penetrations and construction time.

Conclusion

Saint Joseph Patient Care Center is in a high seismic region therefore the building self weight is a major factor when considering different floor systems. The use of steel has highly benefited the amount of lateral resisting frames in the building. And with the braced frames being used as an architectural feature, steel has incorporated itself well into the building. The use of concrete in the building will increase the building self weight; therefore increase the buildings overturning moment in the foundation due to seismic activity.

The hollow core floor system should be altered so that the planks rest on the flanges of steel beams. This would have gained floor height and reduced the floor's self weight by not using huge precast beams. Overall I think this system would be highly ineffective; due to its large floor system self weight, its cost is well above the other floor systems and with the hospitals floor penetrations this system would be inefficient. The use of a two way slab with beams and drop panels could be ruled out for the same reason, its high cost and weight would make this system inefficient for a hospital in seismic region.

Two systems that could be considered now would be a post tensioned slab and waffle slabs. The two systems have minor differences in cost, self weight and 3" to 5" difference in floor height. The two systems are relatively at the same stiffness judging by deflections. The post tensioned system would be highly effective at the 2nd floor court yard and roof due to the extreme loading conditions. The courtyard and roof would have to use deeper waffle slabs which would add construction material cost. Floor penetrations are problems for both systems and would have to be addressed locally.

Judging all the proposed concrete systems, the initial design using a composite steel floor system was the most effective. This can be due to many reasons; one of them is due to a low self weight compared to the concrete structures which means less seismic lateral forces that can affect the lateral force resisting system. Floor to floor height is not a major issue due to the original design being less than 80 ft. Hospitals generally go through a lot of renovations and change in mechanical and medical equipment during the life time of the buildings; therefore a system with the ease of making adjustments that affect the structural system like opening in the floor system, and heavy equipment placed in different locations would need to be addressed. A composite steel floor system offers the easiest and fastest solutions in compared to the concrete floor system when deciding to make floor openings. When heavy equipment layouts are changed steel members can be added to enhance the structural strength capacity of the floor area being changed. These reasons are probably why the structural engineer went with a composite steel floor system.

Appendix

Composite Concrete with Steel Beams Framing System Calculations

Gravity Beam check (W16x31)

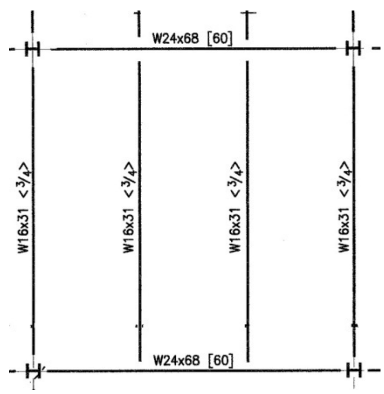


Fig17. Typical 30'-0"x30'-0" bay located on Levels 2, 3 and 4

Computer Loadings	
Live Loads	80 psf
Dead Loads	79 psf
Beams with composite action, deck running perpendicular to beams	f'c = 3000 psi
	¾" Shear Studs @ 12" OC
w_u	2.22 Klf
M_u	250 ft-Kips
V_u	33.3 Kips
Compute Moment Strength Capacity	
Σ Qn from studs	30*17.1=516k
Σ Qn required	456K
beff = min(0.5*span, spacing)	10'
a	1.49"
Y2	5.5"
ΦMp	460 ft-K > Mu **OK**
Check Deflection	
I_{LB}	1200 in^4
$\Delta_{max} = l/360$	1"
$\Delta = 5*wL^4/384EI$	.42" **OK**
Check Deflection before composite action	
w_u (Dead load of Concrete Deck)	.59 Klf
I	384 in^4
$\Delta_{max} = 5*wL^4/384EI$	.99"-.75" Camber = .24" **OK**

Gravity Girder Check (W24x68)

Computer Loadings	
Beams with composite action, deck running perpendicular to beams	f'c = 3000 psi
	(60) ¾" Shear Studs
Pu (@ 1/3 Points)	33.3 Kips
M_u	666 ft-Kips
V_u	33.3 Kips
Compute Moment Strength Capacity	
Σ Qn from studs	60*17.1=1026k
Σ Qn required	1000K
beff = min(0.5*span, spacing)	15'
a	2.18"
Y2	5.16"
ΦMp	1270 ft-K > Mu **OK**
Check Deflection	
I_{LB}	4680
Δmax = l/360	1"
Δ	.29" **OK**
Check Deflection before composite action	
P_u (Dead load of Concrete Deck @ 3rd points)	16.7 Kips
I	1830 in^4
Δ	.52" **OK but overdesigned**

This analysis does not take floor vibrations into account, and since larger steel sections perform better damping, the girder might be increased in size for this reason.

Two Way Post Tensioned Slab Calculations

Ram Concept was used to design this floor system.

Design Criteria
f'c = 5000 psi
fy = 60,000 psi
fpu = 270,000 psi
Unbounded tendons, ½" Diameter, 7 Wire Strands A = 0.153
Lightweight concrete (110 pcf)
Total slab thickness = 8"
Super Imposed Dead Load = 32 psf
Live Load = 80 psf

The following are steps taken while designing the floor system using RAM concept:
Slab Thickness h = L/45 = 30(12)/45 = 8" min, RAM uses 9" due to less failures

Since fpu = 270 ksi, and According to ACI 18.6 the Estimated Prestress Losses = 15ksi
f_{se} = (.7)(270) ksi – 15 ksi = 174 ksi (ACI 18.5.1)
P_{eff} = A*f_{se} = (0.153)(174) = 26.6 Kips/tendon
Precompression Limits >124psi and <300 psi (ACI 18.12.4)

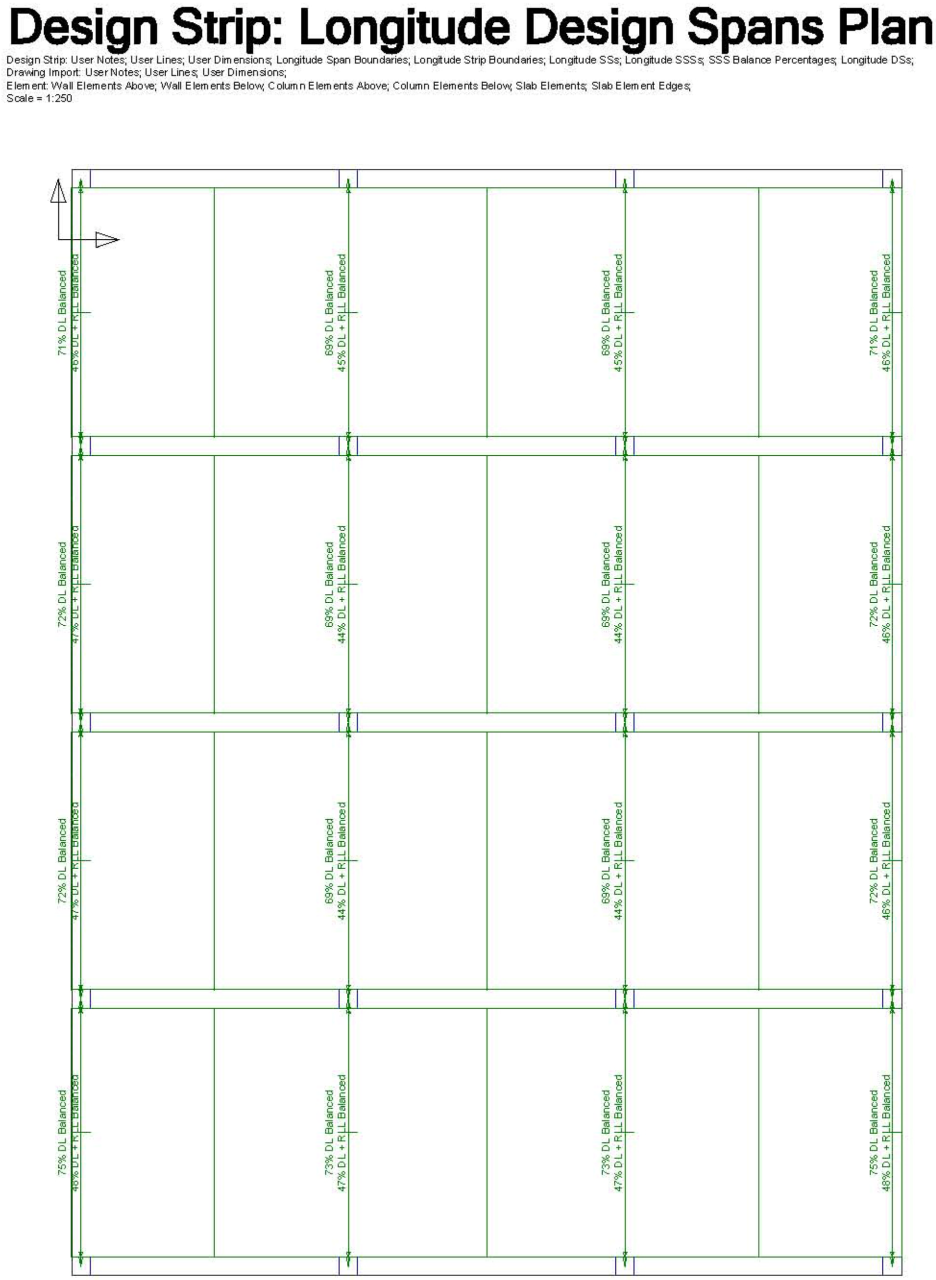
Preliminary Number of Tendons Required per bay running longitudinal
.75% Balance of Self Weight Required
W_{DL}= (150)(9/12)(30)(.75) = 2,531 plf
P = (2531)$(30)^2$/8(7/12) = 488 kips, Tendon 7" from bottom of slab
# Tendons = 433/26.6 = 19 Tendons
P = 19(26.6)1000/(9(30)12) = 156 > 125 psi and < 300 psi, therefore ok

