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S&T Bank
Corporate Headquarters

Indiana, PA

Executive Summary

The following technical report provides a structural study of the existing and four alternative floor systems for the S&T Bank Corporate Headquarters in Indiana PA. The purpose of this report is to evaluate whether or not any of the systems is a viable alternative floor system to the existing floor system.

The four alternative systems that were analyzed are…

- Composite Decking w/ Steel Beams
- One-Way Concrete Joist Slab System
- Pre-cast Pre-stressed Hollowcore Plank
- Two-way Flat Slab w/ Drop Panels

To make an educated decision on whether or not any of the systems would be a good alternative, several factors were considered. Among those considered were…

- Cost
- Time of Construction
- Depth of System
- Vibration Control
- Fire-proofing

After the analysis of each system, a comparison of Pros and Cons helped to determine which systems would be a good alternative.

- The existing system is an adequate system
- Composite is an efficient alternative
- One-Way Joist is not a viable alternative
- Pre-stressed Hollowcore is not a viable alternative
- Two-Way Drop Panel is a viable alternative

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Pro-Con Structural Study of Alternate Floor Systems

Introduction:

This document will first present some general conditions of S&T Bank's Corporate Headquarters in Indiana, PA. Then the existing structural floor system of Bank will be described in full detail. As is the purpose of this report, the following paragraphs will present four alternative structural floor systems. These systems will finally be critiqued according to cost, time for erection, fire-ratings, vibrations, and other various topics.

S&T Bank Corporate Headquarters is a 4 story, steel frame building. At the foundation, spread footings support the building while it rests on site class C soil. The basement floor is a concrete slab and the walls are masonry block. The 1st through 4th floors consist of non-composite decking with a 3" concrete topping that rests on joists. The building is an office building, except for the bank that resides on the first floor. About 50% of the floors above are used as a lobby space for bank customers. For this reason, the live loads determined in IBC 2003 are for an office lobby area. This live load is 100psf. The dead loads for the layout are as follows…

Dead Loads:

Structural Framing Weight:	6.42psf
Slab/Decking Load:	40psf
Superimposed Dead Load:	12psf

The floor layout is very basic and very repetitive from floor to floor. A typical bay is 28′ by 28′. At the intersection of beams and columns, moment connections are used to provide lateral load support. The basic layout of the building is described in the figure below.

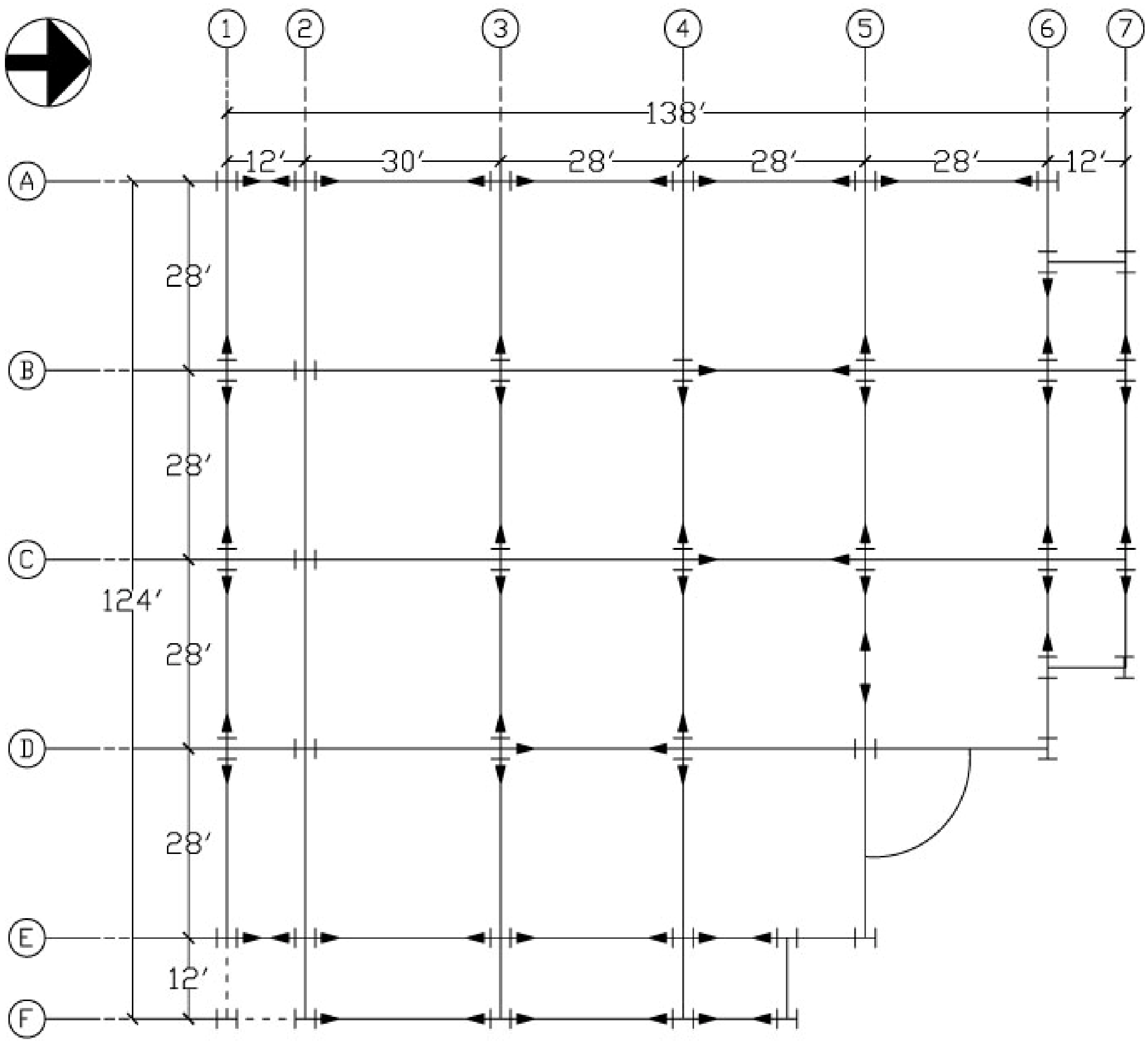


Figure 1a: Typical Floor Layout

Existing Structural Floor System

To describe the existing floor, a typical multiple bay layout is shown below. With the exception of the joist system, this layout will also be used for alternate system analyses later in the report.

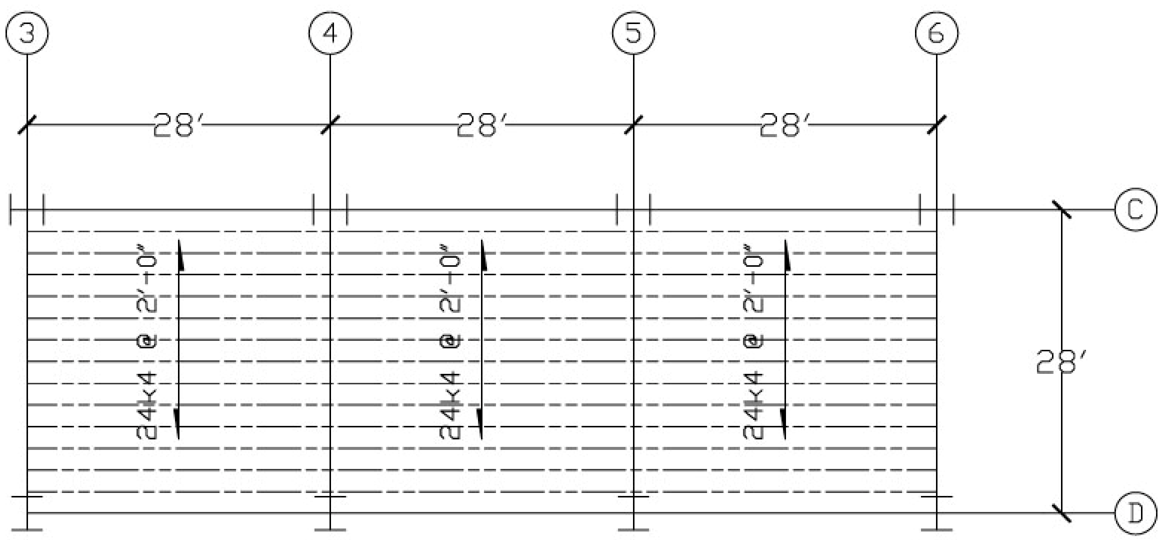


Figure 1b: Typical Floor Layout

The 1st through 4th floor construction is a 3″ concrete topping, reinforced with 6x6 W1.4 x W1.4 WWF and rests on Bowman 28 Gage SF-1 galvanized decking. This non-composite decking then sits on typical 24k4 joists spaced at 2ft o.c. The 28-day strength of the concrete topping is 3000psi. The depth from the bottom of the joist to the top of the slab is 27″, which is coincidently the same depth as the girder in this span (W24x68

plus 3" slab). The total depth of the system is 27". An example of this system can be seen below in figure #1c.

