Hilton Baltimore Convention Center Hotel

Western Podium

Baltimore, MD

CHRIS SIMMONS

Structural Option

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Technical Report 2

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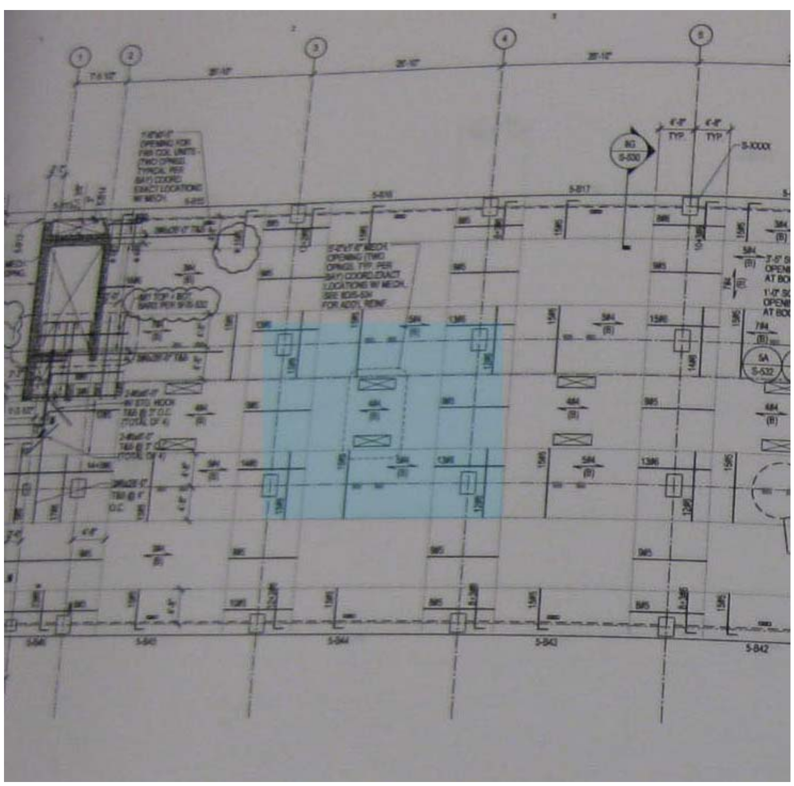
EXECUTIVE SUMMARY

In this pro-con study of floor systems for the Baltimore Hilton four floor systems were analyzed. Those systems were the existing two-way flat plate slab, one-way slab, hollow-core precast panels on concrete beams, and composite steel framing. After the analysis of the systems it was found that the existing two-way flat plate slab was the best option of the four. It had the lowest total system depth, a short lead time and it cost the least out of the four. It was found that a fifth system could be analyzed to see how it compared to the existing system. The fifth system would be a pre-stressed, post-tensioning system. Hopefully the fifth system will provide a viable option for an alternate floor system.

INTRODUCTION

The Hilton Baltimore Convention Center Hotel (HBCCH) is located right in downtown Baltimore next to the Baltimore Orioles stadium Camden Yards, and located blocks away from Inner Harbor. HBCCH is broken up into two podiums, East and West. The eastern podium is a 4-story building that houses a junior ballroom, meeting rooms and a multipurpose restaurant. The western podium is a 21-story building that houses the main hotel lobby, parking garage, grand ballroom with corresponding kitchen, meeting rooms, pool/health club, and 757 hotel rooms. The grand ballroom has moveable partitions located in the ceiling that allow multiple events to take place there. The western podium offers over 900,000 SF of hotel space. The structure of the western podium consists of concrete beams, columns and shear walls to resist lateral loading. The green roof above the grand ballroom is supported by special joists and while the pool above the grand ballroom is supported by steel beams.

For the second technical report of HBCCH an existing bay's floor system was analyzed and compared to three other floor systems. The existing floor system is a two-way flat plate slab and the alternative systems are a one-way slab, hollow-core precast panels on concrete beams, and composite steel framing. Figure 1 shows the selected bay to be analyzed. The bay is on the 5th floor.



FOUNDATION SYSTEMS

The foundation of the western podium consists of caissons and spread footings. The spread footings will bear on firm natural soils and have a minimum bearing capacity of 4ksf. The drilled caissons will have straight shafts to bear on gneiss rock and have a minimum safe bearing capacity of 100ksf. The depths of the bottoms of the caissons vary from 14 feet all the way up to 32 feet below level B2's floor slab. The compressive strength of the drilled caissons and spread footings are 3500 psi, while the caisson caps that the columns bear on have a compressive strength of 4000 psi. A typical caisson section is shown in figure 2.

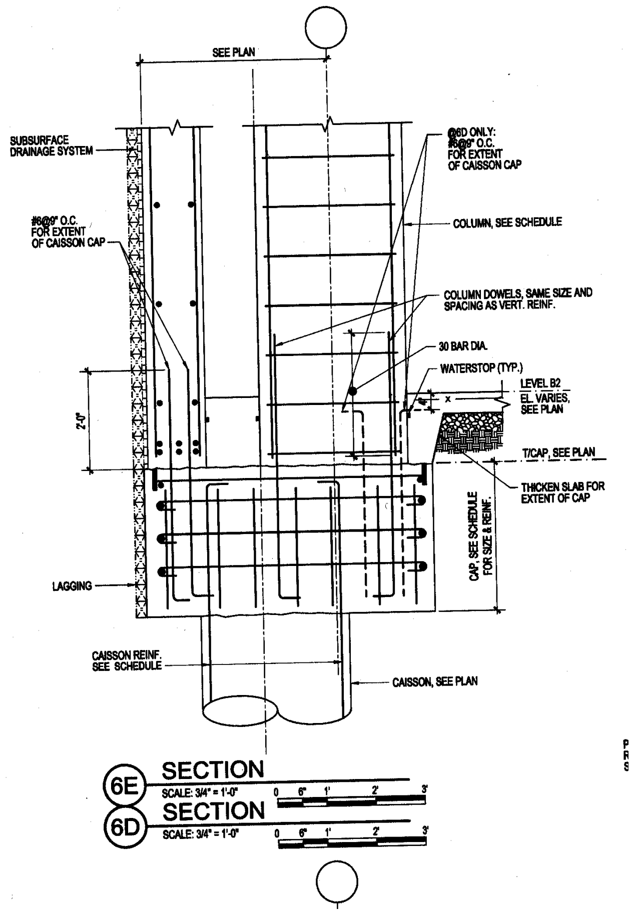


Fig. 2

FLOOR SYSTEM

The floor system consists of two-way slabs whose thicknesses range from 8" thick on the floors with hotel rooms to 11" in the underground parking garage. The slabs shall are reinforced with 6x6-W1.4xW1.4 WWF, except for the slab-on-grade which is reinforced with 6x6-W2.1xW2.1 WWF as seen in figure. 3. Drop panels are located on the B1, 1st, Mezzanine level, 2nd, 3rd, and 15th floors. The drop panels vary from 5" up to 11" in thickness. Typical spans for floors consisting of hotel rooms are 26'-10". The column system layout is a very uniform layout consisting of typical exterior bays of 26'-10" x 18'-8" and interior bays of 26'-10" x 19'-7". All columns consist of either a gravity resisting member or a combination of lateral and gravity resisting members. Column sizes vary from 12” x 18” columns to 44” x 30” Columns. Sloped columns can be found on the second and third floors of the western podium. A typical sloping column is shown in figure 2 as well as a typical concrete column post on beam detail is shown in figure 2.