Preliminary Number of Tendons Required per bay running latitudinal
.75% Balance of Self Weight Required
W_{DL}= (150)(9/12)(30)(.75) = 2,531 plf
P = (2531)$(4)^2$/8(3/12) = 20 kips, Tendon 3.75" from bottom of slab and spaced @ 4'
# Tendons = 20/26.6 = 1 Tendons
P = 1(26.6)1000/(9(4)12) = 61 > 125 psi and < 300 psi, therefore not ok, use 3 Tendons

Results were placed into RAM Concept and Analysis was preformed, the numbers of tendons were then manually changed with the tendon mid span locations to obtain a balance of approximately 75% self weight. Results are as follows.

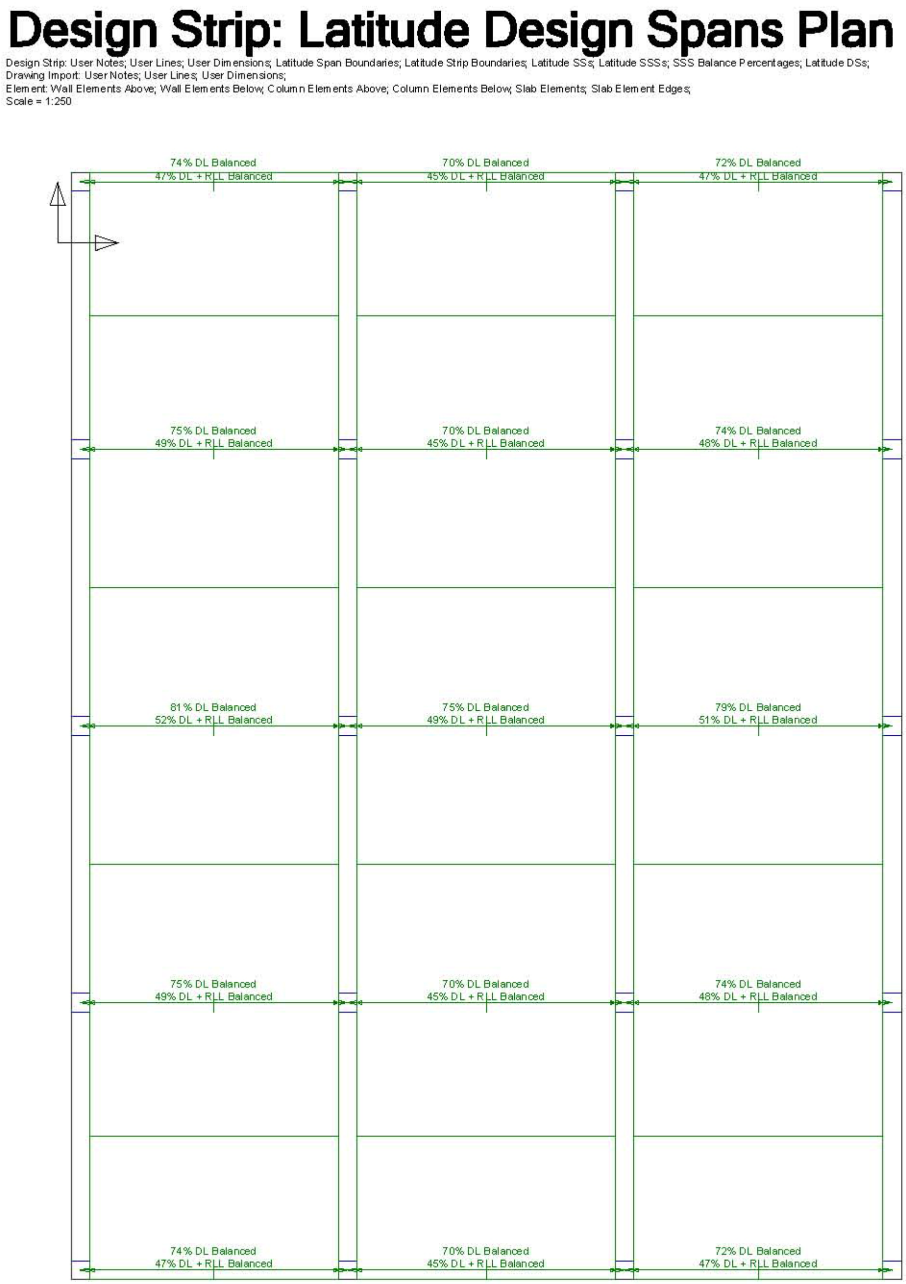
Design Strips Longitudinal

Prestress force is balanced to approx. 75% self weight dead load.



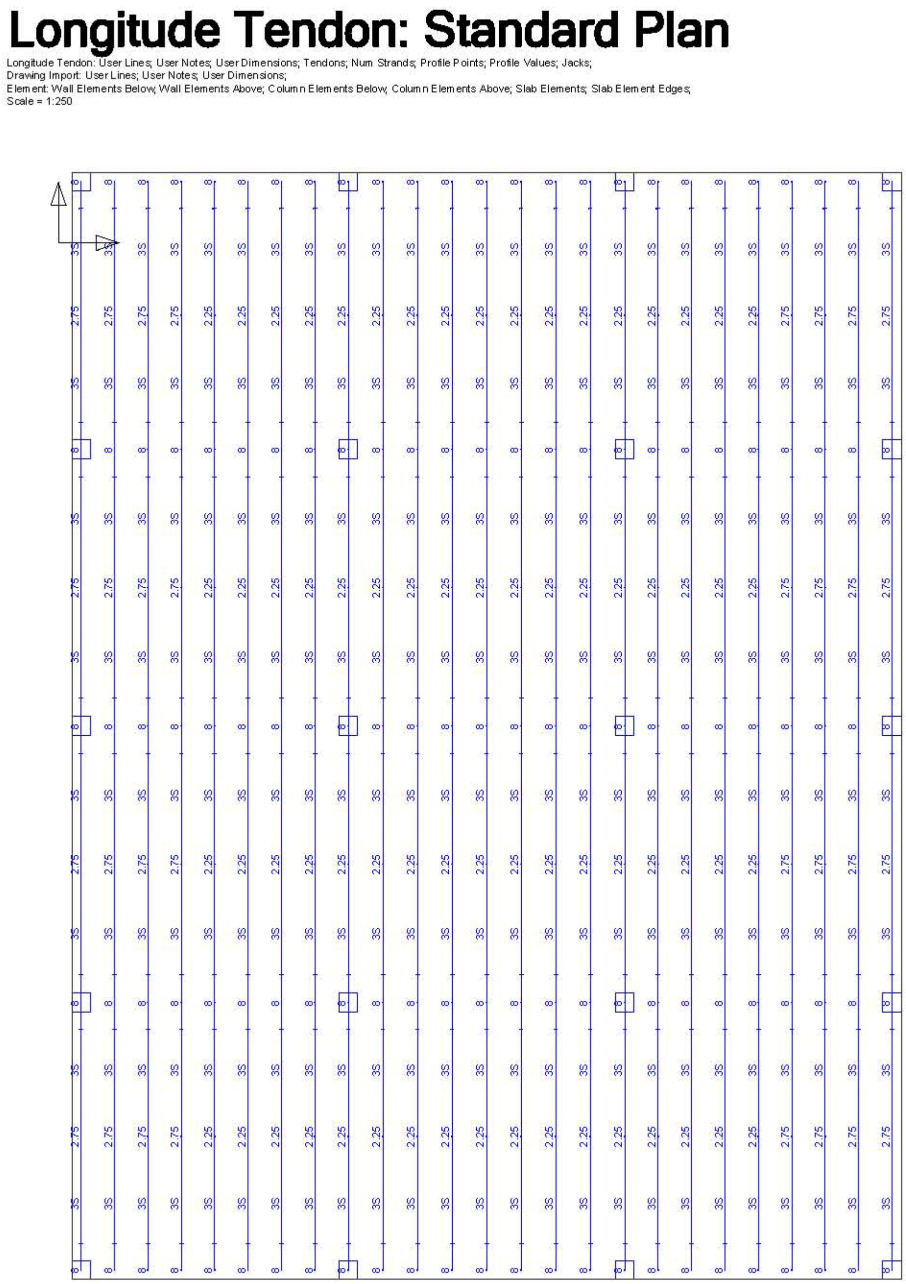
Design Strips Latitudinal

Prestress force is balanced to approx. 75% self weight dead load.