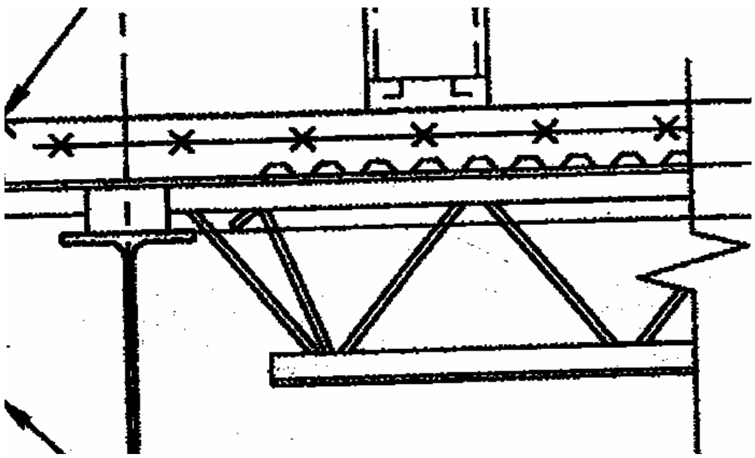


Figure #1c: Floor Construction Detail

This current system is sprayed with fire-proofing so that the system as a whole has a 2 hour fire-rating.

Now that the existing system is described, four alternate systems will be proposed. The four systems that will be investigated are…

1.) Composite Decking

2.) One-Way concrete Joists System

3.) Pre-stressed Pre-Cast Hollow-Core Planks

4.) Two-way Concrete Slab with Drop Panels

Alternate System #1: Composite Decking

Composite decking construction helps to uniformly form together the beam, deck form, and concrete using shear studs placed along the beams. For this system, the joists that are currently present in the existing system will need to be replaced with beams, which will then form into girders. The USD Design Manual and Catalog was used to choose decking when subjected to live load. With the live load of 100psf and a serviceability factor of 1.0, the tables were entered. It was determined that the decking suitable for the load is 22 gage 2″ Lok-Floor with 4 ½″ normal weight concrete topping. With a span from beam to beam of 8′-0″ the decking can carry a live service load of 230psf. However since WWF is not being considered, there must be a 10% reduction of this capacity. Therefore the allowable load for the decking can be 207psf. 207psf > 100psf so the system is adequate.

Next RAM Steel was used to determine the intermediate beam sizes, new girder sizes, and amount of shear studs. This output was then verified by hand calculations. It was found by RAM that intermediate beams should be W8x10 spaced at 7′-0″ which then formed into W18x35 and W21x48 girders. From hand calculations, the beams should be W10x15 spaced at 7′-0″. The girders from RAM matched the girders from hand calculations. I

have decided to use the beams determined from the hand calculation solution. A representation of this can be seen below in Figure #2a.

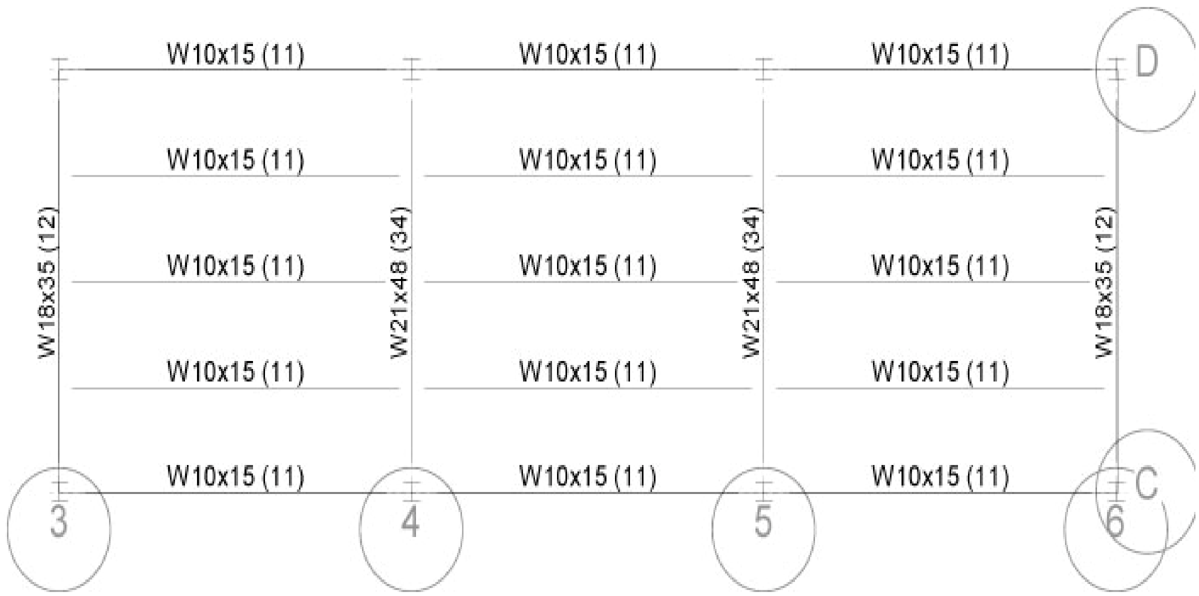


Figure #2a: Composite Member Layout Using Hand Calculations

The numbers in parenthesis are the number of shear studs along the member. The decking in this figure spans from top to bottom.

The total weight of this system is designated at 42psf, according to the USD design manual. The total depth of the system is 25.5″; this includes the depth of the girder and the depth of the decking with the concrete topping. The deflection limitations for the decking and the beams, length/360, are taken care of in the design manual and in the RAM calculations, respectively. Actual deflection calculations and member sizing

can be seen in Appendix A-1. To meet an adequate fire rating, the beams and girders would need to be sprayed with fire-proofing. A 4.5" slab depth is good enough to provide a 2 hour fire-rating. A detail of this system can be seen in figure #2b.