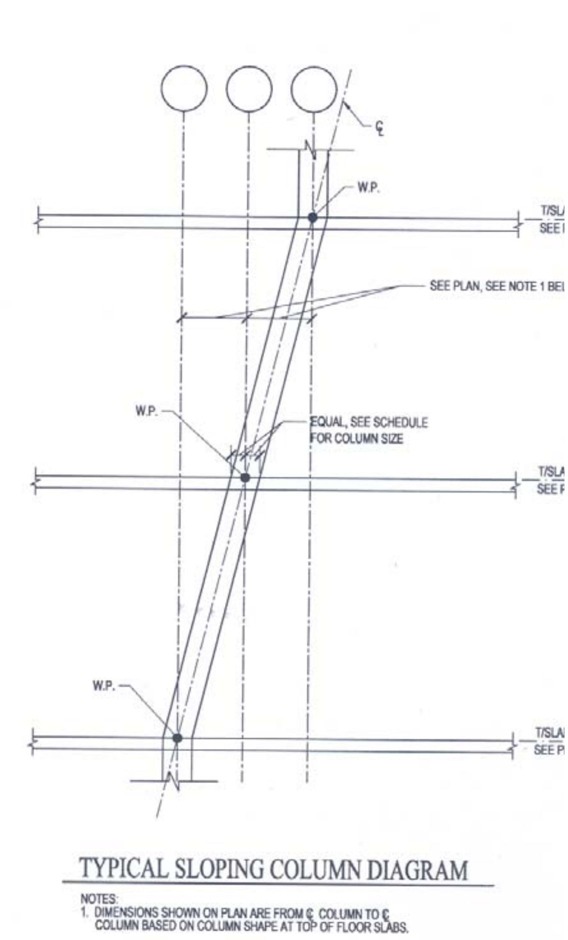


Fig. 3

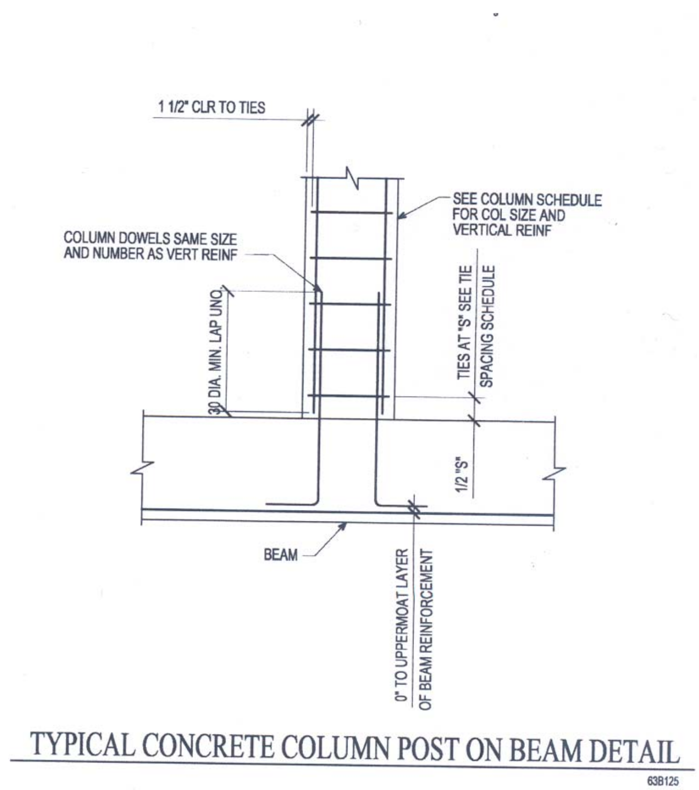


Fig. 4

LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consist of rigid moment frames that are inherent in monolithic concrete-framed construction. A shear wall system is used as well for resisting wind and seismic loads. Shear walls are located around the elevator shafts and stairwells. The shear walls thickness ranges from 12” up to 18”. A shear wall plan for the elevator is shown in figure 5.

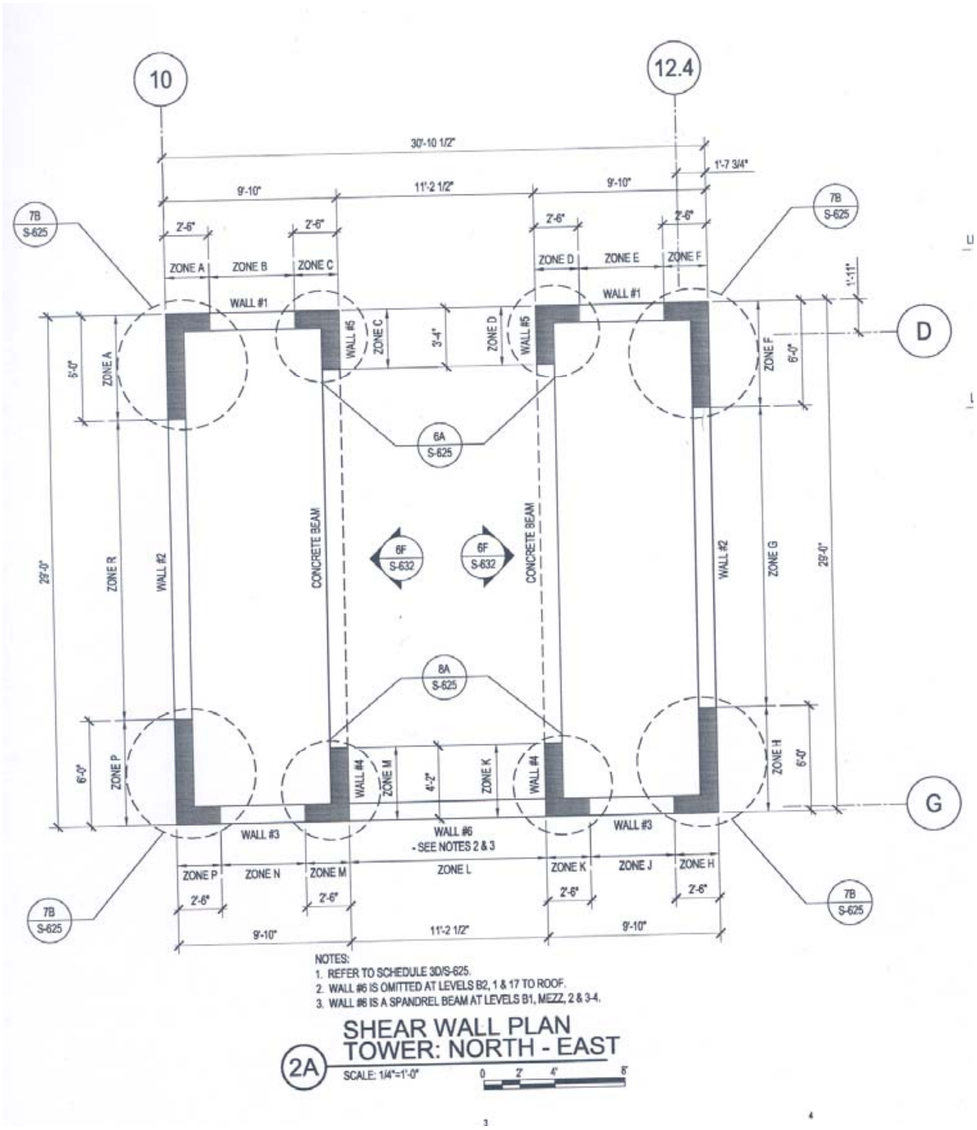


Fig. 5

DESIGN LOADS

All of the design loads for this technical report were calculated referencing ASCE 7-05: *Minimum Design Loads for Buildings and Other Structures*. All dead, live and snow loads can be seen in Table 1 below.

Table 1 - Gravity Loads			
Description	ASCE 7-05	RTKL Value	Design Value
DEAD (DL)			
Concrete	150pcf	150pcf	150pcf
Ceiling, Mech, Ducts, etc.	20-25psf	5-20psf	20psf
LIVE (LL)*			
Private Hotel Rooms	40psf	40psf	40psf
Ballroom	100psf	100psf	100psf
Corridors (first floor, main lobby)	100psf	100psf	100psf
Corridors (serving private hotel rooms)	100psf	40psf	100psf
Aerobic/Exercise Rooms	100psf	100psf	100psf
Pool Deck	75psf	80psf	75psf
Green Roof	100psf	100psf	100psf
Exterior Balconies (East Tower)	100psf	100psf	100psf
Roofing	20psf	30psf	20psf
SNOW (S)			
Snow	20psf	20psf	20psf
* Live load reductions were not taken into consideration in the design.			

Almost all lateral loads were calculated by hand and inserted into the tables on the following pages. The hand calculations can be found on pages 31-35 in the Appendix. After calculating the lateral loads it was concluded that Seismic controlled over Wind.

FLOOR SYSTEMS

Two-Way Reinforced Flat Plate - Existing

This two-way reinforced flat plate system designed by RTKL includes an 8" NWC slab with 15 #5 top reinforcing as well as 5 #4 and #4@10" o.c. mat reinforcing for the bottom reinforcing. The top reinforcing changes to #6 bars around columns to help resist the increased moment.

A typical interior bay analysis, done at the 5th floor, was completed using the Direct Design Method reviewed in *Reinforced Concrete: Mechanics & Design* by Wight and MacGregor with the loads determined by ASCE 7-05. The bay was split into two frames, Frame A and Frame B noted in figure 6, which were checked for minimum slab thickness and reinforcement design. The slab thickness of 8" did not exceed the minimum of 9.27" so deflection was checked using RAM (deflection results can be seen in the appendix under two-way flat plate). The reinforcement was found to be larger than that designed due to mat reinforcing not being taken into account. The typical bay was taken as 27'x20' to simplify calculations rather than looking at the 26'10"x18'8" bays and 26'10"x19'7" bay. There are calculations reviewing the wide beam action and punching shear within the slab which did not prove to be an issue and did not require any additional shear reinforcement. Deflections found using RAM were found to be within the limits of l/480 for long-term deflection. All supporting calculations for this analysis can be found in Appendix A.