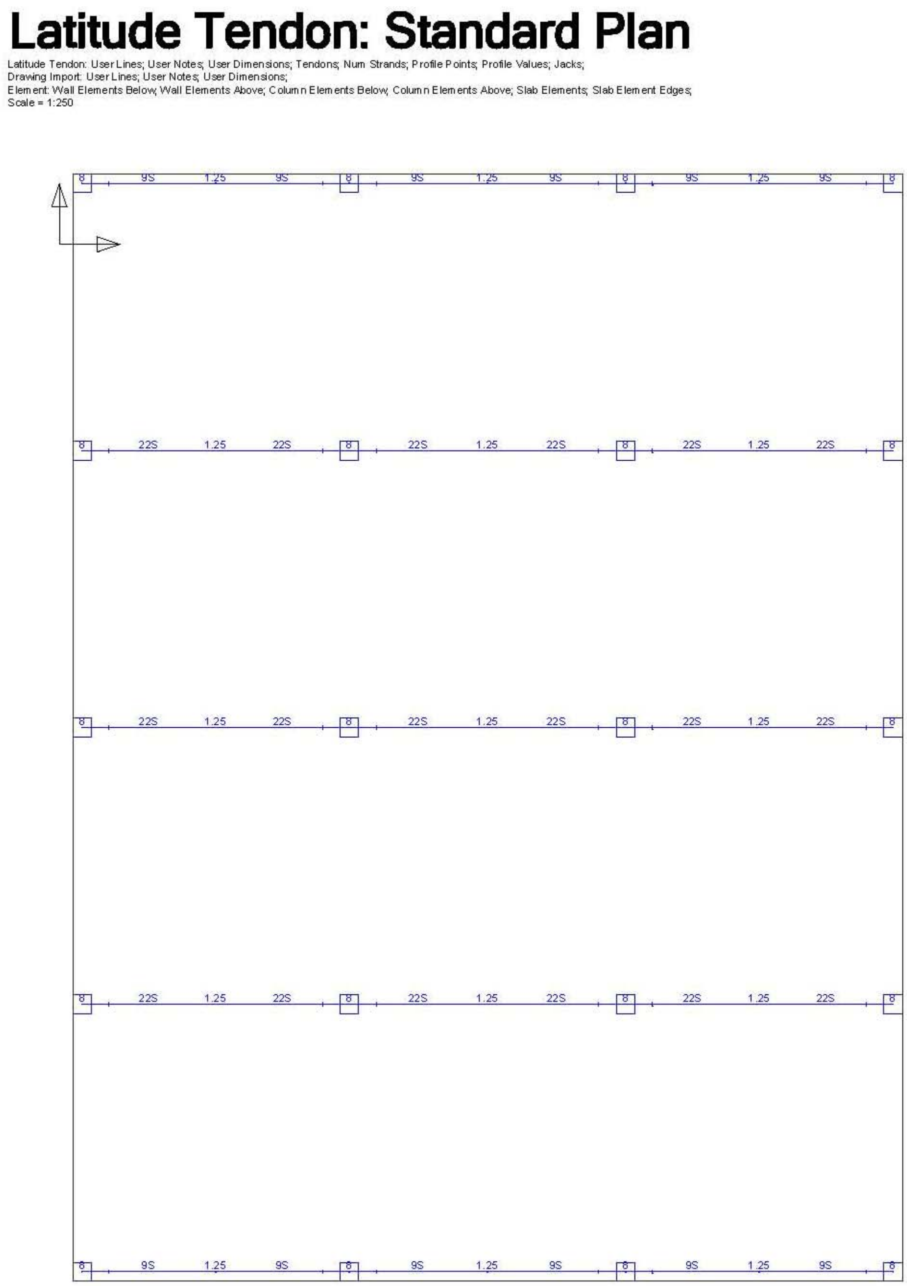
Longitudinal Tendons

Tendon location is shown and altered so that it balances to 75% self weight. 3 Strands @ 3'-4' O.C.

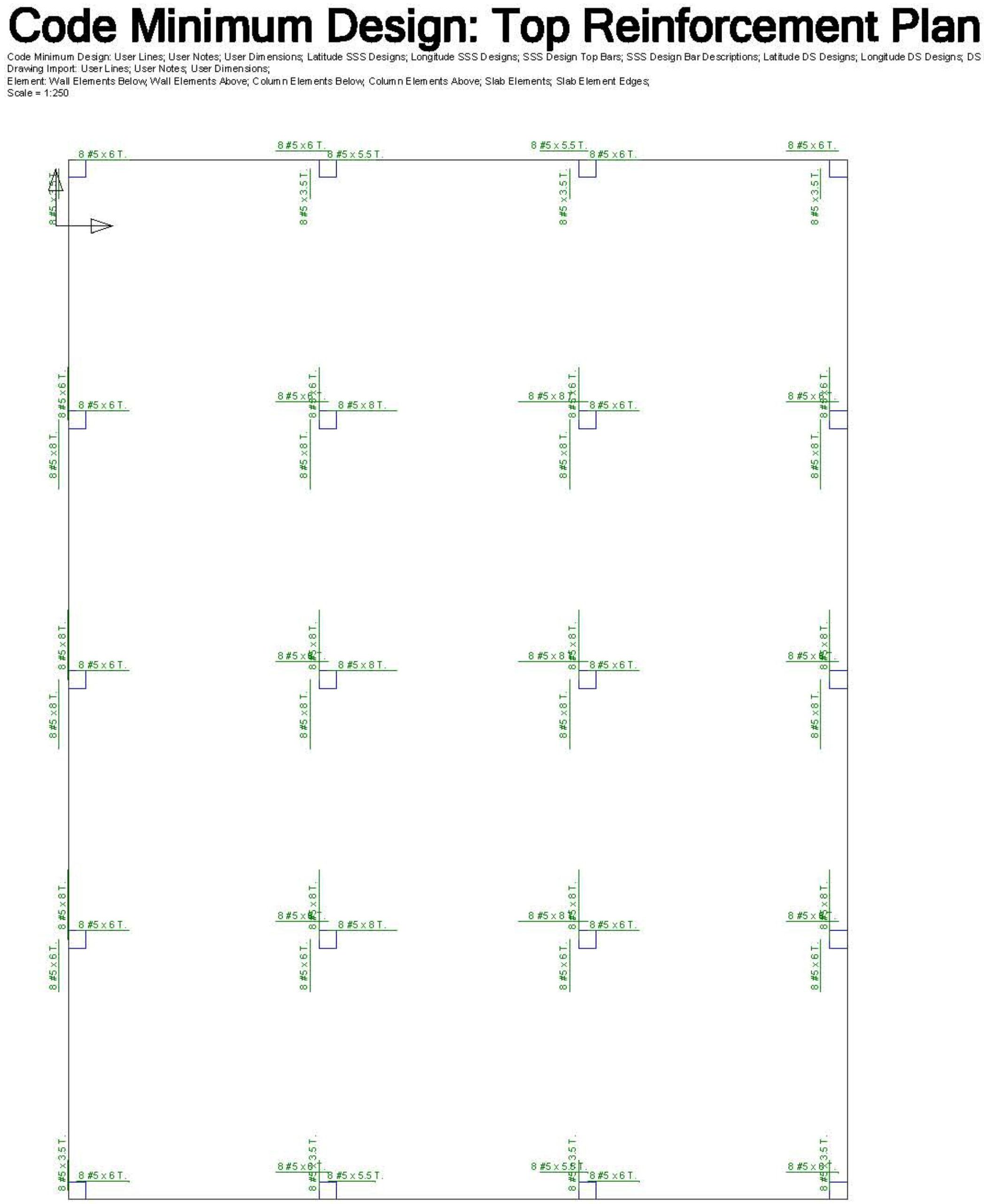


Latitudinal Tendons

Tendon location is shown and altered so that it balances to 75% self weight. 9 Strands Exterior Band, 22 Strands Interior Band.