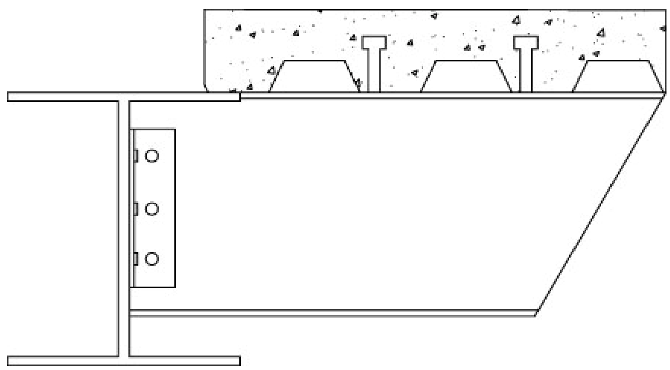


Figure #2b: Composite System Detail

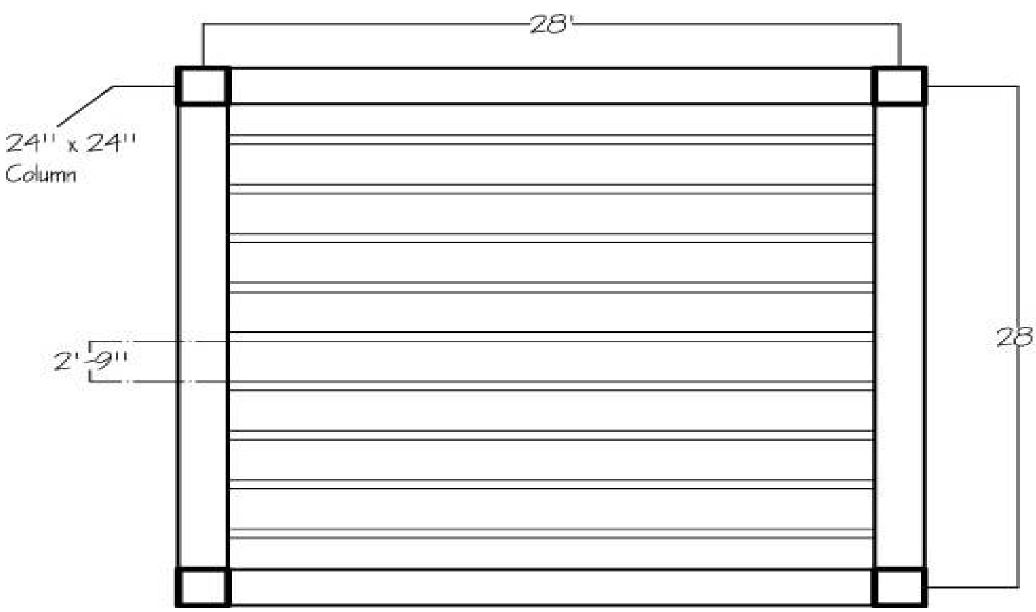
Alternate System #2: One-Way Concrete Joist Construction

Concrete joist construction is a method of construction consisting of a monolithic combination of regularly spaced joists and a thin slab of concrete cast in place to form an integral unit with the supporting beams, columns, or walls. The application of this type of system would require a redesign of steel beams into concrete beams, and the steel columns into concrete columns. Switching from a steel system to a concrete system would also increase the total building weight significantly. Due to the increased

building weight, footing sizes will need to be checked to verify they are still adequate to carry the applied loads.

The existing spans for the non-composite decking run in the E-W direction. The one-way joist system will also span in this direction. The concrete joists will experience less deflection over the shorter non-typical spans; therefore this system will work well with the current layout of the frame (see Figure 1b).

The CRSI Handbook was used to determine an adequate one-way concrete joist floor system that fit multiple spans of 28′ and a uniform load of 186.8psf. A typical layout of this system can be seen below in figure#3a.



Figure#3a: Layout Plan

Entering the table on page 8-30 of the CRSI Handbook with a span of 28′ and a factored load of 186.8psf, it was determined that 30″/6″ one-way joist system with 16″ deep ribs and a 4.5″ top slab would carry a load of 225psf for an interior bay. The factored usable superimposed loads provided by the tables uses a w_u =1.4DL+ 1.7LL and already reduces the system's self-weight. Since 225psf > 186.8psf, the suggested system is adequate. The system for the applied load does not need to be checked for deflection, as says the CRSI Handbook. A diagram of the chosen system can be seen below (Figure #3b). Top reinforcement consists of #5 rebar spaced at 11″ o.c. Bottom reinforcement consists of 2-#5 rebar evenly spaced.

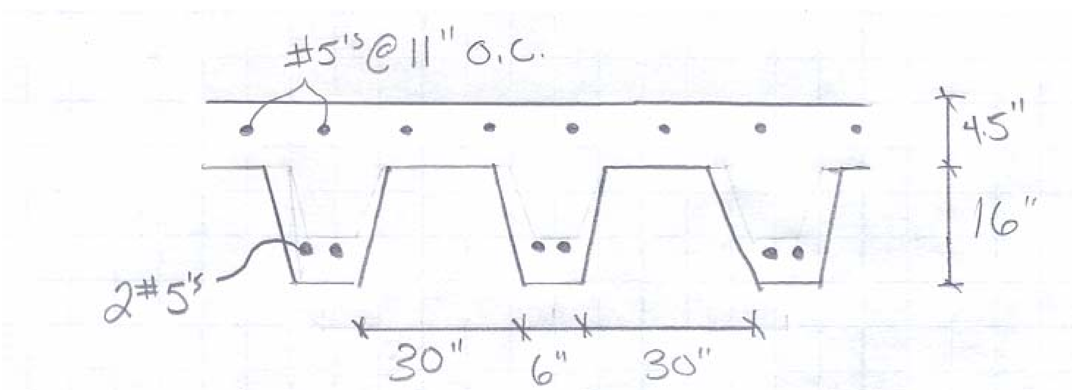


Figure #3b: Description of Chosen One-Way Slab System

The corresponding girder/joist-band beam that would be necessary to carry this type of a floor system is a 20.5″ deep by 24″ wide cast-in-place concrete beam. Top reinforcement consists of 2 #11′s; the bottom

reinforcement is 8 #10's (see Appendix A-2). To provide shear strength, 21 #5 stirrups spaced 1@2", and 20@8" for each end are required. A representation of this can be seen below in Figure #3c.

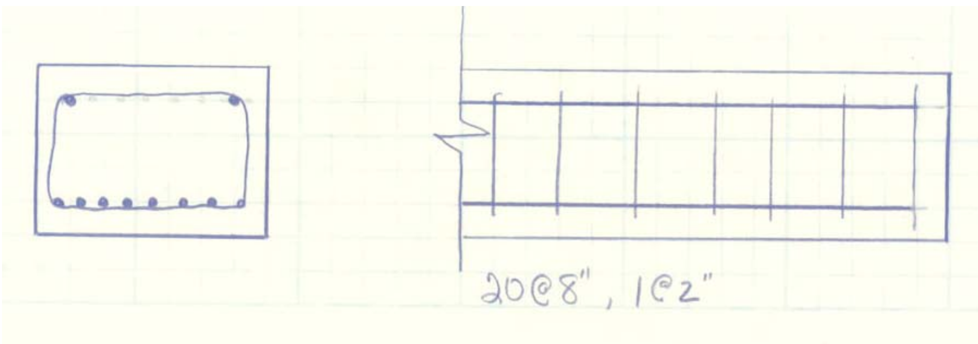


Figure #3c: Joist-Band Beam Section

The total depth of this system is 20.5", which is 6 ½" less than the existing system. As mentioned before, concrete beams and columns would have to be designed for the application of this system. If the material costs are not higher for this system, the labor costs will be. Formwork will need to be purchased and it will require time for it to be placed. Rebar also takes time to be placed in the proper place. It will also require more time to erect since the concrete takes time to set. Fire-proofing the one-way joist system is not needed since the shallowest depth is 4.5" and it is all concrete. The weight of the concrete is 150pcf. The overall weight of this system is much higher than the existing metal deck system. Because of this, vibrations for this system would be lower than the current floor system.

Alternate System #3: Pre-stressed Pre-Cast Hollow-Core Planks

The pre-cast pre-stressed hollow-core plank is a concrete floor system that eliminates useless concrete in the center of the plank which in turn provides a lighter weight floor system. Once the planks are set in place atop of the steel or concrete girders, a concrete topping is poured to make the floor act more uniform. The planks can be attached to the steel in a number of ways. A typical way of attaching the plank to the girders is to have a steel plate integrated on the bottom of the plank and then weld that section of the plank to the girders. However this would create a 36″ deep floor system, which is undesirable. Pre-stressed steel strands in the hollow-core create a negative bending moment in the concrete which helps to counteract the positive bending moment developed in the member from gravity loads. Because of this, the system can carry heavier loads that a simple concrete slab.

The PCI Handbook provided design tables for hollow-core plank floor systems. To determine a correct floor system, the tables must be entered with service loads only, i.e. no load factors. With a span of 28′ and a load of

112psf, an appropriate system was chosen. The floor system chosen can be seen in Figure #4a and Figure #4b.

