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STRUCTURAL OPTION

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AE 481 W

TECHNICAL REPORT TWO

PRO – CON STUDY OF ALTERNATE FLOOR SYSTEMS

EXECUTIVE SUMMARY

This technical report concentrates on the existing floor system of Memorial Sloan-Kettering along with four efficient alternative systems. A detailed analysis of each system is provided, discussing the advantages and disadvantages associated with that particular design. Each alternative is then compared against the original floor design in order to determine how effective of an option it is. All four floor systems chosen for this report appear to be suitable alternatives for MSK. Therefore, these results will help provide a good basis of which systems would be the most beneficial to further investigate.

This report begins by examining the existing composite system found on the second, third, and fourth floors of Memorial Sloan-Kettering. A typical 30’ x 30’ interior bay was analyzed with hand calculations to check the framing members. After confirming those member sizes, this system was slightly modified into a non-composite system and analyzed for a second time. Member sizes were once again designed for and compared to the original.

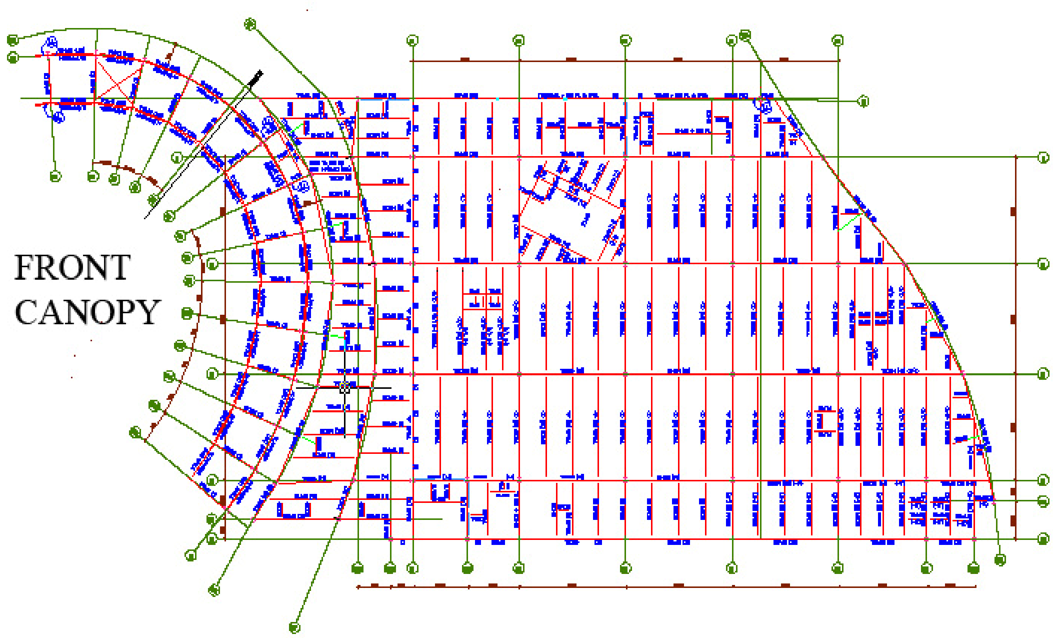
The other three systems investigated for this report were a one-way concrete joist system, a hollow-core precast plank system, and a two-way slab system with drop panels. All three of these designs are considerably different then the original since they deal predominately with concrete instead of structural steel. Because of this, the CRSI 2002 and PCI 2000 handbooks were both referenced to aide in the structural design of these systems. All tables referenced for this report can be found in the appendix. Each system was created for the same interior bay as the original with the same superimposed loads acting on it. For each design, the type of floor system is described and then analyzed to determine the correct concrete member sizes, reinforcement size and placement, and slab properties. In addition, advantages and disadvantages are discussed for that particular system along with how those characteristics would specifically influence Memorial Sloan-Kettering.

After all four alternative floor systems were examined, a comparison chart was created to contrast the cost, weight, floor depth, and construction speed of each system against the others and the original. From this chart, it became apparent which systems would in fact work in MSK and which were simply ineffective. This report acknowledges the original composite design’s efficiency as well as recommends further investigation of both the hollow-core precast plank and one-way joist system as possible floor system alternatives.

EXISTING FLOOR SYSTEM

Memorial Sloan-Kettering Cancer Center is comprised of four stories above grade. The 1st floor is made up of a one-way concrete slab system while the 2nd through 4th floors consist of composite concrete slab on metal decking. For this technical assignment, the latter will be used as the typical floor system in MSK.

Each overall floor area is approximately 20,000 square feet. The second, third, and fourth floors all share similar beam, girder, and column sizes due to fact that bay sizes remain relatively constant throughout the building. The dead and live loads applied to these floors are identical as well. Because of these similarities found in the framing plans of Memorial Sloan-Kettering, it is possible that the same floor system could be used for the entire building.



Typical Floor Framing

The current design of a typical floor system in Memorial Sloan-Kettering is composite concrete slab on metal decking. This system consists of a 4 ½" normal-weight concrete slab poured onto 2" 20-gauge galvanized metal decking. The slab is reinforced with 6x6-W2.9 x W2.9 welded wire fabric. The metal floor deck spans in the E – W direction and is continuous over a minimum of two or more spans. This decking connects into the wide flange steel beams through equally spaced ¾" diameter by 4" long headed shear studs welded into the center of the flange.

A typical floor framing bay, shown between column lines 18-19 and J-K, has been selected for this floor system analysis and will be used for the remainder of the report. An enlarged image of this bay can be found on the following page. This 900 square foot bay is framed in the N – S direction by wide flange steel beams which span 30'. These W6x26 beams are spaced 7.5' feet on center and tie into W24x94 girders which span E – W from column to column.

LOADS AND CALCULATIONS

The current loading found on the floors of Memorial Sloan Kettering are listed below. All calculations used to find these loads are referenced in Appendix B.

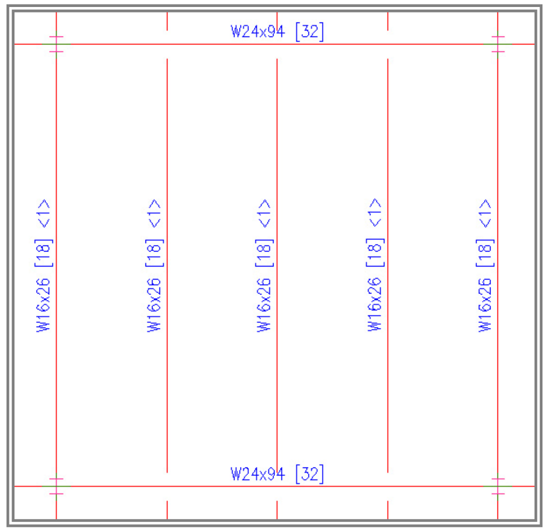
Dead Loads:

Typical Floor:

56 psf	*concrete slab*
2 psf	*metal deck*
15 psf	*steel framing*
15 psf	*superimposed dead load*
88 psf	

Live Loads:

100 psf   Table 4-1 (*ASCE 7-02*)



Typical Bay Framing Plan

This live load value was taken from Table 4-1 found in ASCE 7 – 02. Furthermore, the same live load value was used in the design of MSK. In order to compare alternative floor systems effectively, the same live load will be used in this report.

The typical bay being tested in this analysis follows the following load criteria. The dead load does not include the self-weight of the beam.