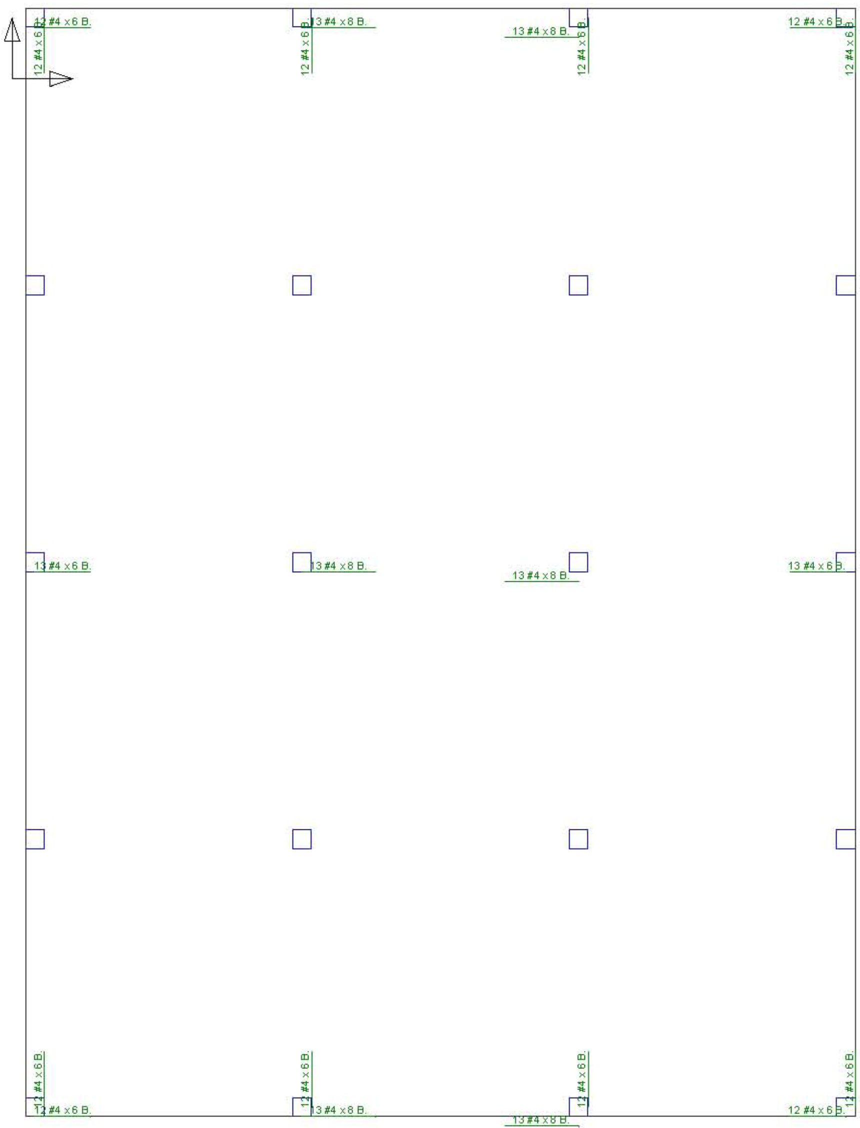
Top Reinforcement



Bottom Reinforcement

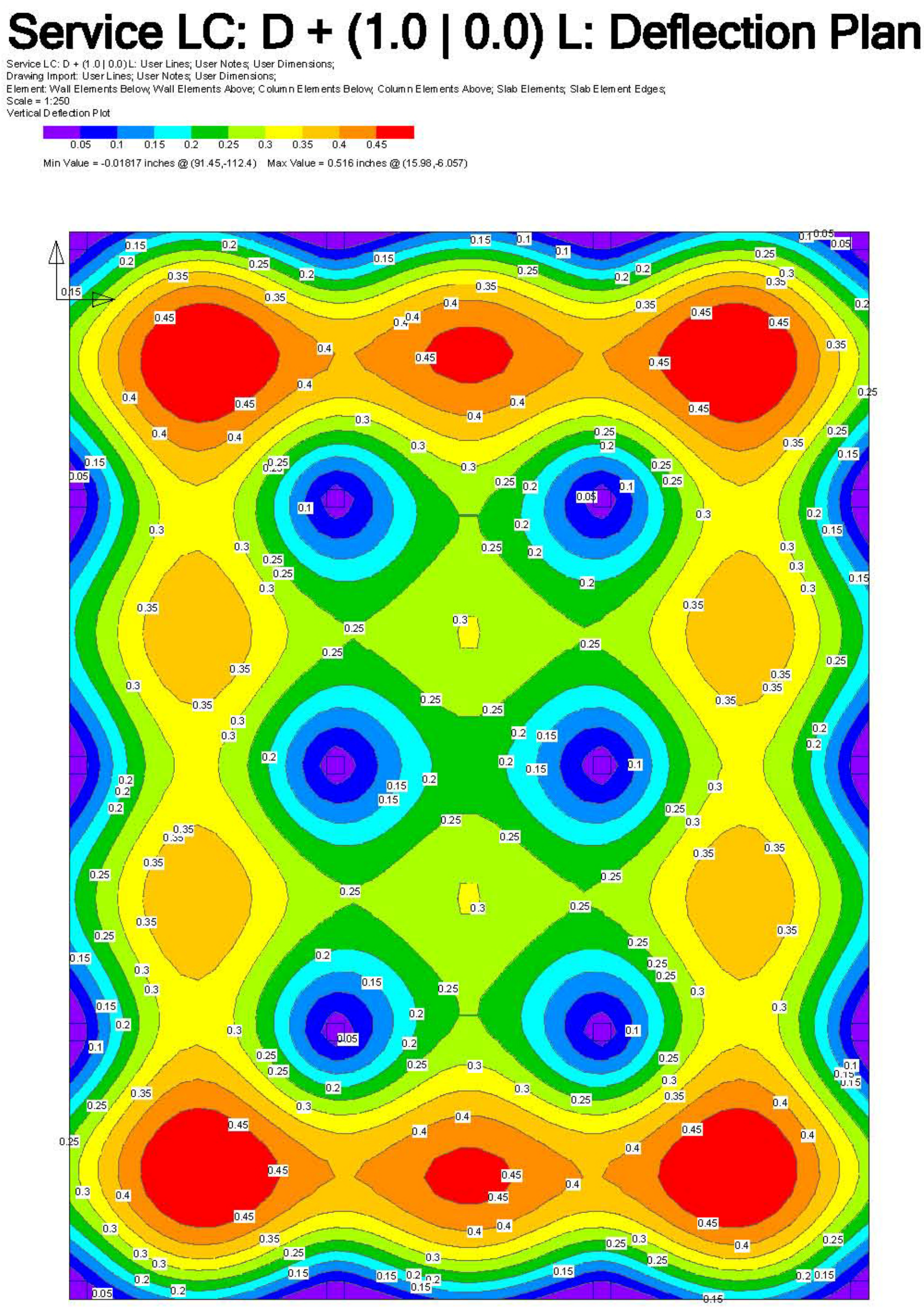
Code Minimum Design: Bottom Reinforcement Plan

Code Minimum Design: User Lines; User Notes; User Dimensions; Latitude SSS Designs; Longitude SSS Designs; SSS Design Bottom Bars; SSS Design Bar Descriptions; Latitude DS Designs; Longitude DS Designs; DS Design Bottom Bars;
Drawing Import: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Edges;
Scale = 1:250



Deflection

All live load deflection < L/360, therefore OK



Punching Shear

Punching shear controls at every column. There drop panels were using hand calculations.

w_u = 1.2[9(150)/12 + 32] + 1.6[80] = 301 psf
Assume 24"x24" Column, with d=8" (1" cover") therefore b_o= 4(24+8) = 128
V_u = [(30)(30) – $(24+8)^2$] x 301= 269 Kips

ΦVc = (.75)(4)$(5000)^{.5}$(128)(8) = 217 kips < V_u, therefore Punching shear controls

Design for drop panel Depth:
ΦVc = (.75)(4)$(5000)^{.5}$(128)(d) = V_u = 269, d= 10"

Determine Drop Panel Size:
V_u = [(30)(30) – $(x)^2$] x 301 = ΦVc = (.75)(4)$(5000)^{.5}$(12x)(10), x = 2.64' use 3'-0"x3'-0" Drop Panel

Final Drop Panel Design, 3'-0"x3'-0"x10"

Edge and Corner Drop Panels not design in this report.