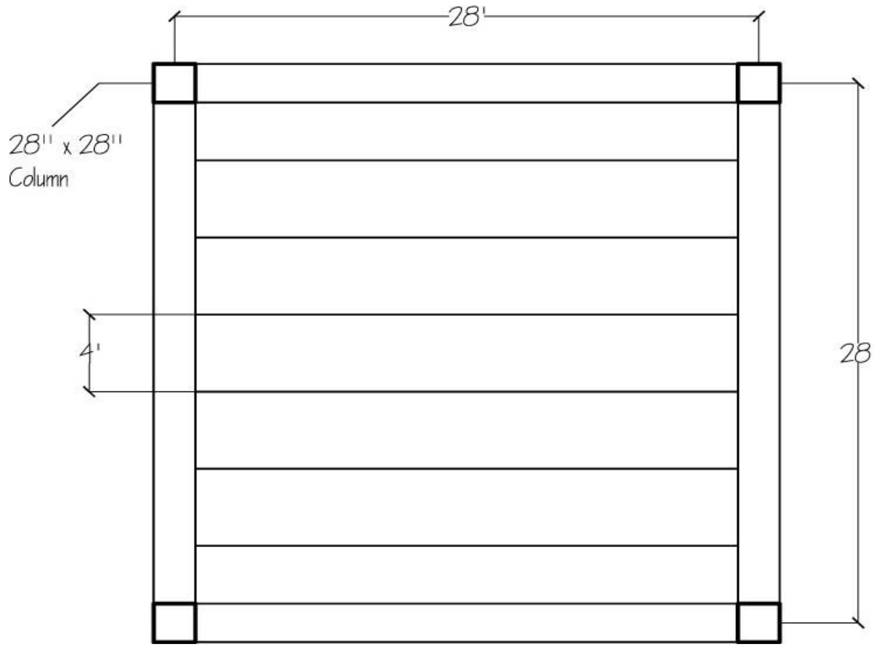


Figure #4a: Layout Plan

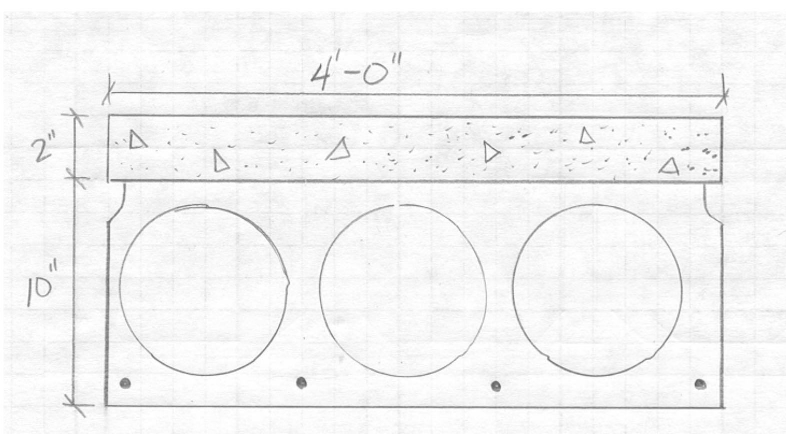


Figure #4b: Typical Hollow-Core Section

Pictured in the figure is a 4HC10+2 plank. This translates to a plank that is 4′ wide and 10″ deep. It has 4 pre-stressed strands in the bottom of the plank. These strands are ½″ in diameter and have strength of 270ksi. On top of the plank is a 2″ concrete slab, where f'_c=5000psi. According to the charts in the PCI Handbook, this system carries a service load of 113psf which is greater than the applied load of 112psf, therefore it is adequate. The weight for this plank is also listed at 93psf. This is more than twice the weight of the existing system, so the girders have to be redesigned. The existing steel girders will be replaced by inverted concrete tee beams. Again, using the PCI Handbook, an appropriate inverted, pre-stressed tee beam was designed (see Appendix A-3). To carry a distributed load of 5.404klf the following tee beam was found (figure #4c).

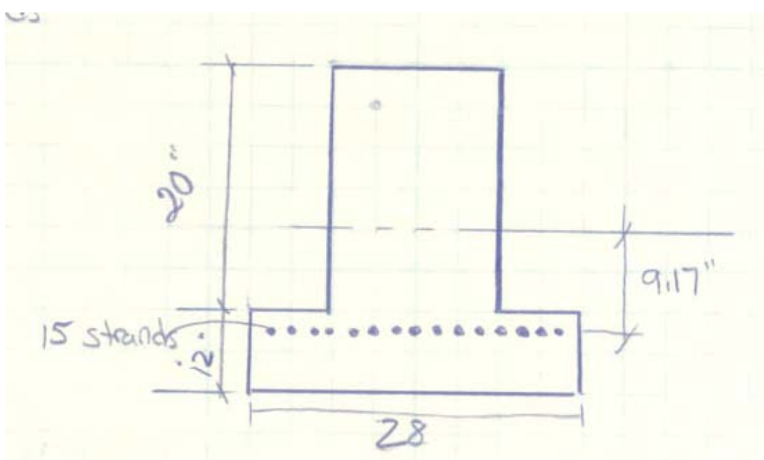


Figure #4c: Tee Beam Detail

The total depth of the system is 32″. The concrete plank is thick enough so that it doesn't need any fire-proofing. This system increases the weight of the building therefore vibrations would be less than the existing system.

Alternate System #4: Two-Way Flat Slab w/ Drop Panels

Two-way flat slabs with drop panels provide the ability to compensate for longer spans and heavier loads, while keeping the slab system itself thin. This system will require the design of concrete columns to replace the existing steel columns. The CRSI Handbook will be an efficient tool used to decipher a compatible two-way slab for the present spans and loads. Compared to the one-way joist system, the two-way flat slab system will have a smaller depth. The system analyzed has a depth of 18″, according to the CRSI manual, as compared to 20.5″ of that in a one-way joist system.

Entering the table on page 10-25 of the CRSI Handbook with a span of 28′ and a factored load of 186.8psf, a two-way flat slab system was chosen (as seen in figure #5a and Figure #5b). Appropriate supporting work can be found in Appendix A-4.

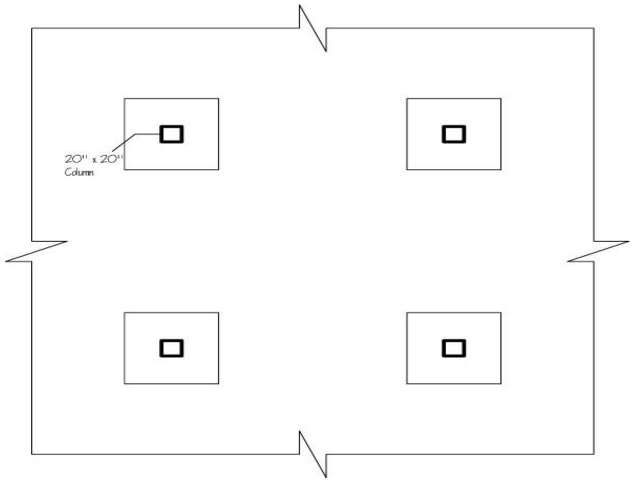


Figure #5a: Layout Plan

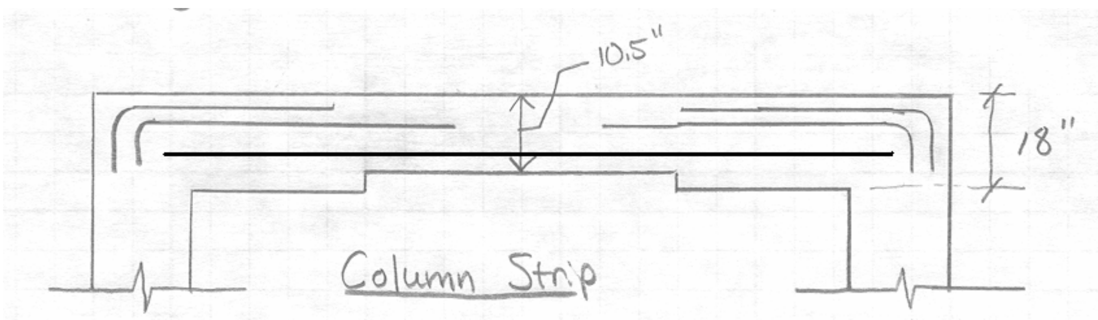


Figure #5b: Two-Way Flat Slab w/ Drop Panels

Panel Specs:

Slab:	**10.5" thick**	
Drop Panel:	**9'-4" by 9'4"**	
	7.5" thick	
Reinforcement:	**Top:**	**15-#6 ———> Column Strip**
	Bottom:	**12-#6**
	Top:	**13-#5————> Middle Strip**
	Bottom:	**11-#5**
Total Steel:	**3.07psf**	
Total Depth:	**18"**	
f'_c:	**4000psi**	
f_y:	**60ksi**	

With a depth of 18", the two-way drop panel system is 9" shallower than the existing joist-deck system. As with the one-way joist system, the labor costs for this system will skyrocket since it must be poured in place. Again, these cost increases are mainly due to formwork cost, material cost, time to set the formwork, and time for the concrete to cure. Since the slab is 10.5" thick, no additional fire-proofing is required. The weight of the concrete is 150pcf. This system increases the weight of the building therefore vibrations would be less than the existing system.