LOAD ON STEEL BEAMS							
Span	Trib. Width	Trib. Area	Live Load	Dead Load	PSF	KLF	M_{MAX}
(feet)	(feet)	(sq. feet)	(psf)	(psf)	1.2(DL) + 1.6(LL)		k - ft
30	7.5	225	100	74	248.8	0.2488	209.925

MODIFIED FLOOR SYSTEM

Non-Composite System

The first alternative floor system analyzed in this report is a concrete slab and metal decking on non-composite wide flange steel beams. This system is simply a modification of the existing floor system because the only alternation would be the lack of shear studs welded between the members and the slab.

There are some advantages to selecting a non-composite system over a composite one. The time of construction would be slightly reduced due to the fact that shear studs would not have to be welded in place prior to the pouring of the concrete slab. This aspect could also lower the installation cost because field welders would not have to be hired to complete this task.

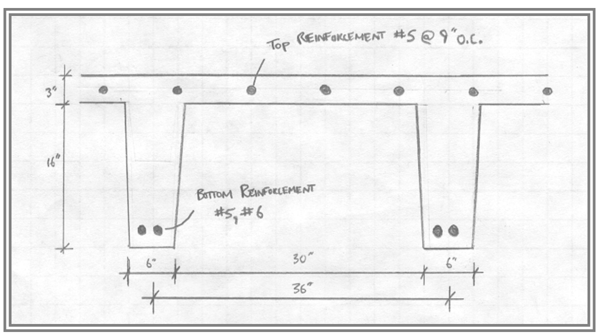
Despite these advantages, a number of disadvantages surface with the decision to use a non-composite system. With this design, the concrete and steel are not working together, causing the steel member to take the entire moment. Because of this, the sizes of the steel beams supporting these floors tend to heavier and larger then those required for composite floors. When calculating the required size of a non-composite beam for this bay, the most economical choice was a W18 x 35. Compared with the existing beam size of W16 x 26, these new structural members are 2” deeper, taking away space from MEP equipment or increasing the floor to floor heights.

ALTERNATIVE FLOOR SYSTEMS

The remaining three systems analyzed in this report change Memorial Sloan-Kettering’s floor system from steel framing to concrete. Both the CRSI 2002 and PCI 2000 design manuals were referenced to aide in the structural design of these systems. Although MSK is currently framed in structural steel, these systems are being looked at to determine whether they would be an effective alternative to the composite slab on steel members. The three concrete systems being investigated are: a one-way joist system; hollow-core precast planks; a two-way slab system with drop panels.

One-Way Concrete Joist System

A one-way concrete joist system was looked at as a possible alternative to Memorial Sloan-Kettering because of its similarities to the current floor system. The joists are arranged in one-direction in between larger, parallel supports much like steel beams in between girders. This system also benefits longer spans like the one chosen for this report. The deep concrete joists allow for adequate stiffness and efficient reinforcement placement while keeping the slab at a minimal thickness, thus reducing potential dead load. Another positive attribute related to the construction of this system is that the pan forms can be re-used multiple times to reduce cost.



The concrete joists were designed to span 30’ in the N – S direction, taking on the role of the current steel beams. After going into the CRSI tables, an adequate design was found on page 8-24. This page can be referred to in Appendix D. This design calls for 16” deep joist ribs supporting 3” of top slab. These ribs are 6” thick and are spaced 36” on center. A cross-section of this system is displayed to the left. Each joist is reinforced with two bottom bars (one #5, one #6) and have top reinforcement of #5 bars spaced 9” on center.

Joist-band beams were also selected to take the place of steel girders for this system. These beams would span from column to column in the E – W direction and transfer the loads taken from the concrete joists into the columns. An effective design calls for a 24.5” deep by 24” wide concrete beam reinforced with two #14 bottom bars and five #14 top bars. Again, this table is referenced in Appendix D.

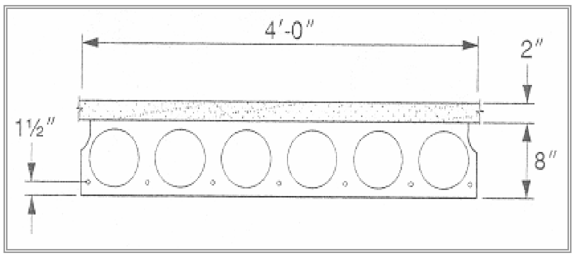
One variable that was considered for this design was keeping the depth of the joists close to the current depth of the beams. A W16 x 26 beam supporting 4.5” of slab has a total depth of approximately 20.5”. The design chosen for this bay has a total depth 19”, reducing the member depth by over an inch and providing more room for the MEP system.   Another advantage to this system is that it meets the required two-hour fire rating without the need to fire-proof.

This one-way joist system also has certain drawbacks not seen in the current system. One of the largest disadvantages of this system is the amount of time required for construction of each floor. Formwork and steel reinforcement must be correctly set before the concrete can been poured. Once the concrete is set in place, construction workers must wait until the concrete cures before removing the pans and beginning the next floor. On top of this, shoring must be set in place to support the formwork until the concrete is able to support its own weight.

Hollow-Core Precast Planks

A hollow-core precast plank system was chosen as the next alternative concrete floor system analyzed for MSK. These precast, pre-stressed planks are created in concrete plants, which allows for higher quality products and quicker assembly once brought on site. Because steel strands are pre-stressed within these planks, load capacity and span ranges are larger then normal reinforced concrete. Deflection can also be controlled by altering the camber of the plank. These hollow-core strips can also rest on steel girders or inverted tee beams depending on the infrastructure of the building.

When designing for a typical bay chosen, the hollow-core precast planks would span 30' in the N-S direction from support to support. Tables from the PCI design handbook were used to assist in this system's design. The sufficient size chosen was a 4'-0" wide by 8" thick precast hollow-core plank with a 2"normal weight concrete topping. This plank is reinforced with six #8 strands which have a straight tendon profile throughout the entire strip. A cross-section of this system is displayed below.



This hollow-core system has some advantages not offered by other concrete systems. The most noticeable would be its rapid construction period. As mentioned earlier in the report, these planks are brought on site fully cured. Only the 2" concrete topping needs to be applied once in place, and because the planks are at full strength, no shoring is required for added support. Another advantage to this system would be its thickness. Even with the 2" concrete topping, these hollow-core planks are only 10" thick allowing for more MEP space, higher ceilings, or decreased floor to floor heights. As with the other concrete systems, no additional fire proofing is required to meet the necessary two-hour fire rating.

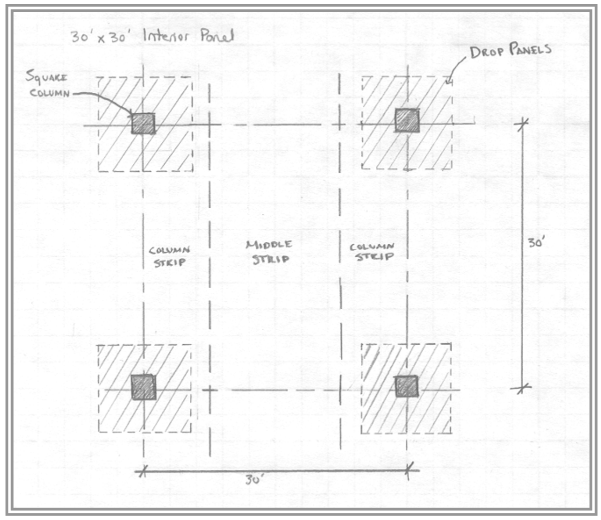
There are some disadvantages to this system as well. First off, hollow-core planks are designed for regular-shaped frames. However, the east and west sides of MSK maintain a curved façade, creating irregularly shaped exterior bays. Another negative aspect to this system would be the need to have cranes on site in order to put the precast strips into place. Furthermore, specialized workmanship is often required to ensure correct placement.