Precast Prestressed Hollow Core Slab. Calculations

PCI Industry Handbook 6th edition was used to design this floor system.

Design of the hollow core floor system:

Safe Super Imposed Service Loads (psf) = 20 (Partitions) + 12 (Super Imposed) + 80 (Live Loads) = 112 psf.

Strand Pattern Designation
48-S

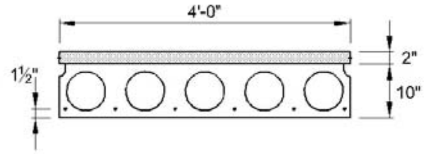
- S = straight
- Diameter of strand in 16ths
- No. of Strand (4)

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

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

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

HOLLOW-CORE
4'-0" x 10"
Normal Weight Concrete



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

Section Properties

	Untopped	Topped
A =	259 in.²	355 in.²
I =	3,223 in.⁴	5,328 in.⁴
y_b =	5.00 in.	6.34 in.
y_t =	5.00 in.	5.66 in.
S_b =	645 in.³	840 in.³
S_t =	645 in.³	941 in.³
wt =	270 plf	370 plf
DL =	68 psf	93 psf
V/S =	2.23 in.	

4HC10 + 2

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

Strand Designation Code	Span, ft																										
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
48-S	308	287	256	228	204	183	165	148	133	119	107	96	86	74	63	52	43	34	26								
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4								
	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.8	–1.0	–1.2	–1.4	–1.7								
58-S	317	298	282	267	252	237	219	198	180	163	148	134	120	105	92	80	69	59	50	41	33	26					
	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	0.4	0.3	0.2	0.2	0.1	0.0	–0.1	–0.3	–0.4					
	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.1					
68-S	326	307	291	273	258	246	234	222	212	202	188	171	153	137	122	108	96	84	74	64	55	46	38	31			
	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.2			
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.3	0.2	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.2			
78-S	335	313	297	279	267	252	240	228	218	208	196	189	181	165	150	135	122	109	97	86	76	67	58	50	42	35	28
	0.6	0.7	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.4	0.3	0.1	0.0	–0.2
	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–1.9	–2.3	–2.8
88-S	344	322	306	288	273	258	246	234	221	211	202	195	184	178	172	158	144	130	118	107	96	87	77	68	60	52	44
	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.8	0.7	0.5	0.3
	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.6	–2.0

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

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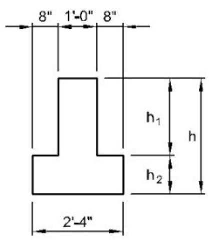
Safe Super Imposed Service Loads (psf) = 148 > 112 psf therefore sufficient to carry the load

Hollow Core Dead Weight = 93 psf + 1/12*150(1" Additional cover) = 105.5 psf

Design of the beams supporting the floor system as taken as follows:

Safe Super Imposed Service Loads (plf) = [112 (Super Imposed Dead load + Live Load) + 105.5(Dead Load)]* 30 = 5850 plf for Inverted Tee

INVERTED TEE BEAMS



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Normal Weight Concrete

Section Properties								
Designation	h in.	h_1/h_2 in./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.

Key

6511 – Safe superimposed service load, plf.
0.2 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

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

Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	98-S	2.44 2.44	6511 0.2 0.1	5076 0.3 0.1	4049 0.4 0.1	3289 0.4 0.1	2711 0.5 0.1	2262 0.5 0.1	1905 0.6 0.0	1617 0.7 0.0	1381 0.7 0.0	1186 0.7 0.0	1022 0.8 –0.1							
28IT24	188-S	2.73 2.73	9612 0.2 0.1	7504 0.3 0.1	5997 0.3 0.1	4882 0.4 0.1	4034 0.4 0.1	3374 0.5 0.1	2850 0.6 0.1	2427 0.6 0.1	2081 0.7 0.1	1795 0.7 0.1	1555 0.7 0.0	1351 0.8 0.0	1178 0.8 –0.1	1029 0.8 –0.2				
28IT28	138-S	3.08 3.08			8353 0.3 0.1	6822 0.3 0.1	5657 0.4 0.1	4750 0.5 0.1	4031 0.5 0.1	3451 0.6 0.1	2976 0.6 0.1	2582 0.7 0.1	2252 0.7 0.1	1973 0.8 0.1	1735 0.8 0.0	1530 0.8 0.0	1352 0.9 –0.1	1197 0.8 –0.2	1061 0.8 –0.2	
28IT32	158-S	3.47 3.47				9049 0.3 0.1	7521 0.4 0.1	5333 0.4 0.1	5389 0.5 0.1	4628 0.5 0.1	4006 0.6 0.1	3490 0.6 0.1	3057 0.7 0.1	2691 0.7 0.1	2379 0.8 0.1	2110 0.8 0.1	1876 0.9 0.0	1673 0.9 0.0	1495 0.9 0.0	1337 0.9 –0.1
28IT36	168-S	3.50 3.50					9832 0.3 0.1	8295 0.4 0.1	7075 0.4 0.1	6092 0.5 0.1	5287 0.5 0.1	4619 0.6 0.1	4060 0.6 0.1	3587 0.7 0.1	3183 0.7 0.1	2835 0.8 0.1	2534 0.8 0.0	2271 0.9 0.0	2040 0.9 0.0	1836 0.9 –0.1
28IT40	198-S	4.21 4.21							8638 0.4 0.1	7440 0.5 0.1	6460 0.5 0.1	5647 0.6 0.1	4966 0.6 0.1	4390 0.7 0.1	3898 0.7 0.1	3474 0.8 0.1	3107 0.8 0.1	2787 0.8 0.1	2506 0.9 0.1	2258 0.9 0.1
28IT44	208-S	4.40 4.40								9186 0.4 0.1	7989 0.5 0.1	6997 0.5 0.1	6165 0.6 0.1	5462 0.6 0.1	4861 0.7 0.1	4344 0.7 0.1	3896 0.7 0.1	3505 0.8 0.1	3162 0.8 0.1	2859 0.8 0.0
28IT48	228-S	4.55 4.55									9719 0.4 0.1	8525 0.5 0.1	7523 0.5 0.1	6676 0.6 0.1	5953 0.6 0.1	5330 0.7 0.1	4791 0.7 0.1	4320 0.8 0.1	3907 0.8 0.1	3542 0.9 0.1
28IT52	248-S	5.17 5.17										9987 0.5 0.1	8823 0.5 0.1	7838 0.6 0.1	6998 0.6 0.1	6274 0.6 0.1	5647 0.7 0.1	4100 0.7 0.1	4619 0.8 0.1	4196 0.8 0.1
28IT56	268-S	5.23 5.23												9307 0.5 0.2	8319 0.6 0.2	7469 0.6 0.2	6731 0.7 0.2	6088 0.7 0.2	5524 0.8 0.2	5026 0.8 0.2
28IT60	288-S	5.57 5.57													9645 0.6 0.2	8668 0.6 0.2	7820 0.7 0.2	7081 0.7 0.2	6432 0.8 0.2	5859 0.8 0.2

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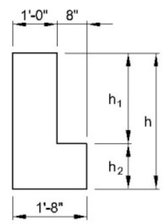
Safe Super Imposed Service Loads (plf) = 5850/2 = 2925 plf for L Beam used at exterior bay.

L-BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Designation	h in.	h_1/h_2 in./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.