System Comparisons:

Although detailed descriptions are beneficial to understanding a systems function, when presented separately the systems are hard to compare. The following chart is a comparison of each of the proposed alternate floor systems. This chart shows the good things the system has to offer, the bad things the system has to offer, and whether it should be considered for further analysis.

S&T Bank
Corporate Headquarters
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System	Pro	Con	Possible Solution
Existing Non-Composite Metal Deck	Foundation size Low Cost Moderate Erection Time	Vibrations Fire-proofing 27″deep	
Alternate#1 Composite	Foundation size 25.5″ Deep Moderate Erection Time	Shear Stud Cost Vibrations Fireproofing	YES
Alternate #2 One-Way Joist	20.5″ Deep Low Vibrations No Fireproofing	Foundation size Long Erection Time Formwork Cost Material Costs	High concrete costs: NO
Alternate #3 Precast Hollowcore	Fast Erection Time Low Vibrations No Fireproofing	Foundation size Higher Cost 32″ Deep	Very Deep System: NO
Alternate #4 Flat Slab w/ Drop Panel	18″ deep Low Vibrations No fireproofing	Foundation size Long Erection Time Formwork Costs	YES

Conclusion:

Every floor system described in the report is a good floor system given certain building conditions. However the problem of design lies in construction issues such as cost, system depth, schedule/time for erection etc. Of the four floor types suggested, only two of them will be looked into with a more scrutinizing eye.

The composite decking system is just like the existing non-composite system except that it is more efficient. The shear studs along the steel members make the concrete and steel work more uniformly.

The other system that has potential as an alternate is the two-way flat slab with drop panels. This system will need formwork and more time to erect because of the curing time of concrete, but the system is 9" thinner than the existing system. Since the concrete slab is 10.5" thick (min) no additional fire-proofing will be required.

A one-way joist system wouldn't be ideal for this building for a couple of reasons. Though it is only 20.5" deep, the system would be heavier than the existing system and require a redesign of the foundation. There would also be added costs and time to form all of the joists and slab. Due to these reasons, this system would not be an ideal choice.

The pre-cast Hollowcore plank floor system also is not a consideration for further analysis. The main reason is that the system is 32" deep. This alone is enough to omit it from an ideal system. There would be lead time for the pre-cast members but if coordinated properly the erection time would be less than any of the formed concrete systems. Pre-cast is also typically more expensive than formed concrete, so overall costs would go up. So, the increased costs and depth of the system makes this a bad choice as a floor system.

Appendix A-1:

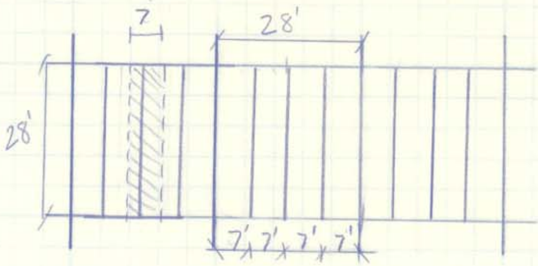
Composite Decking

Composite Decking – Beam Design

2" Lok-Floor – 4.5" conc.

$w_{LL} = 100$ psf

$w_{DL} = 42$ psf

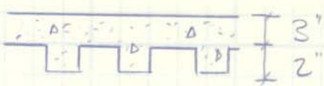


$W_u = 1.2DL + 1.6LL$

$= 1.2(42) + 1.6(100)$

$= 210.4$ psf

beam spacing = 7'



$w_u = 210.4(7') = 1.472$ klf

$M_u = \frac{w_u l^2}{8} = \frac{1.472(28)^2}{8}$

$= 144.256^k$

assume $a = 1"$ ∴ $Y_2 = 5 - \frac{1}{2} = 4.5"$

Try W10X15

$\phi M_n = 148^{1k}$

$\Sigma Q_n = 221^k$

@ Top of Flange

b_{eff} { $\frac{1}{4}$ Span $= 7(12) = 84"$ * controls
Spacing $= 7(12) = 84"$ *

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

$= \frac{221}{.85(3)(84)} = 1.032$

Deflection: $l/360 = .933"$

$Y_2 = 5" - \frac{1.032}{2} = 4.484"$

$\Delta = \frac{5 w l^4}{384 EI} = \frac{5(1.472)(336)^4}{384(29000)(68.9)(144)}$

$= 0.85" < .933"$

∴ OK ✓

interpolate to find actual ϕM_n

$\phi M_n = 147.17^{1k}$

$\Sigma Q_n = 221^k$

Use W10X15 @ 7' spacing

$\#\text{Shear Studs} = \frac{\Sigma Q_n}{Q_n} = \frac{221}{21^k}$

= 11 shear studs per beam

Composite Deck w/ Beams

Decking: 22gage 2" Lok-Floor w/ 4½" normal wt. concrete Slab

Beams: W8x10 spaced @ 8'-0" o.c.

Girders: Exterior: W18x35

Interior: W21x48

Deflection: $\ell/360 = \frac{28(12)}{360} = .933''$

max beam deflection = .692"

$.933'' > .692''$ ∴ ok ✓

· Decking deflections are taken care of in USD manual according to $\ell/360$

If WWF is not present, deduct 10% from the listed loads.

Decking can carry 230psf − 10% = 207psf uniform service Live Load when spanned 8'-0".

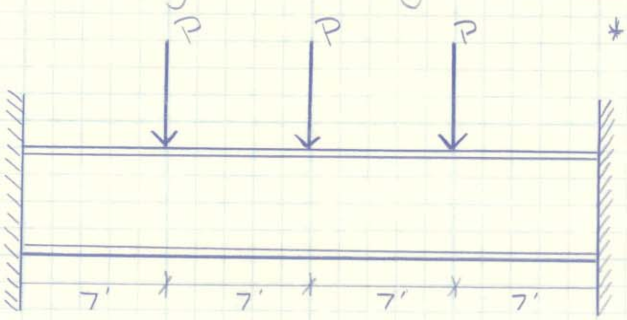
207psf >> 100psf ∴ ok ✓

wt. of System = 42 psf

Total depth of system: Beam = 21"
+
Deck/Slab = 4.5"

Total = 25.5"

Composite Decking – Girder Design



*assume full moment frame

From table 5-16 in AISC

a = .188 Maximum positive moment = aPL
b = .313 Maximum negative moment: bPL

$P = 28' \times (1.472^{klf} + 15_{plf})$
$= 41.636^{K}$
$L = 28'$

$+M_u = .188(41.636)(28')$
$= 219.17^{'K}$

$-M_u = .313(41.636)(28')$
$= 364.89^{'K}$ → controls

use W21x48 for Girders in Composite System

Table 5-2, pg 5-40
$\Sigma Q_n = 705^{K}$
#Shear Studs = 34

$\phi M_p > M_u$
398 > 365 ∴ ok ✓

Composite System:

Girder: W21x48 w/ 34 shear studs @ 28' span
Beam: W10X15 w/ 11 shear studs @ 7' span
Deck: 2" LOK-Floor w/ 4.5" normal wt. concrete

Deflection: $\frac{.010(41.636)(336)^3}{29000 \times 959} = 0.568'' < .993''$ ∴ ok ✓

Table 5-16

Appendix A-2:

One-Way Concrete Joist

One-way Joist Slab | Tech #2

DL: 25 psf superimposed

LL: 100 psf

*Self wt of slab + joists have been reduced from values found in CRSI Tables

Load Combination (CRSI)

$*W_u = 1.4D + 1.7L$

$W_u = 1.4(25) + 1.7(100)$

$= 205$ psf

Clear Span = 28'

CRSI Handbook
Page 8-30

Use: 30" Forms + 6" Ribs @ 36" c-c
16" Deep Rib + 4.5" top Slab

Total depth = 20.5"

w/ 28'-0" span, Factored Usable Superimposed Load (psf)
for Interior Span 225 > 205 psf

Use #5 top reinforcing spaced 11"
#5, #5 bottom reinforcing

$f'_c = 4000$ psi
$f_y = 60$ ksi

Span Deflection coefficient: 4.554

$t \geq \frac{l_n}{21}$ → interior spans

$20.5'' \geq \frac{28'(12'')}{21}$

$20.5'' \geq 16''$ ∴ Deflection is okay

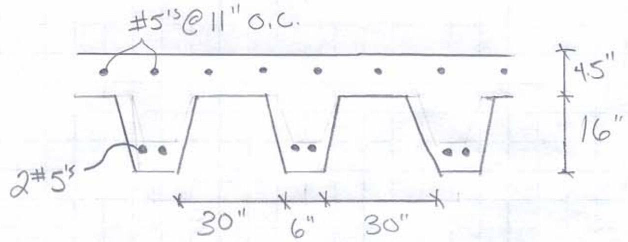


Table 8.1 pg 8-13 from CRSI HandBook

weight = 97 psf

One Way System - Girder Design

weight of joist slab = 97 psf   use 1.4D + 1.7L

weight on Girder = (97 psf × 28') + (78.57 psf × 28') f'_c = 4000 psi, f_y = 60 ksi

$w_u = 1.4(2.716) + 1.7(2.2)$
$= 7.54$ klf

Live Load Reduction

$L = 100\left(.25 + \frac{15}{\sqrt{2(784)}}\right) = 78.57$ psf

$A_T = 28' \times 28'$
$= 784\ ft^2$

For an interior Bay: CRSI 12-103

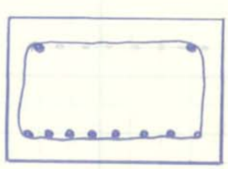
use a Joist-Band Beam ⇒ h = 20.5", b = 24"   2#11 top Reinf., 4#14 bottom reinf.

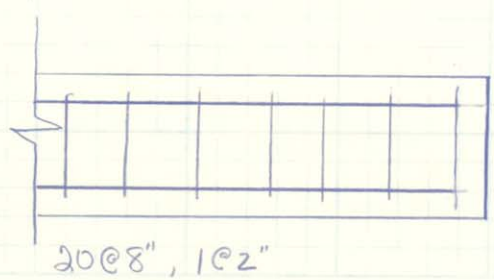
* Suggested reinf. is very large, redesign

$A_s = 9\ in^2$ → current
$A_s = 10.16\ in^2$ → 8-#10's
$b = 8(1.27) + 7(1.27) + 4$
$= 23.05" < 24"$ ∴ ok ✓ use 8#10's as bottom reinf.

allowable load:
8.2 klf > 7.54 klf ∴ ok ✓

Stirrups: 21 #5's: 1@ 2", 20@ 8" for each end





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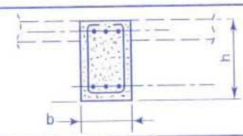
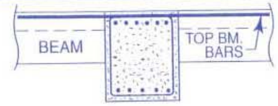
f'_c = 4,000 psi
f_y = 60,000 psi

JOIST-BAND BEAMS, INTERIOR SPANS





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

STEM h in.	STEM b in.	BARS[1] BOTTOM ℓ_n + 12 in.	BARS[1] BOTTOM 0.875 ℓ_n	Layers (2)	TOP	28 ft LOAD (4) k/ft	28 ft STIR. TIES (5)	28 ft ϕT_n ft-kips	28 ft $A\ell$ sq. in.	28 ft STEEL WGT lb.	30 ft LOAD (4) k/ft	30 ft STIR. TIES (5)	30 ft ϕT_n ft-kips	30 ft $A\ell$ sq. in.	30 ft STEEL WGT lb.	32 ft LOAD (4) k/ft	32 ft STIR. TIES (5)	32 ft ϕT_n ft-kips	32 ft $A\ell$ sq. in.	32 ft STEEL WGT lb.	34 ft LOAD (4) k/ft	34 ft STIR. TIES (5)	34 ft ϕT_n ft-kips	34 ft $A\ell$ sq. in.	34 ft STEEL WGT lb.	$+\phi M_n$ $-\phi M_n$ (6) ft-kip	DEFL. (C) (7) $\times 10^{-9}$ in.
20.5	24	2#8	1#8	1	4#8	3.4	113G	14	-	494	2.9	113G	14	-	523	2.6	113G	13	-	552	2.3	103G	13	-	574	182	307
				1			343C	55	1.6	741		363C	54	1.5	789		383C	54	1.5	837		413C	53	1.5	895	239	
		2#9	1#9	1	4#10	4.6	153F	14	-	705	4.0	153F	14	-	746	3.6	153F	13	-	788	3.1	163F	13	-	837	227	338
				1			214F	55	1.5	992		363C	54	1.5	981		383C	54	1.5	1042		413C	53	1.5	1113	365	
		2#11	1#11	1	5#11	6.9	173F	14	-	1046	6.0	183F	14	-	1118	5.3	193F	13	-	1190	4.7	193F	13	-	1255	338	284
				1			215F	55	1.5	1569		235F	54	1.5	1696		245F	54	1.5	1792		265F	53	1.5	1919	523	
		2#11	2#11	1	4#14	8.2	184F	14	-	1371	7.1	194F	14	-	1464	6.2	203F	13	-	1411	5.5*	213F	13	-	1497	435	253
				1			215F	55	1.5	1756		235F	54	1.5	1896		245F	54	1.5	2005		265F	53	1.5	2145	580	
	36	2#9	2#9	1	5#9	5.3	123F	24	-	761	4.6	123F	24	-	808	4.0	123F	24	-	854	3.6	123F	24	-	901	305	210
				1			563A	97	2.3	1300		603A	96	2.3	1392		643A	95	2.3	1484		683A	95	2.3	1577	375	
		2#10	2#10	1	5#11	7.8	163F	24	-	1065	6.8	163F	24	-	1130	5.9	173F	24	-	1204	5.3	173F	24	-	1270	379	227
				1			284D	97	2.3	1552		603A	96	2.3	1680		643A	95	2.3	1791		683A	95	2.3	1903	557	
		2#11	2#11	1	5#14	9.3	173F	24	-	1392	8.1	173F	24	-	1481	7.1	183F	24	-	1577	6.3	193F	24	-	1674	456	198
				1			215F	97	2.3	2001		304D	96	2.3	2004		324D	95	2.3	2137		344D	95	2.2	2270	754	
		2#14	2#14	1	6#14	12.2	184F	24	-	1923	10.7	194F	24	-	2055	9.4	204F	24	-	2186	8.3*	214F	24	-	2318	625	172
				1			345C	97	2.3	2845		365C	96	2.3	3032		385C	95	2.2	3219		265F	95	2.2	2902	871	
	48	3#8	3#8	1	6#10	7.5	123G	35	-	998	6.5	N/A	35	-	945	5.7	N/A	35	-	1007	5.1	N/A	35	-	1070	365	166
				1			**	142	3.2	882		**	141	4.3	945		**	140	0.0	1007		**	139	0.0	1070	564	
		3#9	3#9	1	6#11	9.3	153F	35	-	1250	8.1	153F	35	-	1328	7.1	153F	35	-	1406	6.3	163F	35	-	1494	454	174
				1			**	142	3.2	1105		**	141	3.1	1183		**	140	3.1	1261		**	139	2.8	1340	677	
		3#11	3#11	1	6#14	12.9	174F	35	-	1970	11.3	184F	35	-	2105	9.9	184F	35	-	2222	8.8	193F	35	-	2188	677	150
				1			245E	142	3.0	2634		265E	141	3.0	2833		384C	140	3.0	2859		414C	139	3.0	3053	921	
		4#11	3#11	1	8#14	15.8	185F	35	-	2625	13.8	195F	35	-	2803	12.1	204F	35	-	2761	10.7*	204F	35	-	2909	775	131
				1			425B	142	3.0	3809		455B	141	3.0	4080		485B	140	3.0	4350		295E	139	3.0	3722	1161	

(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 ℓ_n/360 are designated thus: * — ℓ_n/360 < deflection < ℓ_n/240
X — ℓ_n/240 < deflection < ℓ_n/180
Y — deflection > ℓ_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.