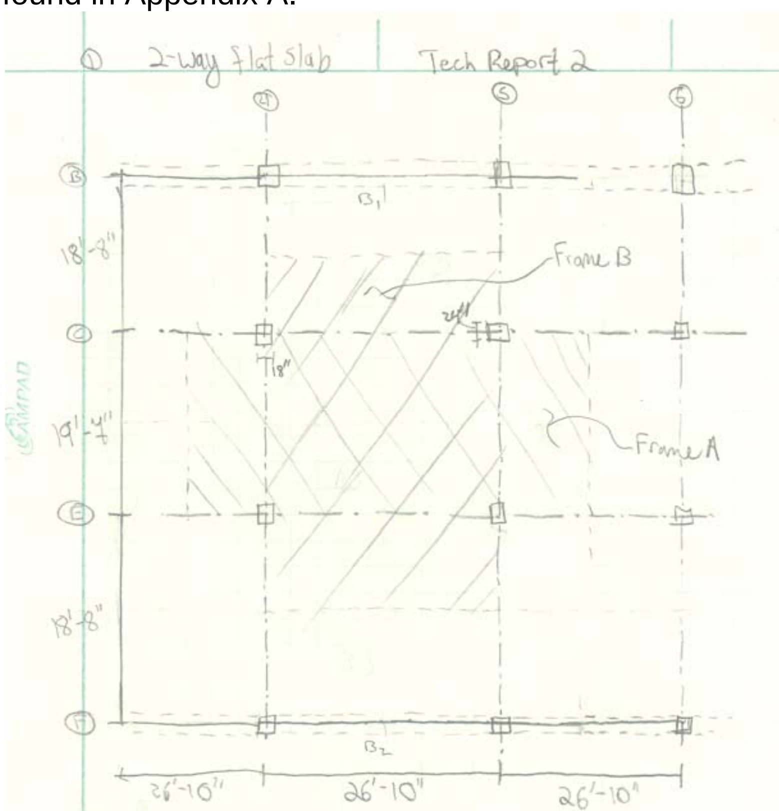


Fig. 6

ONE-WAY SLAB – OPTION #1

This system was designed by hand calculations per ACI 318-05 using an f'c of 4,000psi and a f_y of 60,000psi. The slab thickness was calculated to be 9" with cracking controlling the flexural rebar placement. #4 rebar was placed at 12" o.c. at the top and bottom of the slab for flexural strength. To allow for a 2 hour fire rating between floors a clear cover of ¾" was used as per ACI 318-05. The slab spans over the short direction of the 26'10"x19'9" and 26'10"x18'8" bays to a concrete beam on either side. The bays were used to be 27'x20' for simplification of calculations. The concrete beam was designed to have a depth of 21" and a width of 16". The beam contains 14 #11 bars for flexural strength. Additional details and calculations can be found in the Appendix section of this report.

HOLLOW-CORE PRECAST PANEL ON CONCRETE – OPTION #2

This system was designed using PCI Edition 6. As per the design guide it was found that a 96-S, 4'-0"x6" with a 2" normal weight topping is appropriate. The superimposed service load in psf for that particular member is 68psf which is more than the calculated load. The precast slab is supported by concrete beams that frame into the concrete columns. The concrete beam was sized using ACI 318-08 and found to be 19” in depth and 14” in width.

Fig. 7 – Precast Hollow Core Planks

COMPOSITE STEEL FRAMING – OPTION #3

This system was designed by using RAM Steel and the Vulcraft Steel Deck Design Guides, 2008. The bay designed consists 2” normal weight concrete (145pcf) on 2VLI18 3”x18Ga. steel decking. The composite steel decking was found to hold 228psf which is more than the required load. The beams were designed using RAM Steel and found to be W8x10, W16x26 and W10x12 with the beams requiring a maximum of 18 shear studs, 22 shear studs, and 19 shears studs, respectively. A camber of 1-1/4” was used.

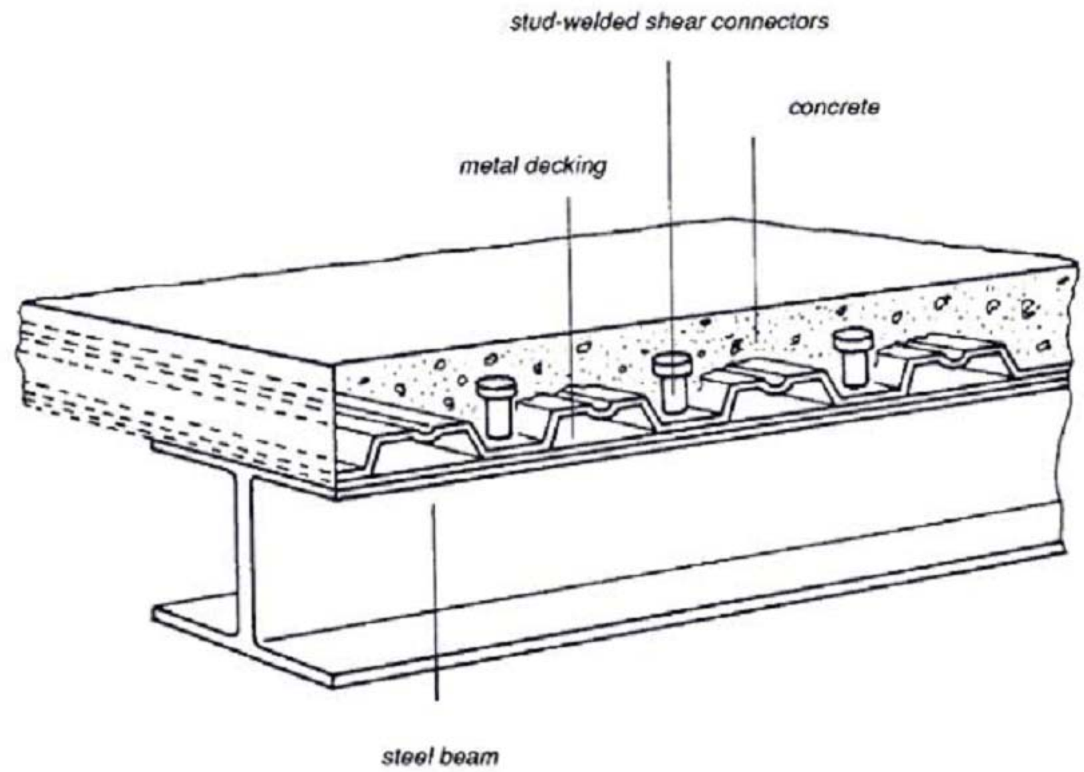


Fig. 8 -Typical Composite Steel Detail

FLOOR SYSTEM COMPARISONS

A comparison of all of the systems was performed for this report. Theses comparisons consisted of factors such as weight, slab depth, system depth, foundation impact, fire rating, fire protection, column size, effect on the column grid, architectural impact, constructability, formwork, lead time, and system cost. The comparisons were put into the following table. Strong advantages for the system were highlighted in green, while the strong disadvantages were listed in red. Values that were neither a strong advantage nor a strong disadvantage were left blank.

Comparison Criteria	Existing	Option #1	Option #2	Option #3
	Two-way Reinforced Flat Plate	One-way Reinforced Slab	Hollow Core Precast Panel on Concrete	Composite Steel Frame
Slab Self Weight	112.5 psf	112.5 psf	74 psf	62.5 psf
Slab Depth	9"	9"	8"	5"
System Depth	9"	21"	27"	20.7"
Foundation Impact	-	Little	Moderate	Moderate
Fire Rating	2 hour	2 hour	1.5-2 hour	1.5-2 hour
Fire Protection	None	None	Spray	Spray
Column Size	18x24	18x24	18x24	W10x33
Effect on Column Grid	-	None	Little	None
Architectural Impact	Existing	Greater Floor Depths	Deepest Girders	Greater Floor Depths
Constructability	Medium	Medium	Easy	Easy
Formwork	Yes	Yes	No	No
Lead Time	Short	Short	Long	Long
System Cost	$8.78/SF	$9.04/SF	$14.23/SF	$10.15/SF
More In Depth Analysis	Yes	No	No	No

CONCLUSION

After analyzing the different floor systems it was found that the current two-way flat plate system is the best for the building. The only other options that have potential to be considered are a two-way pre-stressed, post-tensioning system. The only downside of the existing system is the weight of the system which impacts seismic loads in the long run. The two-way system is the cheapest of the four systems analyzed, has the least overall system depth and a short lead time.

The one-way slab turned out not to be a viable option even though it's cost is low and the slab thickness is the same as the two-way flat plate system. Also there wouldn't be an effect on the column grid and a minimal effect on the foundation system. The downside to the one-way slab is the system depth is about twice as much as that of the two-way system which is going to cause greater floor depths which causes lower floor-to-ceiling heights.

The hollow-core precast panel on concrete beam system also turned out to not be a viable option. Even though the precast system weighed less, had a lower slab depth and easier constructability compared to the existing system it had a higher cost, higher total system depth. It would impact the foundation system, require spray on fire-protection and has a long lead time and some effect on the column grid.

The composite steel frame system turned out to not to be a viable option as well. Even though it has least weight and slab depth out of the four systems analyzed it has the second highest cost, third highest total slab depth and a long lead time compared to the other systems which makes it not viable.