Two-Way Slab System with Drop Panels

A two-way slab system with drop panels was the last alternative analyzed for Memorial Sloan-Kettering's floor design. This system is typically used for square bays with longer spans, much like the one chosen for this report. By adding a drop panel around each column, punching shear is avoided and more moment can be taken at the supports. This in turn reduces the overall slab thickness and steel reinforcement in the bay. Smaller columns can also be used compared to a two-way system without drop panels.

Once again, the CRSI tables were referenced for this system and an appropriate design was found on page 10-29. This design calls for an 11.5" thick slab with 10' x 10' drop panels, 9" deep around each support. Refer to the image below. Reinforcement for the slab is broken down between the column strip and moment strip. The column strips requires (17) #6 bars as top reinforcement and (14) #6 bars as bottom reinforcement. The middle strip requires (15) #5 bars as top reinforcement and (13) #5 bars as bottom reinforcement. All reinforcement must also be distributed equally throughout each assigned strip. A minimum column size of 19" x 19" must be provided for this two-way system as well.

There are a number of structural advantages by using a two-way system with drop panels. Like the hollow-core planks, this system offers minimum thickness for a bay this large. Compared to the existing composite system that has a depth of 20.5", this two-way slab is only 11.5" for most of the bay. The drop panels descend an additional 9" around each column, but even at those points the slab becomes 20.5". Another benefit of the two-way system would be its lack of structural members. Because there are no beams or girders, MEP equipment can run in either direction without anything hindering its path. This could allow for a more effective MEP layout throughout the building.



Like all floor alternatives, this two-way slab system comes with a few disadvantages. The first would be its increased construction period. Like the one-way joists, this system must have formwork and steel reinforcement placed before the concrete can be poured. Although the two-way slab is flat, additional formwork must be produced for each drop panel. Shoring must also be provided to support the slab until it is fully cured. Another negative aspect for this system would be the added dead weight applied to the structure. An 11.5" slab has a self-weight of 144 psf, and that does not even take into consideration the weight of the drop panels. This additional weight will affect both the infrastructure's column sizes and foundation.

COMPARISON CHART

Beam Comparison Chart								
	System	Description	Depth	Weight	Construction Speed	Approximate Cost		
Steel			(inches)	(psf)	(1-4) 1=quickest	Mat.	Inst.	*Total*
	Original	Composite Slab/Beam	20.5	74	3	$8.80	$4.61	$13.41
	Modified	Non-Composite Slab/Beam	22.5	76	2	$11.45	$6.20	$17.65
Concrete								
	Alternative #1	One-Way Joist System	19	78	4	$7.10	$9.45	$16.55
	Alternative #2	Hollow-Core Precast Planks	10	68	1	$14.35	$4.93	$19.28
	Alternative #3	Two-Way Slab w/ Drop Panels	11.5	144	4	$7.00	$8.25	$15.25

CONCLUSION

After analyzing the four alternative floor systems and comparing each to the existing design, certain advantages and disadvantages become evident with each option. The original composite floor system offers both economical and structural advantages to MSK, making it apparent why this design was initially chosen. The modified version of this system saves a small amount of construction time but is also more expensive. Because no clear advantages exist with this non-composite design, there is no reason to consider it as a possible alternative. Despite a longer construction period, the one-way concrete joist system offers a smaller overall floor depth and seems to be an effective alternative at this time. The hollow-core precast plank system also reduces floor depth as well as increases construction speed. As a result, this system would also be an efficient choice and should be further investigated. The final option of a two-way slab with drop panels adds to much additional dead weight to the structure and therefore is not worth considering in further designs of Memorial Sloan-Kettering. At this time, both the one-way joist and hollow-core plank systems appear to be the two best alternatives.

Appendix A: References

CRSI Design Handbook 2002

PCI Design Handbook – 5th edition

Manual of Steel Construction – 3rd edition

RS Means 2005

APPENDIX B: LOAD CALCULATIONS

LOAD CALCULATIONS

DEAD LOADS

1st Floor

$(6''/12'')(150\ PCF) = 75\ PSF$ SLAB

$= 15\ PSF$ SUPER IMPOSED

$(18''/12)(24''/12)(150)(1/10') = 45\ PSF \rightarrow$ BEAM $\rightarrow$ (450 PLF @ 10' O.C)

$(24/12)(30/12)(150)(1/30') = 50\ PSF \rightarrow$ GIRDER $\rightarrow$ (750 PLF @ 30' O.C; FOR BOTH DIRECTIONS)

185 PSF

2ND FLOOR – 4TH FLOOR

$(4.5''/12'')(150\ PCF) = 56.25\ PSF$ SLAB

$= 2\ PSF$ METAL DECK

$= 15\ PSF$ STEEL FRAMING

$= 15\ PSF$ SUPERIMPOSED

88.25 PSF

ROOF

$(3.5/12)(150) = 43.8\ PSF$ CONCRETE

2 PSF METAL DECK

15 PSF STEEL FRAMING

20 PSF MECHANICAL

80.75 PSF

LIVE LOADS

ALL FLOORS $\rightarrow$ 100 PSF $\rightarrow$ TABLE 4-1 ASCE 7-02

SNOW LOADS

GROUND SNOW LOAD $P_g = 30\ PSF \rightarrow$ TABLE

FLAT ROOF SNOW LOADS

$P_f = 0.7 C_e C_t I p_g \quad \therefore \quad C_E = 0.9$ TABLE 7-2

$C_T = 1.0$ TABLE 7-3

$I = 1.2$ TABLE 7-4

$P_f = (.7)(0.9)(1.0)(1.2)(30)$

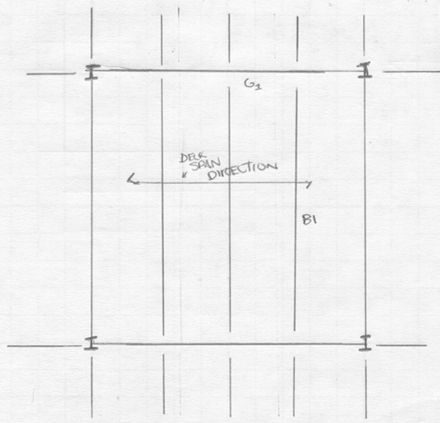
$P_f = 22.68\ psf$

APPENDIX C: EXISTING FLOOR SYSTEM

Composite Slab on Deck Check:

1

COMPOSITE SLAB ON METAL DECK



B1
BEAM → W16x26
→ 30' SPAN
7.5' TRIBUTARY AREA
<15> 3/4" ϕ DIAMETER SHEAR STUDS

G_1 → W24 x 94
→ 30' SPAN
30' TRIBUTARY AREA
<32> 3/4" ϕ SHEAR STUDS

SLAB
4½" NORMAL WEIGHT GALVANIZED CONCRETE ON 2" METAL DECK

CHECK GRAVITY LOADS

DL
CONCRETE = (.150 PCF)(4.5/12) = 56.25 psf
METAL DECK = 2 psf
SUPERIMPOSED 15 psf
73.25

LL = 100 psf (ENTIRE BUILDING)