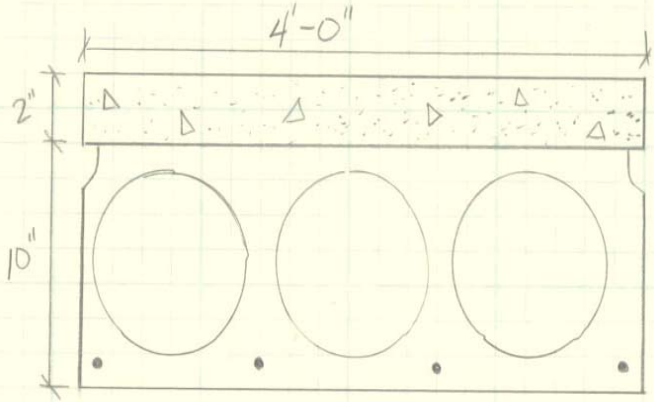
Other notation: N/A — STIRRUPS ARE NOT REQUIRED
** — MAXIMUM SPACING IS LESS THAN 3 INCHES. NOT RECOMMENDED
*** — SHEAR STRESS IS GREATER THAN $10\sqrt{f'_c}$
**** — TORSION STRESS EXCEEDS ALLOWABLE

(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.

Appendix A-3:

Pre-Stressed HollowCore Plank

Pre Stressed PreCast | Hollow-Core Plank | PCI Design Handbook



DL = 12 psf

LL = 100 psf

Serviceability = 1.0 DL + 1.0 LL = 112 psf

@ 28' span, 112 psf

⟹ use 4HC10+2 48-S strand Designation

4' wide plank
10" Plank depth
2" concrete topping

4 Strands
½" Ø Strands
Straight

Carries 113 psf > 112 psf ✓ ok

Camber = 0.3 in @ time of erection
0.0 in long-time camber

$I = 5328 \text{ in}^4$
$w_t = 370$ plf
93 psf
$f'_c = 5000$ psi

Deflection:

Strand Strength = f_y = 270 ksi

Strand Pattern Designation

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

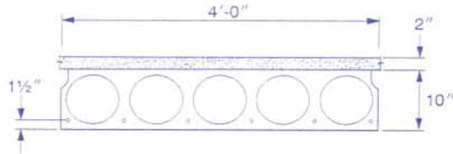
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

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

Key
239 — 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'_{ci} = 3,500 psi

Section Properties

		Untopped	Topp
A	=	259 in²	—
I	=	3,223 in⁴	5,328
y_b	=	5.00 in.	6.34
y_t	=	5.00 in.	5.66
S_b	=	645 in³	840
S_t	=	645 in³	941
b_w	=	10.50 in.	10.50
wt	=	270 plf	370
		68 psf	93
V/S	=	2.23 in.	

4HC10

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

Strand Designation Code	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42
48-S	239	212	188	168	150	134	120	107	96	86	76	68	61	54	48	42							
	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							
	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.3	−0.5							
58-S	280	263	245	219	197	177	160	144	130	118	107	96	87	79	71	64	58	52	46	41			
	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			
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	0.0	−0.1	−0.3	−0.5	−0.7			
68-S	289	272	255	242	231	217	199	180	164	149	136	124	113	103	94	86	78	71	64	58	53	48	43
	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
	0.7	0.7	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	−0.3	−0.6	−0.8
78-S	298	278	264	248	237	223	214	203	193	179	164	150	138	126	116	106	98	90	82	75	69	63	57
	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
	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.2	1.2	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.6	0.5	0.3	0.1	−0.1
88-S		287	270	257	243	229	220	209	199	189	183	174	162	149	137	126	117	107	99	91	84	78	71
		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
		1.0	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.5	1.5	1.5	1.5	1.4	1.4	1.3	1.3	1.2	1.1	0.9	0.8	0.6

Span, ft

4HC10+2

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

Strand Designation Code	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42
48-S	293	258	229	203	181	161	143	127	113	101	89	79	69	60	50								
	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								
	0.3	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								
58-S			297	268	241	216	194	175	157	142	128	115	103	92	79	68	58	48					
			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					
			0.4	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					
68-S				286	272	259	244	221	200	182	165	150	136	123	109	96	84	73	63	54			
				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			
				0.6	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			
78-S				295	278	265	250	239	226	218	201	184	168	154	138	124	111	98	87	77	67	58	49
				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
				0.8	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
88-S					287	271	259	245	232	224	213	202	193	179	163	148	134	121	110	99	88	78	69
					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
					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	0.5	0.3	0.1	−0.1	−0.3	−0.6

Span, ft

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

Inverted Tee Beam Design.

$W_u = 1.0(100psf \times 28ft) + 1.0(93psf \times 28)$

$= 5.404$ k/f

LL = 100psf
Reducible = 78.57
DL = 93psf

$L = L_o\left(.25 + \frac{15}{\sqrt{2(784)}}\right)$

$= 78.57$ psf

$A_T = 28 \times 28 = 784$

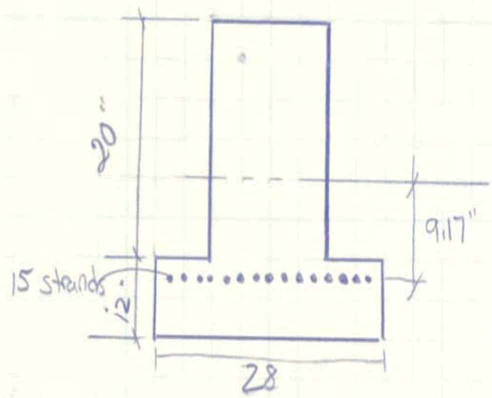
From PCI 2-44

Span of 28ft

use: 28IT32 w/ 15 strands

e = 9.17"

Area = 576in²



Safe Service Load:

5474 plf > 5404 plf ∴ Strong enough

use Precast, Prestressed 28IT32

$f'_c = 5000psi$

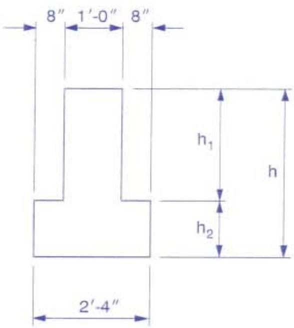
$f_{pu} = 270$ ksi

1/2" Ø

low relaxation Strand

INVERTED TEE BEAMS

Normal Weight Concr



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

½ in. diameter
low-relaxation strand

Section Properties								
Designation	h in.	h_1/h_2 in.	A in^2	I in^4	y_b in.	S_b in^3	S_t in^3	w pl
28IT20	20	12/8	368	11,688	7.91	1,478	967	3
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	5
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	5
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	6
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	6
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	7
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	8
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	8
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	9
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	9
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,0

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 80
top tension has been allowed, therefore additional top reinforcement is required.