Key

6566 – Safe superimposed service load, plf.
0.3 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

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

Designation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
20LB20	98-S	2.44 2.44	6566 0.3 0.1	5131 0.4 0.2	4105 0.5 0.2	3345 0.6 0.2	2768 0.7 0.2	2318 0.8 0.2	1961 0.9 0.3	1674 1.0 0.3	1438 1.0 0.3	1243 1.1 0.3	1079 1.2 0.2							
20LB24	108-S	2.80 2.80	9577 0.3 0.1	7495 0.3 0.1	6006 0.4 0.1	4904 0.5 0.1	4066 0.5 0.1	3414 0.6 0.2	2896 0.7 0.2	2479 0.8 0.2	2137 0.9 0.2	1854 0.9 0.2	1617 1.0 0.1	1416 1.0 0.1	1244 1.1 0.1	1097 1.1 0.0	969 1.2 0.0			
20LB28	128-S	3.33 3.33			8228 0.4 0.1	6733 0.4 0.1	5596 0.5 0.2	4711 0.6 0.2	4009 0.6 0.2	3443 0.7 0.2	2979 0.8 0.2	2595 0.9 0.2	2273 0.9 0.2	2000 1.0 0.2	1768 1.1 0.2	1567 1.1 0.2	1394 1.2 0.1	1243 1.2 0.1	1110 1.2 0.0	992 1.3 0.0
20LB32	148-S	3.71 3.71				8942 0.4 0.1	7446 0.5 0.2	6281 0.5 0.2	5356 0.6 0.2	4611 0.7 0.2	4001 0.7 0.2	3495 0.8 0.2	3071 0.9 0.3	2712 1.0 0.3	2406 1.0 0.3	2143 1.1 0.2	1914 1.2 0.2	1715 1.2 0.2	1540 1.3 0.2	1386 1.3 0.1
20LB36	168-S	4.25 4.25					9457 0.4 0.2	7988 0.5 0.2	6823 0.5 0.2	5883 0.6 0.2	5113 0.7 0.2	4476 0.8 0.3	3941 0.8 0.3	3489 0.9 0.3	3103 1.0 0.3	2771 1.1 0.3	2483 1.1 0.3	2231 1.2 0.3	2011 1.2 0.3	1816 1.3 0.2
20LB40	188-S	4.89 4.89						9812 0.4 0.2	8386 0.5 0.2	7235 0.6 0.2	6293 0.6 0.2	5513 0.7 0.2	4858 0.8 0.2	4305 0.8 0.3	3832 0.9 0.3	3425 1.0 0.3	3073 1.0 0.3	2765 1.1 0.3	2495 1.1 0.3	2257 1.2 0.3
20LB44	198-S	5.05 5.05								8959 0.5 0.2	7803 0.6 0.2	6845 0.6 0.2	6042 0.7 0.2	5363 0.8 0.2	4783 0.8 0.2	4284 0.9 0.2	3851 0.9 0.2	3474 1.0 0.2	3143 1.1 0.2	2850 1.1 0.2
20LB48	218-S	5.81 5.81									9226 0.5 0.2	8100 0.6 0.2	7158 0.6 0.2	6360 0.7 0.2	5678 0.8 0.2	5092 0.8 0.2	4584 0.9 0.3	4140 0.9 0.3	3751 1.0 0.3	3408 1.1 0.3
20LB52	238-S	6.17 6.17										9634 0.6 0.2	8521 0.6 0.2	7578 0.7 0.2	6774 0.7 0.3	6082 0.8 0.3	5482 0.9 0.3	4958 0.9 0.3	4499 1.0 0.3	4094 1.0 0.3
20LB56	258-S	6.64 6.64											9954 0.6 0.2	8860 0.7 0.2	7927 0.7 0.3	7124 0.8 0.3	6427 0.8 0.3	5820 0.9 0.3	5287 1.0 0.3	4816 1.0 0.3
20LB60	278-S	7.33 7.33													9089 0.7 0.3	8173 0.7 0.3	7380 0.8 0.3	6688 0.9 0.3	6080 0.9 0.3	5544 1.0 0.3

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Computing Deflections:

E= 57000(f'c)^.5 = 4,030,508

I_{LBeam} =59,119 in^4

Δ_{max} = 5/384*(15x80)*30^4*12^3/(59119*4030508) =.09" for L Beam

$I_{Inverted\ Tee}$ = 68,101 in^4

Δ_{max} = 5/384*(30x80)*30^4*12^3/(68101*4030508) =.159" for Inverted Tee

Two Way Concrete Waffle Flat Slab System Calculations

CRSI Design Handbook 2002 is used to design waffle flat slab system.

Factored Super Imposed Load (psf) = 1.4(12(Super Imposed) + 20(Partitions)) + 1.7 (80 (Live Load)) = 181 psf

Using Waffle Flat Slab System 30" x 30" Voids: 6" Ribs @ 36" @ Pg 11-20
Total Depth = 13", Rib Depth = 10", Total Slab Depth = 3"

Concrete Volume per SF=.624 CF/SF
$Gamma_f$ = .626
-M_{edge} = 255 ft-kips
+ M_{bot} = 604 ft-kips
-M_{int} = 686 ft-kips

$w_{udeadload}$ = 1.4(.624)(150) =131 psfwu
w_u = 131 + 181 = 312 psf
V_u = 312 (30*30/2) = 140 kips
M_o = (255 + 604)/2 + 686 = 1115.5 kips-ft
.3M_o = 335 kips-ft

Shear Check:
Using a 24"x24" Column size, 13" Slab table 11-4 gives us:
C_{ab} = 9.31
A_c = 1338
J_c = 115,395
Vu = 140,000/(0.85*1338)+ 0.626(335)(12000)(9.31)/.85(115,395) = 123 + 239 = 362 psi
4(4000)^.5 = 252 < 362 < 7(4000)^.5 = 379 therefore OK

Deflection Check:
Pg 11-4
$\Delta_{max} = kwL^4/Ect^3_e$ = 0.1028*80*(30)^4*12^2/(3,500,000*10.18^3) = 0.259"

Design Summary:
Interior Bay
Col. Strip:
Top: 22-#6 Therefore #6 @ 6" O.C.
Bottom: 2 # 7 Bars per Rib
Middle Strip:
Top: 10 #5 therefore #5 @ 18" O.C.
Bottom: #5 Long Bar and #6 Short Bar