* BEAM WT ALREADY CALC'D INTO TABLE

$w_u = (1.2)(73.25\ psf) + 1.6(100) = 247.9\ psf$

b_{eff}

$(30 \cdot 12)/4 = 90"$

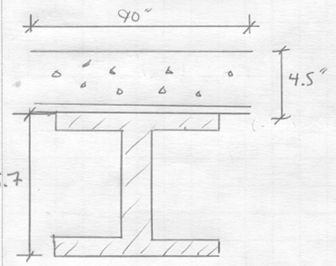
or $b_{eff} = 90"$

$(7.5)(4) = 90"$

$W_u = (247.9)(7.5') = 1.86\ k/ft$

$M_u = \frac{(1.86)(30')^2}{8} = 209.3\ 'k$

D = 15.7



D = 15.7

→ Assume W16x26

$C = (.85)(4.5)(90)(4) = 1377\ ksi$

$T_s = (7.68)(50) = 384\ ksi$

STEEL CONTROLS

$a = \frac{384}{(.85)(4.5")(90)} = 1.115"$ → PNA

$y_1 = 1.115$

$y_2 = 4.5 - 1.115/2 = 3.94$

→ LOOK @ TABLE 5-14 ON pg 5-143

@ $y_2 = 3.5$ (conservative)

y_1 = @ #6

$M_n = 243\ ft \cdot k > 209.25\ ft \cdot k$

WORKS

∴ CHECK SHEAR

Composite Slab on Deck (continued)

2

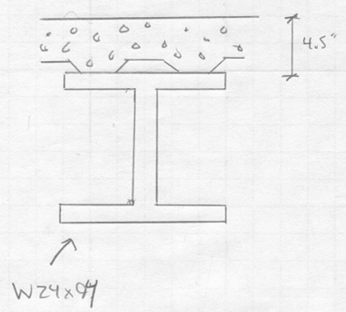
CHECK SHEAR

ASSUME (1) SHEAR STUD $Q_n = 26.1$ kips → TABLE 5-13

TABLE 5-14 $\sum Q_n = 145 / 26.1$ kips $= 5.55 \therefore 6 \cdot 2 = 12$ STUDS REQUIRED FOR FULL MOMENT DEVELOPMENT ✓

∴ a W16x26 works in this design

→ GIRDER → use same bay as shown before



$d = 24.3$

DL = 73.25 psf (slab and deck)

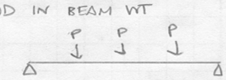
LL = 100 psf

$w_u = (1.2)(73.25) + (1.6)(100) = 247.9$ psf

$W_u = (247.9$ psf$)(30') = 7.44$ klf

$M_u = \frac{(7.44)(30')^2}{8} = 837$ 'K

ADD IN BEAM WT



$P = (26$ plf$)(30') = .78$ K

TABLE 5-16 in LRFD

Max Moment $= (.5)(.78^K)(30') = 11.7$ k'

$M = 837 + 11.7 = 848.7$ 'K

→ FIND M_n

$b_{eff} = (30 \cdot 12)/4 = 90"$

$C_c = (.85)(4.5)(90)(4) = 1377$ ksi

$T_s = (27.7)(50) = 1385$ ksi

} P.N.A. is in steel

$a = \frac{1385}{(.85)(4)(90)} = 4.52$

T = C

$T_s = C_c + C_s$

$C_s = 8$

* PNA IS in FLANGE

$8/2 = (9.07)(50)(x)$

$x = .001$ ∴ in Flange

Composite Slab on Deck (continued)

3

→ LOOK @ TABLE 5-14 ON PG 5-140

$M_n = 1687$ 'k > 788 'k works

Check Shear

$Q_n = 26.1$ k

32 studs / 2 = 16

$\Sigma Q_n = 16(26.1) = 417.6^k > 394^k$ ∴ full moment developed

APPENDIX D: ALTERNATIVE FLOOR SYSTEMS

Non-Composite Slab on Deck

Modified System | Non-Composite Slab on Metal Deck | TECH 2

→ Non-Composite SLAB ON METAL DECK

→ Same framing plan as Composite SLAB ON METAL DECK

→ maintain 4.5" slab on metal decking

LOADS:

DEAD → $(150\ pcf)(4.5/12) = 56.25\ psf$ concrete slab

$2\ psf$ metal decking

$15\ psf$ Superimposed

$73.25\ psf$

LIVE → 100 psf

Beams

tributary width = 7.5'

$w_u = (1.2)(73.25\ psf) + 1.6(100\ psf) = 247.9\ psf$

$W_u = (247.9\ psf)(7.5') = 1.86\ k/ft$

$M_u = \frac{(1.86)(30')^2}{8} = 209.25\ 'k$

BEAM Moment = 209.25'k

→ Find adequate member size

→ Table 5-3 (W-Shape Selection by Z_x)

→ find Z_x

$Z_{x\,reqd} = \frac{M_u}{\phi_b \cdot F_y} = \frac{(209.25)(12")}{(.9)(50)} = 55.8\ in^3$

→ Current member

W14x26 $Z_x = 39.9\ in^3$

∴ no good

→ Most economical member to work

W18 x 35 ∴ $Z_x = 66.5\ in^3 > 55.8\ in^3$

$\phi M_n = 249\ k\text{-}ft > 209.25\ k\text{-}ft$

→ USE W18 x 35 as non-composite member for typical bay

One-Way Concrete Joist

TECH 2 | ONE-WAY JOIST ANALYSIS

→ Using CRSI

→ Typical Bay 30' × 30'

∴ Clear span = 30'

→ Factors considered

- multiple spans
- concrete joists will span in same direction as beams span in existing design (shown below)

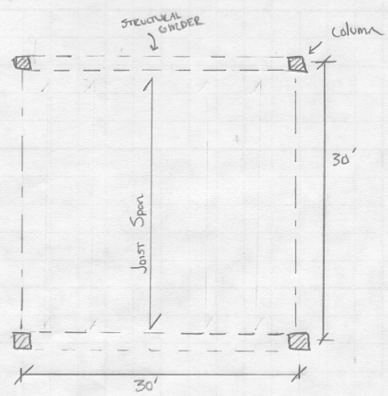
- current depth of existing system → W16×26 w/ 4½" slab on metal decking

→ Want depth of joist system to be ≤ current depth to allow room for MEP system, etc.

W16×26 → d = 15.7" (TABLE 1-1 from AISC)

SLAB = 4.5"

20.2" → current depth



Interior typical bay

FACTORED LOADS: → use factored loads of 1.4(DL) and 1.7(LL)

LL = 100 psf

DL = 15 psf (superimposed)

w_u = 1.4 (15 psf) 1.7 (100 psf) = 191 psf

→ Refer to CRSI 2002 (pg 8-24)

→ INTERIOR Span

→ Try 16" Deep Ribs w/ 3" top Slab

∴ Total Depth: 19"

→ Factored Usable Superimposed Load (PSF) = 239 PSF

∴ 239 PSF > 191 psf ∴ ✓

Joist Information

30" forms and 6" Ribs @ 36" o.c.

f'_c = 4000 psi

Required Reinforcement: Bottom Bars: (1) #5, (1) #6 A_s = 1.42

Top Bars: #5 bars spaced 9" o.c.