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

Key
6,929 — 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

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

2–44 PCI Design Handbook/Fifth Edition

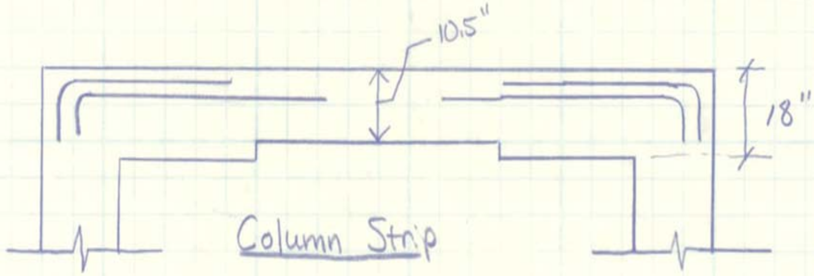
Appendix A-4:

One-Way Concrete Joist

Daniel Hancock
Structural Option
Dr. Hanagan

S&T Bank
Corporate Headquarters
Indiana, PA

Two-Way Solid Flat Slabs – w/ drop Panels CRSI Handbook



Superimposed DL = 12 psf

LL = 100 psf

$w_u = 1.4DL + 1.7LL$ (for CRSI tables)

$= 1.4(12) + 1.7(100)$

$= 186.8$ psf

Enter tables w/ span = 28', load = 186.8 psf

⟹ use

Slab: 10.5"

Drop Panel: 9'-4" x 9'-4"
depth: 7.5"

Column: 18" x 18"

Reinforcement:

Top: 15-#6, Bottom: 12-#6 } Column Strip

Top: 13-#5, Bottom: 11-#5 } middle Strip

total steel: 3.07 psf

Total depth: 18"

$f'_c = 4000$ psi

$f_y = 60$ ksi

S&T Bank
Corporate Headquarters
Indiana, PA

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL — With Drop Panels — No Beams

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.)	(3) Square Column γ_f	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)
h = 10.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS														
26	100	6.00	8.67	12	0.760	12-#5 2	15-#5	15-#5	10-#5	10-#5	2.46	151.6	303.2	408.1
26	200	6.00	8.67	15	0.798	12-#5 4	11-#7	14-#6	13-#5	11-#5	3.08	198.2	396.4	533.6
26	300	7.50	8.67	18	0.679	12-#5 2	18-#6	12-#7	9-#7	10-#6	3.83	244.7	489.4	658.8
26	400	9.00	8.67	20	0.632	12-#5 2	16-#7	13-#7	14-#6	9-#7	4.39	291.2	582.3	783.9
26	500	9.00	10.40	22	0.707	14-#5 2	12-#9	12-#8	12-#7	10-#7	5.17	336.6	673.1	906.1
26	600	9.00	10.40	26	0.701	16-#5 3	17-#8	13-#8	9-#9	9-#8	6.00	379.8	772.7	1022.5
27	100	6.00	9.00	12	0.797	12-#5 3	9-#7	12-#6	12-#5	10-#5	2.66	170.3	340.6	458.5
27	200	7.50	9.00	16	0.651	12-#5 1	12-#7	20-#5	15-#5	9-#6	3.25	222.6	445.2	599.3
27	300	9.00	9.00	18	0.634	12-#5 2	15-#7	12-#7	10-#7	11-#6	3.96	274.9	549.8	740.1
27	400	9.00	9.00	20	0.741	14-#5 4	14-#8	12-#8	9-#8	10-#7	4.88	327.9	655.8	882.8
27	500	9.00	10.80	25	0.694	16-#5 3	13-#9	13-#8	9-#9	15-#6	5.70	375.4	750.8	1010.7
28	100	7.50	9.33	12	0.750	13-#5 2	19-#5	18-#5	13-#5	11-#5	2.74	191.0	382.0	514.2
*28	200	7.50	9.33	16	0.767	13-#5 4	18-#6	16-#6	12-#6	10-#6	3.50	249.3	498.5	671.1
28	300	9.00	9.33	18	0.745	13-#5 5	13-#8	26-#5	11-#7	17-#5	4.32	308.1	616.1	829.4
28	400	9.00	11.20	23	0.722	15-#5 4	13-#9	16-#7	10-#8	11-#7	5.20	365.1	730.3	983.1
28	500	9.00	11.20	28	0.644	17-#5 2	18-#8	14-#8	12-#8	10-#8	5.95	415.8	831.6	1119.4
29	100	7.50	9.67	12	0.787	13-#5 3	22-#5	14-#6	10-#6	12-#5	2.88	212.8	425.5	572.8
29	200	9.00	9.67	16	0.702	13-#5 3	15-#7	23-#5	10-#7	11-#6	3.67	277.7	555.4	747.6
29	300	9.00	9.67	19	0.763	14-#5 5	12-#9	15-#7	10-#8	19-#5	4.75	342.7	685.5	922.7
29	400	9.00	11.60	25	0.702	17-#5 3	14-#9	14-#8	12-#8	10-#8	5.68	405.3	810.5	1091.1
30	100	9.00	10.00	12	0.722	14-#5 1	17-#6	14-#6	16-#5	13-#5	3.00	236.8	473.6	637.6
30	200	9.00	10.00	16	0.763	14-#5 4	13-#8	18-#6	11-#7	17-#5	3.99	308.5	617.1	830.7
30	300	9.00	10.00	22	0.691	16-#5 3	13-#9	17-#7	18-#6	15-#6	5.07	377.6	755.2	1016.6
30	400	9.00	12.00	28	0.700	18-#5 5	16-#9	15-#8	10-#9	18-#6	5.96	444.1	888.3	1195.7
31	100	9.00	10.33	12	0.777	14-#5 3	11-#8	16-#6	13-#6	15-#5	3.29	261.9	523.8	705.1
31	200	9.00	10.33	18	0.749	14-#5 5	12-#9	15-#7	12-#7	19-#5	4.29	339.6	679.2	914.3
31	300	9.00	10.33	24	0.731	17-#5 6	18-#8	14-#8	12-#8	13-#7	5.38	416.0	832.0	1120.0
31	400	9.00	12.40	31	0.697	14-#6 4	17-#9	14-#9	11-#9	12-#8	6.43	483.9	967.9	1302.9

SQUARE INTERIOR PANEL
With Drop Panels(2) — No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	(3) Square Column Size (in.)	Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
h = 10.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS								
26	100	12	14-#5	10-#5	10-#5	10-#5	2.29	0.931
26	200	18	18-#5	13-#5	10-#5	10-#5	2.67	0.931
26	300	21	15-#6	9-#7	9-#6	11-#5	3.39	0.944
26	400	23	12-#7	14-#6	15-#5	13-#5	3.82	0.958
26	500	26	26-#5	12-#7	10-#7	15-#5	4.41	0.995
26	600	26	12-#8	11-#8	11-#7	18-#5	5.19	0.995
27	100	12	16-#5	12-#5	10-#5	10-#5	2.40	0.931
27	200	18	14-#6	15-#5	12-#5	10-#5	2.85	0.944
27	300	22	15-#6	13-#6	10-#6	12-#5	3.40	0.958
27	400	23	18-#6	9-#8	9-#7	15-#5	4.24	0.958
27	500	26	12-#8	11-#8	14-#6	9-#7	4.93	0.995
28	100	12	16-#5	13-#5	11-#5	11-#5	2.48	0.944
*28	*200	18	15-#6	12-#6	13-#5	11-#5	3.07	0.944
28	300	22	13-#7	21-#5	16-#5	10-#6	3.75	0.958
28	400	24	15-#7	18-#6	10-#7	12-#6	4.51	0.995
28	500	27	13-#8	12-#8	12-#7	10-#7	5.25	0.995
29	100	12	18-#5	14-#5	11-#5	11-#5	2.52	0.944
29	200	19	15-#6	19-#5	10-#6	12-#5	3.13	0.958
29	300	22	26-#5	17-#6	10-#7	15-#5	4.01	0.958
29	400	24	13-#8	12-#8	12-#7	18-#5	4.95	0.995
30	100	12	18-#5	16-#5	12-#5	11-#5	2.57	0.958
30	200	19	17-#6	21-#5	16-#5	10-#6	3.43	0.958
30	300	22	16-#7	11-#8	14-#6	12-#6	4.48	0.958
30	400	26	14-#8	10-#9	10-#8	20-#5	5.16	0.995
31	100	12	20-#5	18-#5	14-#5	12-#5	2.77	0.958
31	200	19	26-#5	23-#5	13-#6	15-#5	3.60	0.958
31	300	22	17-#7	21-#6	12-#7	19-#5	4.68	0.958
31	400	29	16-#8	11-#9	11-#8	12-#7	5.65	0.995

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