APPENDIX

Two-way Flat Slab (Existing)

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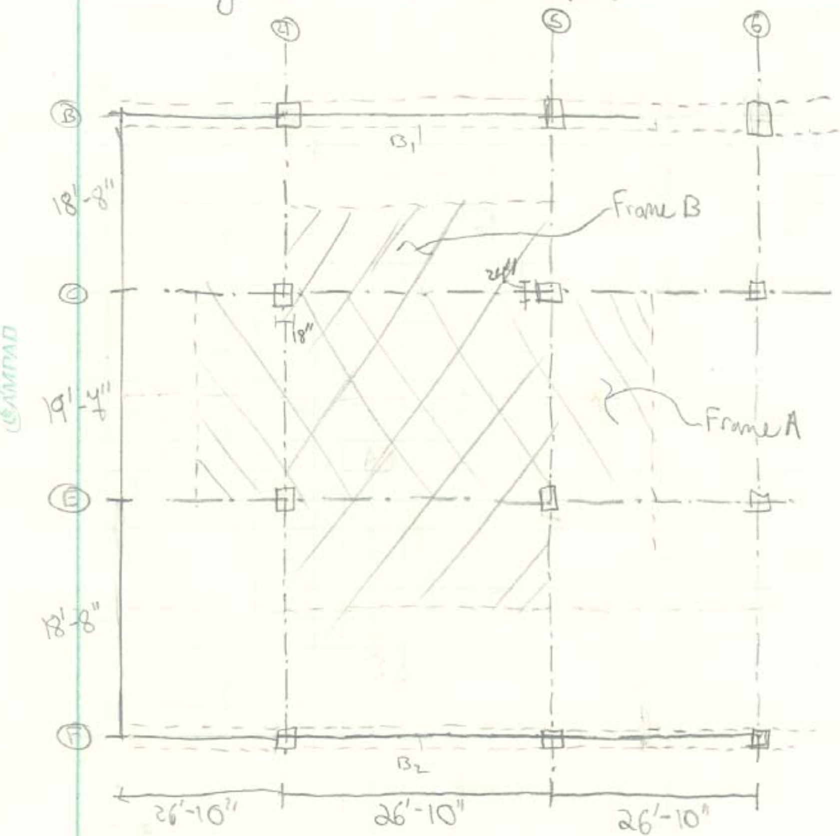
1 2-Way flat slab | Tech Report 2



$f'_c = 4000$ psi
$f_y = 60000$ psi

$w_L = 40$ psf
$w_D^{\text{superimposed}} = 20$ psf
$w_{conc} = 150$ pcf

Columns are 18"x24"
8" thick slab
two-way slab, no int. beam

#4 @ uniform spacing

Direct Design Method (DDM)

Assuming bays are 24' x 20' to simplify calculation

$w_u = 1.2(20) + 1.2(\frac{10}{12} \times 150) + 1.6(40) = 238$ psf

(2)

Panel A

18" x 24" columns

§ 13.2.1 ⇒ Column Strip is a width on each side of a column centerline equal to the smaller of $0.25\ell_2$ or $0.25\ell_1$.

$0.25\ell_2 = 0.25(20) = 5'$ ⟶ 10' column strip

$0.25\ell_1 = 0.25(27) = 6.75'$

$M_u = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(20)(27 - \frac{18}{12})^2(.238) = 386.9^{1k}$

$\alpha = 0$ since $I_b = 0$ for no beams

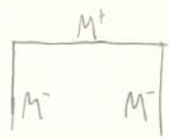
Min. thickness of slab without beams [Table 9.5c]

Interior Panel without drop panels → $\ell_n/33 = \frac{(27 - 18/12)(12)}{33} = 9.27" >$ design 8" ∴ need to check deflection

Δ_{live} from PCA Slab = 0.064"

$\ell/360 = \frac{27 \times 12}{360} = .9" > 0.064"$ ∴ ok

Frame A



$M^- = 0.65 M_u = -251.5^{1k}$

$M^+ = 0.35 M_u = +135.4^{1k}$

$\ell_2/\ell_1 = \frac{20}{27} = .74$, $\alpha\frac{\ell_2}{\ell_1} = 0$

§13.6.4.1	ℓ_2/ℓ_1	0.5	0.74	1.0
	$\alpha\ell_2/\ell_1 = 0$	75	$\boxed{75}$	75

75% of M^- to CS = -188.6^{1k}

25% of M^- to MS = -62.9^{1k}

3

§13.6.4.4

l_2/l_1	0.5	0.74	1.0
$\alpha l_2/l_1$	60	$\boxed{60}$	60

60% of M^+ to C.S. = $81.2^{'k}$
40% of M^+ to MS. = $54.2^{'k}$

	M^-	M^+	M^-
Total Moment	-251.5	135.4	-251.5
CS	-188.6	81.2	-188.6
MS	-62.9	54.2	-62.9

Panel A Total Width = 20'
CS = 10' = 120"
MS = 10' = 120"
#4 $d_s = 8 - \frac{3}{4} - \frac{1}{2}(.5) = 7"$
$d_L = 7 - 0.5 = 6.5"$

Design Slab Reinforcement in CS

Item	Description	Left	Interior Span Mid	Right
1	M_u (1k)	-188.6	81.2	-188.6
2	CS width b (in)	120	120	120
3	Effective depth, d (in)	6.5	6.5	6.5
4	$M_n = M_u/0.9$	-209.6	90.2	-209.6

$* \text{Check } d_{min} = \sqrt{\frac{M_n(1000)(12)}{\rho f_y b(1-0.59\rho\frac{f_y}{f_c})}}$ where $\rho_{max} = .0206$ for $f'_c = 4$ ksi and $f_y = 60$ ksi, $\epsilon_t = 0.004$

$d_{min} = \sqrt{\frac{188.6(12000)}{(0.0206)(60000)(120)(1-0.59(.0206)(\frac{60}{4})}} = 1.02" < 6.5" \therefore ok$

Item	Description	Left	Interior Span Mid	Right
5	$R_u = \frac{\lvert M_n \rvert}{bd^2}$	497	214	497
6	$\rho_{req'd}$ (Table A-3 woo)	0.009	.0037	.009
7	$A_{s\,req'd} = \rho bd$	$\boxed{7.02}$	$\boxed{2.89}$	$\boxed{7.02}$
8	$A_{s\,min} = 0.002bt$	1.92	1.92	1.92
9	$N = \frac{\text{larger 7) or 8)}}{A_{s\,bar}}$	$\boxed{36}$ 35.1	$\boxed{15}$ 14.45	$\boxed{36}$ 35.1
10	$N_{min} = \frac{\text{width of slab}}{2t}$	8 7.5	8	8

(4)

Calculated	RTKL
Int: $M_L^- = 36\#4$	5#4 + #4@10 E.W. Continuous bottom mat reinforcing
$M^+ = 15\#4$	5#4 + #4@10 E.W. Continuous bottom mat reinforcing
$M_R^- = 36\#4$	5#4 + #4@10 E.W. Continuous bottom mat reinforcing

The calculated values are all larger than what RTKL used but that could be reduced with the introduction of bottom mat reinforcing.

Design of Slab Reinforcement in MS

Item	Description	M^-	M^+
1	M_u (1K)	-62.9	54.2
2	MS width b (in)	120"	120"
3	effective depth	6.5"	6.5"
4	$M_n = M_u/0.9$	-69.9	60.2
5	$R = \frac{\lvert M_n \rvert}{bd^2}$	166	143
6	ρ from Table A-3 (4000)	.0029	.0025
7	$A_{s\,req'd} = \rho bd$	$\boxed{2.26}$	$\boxed{1.95}$
8	$A_{s\,min} = .002bt$	1.92	1.92
9	$N = \frac{\text{larger 7) or 8)}}{\text{area}}$	$\boxed{12}$ 11.3	$\boxed{10}$ 9.6
10	$N_{min} = \frac{\text{width of slab}}{2t}$	7.5→8	8

Calculated	RTKL
12#4	5#4 + Bottom mat reinforcement

My calculated values are larger because no mat reinforcement was taken into account in calculations

5

Panel B

18x24 columns
CS = 120"
MS = 14' = 204"

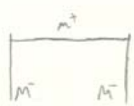
$M_u = \frac{1}{8} w l_2 l_n^2 = \frac{1}{8}(24)(.238)(20 - \frac{24}{12})^2 = 260'^k$

$\alpha = 0$ since $I_b = 0$ No interior beams

Minimum thickness of slab without interior beams [Table 9.5c]

$\frac{l_n}{33} = \frac{(20-\frac{24}{12})(12)}{33} = 6.5" < 8"$ ∴ ok

Frame B



$M^- = 0.65 M_o = -169'^k$

$M^+ = 0.35 M_o = 91'^k$

§13.6.4.1

l_2/l_1	1.0	1.35	2.0
$\alpha l_2/l_1 = 0$	75	$\boxed{75}$	75

75% of M^- to CS = -126.75

25% of M^- to MS = -42.25

§13.6.4.4

l_2/l_1	1.0	1.35	2.0
$\alpha l_2/l_1 = 0$	60	$\boxed{60}$	60

60% of M^+ to CS = $54.6'^k$

40% of M^+ to MS = $36.4'^k$

	M^-	M^+
Total Moment	-169	91
CS	-126.75	54.6
MS	-42.25	36.4

Panel B: Total Width = 24'
CS = 10' = 120"
MS = 14' = 204"

$d_s = 7"$
$d_L = 6.5"$

(6)

Design of Slab Reinforcement in CS

Item	Description	Int M^-	Int M^+
1	M_u (lk)	-126.45	54.6
2	CS width, b (in)	120"	120"
3	Effective depth	6.5"	6.5"
4	$M_n = M_u/0.9$	-140.83	60.67
5	$R = \lvert M_n \rvert / bd^2$	334	144
6	ρ from Table A-3 (UDD)	0.006	.002
7	$A_{s\,req'd} = \rho bd$	$\boxed{4.68}$	1.56
8	$A_{s\,min} = .002bt$	1.92	$\boxed{1.92}$
9	N = larger of 7) or 8) / area	23.4 → $\boxed{24}$	9.6 → $\boxed{10}$
10	$N_{min} = \frac{\text{width of CS}}{24}$	7.5 → 8	8

Calculated

Int: $M^- = 24$ #4

$M^+ = 10$ #4

RTKL

5#4 + Bottom Mat Reinforcement

5#4 + Bottom Mat Reinforcement

My calculated values are larger than RTKL because bottom mat reinforcement was not used in calcs.