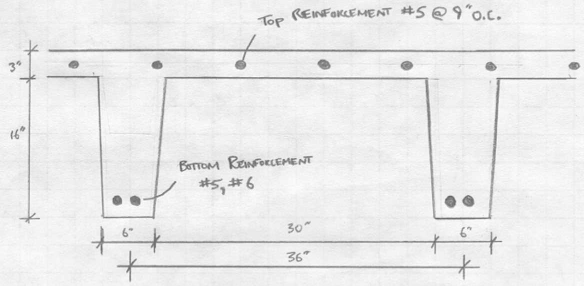
f_y = 60000 psi

Check Deflection

$\ell_n/21$ (interior spans)

$\frac{(30)(12)}{21}$ = 17.14" < 19" (Depth)

∴ ✓



Joist-Band Beam Design

TECH 2 | ONE-WAY JOIST ANALYSIS
JOIST BAND BEAM DESIGN

→ DESIGN FOR INTERIOR Span JOIST-BAND BEAM

→ w_u on joists ∴ 191 psf (FACTORED)

→ find DISTRIBUTED WEIGHT for joist system
→ Refer to TABLE 8-1 in CRSI 2002

→ 3" Top SLAB
→ 16" RIB DEPTH
→ 6" RIB WIDTH
→ 30" PAN WIDTH

→ Joist System Wt → 78 psf (Unfactored)
* SEE TABLE IN APPENDIX

(1.4)(78 psf) = 109.2 psf

Total w_u = 191 psf + 109.2 psf = 300.2 psf

(300.2 psf)(30') = 9.01 k/ft

→ Go INTO CRSI TABLES
- JOIST-BAND BEAMS, INTERIOR SPANS
* FACTORED DEAD WT OF BEAM NOT DEDUCTED FROM TABLES

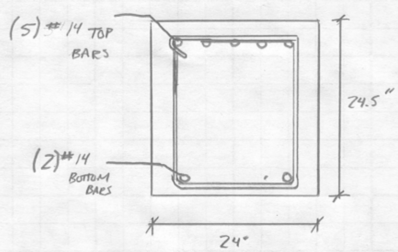
- 30 ft span

→ Try 24.5" deep by 24" wide Beam pg 12-105
LOAD → 10.4 k/ft ∴ OK

→ ADD in WT OF BEAM

= (150 lb/ft³)(24.5/12)(24/12) = (.6125 k/ft)(1.4) = .860 k/ft

Total load = 9.01 k/ft + .86 k/ft = 9.87 k/ft < 10.4 k/ft ∴ OK



→ Required Reinforcement
2 #14 BOTTOM BARS
5 #14 TOP BARS

One-Way Concrete Joist Design Table

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS

30" Forms + 6" Rib @ 36" c.-c. (2)

FACTORED USABLE SUPERIMPOSED LOAD (PSF)

f'_c = 4,000 psi, f_y = 60,000 psi

16" Deep Rib + 3.0" Top Slab = 19.0" Total Depth

TOP BARS	Size @	# 4 12	# 4 10	# 4 8	# 5 10.5	# 5 8.5	End Span Defl. Coeff. (3)	# 4 11	# 4 9	# 5 11	# 5 9	# 6 11	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7		# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	
Steel (psf)		.69	.85	1.04	1.24	1.46	(3)	.75	.93	1.18	1.42	1.70	(3)
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		127	177	236	295	330*	4.703	161	234	308	375*	379*	2.894
		0	0	0	0	365		0	0	0	393	478*	
26'-0"		109	156	210	264	308*	5.502	141	208	276	353*	356*	3.386
		0	0	0	0	330		0	0	0	355	434	
27'-0"		93	136	186	237	289*	6.398	123	185	248	321	335*	3.938
		0	0	0	0	298		0	0	0	0	395	
28'-0"		79	119	166	213	269	7.400	106	164	223	291	316*	4.554
		0	0	0	0	0		0	0	0	0	359	
29'-0"		66	104	147	191	243	8.516	92	146	201	264	298*	5.240
		0	0	0	0	0		0	0	0	0	327	
30'-0"		55	90	130	171	220	9.752	79	129	180	239	282*	6.001
		0	0	0	0	0		0	0	0	0	299	
31'-0"		44	77	115	153	199	11.119	67	114	162	217	267*	6.842
		0	0	0	0	0		0	0	0	0	273	
32'-0"			65	101	137	180	12.625	56	100	145	197	249	7.769
			0	0	0	0		0	0	0	0	0	
33'-0"			55	88	122	163	14.278	46	88	130	179	228	8.787
			0	0	0	0		0	0	0	0	0	
34'-0"			45	77	109	147	16.089		76	116	162	208	9.901
			0	0	0	0			0	0	0	0	
35'-0"				66	97	133	18.067		66	103	147	190	11.118
				0	0	0			0	0	0	0	
36'-0"				57	85	119	20.222		56	92	133	174	12.445
				0	0	0			0	0	0	0	
37'-0"				48	75	107	22.565		47	81	120	159	13.886
				0	0	0			0	0	0	0	
38'-0"				40	65	96	25.105			71	108	145	15.449
				0	0	0				0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .52 CF/SF) (4)

NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.60	.72	.90	1.06	1.31		.65	.80	1.01	1.24	1.44
STEEL % (UNIFORM)	.45	.54	.68	.80	.99		.49	.60	.77	.94	1.09
(TAPERED)	.27	.33	.41	.48	.60		.30	.36	.46	.56	.66
EFF. DEPTH, IN.	17.8	17.8	17.8	17.7	17.7		17.8	17.8	17.7	17.7	17.6
− ICR/IGR	.144	.167	.199	.225	.263		.155	.181	.217	.252	.279
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.51	.62	.75	.88	1.04		.40	.51	.62	.75	.88
STEEL %	.08	.10	.12	.14	.16		.06	.08	.10	.12	.14
EFF. DEPTH, IN.	17.7	17.7	17.6	17.6	17.6		17.8	17.7	17.7	17.6	17.6
+ICR/IGR	.153	.184	.217	.251	.290		.123	.153	.184	.217	.251

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS

30" Forms + 7" Rib @ 37" c.-c. (2)

FACTORED USABLE SUPERIMPOSED LOAD (PSF)

f'_c = 4,000 psi, f_y = 60,000 psi

16" Deep Rib + 3.0" Top Slab = 19.0" Total Depth

TOP BARS	Size @	# 4 10	# 4 8.5	# 5 11	# 5 9	# 5 8	End Span Defl. Coeff. (3)	# 4 9	# 5 11	# 5 9.5	# 5 8	# 6 9.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7		# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	
Steel (psf)		.83	1.00	1.20	1.41	1.63	(3)	.92	1.16	1.36	1.63	1.93	(3)
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		163	219	277	346	366*	4.834	218	290	372	419*	425*	2.975
		0	0	0	0	416		0	0	0	456	536*	
26'-0"		142	194	247	311	342*	5.655	193	259	335	394*	399*	3.480
		0	0	0	0	376		0	0	0	413	498*	
27'-0"		123	171	221	280	321*	6.576	170	232	302	371*	376*	4.047
		0	0	0	0	340		0	0	0	374	460	
28'-0"		106	151	197	252	301*	7.606	150	207	273	340	354*	4.681
		0	0	0	0	308		0	0	0	0	420	
29'-0"		91	133	176	227	279	8.752	132	185	247	309	335*	5.386
		0	0	0	0	0		0	0	0	0	383	
30'-0"		77	117	157	205	253	10.023	116	165	223	281	317*	6.168
		0	0	0	0	0		0	0	0	0	351	
31'-0"		65	102	139	184	230	11.428	101	148	201	256	300*	7.033
		0	0	0	0	0		0	0	0	0	321	
32'-0"		54	88	124	166	208	12.975	88	131	182	233	285*	7.985
		0	0	0	0	0		0	0	0	0	294	
33'-0"		44	76	109	149	189	14.675	75	116	164	212	269	9.031
		0	0	0	0	0		0	0	0	0	0	
34'-0"			65	96	133	171	16.536	64	103	148	193	247	10.176
			0	0	0	0		0	0	0	0	0	
35'-0"			55	84	119	155	18.569	54	91	133	175	227	11.427
			0	0	0	0		0	0	0	0	0	
36'-0"			45	73	106	140	20.784	45	79	119	159	208	12.790
			0	0	0	0		0	0	0	0	0	
37'-0"				63	94	126	23.191		69	106	145	190	14.272
				0	0	0			0	0	0	0	
38'-0"				54	83	114	25.802		59	95	131	174	15.878
				0	0	0			0	0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .55 CF/SF) (4)

NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.74	.87	1.04	1.27	1.43		.82	1.04	1.21	1.43	1.71
STEEL % (UNIFORM)	.49	.58	.70	.85	.96		.55	.70	.81	.96	1.15
(TAPERED)	.31	.36	.44	.53	.60		.34	.44	.51	.60	.72
EFF. DEPTH, IN.	17.8	17.8	17.7	17.7	17.7		17.8	17.7	17.7	17.7	17.6
− ICR/IGR	.159	.181	.207	.241	.264		.173	.207	.232	.264	.298
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.62	.75	.88	1.04	1.20		.51	.62	.75	.88	1.04
STEEL %	.09	.12	.13	.16	.18		.08	.09	.12	.13	.16
EFF. DEPTH, IN.	17.7	17.6	17.6	17.6	17.6		17.7	17.7	17.6	17.6	17.6
+ICR/IGR	.166	.197	.227	.262	.298		.139	.166	.197	.227	.262

One-Way Joist Properties

TABLE 8-1 CROSS SECTION PROPERTIES — STANDARD JOIST CONSTRUCTION (1)

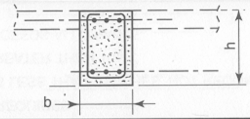
Joist (2)	3-Inch Top Slab: Gross Area(3) (in.^{2})	Wt.(4) (psf)	Y_{cg}(3) (in.)	I_g(3) (in.^{4})	$+M_{cr}$ (ft-k)	$-M_{cr}$(3) (ft-k)	4.5-Inch Top Slab: Gross Area(3) (in.^{2})	Wt.(4) (psf)	Y_{cg}(3) (in.)	I_g(3) (in.^{4})	$+M_{cr}$ (ft-k)	$-M_{cr}$(3) (ft-k)
8 + 5 + 20	120.3	60	7.49	1,104	5.8	12.4	157.8	79	8.50	1,630	7.6	16.1
	152.3		6.75	1,582		14.7	189.8		7.74	2.340		19.4
8 + 6 + 20	131.3	63	7.32	1,254	6.8	13.5	170.3	82	8.33	1,852	8.8	17.6
	163.3		6.67	1,709		15.6	202.3		7.65	2,528		20.6
10 + 5 + 20	133.3	67	8.76	1,826	8.2	17.0	170.8	85	9.86	2,561	10.3	21.8
	173.3		7.89	2,594		20.1	210.8		8.93	3,659		26.0
10 + 6 + 20	146.3	70	8.56	2,069	9.6	18.4	185.3	89	9.65	2,906	11.9	23.7
	186.3		7.80	2,801		21.3	225.3		8.83	3,951		27.5
12 + 5 + 20	147.0	74	9.99	2,799	11.1	22.1	184.5	92	11.16	3,797	13.4	28.1
	195.0		9.01	3,951		26.1	232.5		10.10	5,388		33.3
12 + 6 + 20	162.0	78	9.76	3,165	12.8	23.9	201.0	97	10.92	4,300	15.6	30.5
	210.0		8.90	4,264		27.6	249.0		9.97	5,815		35.2
8 + 5 + 30	150.3	54	7.89	1,223	6.1	15.5	202.8	72	8.89	1,813	8.1	19.8
	190.3		7.07	1,914		19.3	242.8		8.08	2,825		25.3
8 + 6 + 30	161.3	56	7.73	1,393	7.1	16.8	215.3	75	8.74	2,058	9.3	21.6
	201.3		6.99	2,051		20.2	255.3		7.99	3,028		26.6
10 + 5 + 30	163.3	58	9.26	2,032	8.7	21.5	215.8	77	10.35	2,841	10.8	27.1
	213.3		8.26	3,145		26.2	265.8		9.35	4,422		33.9
10 + 6 + 30	176.3	61	9.06	2,307	10.1	23.1	230.3	80	10.16	3,227	12.6	29.4
	226.3		8.16	3,366		27.5	280.3		9.24	4,737		35.6
12 + 5 + 30	177.0	63	10.58	3,128	11.7	28.0	229.5	82	11.77	4,219	14.2	35.2
	237.0		9.42	4,790		34.0	289.5		10.57	6,520		43.5
12 + 6 + 30	192.0	67	10.34	3,541	13.5	30.1	246.0	85	11.53	4,783	16.4	38.0
	252.0		9.31	5,124		35.6	306.0		10.45	6,979		45.6
14 + 5 + 30	191.3	68	11.86	4,549	15.2	35.0	243.8	87	13.13	5,986	18.0	44.1
	261.3		10.56	6,905		42.4	313.8		11.76	9,174		53.8
14 + 6 + 30	208.3	72	11.59	5,135	17.5	37.5	262.3	91	12.86	6,773	20.8	47.4
	278.3		10.44	7,382		44.4	332.3		11.62	9,812		56.4
16 + 6 + 30	225.3	78	12.81	7,127	22.0	45.5	279.3	97	14.15	9,238	25.8	57.5
	305.3		11.55	10,197		54.1	359.3		12.78	13,295		68.1
16 + 7 + 30	244.3	83	12.55	7,890	24.9	48.3	299.8	101	13.88	10,246	29.2	61.2
	324.3		11.43	10,844		56.6	379.8		12.64	14,137		71.1
20 + 6 + 30	261.3	91	15.18	12,469	32.5	63.0	315.3	109	16.65	15.768	37.4	79.4
	361.3		13.74	17,741		75.8	415.3		15.05	22,454		93.9
20 + 7 + 30	284.3	96	14.88	13,769	36.6	67.0	339.8	115	16.33	17,433	42.2	84.3
	384.3		13.61	18,864		79.4	439.8		14.89	23,861		98.1

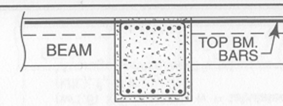
(1) f'_c = 4,000 psi, rib side slope = 1 to 12.
(2) First value is the pan depth, second value is the rib width, and the third value is the pan width (in.)
(3) First value is for a standard section; second value is at a tapered end.
(4) For normal-weight concrete, w = 150 pcf (added weight of tapers is neglected.)