11-20

WAFFLE FLAT SLAB SYSTEM 30″ X 30″ Voids: 6″ Ribs @ 36″

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANELS																	SQUARE INTERIOR PANELS									
Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Edge Column $c_1 = c_2$ (in.)	Square Edge Column γ_f	Square Edge Column (2) Stirrups	Column Strip Top Edge No.-size +	Column Strip Bottom No. Ribs	Column Strip Bottom Bars per Rib	Column Strip Top Interior No.-size	Middle Strip Bottom No. Ribs	Middle Strip Bottom Long Bars	Middle Strip Bottom Short Bars	Middle Strip Top Interior No.-size	Moments −M Edge (ft-k)	Moments +M Bot. (ft-k)	Moments −M Int. (ft-k)	(1) Steel (psf)	Square Interior Column $c_1 = c_2$ (in.)	Square Interior Column (2) Stirrups	Column Strip Bottom No. Ribs	Column Strip Bottom Bars per Rib	Column Strip Top Interior No.-size	Middle Strip Bottom No. Ribs	Middle Strip Bottom Long Bars	Middle Strip Bottom Short Bars	Middle Strip Top Interior No.-size
Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.																	Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.									
18'- 0" D= 6.500 RIB ON COLUMN LINE 0.597 CF/SF	50	1.86	12	0.664		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	28	56	76	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	100	1.86	12	0.687		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	37	77	98	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	150	1.92	12	0.711		13-#5+ 0	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5	45	106	121	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	200	1.98	12	0.735		13-#5+ 0	3	2-#5	13-#5	3	#4	#4	5-#5	53	132	144	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	300	2.16	12	0.782		13-#5+ 0	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5	70	185	189	1.95	12	3 S 4 1	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5
	400	2.61	12	0.830		13-#5+ 0	3	1-#6 and 1-#7	13-#5	3	#5	#6	5-#5	87	236	235	2.28	12	3 S 4 1	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5
21'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.637 CF/SF	50	1.84	12	0.735		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	47	93	125	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	100	1.84	12	0.763		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	60	128	162	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	150	2.01	12	0.791		15-#5+ 0	4	2-#5	15-#5	3	#4	#5	6-#5	74	174	199	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	200	2.19	12	0.818		15-#5+ 0	4	1-#5 and 1-#6	15-#5	3	#5	#5	6-#5	87	218	236	1.93	12		4	1-#4 and 1-#5	15-#5	3	#4	#4	6-#5
	300	2.63	12 *	0.874		15-#5+ 0	4	1-#6 and 1-#7	15-#5	3	#6	#6	6-#5	115	302	309	2.22	12	3 S 4 1	4	2-#5	15-#5	3	#5	#5	6-#5
	400	3.13	12 *	0.634		15-#5+ 0	4	2-#7	18-#5	3	#7	#7	6-#5	142	385	383	2.69	12 *	3 S 4 1	4	2-#6	17-#5	3	#5	#6	6-#5
24'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.613 CF/SF	50	1.92	12	0.801		18-#5+ 0	4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5	69	138	186	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	100	1.98	12	0.831		18-#5+ 0	4	2-#5	18-#5	4	#4	#4	7-#5	90	190	242	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	150	2.32	12	0.862		18-#5+ 0	4	2-#6	18-#5	4	#5	#5	7-#5	111	260	298	2.05	12		4	2-#5	18-#5	4	#4	#4	7-#5
	200	2.57	12 *	0.892		18-#5+ 0	4	1-#6 and 1-#7	18-#5	4	#5	#6	7-#5	131	325	353	2.25	12	3 S 4 1	4	1-#5 and 1-#6	18-#5	4	#4	#5	7-#5
	300	3.32	12 *	0.634		18-#5+ 0	4	2-#8	16-#6	4	#6	#7	7-#5	172	453	464	2.78	12 *	3 S 4 1	4	1-#6 and 1-#7	21-#5	4	#5	#5	7-#5
	400	4.07	16 *	0.628		18-#5+ 1	4	2-#9	20-#6	4	#7	#7	9-#5	210	564	565	3.49	13 *	3 S 4 1	4	2-#7	18-#6	4	#6	#6	9-#5
27'- 0" D= 9.500 RIB NOT RIB NOT ON COLUMN LINE 0.597 CF/SF	50	1.96	13	0.824		20-#5+ 0	4	2-#5	20-#5	5	#4	#4	8-#5	98	196	264	1.86	13		4	2-#4	20-#5	5	#4	#4	8-#5
	100	2.22	13	0.860		20-#5+ 0	4	2-#6	20-#5	5	#4	#5	8-#5	127	263	343	2.02	13		4	2-#5	20-#5	5	#4	#4	8-#5
	150	2.49	13 *	0.896		20-#5+ 0	4	2-#7	20-#5	5	#5	#5	8-#5	157	363	422	2.13	13	3 S 4 1	4	1-#5 and 1-#6	20-#5	5	#4	#4	8-#5
	200	3.03	13 *	0.632		20-#5+ 0	4	2-#8	25-#5	5	#5	#6	8-#5	186	455	502	2.61	13 *	3 S 4 1	4	1-#6 and 1-#7	23-#5	5	#4	#5	8-#5
	300	3.99	18 *	0.626		20-#5+ 1	4	2-#9	23-#6	5	#6	#7	8-#6	240	617	647	3.55	14 *	3 S 4 2	4	1-#7 and 1-#8	21-#6	5	#5	#6	10-#5
30'- 0" D=12.500 RIB ON COLUMN LINE 0.624 CF/SF	50	2.12	15	0.829		22-#5+ 0	5	1-#5 and 1-#6	22-#5	5	#4	#5	9-#5	136	272	367	1.94	15		5	1-#4 and 1-#5	22-#5	5	#4	#4	9-#5
	100	2.31	15 *	0.887		22-#5+ 3	5	2-#6	23-#5	5	#5	#5	9-#5	177	353	475	2.03	15		5	2-#5	22-#5	5	#4	#4	9-#5
	150	2.95	15 *	0.933		22-#5+ 6	5	2-#7	28-#5	5	#6	#6	9-#5	217	483	584	2.63	15 *	3 S 4 1	5	2-#6	26-#5	5	#5	#5	9-#5
	200	3.53	18 *	0.626		22-#5+ 3	5	2-#8	24-#6	5	#6	#7	11-#5	255	604	686	3.12	15 *	3 S 4 1	5	1-#6 and 1-#7	22-#6	5	#5	#6	10-#5
33'- 0" D=12.500 RIB ON COLUMN LINE 0.609 CF/SF	50	2.25	16 *	0.867		25-#5+ 2	5	2-#6	25-#5	6	#4	#5	10-#5	180	360	485	2.06	16		5	2-#5	25-#5	6	#4	#4	10-#5
	100	2.71	16 *	0.922		25-#5+ 6	5	2-#7	31-#5	6	#5	#5	10-#5	234	467	629	2.46	16 *	3 S 4 1	5	2-#6	28-#5	6	#4	#5	10-#5
	150	3.42	19 *	0.931		25-#5+10	5	2-#8	27-#6	6	#6	#6	13-#5	285	614	768	3.07	16 *	3 S 4 1	5	1-#6 and 1-#7	25-#6	6	#5	#5	12-#5
	200	4.05	25 *	0.621	3 S 4 1	25-#5+ 3	5	2-#9	32-#6	6	#6	#7	15-#5	•331	711	892	3.63	18 *	3 S 4 2	5	2-#7	30-#6	6	#5	#6	10-#6
36'- 0" D=12.500 RIB ON COLUMN LINE 0.597 CF/SF	50	2.52	18 *	0.875		27-#5+ 3	5	2-#7	31-#5	7	#5	#5	11-#5	231	463	623	2.16	18	3 S 4 1	5	1-#5 and 1-#6	28-#5	7	#4	#4	11-#5
	100	3.15	19 *	0.926		27-#5+ 9	5	2-#8	40-#5	7	#5	#6	13-#5	300	601	808	2.79	18 *	3 S 4 1	5	1-#6 and 1-#7	37-#5	7	#4	#5	12-#5
	150	3.93	27 *	0.620	3 S 4 1	27-#5+ 5	5	2-#9	35-#6	7	#6	#6	11-#6	362	725	976	3.62	18 *	3 S 4 2	5	1-#7 and 1-#8	33-#6	7	#5	#5	15-#5

See the notes on page 11-19.

CONCRETE REINFORCING STEEL INSTITUTE

Two Way Concrete Slab with Beams Calculation

The design of this floor system was done using pcaSlab. Additional computer analysis output is available upon request.

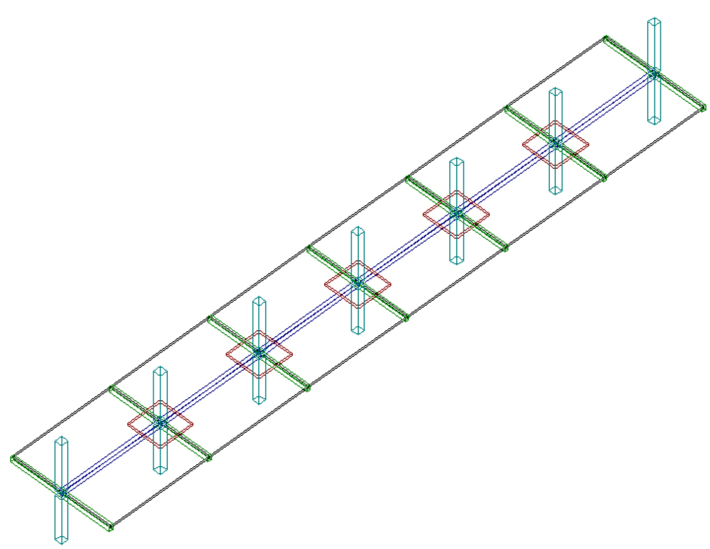
Design Criteria
f'c = 5,000 psi
Normal Weight Concrete (150pcf)
fy = 60,000 psi
Typical 30'-0"x30'x0" Bay
Super Imposed DL = 32 psf
LL = 80 psf

Computing Min. Floor thickness based on 30'x30' bay with 24"x24" Columns.

h= 28(12)(.8+60,000/200,000)/(36+9(1)) = 8.213 use 8.25"

To obtain this slab thickness, $\alpha_m = 2$ is required.
I_{slab} = 30(12)8.25^3/12 = 16,845 in^4
Maintain aspect ratio of b=2/3h, $\alpha_m = 2 = EI_{slab}/EI_{beam,}$ Assume Exterior Bay for Conservative Thickness
I_{beam} = (1.5)(2)(2/3h*h*h^3/12) = (16,845) , solving for h = 17.8 use 18" and b = 12"

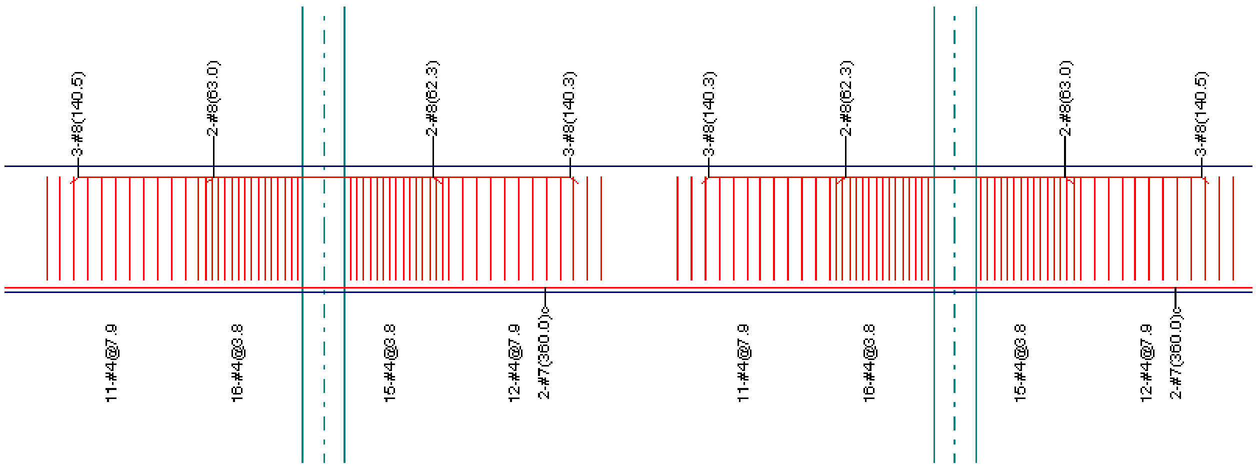
Using pcaSlab, the following is the design output.