Design of Slab Reinforcement in MS

Item	Description	M^-	M^+
1	M_u (lk)	-42.25	36.4
2	MS width, b	204"	204"
3	Effective depth	6.5"	6.5"
4	$M_n = M_u/0.9$	-46.9	40.4
5	$R = \frac{\lvert M_n \rvert}{bd^2}$	66	57
6	ρ from table A-3 (NDD)	.00112	.00096
7	$A_s = \rho bd$	1.49	1.24
8	$A_{s_{min}} = .002bt$	$\boxed{3.26}$	$\boxed{3.26}$
9	$N = \frac{\text{larger of 7) or 8)}}{\text{area}}$	16.3 → $\boxed{17}$	16.3 → $\boxed{17}$
10	$N_{min} = \frac{\text{width of MS}}{24}$	12.95 → 13	13

7

Calculated	RIKL
17 #4	5#4

Calculated values are larger due to not accounting for bottom mat reinforcing

Shear Checks

- Wide Beam Action

$\frac{27'}{2} - \left(\frac{6.5}{12}\right) = 12.96'$ critical section width

$w_u = 238 \text{ psf} = .238 \text{ ksf}$

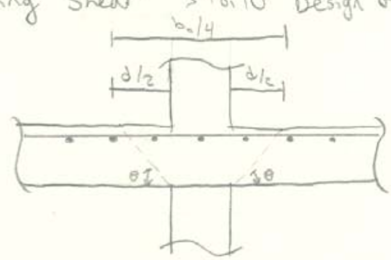
$V_u = w_u \times \text{Area} = .238(12.96 \times 20) = 61.7^k$

$V_n = 2\sqrt{f'_c}\, bd = 2\sqrt{4000}(20)(6.5)(12) = 197.33^k$

$\phi V_n = 0.75(197.33) = 148^k$

$\phi V_n = 148^k > V_u = 61.7^k \therefore ok$

- Punching Shear §18.10 Design of concrete structures



Checking Column
4C + 8" slab
- 18×24
- #4 in both directions
- $f'_c = 4$ ksi

$V_c = 4\sqrt{f'_c}\, b_o d$ [Eq 13.11a]

$d = 8" - \frac{3}{4} - \frac{1}{2}(.5) = 7"$

$\frac{b_o}{4} = 2(d/2) + 24" = 2(7/2) + 24 = 31 \rightarrow 124"$

$V_c = 4\sqrt{4000}(124")(7) = 219.6^K \leftarrow$ Controls

8

Must be $\le V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$

$\alpha_s = 40$ for int. columns

$= \left(\frac{40(7)}{124} + 2\right)\sqrt{4000}\,(124)(7) = 233.76^k$

Must be $\le V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1.0}\right)\sqrt{4000}(124)(7) = 329.4^k$

$\beta_c = 1.0$ sq. col.

$V_u = (.238)\left(20 \times 27 - \frac{18 \times 24}{12}\right) = 77.1^k$

$\phi V_c = 0.75(219.6) = 164.7 > V_u = 77.1^k \therefore$ ok

No additional shear reinforcement needed

Long Direction Deflection

pcaSlab v1.51 © Portland Cement Association 10-23-2009, 04:27:55 PM
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Page 2

8.500	-0.065	-0.015	-0.080
8.750	-0.066	-0.015	-0.081
9.000	-0.066	-0.016	-0.082
9.250	-0.066	-0.016	-0.082
9.500	-0.067	-0.016	-0.082
9.750	-0.067	-0.016	-0.083
10.000	-0.067	-0.016	-0.083
10.250	-0.067	-0.016	-0.083
10.500	-0.067	-0.016	-0.082
10.750	-0.066	-0.016	-0.082
11.000	-0.066	-0.016	-0.082
11.250	-0.066	-0.015	-0.081
11.500	-0.065	-0.015	-0.080
11.750	-0.064	-0.015	-0.079
12.000	-0.063	-0.015	-0.078
12.250	-0.063	-0.015	-0.077
12.500	-0.062	-0.014	-0.076
12.750	-0.061	-0.014	-0.075
13.000	-0.059	-0.014	-0.073
13.250	-0.058	-0.014	-0.072
13.500	-0.057	-0.013	-0.070
13.750	-0.055	-0.013	-0.068
14.000	-0.054	-0.013	-0.066
14.250	-0.052	-0.012	-0.064
14.500	-0.050	-0.012	-0.062
14.750	-0.049	-0.011	-0.060
15.000	-0.047	-0.011	-0.058
15.250	-0.045	-0.011	-0.055
15.500	-0.043	-0.010	-0.053
15.750	-0.041	-0.010	-0.050
16.000	-0.039	-0.009	-0.048
16.250	-0.036	-0.009	-0.045
16.500	-0.034	-0.008	-0.042
16.750	-0.032	-0.007	-0.039
17.000	-0.029	-0.007	-0.036
17.250	-0.027	-0.006	-0.033
17.500	-0.025	-0.006	-0.031
17.750	-0.022	-0.005	-0.028
18.000	-0.020	-0.005	-0.024
18.250	-0.017	-0.004	-0.021
18.500	-0.015	-0.003	-0.018
18.750	-0.012	-0.003	-0.015
19.000	-0.010	-0.002	-0.012
19.250	-0.007	-0.002	-0.009
19.250	-0.007	-0.002	-0.009
19.500	-0.005	-0.001	-0.006
19.750	-0.002	-0.001	-0.003
20.000	-0.000	-0.000	-0.000

Short Direction Deflection

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Page 2

8.500	-0.145	-0.072	-0.217
8.750	-0.146	-0.073	-0.219
9.000	-0.147	-0.073	-0.221
9.250	-0.148	-0.074	-0.222
9.500	-0.149	-0.074	-0.223
9.750	-0.149	-0.074	-0.223
10.000	-0.149	-0.074	-0.223
10.250	-0.149	-0.074	-0.223
10.500	-0.149	-0.074	-0.223
10.750	-0.148	-0.074	-0.222
11.000	-0.147	-0.073	-0.221
11.250	-0.146	-0.073	-0.219
11.500	-0.145	-0.072	-0.217
11.750	-0.143	-0.071	-0.215
12.000	-0.142	-0.070	-0.212
12.250	-0.140	-0.069	-0.209
12.500	-0.137	-0.068	-0.205
12.750	-0.135	-0.067	-0.202
13.000	-0.132	-0.065	-0.198
13.250	-0.129	-0.064	-0.193
13.500	-0.126	-0.062	-0.189
13.750	-0.123	-0.061	-0.184
14.000	-0.120	-0.059	-0.178
14.250	-0.116	-0.057	-0.173
14.500	-0.112	-0.055	-0.167
14.750	-0.108	-0.053	-0.161
15.000	-0.104	-0.051	-0.155
15.250	-0.099	-0.049	-0.148
15.500	-0.095	-0.046	-0.141
15.750	-0.090	-0.044	-0.134
16.000	-0.085	-0.042	-0.127
16.250	-0.081	-0.039	-0.120
16.500	-0.076	-0.037	-0.112
16.750	-0.070	-0.034	-0.105
17.000	-0.065	-0.032	-0.097
17.250	-0.060	-0.029	-0.089
17.500	-0.055	-0.026	-0.081
17.750	-0.049	-0.024	-0.073
18.000	-0.044	-0.021	-0.065
18.250	-0.038	-0.018	-0.056
18.500	-0.033	-0.016	-0.048
18.750	-0.027	-0.013	-0.040
19.000	-0.022	-0.010	-0.032
19.000	-0.022	-0.010	-0.032
19.250	-0.016	-0.008	-0.024
19.500	-0.011	-0.005	-0.016
19.750	-0.005	-0.003	-0.008
20.000	-0.000	-0.000	-0.000

One-way Slab

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① One-Way Slab

I. Interior Slab
Note: Slab designed first so dead load of the slab may be used in beam design

Assuming Columns are 18x24 + Beams or 12" wide

Loading
Dead: 20 psf
Live: 40 psf

self-weight trial:

minimum thickness (h) of nonprestressed one-way slabs both ends continuous $\ell/28$

$\ell = 19'\text{-}7''$ $19.583 \times 12 / 28 = 8.39''$ ← No deflection calcs needed