Joist-Band Beam Design Table

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

JOIST-BAND BEAMS, INTERIOR SPANS





TOTAL CAPACITY $U = 1.4D + 1.7L$(3)

STEM		BARS(1)				SPAN, ℓ_n = 28 ft					SPAN, ℓ_n = 30 ft					SPAN, ℓ_n = 32 ft					SPAN, ℓ_n = 34 ft					$+\phi M_n$ $-\phi M_n$	DEFL (C)
h	b	BOTTOM		Layers	TOP	LOAD (4)	STIR. TIES (5)	ϕT_n	$A\ell$	STEEL WGT	LOAD (4)	STIR. TIES (5)	ϕT_n	$A\ell$	STEEL WGT	LOAD (4)	STIR. TIES (5)	ϕT_n	$A\ell$	STEEL WGT	LOAD (4)	STIR. TIES (5)	ϕT_n	$A\ell$	STEEL WGT	(6)	(7) $\times 10^{-9}$
in.	in.	ℓ_n + 12 in.	0.875 ℓ_n	(2)		k/ft		ft-kips	sq. in.	lb.	k/ft		ft-kips	sq. in.	lb.	k/ft		ft-kips	sq. in.	lb.	k/ft		ft-kips	sq. in.	lb.	ft-kip	in.
		2# 9	1# 9	1	4# 9	5.2	123H	18	-	626	4.5	123H	18	-	663	4.0	123H	18	-	700	3.5	123H	18	-	737	281	187
				1			175H	73	1.9	1099		185H	73	1.9	1169		195H	72	1.8	1240		215H	72	1.8	1345	368	
		2#10	1#10	1	4#11	7.2	133H	18	-	849	6.2	143H	18	-	910	5.5	143H	18	-	962	4.9	153H	18	-	1022	350	192
				1			175H	73	1.9	1313		185H	73	1.9	1399		195H	72	1.8	1485		215H	72	1.8	1604	547	
	24	2#11	2#11	1	4#14	10.4	154H	18	-	1354	9.1	164H	18	-	1448	8.0	164H	18	-	1526	7.1	174H	18	-	1621	547	157
				1			245E	73	1.9	1916		265E	73	1.8	2062		285E	72	1.8	2208		295E	72	1.8	2320	742	
		2#14	1#14	1	5#14	11.9	155H	18	-	1720	10.4	164H	18	-	1679	9.1	174H	18	-	1789	8.1	184H	18	-	1899	582	143
				1			245E	73	1.9	2132		265E	73	1.8	2293		285E	72	1.8	2454		295E	71	1.8	2581	886	
		2# 9	2# 9	1	5#10	7.7	113H	33	-	848	6.7	123H	33	-	910	5.9	123H	33	-	962	5.2	123H	32	-	1015	377	125
				1			563A	133	2.8	1440		603A	131	2.7	1542		643A	130	2.7	1645		683A	129	2.7	1747	580	
		2#11	2#11	1	5#11	9.8	133H	33	-	1153	8.5	133H	33	-	1225	7.5	143H	32	-	1308	6.6	143H	32	-	1380	569	128
				1			245E	132	2.7	1958		185H	131	2.7	1798		195H	130	2.7	1909		683A	129	2.7	2093	697	
24.5	36	2#14	2#14	1	6#14	15.6	155H	33	-	2065	13.6	165H	33	-	2209	12.0	164H	32	-	2149	10.6	174H	32	-	2282	787	106
				1			285D	132	2.7	2707		305D	131	2.7	2899		285E	130	2.7	2937		295E	129	2.7	3090	1114	
		3#14	2#14	1	7#14	17.7	155H	33	-	2430	15.4	165H	33	-	2600	13.5	175H	32	-	2769	12.0	185H	32	-	2939	956	94
				1			345C	132	2.7	3304		305D	131	2.7	3290		325D	130	2.7	3507		345D	129	2.7	3725	1261	
		3# 9	3# 9	1	6#11	11.5	123H	49	-	1233	10.0	133H	49	-	1322	8.8	133H	48	-	1400	7.8	133H	48	-	1478	562	99
				1			**	197	3.9	1105		**	195	3.8	1183		**	193	3.8	1261		**	192	3.7	1340	845	
		3#10	3#10	1	6#14	14.3	134H	49	-	1768	12.5	144H	49	-	1894	11.0	143H	48	-	1864	9.7	153H	48	-	1981	701	94
				1			245E	196	3.7	2549		364C	195	3.6	2595		384C	193	3.6	2756		414C	191	3.6	2945	1164	
	48	4#11	3#11	1	7#14	18.7	145H	49	-	2403	16.2	155H	49	-	2573	14.3	165H	48	-	2744	12.7	164H	48	-	2691	972	85
				1			425B	196	3.6	3783		455B	194	3.6	4052		485B	193	3.6	4321		295E	191	3.6	3629	1329	
		3#14	3#14	1	9#14	22.9	175EfH	49	-	2982	20.0	175EeH	49	-	3154	17.6	185EcH	48	-	3359	15.5	175H	48	-	3499	1164	74
				1			425B	196	3.6	4265		455B	194	3.6	4568		485B	193	3.6	4871		515B	191	3.6	5174	1633	

(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.

CONCRETE REINFORCING STEEL INSTITUTE

12-105

Hollow-Core Precast Planks System

TECH 2 | HOLLOW-CORE PRE CAST SYSTEM | ALTERNATIVE #

→ Using PCI Design HANDBOOK - FIFTH Edition.

→ Find Service loads acting on system:

LL = 100 psf
DL = 15 psf (superimposed)
115 psf

→ Go into tables

– page 2-27

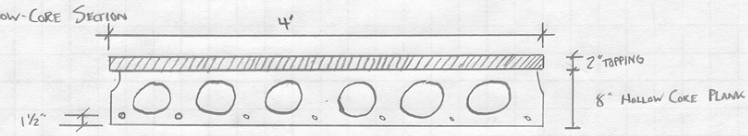
4'-0" wide x 8" thick precast hollow-core
WITH 2" NORMAL WEIGHT TOPPING

Span → 30 ft

→ **Use 4'-0" x 8" PRECAST HOLLOW CORE PLANKS W/ 2" TOPPING**
REINFORCED W/ (6) #8 prestressed strands
∴ Straight tendon profile

SAFE SERVICE LOAD = 125 psf > 115 psf ∴ ✓

HOLLOW-CORE SECTION



– Concrete Compressive Strength
– f'_c = 5000 psi

Hollow-Core Plank Design Table

Strand Pattern Designation

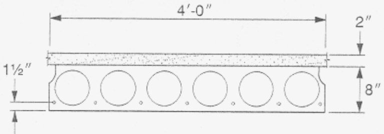
76–S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

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.

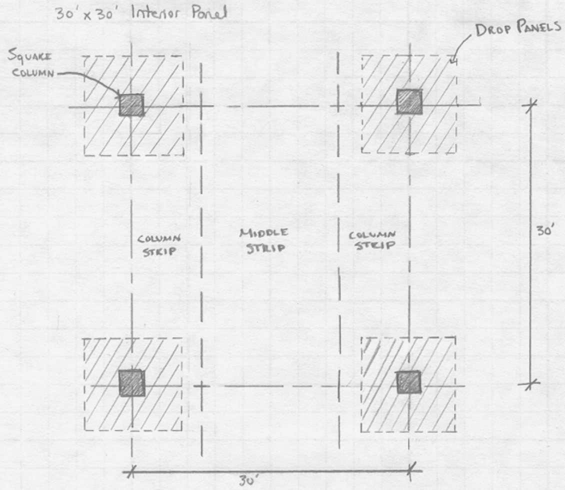
Two-Way Slab System with Drop Panels

Tech 2 | Two-Way Drop Panel System

→ Design for two-way concrete slab w/ Drop panels

→ Reference Concrete Steel Reinforcing Institute (CRSI 2002)

30' x 30' Interior Panel



Factored Superimposed Loads

LL = 100 psf
DL = 15 psf

$w_u = 1.4(15) + 1.7(100) = 191$ psf

→ Go into tables

→ pg 10-29

→ Use 11.5" thick slab w 10'x10' Drop panels, 9" Deep

Factored Load = 200 psf > 191 psf ∴ ✓

Required Reinforcement : (Each Way)

Column Strip → (1/4)(30') = 7.5' Each Strip ∴ Totals 15'
Middle Strip → 30' − 2(7.5') = 15' Strip

Column Strip Reinforcement

→ (17) #6 Bars (Total Top Reinforcement)
→ (14) #6 Bars (Total Bottom Reinforcement)