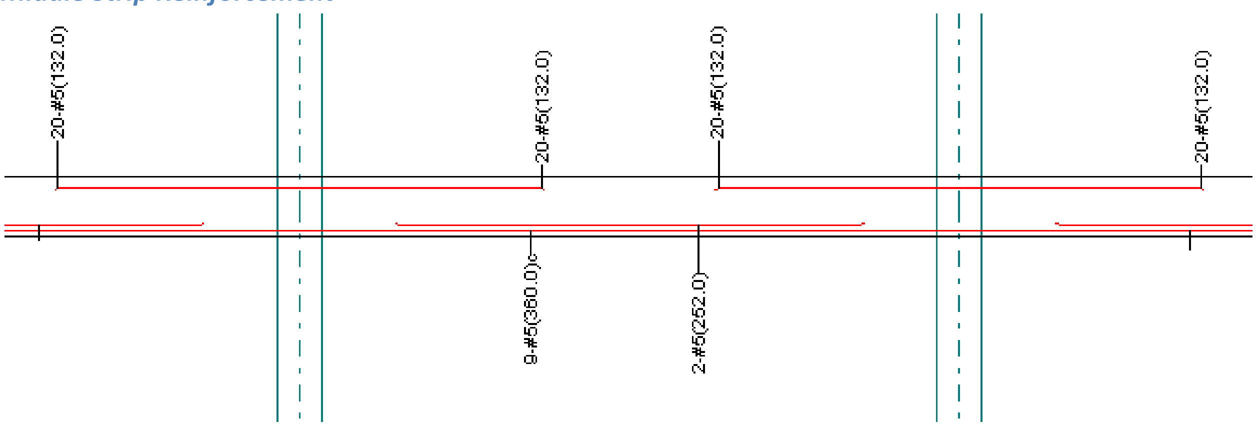
The following structure was modeling into pcaSlab, although the interior bay was only analyzed in this report.

Reinforcement Diagram

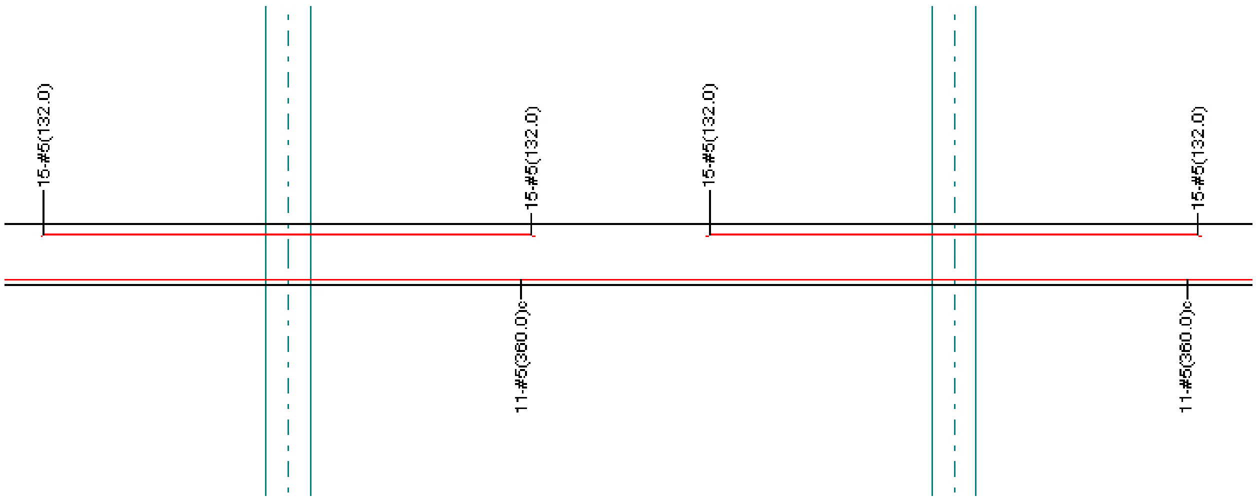
Beam Reinforcement



Middle Strip Reinforcement



Column Strip Reinforcement



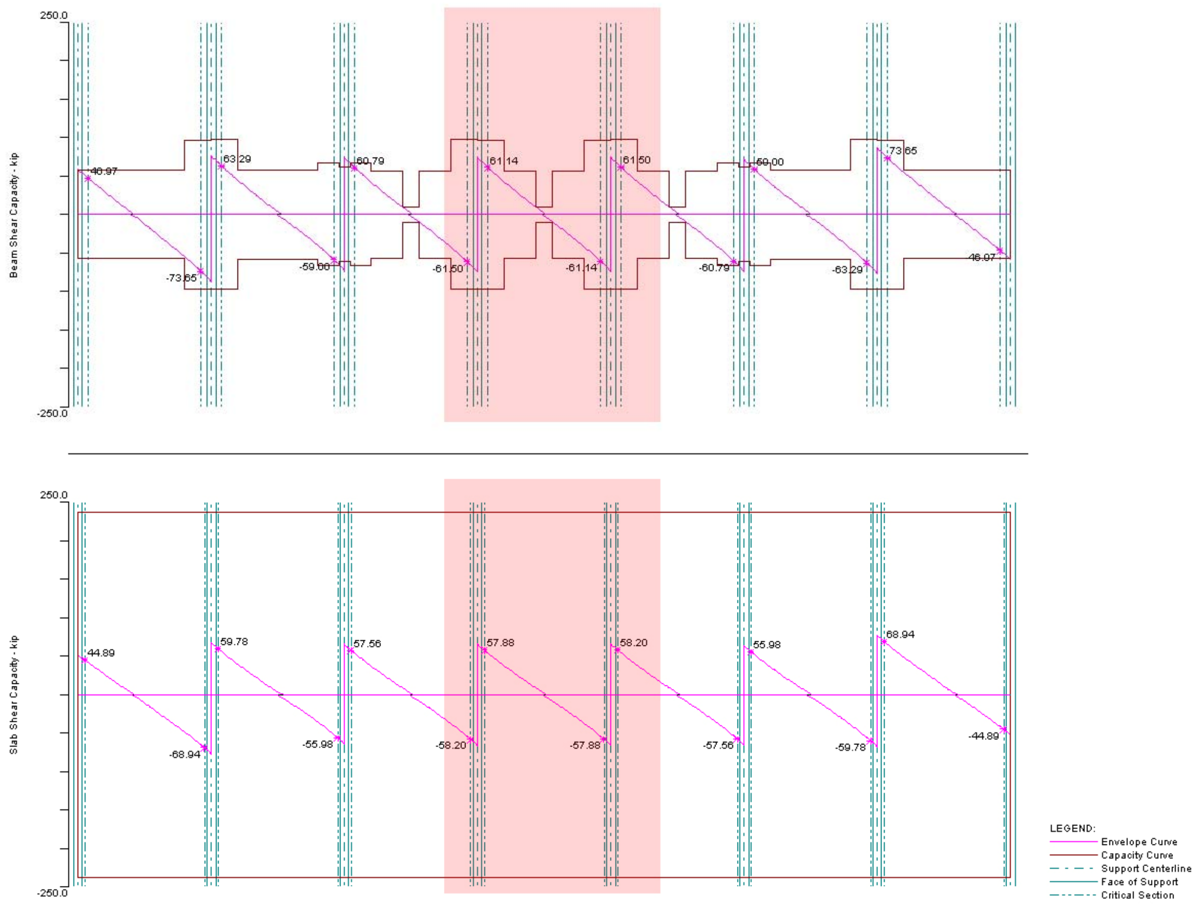
Deflections (in)

All deflections < L/360 = 1" therefore **OK**

Frame			Column Strip			Middle Strip		
Dead	Live	Total	Dead	Live	Total	Dead	Live	Total
0.104	0.194	0.298	0.110	0.205	0.316	0.095	0.176	0.271

Shear Capacity

Shear Capacity is sufficient at d/2 from column face.



Moment Capacity

The moment capacity is sufficient for the loading therefore the reinforcement is effective.