9" used   Assume 3/4 clear cover + No. 4 bars

$150\ \text{lb/ft}^3 \times 9/12 = 112.5\ \text{psf}$ $d = 9 - 0.75 - 0.5/2 = 8''$

$W_u = 132.5(1.2) + 1.6(40) = 223\ \text{psf}$

Factored Moments to check slab thickness

$\frac{1}{11} W_u \ell_n^2$ $\ell_n = 19'\text{-}7'' - \frac{24}{12} = 17.583'$

$\frac{1}{11}(.223)(17.583)^2 = 6.27\ \text{k/ft}$

$A_s \geq \frac{M_u}{\phi f_y (jd)}$ where $jd = 0.95d$

$A_s \geq \frac{6.27 \times 12}{0.9(60) \times 0.95 \times 8} = .183\ \text{in}^2/\text{ft}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{.183(60)}{0.85 \times 4 \times 12} = 0.270\ \text{in}$

$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{6.27 \times 12}{0.9 \times 60 (8 - .135)} = .177\ \text{in}^2/\text{ft}$

② One way Slab

$$\rho = \frac{A_s/ft}{bd} = \frac{.177 in^2}{12 \times 8} = .00185$$

Check whether thickness is adequate for Shear

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15 \times .223 \times 17.583}{2} = 2186 \text{ lb per 1-ft width of slab}$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2 \times \sqrt{4000} \times 12 \times 8 = 12143 \text{ lb per 1-ft width of slab}$$

$\lambda = 1$ for N.W.C.

$\phi V_c > V_u$

$0.75(12143) > 2186$ lb

$9107.4 > 2186$ therefore use $h = 9''$

$A_s = .177 in^2/ft$ use #4 @ 12"

Check Reinforcement spacing for Crack control

$$s = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c \le 12\left(\frac{40000}{f_s}\right) \qquad \text{where } f_s = \tfrac{2}{3} f_y = 40000 \text{ psi}$$

$$s = 15\left(\frac{40000}{40000}\right) - 2.5(.75) \le 12\left(\frac{40000}{40000}\right)$$

$13.1'' \le \boxed{12''}$ ← controls

$s_{used} = 12''$ ∴ ok

Determine the shrinkage and temperature reinforcement

$A_s(S+T) = 0.0018 \times b \times h = 0.0018 \times 12 \times 9 = .194 in^2/ft$

Maximum Spacing $\le 5 \times h$ and $\le 18 in$ $\quad 5 \times h = 5 \times 9 = 45'' > 18''$ ∴ 18" governs

Use #4 @ 12" o.c.

③ One-way Slab

$h = l/16$ ACI 318-08 Table 9.5a

$h = \frac{27 \times 12}{16} = 20.25'' \Rightarrow h = 21''$

$b_w = 16'' \leftarrow \approx 75\%$ of h

$w(\text{beam web}) = \frac{(21-9) \times 18}{144} \times 150\ ^{lb}/_{ft^3} = 225$ plf

$w_{slab+SDL} = (9/12 \times 150 + 20) = \frac{132.5\text{psf} \times 19.583'}{1000} = 2.59\ ^k/_{ft}$

$w_D = 2.59 + .225 = 2.82\ ^k/_{ft}$

$w_L = \frac{60 \times 19.583}{1000} = 1.175\ ^k/_{ft}$

$w_u = 1.2D + 1.6L = 1.2(2.82) + 1.6(1.175) = 5.26\ ^k/_{ft}$

$$b_{eff} \le \begin{cases} b_w + 16h_f = 16'' + 16(9) = 160'' \\ b_w + 2(\tfrac{1}{2}\text{clear distance}) = 16 + (26 \times 12 - 16) = 312'' \\ \tfrac{1}{4}\text{span length} = \tfrac{1}{4}(19.583 \times 12) = 59'' \end{cases}$$

Use #11

$b_{eff} = 59''$ $d = 21 - 0.75 - \frac{1}{2}(11/8) = 19.6''$

1) Check T-BM Behavior

$M_u = \frac{w_u l_n^2}{11} = \frac{5.26^k/_{ft} \times 25.33}{11} = 12.11^{1k}$

$M_{n_{T,BM}} = \phi(.85)f'_c \cdot b \cdot h_f(d - h_f/2) = 0.65(.85)(4)(59'')(9)(19.6 - 9/2)$
$= 17.68^{1k}$

$M_u < M_{n_{T,BM}}$ design as reg. beam

$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_{t,.005}} = 0.85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right) = .01806$

$A_s = 0.01806(59)(19.6) = 20.9\ in^2$ use (14) #11 2 layers
$A_s = 21.8\ in^2$

4 One-way slab

$$a = \frac{A_s f_y}{0.85 f'_c \cdot b} = \frac{20.9(60)}{0.85(4)(59)} = 6.25''$$

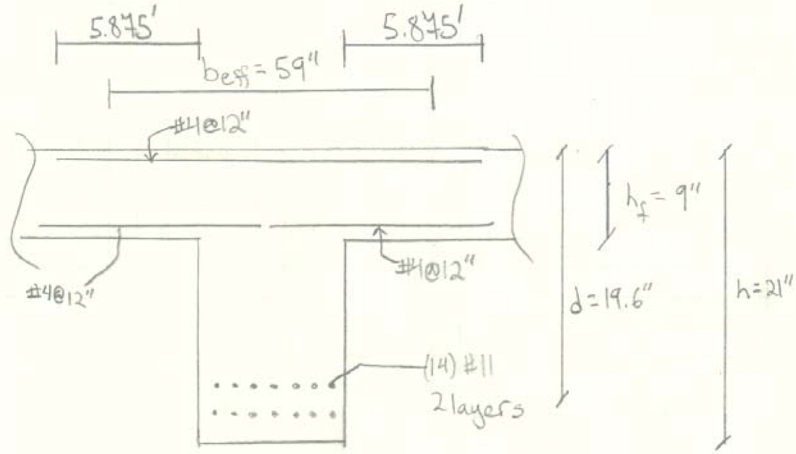
$$c = 6.25 / \beta_1 = .85 = 7.35''$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 21.8(60)\left(19.6 - \frac{6.25}{2}\right) = 21.59'^k$$

$$\varepsilon_t = 0.003\left(\frac{19.6 - 7.35}{7.35}\right) = .005 = .005 \therefore \phi = 0.9$$

$$\phi M_n = 0.9(21.59) = 19.43'^k > 12.11'^k = M_u \therefore \text{ok}$$

Use 21"x16" beam with 2 layers of (7)#11



Hollow-core Precast Panel on Concrete Beams (Option #2)

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Strand Pattern Designation
76-S

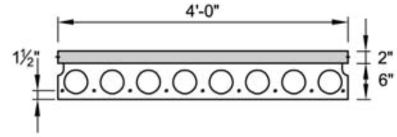
- S = straight
- Diameter of strand in 16ths
- No. of Strand (7)

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

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



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

Section Properties

	Untopped	Topped
A =	187 in.^{2}	283 in.^{2}
I =	763 in.^{4}	1,640 in.^{4}
y_b =	3.00 in.	4.14 in.
y_t =	3.00 in.	3.86 in.
S_b =	254 in.^{3}	396 in.^{3}
S_t =	254 in.^{3}	425 in.^{3}
wt =	195 plf	295 plf
DL =	49 psf	74 psf
V/S =	1.73 in.	

4HC6

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

Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	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	0.3	0.3	0.1	0.0	–0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

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

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

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

PCI Design Handbook/Sixth Edition
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2–31

1 Hollow Core Plank

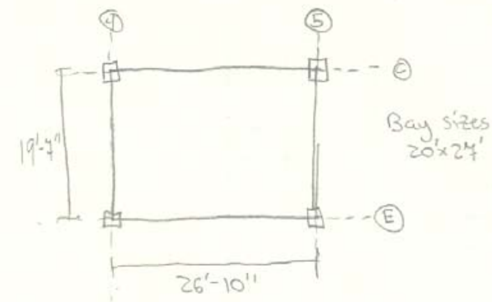
96-S Topped   4'-0" × 6" 4HC6+2 Span = 27 ft $w_u = w_D + w_L = 20 + 40 = 60$ psf

68 psf - safe superimposed service load, psf

Self-weight = 74 psf

$w_D = 74 + 20 = 94$ psf

$w_L = 40$ psf



Beam Design

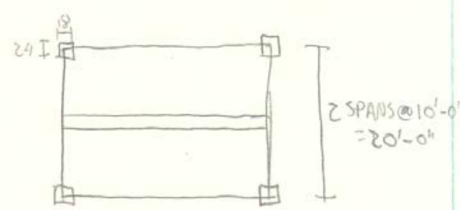
$f'_c = 4000$ psi   All columns are 18×24"

$f_y = 60$ ksi

6" hollow core concrete planks w/ 2" topping (74 psf)

20 psf superimposed dead load

40 psf live load



1) Compute Loads and Moment

- Dead Load

Concrete Planks	74 psf
Misc. D	20 psf
	94 psf (Not including self-wt of beam)

- Live Load

Hotel Room	40 psf

$$W_u = 1.2D + 1.6L = [1.2(94) + 1.6(40)]\left(\frac{10+10}{2}\right) = 1768 \text{ plf}$$

(12)