Middle Strip Reinforcement

→ (15) #5 Bars (Total Top Reinforcement)
→ (13) #5 Bars (Total Bottom Reinforcement)

→ Square Column is a minimum of 19 in x 19 in and must be the same above and below the slab

→ Total Steel → 3.31 psf

Two-Way Slab Design Table

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL With Drop Panels
No Beams

h = 11.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	(3) Square Column Size (in.)	γ_f	Reinforcing Bars (E. W.) Column Strip (1) Top Ext.	+	Column Strip (1) Bottom	Column Strip (1) Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (–) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (–) (ft-k)
29	100	7.00	9.67	12	0.786	13-#5	3	11-#7	14-#6	10-#6	12-#5	2.94	225.5	451.0	607.1
29	200	9.00	9.67	16	0.673	13-#5	3	11-#8	16-#6	10-#7	15-#5	3.61	290.2	580.4	781.3
29	300	9.00	9.67	19	0.717	14-#5	4	11-#9	15-#7	12-#7	10-#7	4.50	355.0	710.0	955.8
29	400	11.00	9.67	21	0.640	15-#5	3	13-#9	16-#7	18-#6	15-#6	5.11	420.6	841.2	1132.4
29	500	11.00	11.60	23	0.707	17-#5	3	15-#9	14-#8	12-#8	10-#8	5.87	485.4	970.8	1306.9
30	100	9.00	10.00	12	0.698	14-#5	1	12-#7	14-#6	15-#5	13-#5	2.99	251.2	502.3	676.2
30	200	9.00	10.00	16	0.721	14-#5	3	12-#8	18-#6	14-#6	16-#5	3.79	322.4	644.9	868.1
30	300	11.00	10.00	19	0.636	14-#5	3	12-#9	15-#7	10-#8	20-#5	4.61	395.5	791.0	1064.8
30	400	11.00	10.00	21	0.698	17-#5	3	18-#8	14-#8	12-#8	10-#8	5.55	467.8	935.7	1259.6
30	500	11.00	12.00	25	0.757	19-#5	6	17-#9	16-#8	11-#9	12-#8	6.52	536.9	1073.9	1445.6
31	100	9.00	10.33	12	0.740	14-#5	2	18-#6	16-#6	12-#6	14-#5	3.16	277.8	555.5	747.8
31	200	9.00	10.33	16	0.777	14-#5	5	11-#9	15-#7	12-#7	13-#6	4.17	357.0	713.9	961.1
31	300	11.00	10.33	19	0.678	16-#5	3	17-#8	22-#6	11-#8	12-#7	5.01	438.1	876.2	1179.5
31	400	11.00	12.40	23	0.749	18-#5	6	16-#9	15-#8	13-#8	11-#8	5.89	517.8	1035.7	1394.2
31	500	11.00	12.40	28	0.746	15-#6	4	19-#9	14-#9	12-#9	16-#7	6.92	588.5	1176.9	1584.3
32	100	9.00	10.67	12	0.803	15-#5	5	15-#7	17-#6	13-#6	11-#6	3.36	306.2	612.4	824.3
32	200	11.00	10.67	16	0.651	15-#5	2	12-#9	15-#7	13-#7	14-#6	4.31	394.9	789.8	1063.2
32	300	11.00	10.67	19	0.781	17-#5	7	15-#9	18-#7	12-#8	13-#7	5.30	483.7	967.4	1302.3
32	400	11.00	12.80	26	0.718	20-#5	5	18-#9	17-#8	12-#9	12-#8	6.49	567.2	1134.4	1527.1
32	500	11.00	12.80	31	0.692	16-#6	3	21-#9	15-#9	13-#9	11-#9	7.31	639.5	1278.9	1721.6
33	100	11.00	11.00	12	0.710	15-#5	2	22-#6	17-#6	20-#5	12-#6	3.48	337.7	675.5	909.3
33	200	11.00	11.00	16	0.754	15-#5	5	17-#8	16-#7	11-#8	12-#7	4.63	434.4	868.8	1169.5
33	300	11.00	11.00	22	0.734	19-#5	5	17-#9	15-#8	13-#8	11-#8	5.84	528.0	1056.0	1421.5
33	400	11.00	13.20	29	0.711	22-#5	6	20-#9	18-#8	13-#9	13-#8	6.83	616.2	1232.4	1659.0
34	100	11.00	11.33	12	0.762	16-#5	4	14-#8	26-#5	12-#7	19-#5	3.77	370.1	740.2	996.4
34	200	11.00	11.33	18	0.752	17-#5	6	18-#8	18-#7	12-#8	13-#7	4.84	473.9	947.8	1275.9
34	300	11.00	11.33	24	0.757	20-#5	8	18-#9	17-#8	12-#9	12-#8	6.19	576.4	1152.8	1551.9
34	400	11.00	13.60	31	0.689	17-#6	3	21-#9	16-#9	14-#9	14-#8	7.18	667.9	1335.8	1798.2

SQUARE INTERIOR PANEL
With Drop Panel(2)
No Beams

h = 11.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	(3) Square Column Size (in.)	Reinforcing Bars (E. W.) Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
29	100	12	13-#6	14-#5	12-#5	12-#5	2.64	1.023
29	200	19	15-#6	18-#5	10-#6	12-#5	3.08	1.042
29	300	22	26-#5	12-#7	12-#6	10-#6	3.86	1.042
29	400	24	15-#7	18-#6	14-#6	12-#6	4.40	1.060
29	500	27	13-#8	12-#8	12-#7	14-#6	5.10	1.105
30	100	12	18-#5	15-#5	13-#5	13-#5	2.67	1.042
30	200	19	17-#6	14-#6	15-#5	13-#5	3.31	1.042
30	300	22	26-#5	17-#6	10-#7	16-#5	3.92	1.060
30	400	24	16-#7	15-#7	12-#7	10-#7	4.78	1.060
30	500	27	15-#8	11-#9	18-#6	12-#7	5.67	1.105
31	100	12	20-#5	12-#6	13-#5	13-#5	2.78	1.042
31	200	19	26-#5	12-#7	12-#6	14-#5	3.56	1.042
31	300	22	15-#7	11-#8	11-#7	18-#5	4.35	1.060
31	400	25	14-#8	13-#8	13-#7	11-#7	5.12	1.105
31	500	28	16-#8	12-#9	12-#8	13-#7	6.02	1.105
32	100	12	16-#6	13-#6	14-#5	13-#5	2.90	1.042
32	200	19	26-#5	17-#6	13-#6	11-#6	3.62	1.060
32	300	22	17-#7	21-#6	12-#7	19-#5	4.52	1.060
32	400	25	15-#8	12-#9	11-#8	13-#7	5.66	1.105
32	500	31	17-#8	13-#9	16-#7	11-#8	6.30	1.105
33	100	12	16-#6	11-#7	11-#6	14-#5	3.06	1.060
33	200	19	15-#7	11-#8	20-#5	17-#5	3.97	1.060
33	300	22	15-#8	11-#9	13-#7	15-#6	5.13	1.060
33	400	27	17-#8	13-#9	12-#8	18-#6	5.91	1.105
34	100	12	18-#6	12-#7	17-#5	15-#5	3.25	1.060
34	200	19	22-#6	21-#6	12-#7	19-#5	4.15	1.060
34	300	22	16-#8	19-#7	12-#8	13-#7	5.40	1.060
34	400	30	18-#8	14-#9	13-#8	12-#8	6.29	1.105

NOTES: (1) 50 percent of these bars may be place in the middle third of column strip. (2) Drop panels same size as for edge panels. (3) Same column size above and below slab.