$M_u = \frac{w_u \cdot l_n^2}{8} = 1.768 \times \frac{(27-1.5)^2}{8} \times 1.1 = 158.1^{1k}$

self-wgt estimate

2) Estimate Size: $bd^2 = 20 M_u$

Try $b = \frac{4}{5} d$ $d^3 = 20(158.1)\frac{5}{4} \rightarrow d = 15.8''$

$h = d + 2.5$ use $h = 19''$ + $b = 14''$

$bd^2 = 14 \times (16)^2 = 3584 \text{ in}^3$

3) Compute Self Wt Effects:

$w_{sw} = \frac{14 \times 19}{144} \times 150 = 277.1 \text{ plf}$

$w_u = 1768 + 1.2(277.1) = 2101 \text{ plf}$

$M_u = 2.10 \times \frac{25.5^2}{8} = 170.7^{1k}$ $20 \times 170.73 = 3414.7 \text{ in}^3 < 3584$

$\therefore$ ok

4) Required Steel

$A_s = \frac{M_u}{4d} = \frac{170.7}{4(16)} = 2.67 \text{ in}^2$ (3) #9 = 3(1) = 3 in^2

$a = \frac{3 \times 60}{0.85(4)14} = 3.78$

$c = a/\beta_1 = 4.45$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{.003}{4.45}(16-4.45) = .00779 > .005$ $\therefore$ ok + $\phi = 0.9$

$M_n = \frac{3 \times 60 \left(16 - \frac{3.78}{2}\right)}{12} = 211.65^{1k}$

$\phi M_n = .9(211.65) = 190.5 > M_u = 170.7$ $\therefore$ ok

Composite Steel Framing (Option #3)

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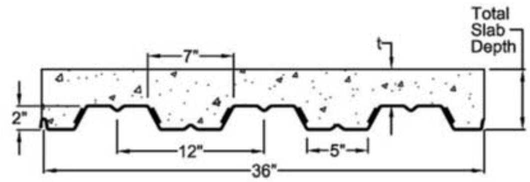
VULCRAFT

2 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



Interlocking side lap is not drawn to show actual detail.

STEEL SECTION PROPERTIES

Deck Type	Design Thickness in	Deck Weight psf	Section Properties I_p in^4/ft	S_p in^3/ft	I_n in^4/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
2VLI22	0.0295	1.62	0.324	0.263	0.321	0.266	1832	50
2VLI20	0.0358	1.97	0.409	0.341	0.406	0.346	2698	50
2VLI19	0.0418	2.30	0.492	0.420	0.489	0.426	3190	50
2VLI18	0.0474	2.61	0.559	0.495	0.558	0.504	3608	50
2VLI16	0.0598	3.29	0.704	0.653	0.704	0.653	3618	40

(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span 1 SPAN	2 SPAN	3 SPAN	Superimposed Live Load, PSF Clear Span (ft.-in.) 5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6
	2VLI22	7'-4	9'-6	9'-9	274	239	211	188	145	129	115	104	94	85	78	71	65	59	54
4.00	2VLI20	8'-7	10'-10	11'-2	310	269	236	210	188	170	155	117	106	96	87	80	73	67	61
(t=2.00)	2VLI19	9'-9	11'-11	12'-4	344	298	261	231	207	186	169	155	142	106	97	88	81	74	68
39 PSF	2VLI18	10'-9	12'-9	12'-9	373	324	285	253	228	206	188	172	159	147	137	103	95	87	81
	2VLI16	11'-1	13'-2	13'-5	400	376	330	292	261	235	214	195	180	166	154	143	109	100	93
	2VLI22	6'-11	9'-0	9'-4	319	278	245	190	168	150	134	121	109	99	90	83	76	69	63
4.50	2VLI20	8'-2	10'-3	10'-7	361	313	275	244	219	198	152	136	123	112	102	93	85	78	72
(t=2.50)	2VLI19	9'-2	11'-5	11'-9	400	346	303	268	240	216	196	180	136	124	113	103	94	86	79
45 PSF	2VLI18	10'-2	12'-4	12'-4	400	376	331	295	264	239	218	200	184	171	130	119	110	102	94
	2VLI16	10'-5	12'-6	12'-11	400	400	383	339	303	274	248	227	209	193	150	137	126	117	108
	2VLI22	6'-7	8'-7	8'-11	364	317	279	217	192	171	153	138	125	113	103	94	86	79	72
5.00	2VLI20	7'-9	9'-10	10'-2	400	356	313	278	249	193	173	156	141	128	116	106	97	89	82
(t=3.00)	2VLI19	8'-9	10'-11	11'-3	400	394	345	306	273	247	224	172	156	141	128	117	107	99	91
51 PSF	2VLI18	9'-7	11'-10	11'-11	400	400	377	336	301	273	249	228	210	162	148	136	126	116	107
	2VLI16	9'-11	12'-0	12'-4	400	400	400	386	346	312	283	259	238	187	171	157	144	133	123
	2VLI22	6'-4	8'-0	8'-6	400	355	278	244	216	192	172	155	140	127	116	106	97	89	81
5.50	2VLI20	7'-5	9'-5	9'-9	400	400	351	312	244	217	194	175	158	143	131	119	109	100	92
(t=3.50)	2VLI19	8'-4	10'-5	10'-9	400	400	388	343	307	277	215	193	175	159	144	132	121	111	102
57 PSF	2VLI18	9'-2	11'-4	11'-7	400	400	400	377	338	306	279	256	199	182	167	153	141	130	121
	2VLI16	9'-5	11'-6	11'-10	400	400	400	400	388	350	318	290	230	210	192	176	162	150	138
	2VLI22	6'-1	7'-5	8'-2	400	394	308	270	239	213	191	172	156	141	129	118	108	99	90
6.00	2VLI20	7'-1	9'-1	9'-4	400	400	390	346	271	241	215	194	175	159	145	132	121	111	102
(t=4.00)	2VLI19	8'-0	10'-1	10'-5	400	400	400	381	340	307	239	215	194	176	160	146	134	123	113
63 PSF	2VLI18	8'-10	10'-11	11'-3	400	400	400	400	375	339	309	243	221	202	185	170	157	145	134
	2VLI16	9'-1	11'-1	11'-5	400	400	400	400	400	388	352	322	255	233	213	195	180	166	154
	2VLI22	5'-11	6'-11	7'-11	400	390	339	297	263	234	210	189	171	155	141	129	118	108	99
6.50	2VLI20	6'-11	8'-9	9'-0	400	400	400	337	297	264	237	213	193	175	159	145	133	122	112
(t=4.50)	2VLI19	7'-10	9'-8	10'-0	400	400	400	400	374	293	262	236	213	193	176	161	147	135	124
69 PSF	2VLI18	8'-7	10'-6	10'-11	400	400	400	400	400	373	340	268	243	222	203	187	172	159	147
	2VLI16	8'-10	10'-8	11'-0	400	400	400	400	400	400	387	309	280	256	234	215	198	183	169

Notes:
1. Minimum exterior bearing length required is 2.00 inches. Minimum interior bearing length required is 4.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.

COMPOSITE

NUCOR VULCRAFT GROUP

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab 10/25/09 18:28:14
Building Code: IBC Steel Code: AISC360-05 ASD

Floor Type: Floor 15 **Beam Number = 23**

SPAN INFORMATION (ft): I-End (26.83,18.67) J-End (26.83,38.25)

Beam Size (Optimum) = W8X10 Fy = 60.0 ksi
Total Beam Length (ft) = 19.58

COMPOSITE PROPERTIES (Not Shored):

				Left		Right
Concrete thickness (in)				3.00		3.00
Unit weight concrete (pcf)				145.00		145.00
f'c (ksi)				4.00		4.00
Decking Orientation				perpendicular		perpendicular
Decking type				VULCRAFT 2.0VL		VULCRAFT 2.0VL
beff (in)	=	58.75	Y bar(in)		=	10.51
Mnf (kip-ft)	=	125.81	Mn (kip-ft)		=	85.01
C (kips)	=	68.92	PNA (in)		=	7.32
Ieff (in4)	=	131.30	Itr (in4)		=	192.14
Stud length (in)	=	3.50	Stud diam (in)		=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
\# of studs: Max = 19 Partial = 8 Actual = 8
Number of Stud Rows = 1 Percent of Full Composite Action = 38.80

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.456	0.456	0.000	---	NonR	0.000
	19.583	0.456	0.456	0.000			0.000
2	0.000	0.179	0.000	0.358	---	NonR	0.000
	19.583	0.179	0.000	0.358			0.000
3	0.000	0.010	0.010	0.000	---	NonR	0.000
	19.583	0.010	0.010	0.000			0.000

SHEAR: Max Va (DL+LL) = 9.81 kips Vn/1.50 = 32.19 kips

MOMENTS:

Span	Cond	LoadCombo	Ma	@	Lb	Cb	Ω	Mn / Ω
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	DL	22.3	9.8	0.0	1.00	1.67	25.62
	Init DL	DL	22.3	9.8	---	---		
	Max +	DL+LL	48.1	9.8	---	---	1.67	50.90
Controlling		DL+LL	48.1	9.8	---	---	1.67	50.90

REACTIONS (kips):

	Left	Right
Initial reaction	4.56	4.56
DL reaction	6.31	6.31
Max +LL reaction	3.50	3.50
Max +total reaction (factored)	9.81	9.81

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	9.79 ft =	-1.726	L/D =	136

Gravity Beam Design

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Live load (in)	at	9.79 ft =	-0.311	L/D =	756
Post Comp load (in)	at	9.79 ft =	-0.466	L/D =	504
Net Total load (in)	at	9.79 ft =	-0.942	L/D =	249

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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Steel Code: AISC360-05 ASD

Floor Type: Floor 15 **Beam Number = 19**

SPAN INFORMATION (ft): I-End (26.83,18.67) J-End (53.67,18.67)

Beam Size (Optimum) = W16X26 Fy = 60.0 ksi
Total Beam Length (ft) = 26.83

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.00	3.00
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	4.00	4.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 80.50	Y bar(in)	=	16.91
Mnf (kip-ft)	= 461.11	Mn (kip-ft)	=	347.58
C (kips)	= 150.76	PNA (in)	=	12.88
Ieff (in4)	= 765.63	Itr (in4)	=	1113.30
Stud length (in)	= 3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 21.5 Rg = 1.00 Rp = 0.75

# of studs per stud segment: Full = 22,1,22
Partial = 6,3,6
Actual = 6,3,6

Number of Stud Rows = 1 Percent of Full Composite Action = 28.12

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
8.945	6.33	4.58	0.00	0.0	3.50	0.00	0.0	0.00	Snow	0.00
8.945	6.02	4.35	0.00	0.0	3.34	0.00	0.0	0.00	Snow	0.00
17.889	6.33	4.58	0.00	0.0	3.50	0.00	0.0	0.00	Snow	0.00
17.889	6.02	4.35	0.00	0.0	3.34	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.026	0.026	0.000	---	NonR	0.000
	26.833	0.026	0.026	0.000			0.000

SHEAR: Max Va (DL+LL) = 19.54 kips Vn/1.67 = 80.52 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	PreCmp+	DL	82.2	13.4	8.9	1.00	1.67	89.81
	Init DL	DL	82.2	13.4	---	---		
	Max +	DL+LL	174.0	13.4	---	---	1.67	208.13
Controlling		DL	82.2	13.4	8.9	1.00	1.67	89.81

REACTIONS (kips):

	Left	Right
Initial reaction	9.28	9.28
DL reaction	12.70	12.70

Gravity Beam Design

RAM Steel v12.1
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	Left	Right
Max +LL reaction	6.84	6.84
Max +total reaction (factored)	19.54	19.54

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	13.42 ft =	-1.247	L/D =	258
Live load (in)	at	13.42 ft =	-0.365	L/D =	882
Post Comp load (in)	at	13.42 ft =	-0.548	L/D =	588
Net Total load (in)	at	13.42 ft =	-1.044	L/D =	308

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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Floor Type: Floor 15 **Beam Number = 20**

SPAN INFORMATION (ft): I-End (26.83,38.25) J-End (53.67,38.25)
Beam Size (Optimum) = W16X26 Fy = 60.0 ksi
Total Beam Length (ft) = 26.83

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.00	3.00
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	4.00	4.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in) =	80.50	Y bar(in) =	16.91
Mnf (kip-ft) =	461.11	Mn (kip-ft) =	347.58
C (kips) =	150.76	PNA (in) =	12.88
Ieff (in4) =	765.63	Itr (in4) =	1113.30
Stud length (in) =	3.50	Stud diam (in) =	0.75

Stud Capacity (kips) Qn = 21.5 Rg = 1.00 Rp = 0.75
# of studs per stud segment: Full = 22,1,22
Partial = 6,3,6
Actual = 6,3,6
Number of Stud Rows = 1 Percent of Full Composite Action = 28.12

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
8.945	6.02	4.35	0.00	0.0	3.34	0.00	0.0	0.00	Snow	0.00
8.945	6.33	4.58	0.00	0.0	3.50	0.00	0.0	0.00	Snow	0.00
17.889	6.02	4.35	0.00	0.0	3.34	0.00	0.0	0.00	Snow	0.00
17.889	6.33	4.58	0.00	0.0	3.50	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.026	0.026	0.000	---	NonR	0.000
	26.833	0.026	0.026	0.000			0.000

SHEAR: Max Va (DL+LL) = 19.54 kips Vn/1.67 = 80.52 kips

MOMENTS:

Span	Cond	LoadCombo	Ma	@	Lb	Cb	Ω	Mn / Ω
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	DL	82.2	13.4	8.9	1.00	1.67	89.81
	Init DL	DL	82.2	13.4	---	---		
	Max +	DL+LL	174.0	13.4	---	---	1.67	208.13
Controlling		DL	82.2	13.4	8.9	1.00	1.67	89.81

REACTIONS (kips):

	Left	Right
Initial reaction	9.28	9.28
DL reaction	12.70	12.70

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
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	Left	Right
Max +LL reaction	6.84	6.84
Max +total reaction (factored)	19.54	19.54

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	13.42 ft =	-1.247	L/D =	258
Live load (in)	at	13.42 ft =	-0.365	L/D =	882
Post Comp load (in)	at	13.42 ft =	-0.548	L/D =	588
Net Total load (in)	at	13.42 ft =	-1.044	L/D =	308

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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Floor Type: Floor 15 **Beam Number = 37**

SPAN INFORMATION (ft): I-End (35.78,18.67) J-End (35.78,38.25)

Beam Size (Optimum) = W10X12 Fy = 50.0 ksi
Total Beam Length (ft) = 19.58

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.00	3.00
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 58.75	Y bar(in)	= 12.21
Mnf (kip-ft)	= 140.01	Mn (kip-ft)	= 89.16
C (kips)	= 51.69	PNA (in)	= 7.66
Ieff (in4)	= 180.13	Itr (in4)	= 287.57
Stud length (in)	= 3.50	Stud diam (in)	= 0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
# of studs: Max = 19 Partial = 6 Actual = 6
Number of Stud Rows = 1 Percent of Full Composite Action = 29.20

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.456	0.456	0.000	---	NonR	0.000
	19.583	0.456	0.456	0.000			0.000
2	0.000	0.179	0.000	0.358	---	NonR	0.000
	19.583	0.179	0.000	0.358			0.000
3	0.000	0.012	0.012	0.000	---	NonR	0.000
	19.583	0.012	0.012	0.000			0.000

SHEAR: Max Va (DL+LL) = 9.83 kips Vn/1.50 = 37.51 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	PreCmp+	DL	22.4	9.8	0.0	1.00	1.67	31.21
	Init DL	DL	22.4	9.8	---	---		
	Max +	DL+LL	48.1	9.8	---	---	1.67	53.39
Controlling		DL+LL	48.1	9.8	---	---	1.67	53.39

REACTIONS (kips):

	Left	Right
Initial reaction	4.58	4.58
DL reaction	6.33	6.33
Max +LL reaction	3.50	3.50
Max +total reaction (factored)	9.83	9.83

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	9.79 ft =	-0.992	L/D = 237

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
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Live load (in)	at	9.79 ft =	-0.227	L/D = 1037
Post Comp load (in)	at	9.79 ft =	-0.340	L/D = 691
Net Total load (in)	at	9.79 ft =	-0.582	L/D = 404

Gravity Beam Design

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DataBase: Composite Slab 10/25/09 18:28:14
Building Code: IBC Steel Code: AISC360-05 ASD

Floor Type: Floor 15 **Beam Number = 36**

SPAN INFORMATION (ft): I-End (44.72,18.67) J-End (44.72,38.25)

Beam Size (Optimum) = W10X12 Fy = 50.0 ksi
Total Beam Length (ft) = 19.58

COMPOSITE PROPERTIES (Not Shored):

			Left		Right
Concrete thickness (in)			3.00		3.00
Unit weight concrete (pcf)			145.00		145.00
f'c (ksi)			4.00		4.00
Decking Orientation			perpendicular		perpendicular
Decking type			VULCRAFT 2.0VL		VULCRAFT 2.0VL
beff (in)	=	58.75	Y bar(in)	=	12.21
Mnf (kip-ft)	=	140.01	Mn (kip-ft)	=	89.16
C (kips)	=	51.69	PNA (in)	=	7.66
Ieff (in4)	=	180.13	Itr (in4)	=	287.57
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
# of studs: Max = 19 Partial = 6 Actual = 6
Number of Stud Rows = 1 Percent of Full Composite Action = 29.20

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.456	0.456	0.000	---	NonR	0.000
	19.583	0.456	0.456	0.000			0.000
2	0.000	0.179	0.000	0.358	---	NonR	0.000
	19.583	0.179	0.000	0.358			0.000
3	0.000	0.012	0.012	0.000	---	NonR	0.000
	19.583	0.012	0.012	0.000			0.000

SHEAR: Max Va (DL+LL) = 9.83 kips Vn/1.50 = 37.51 kips

MOMENTS:

Span	Cond	LoadCombo	Ma	@	Lb	Cb	Ω	Mn / Ω
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	DL	22.4	9.8	0.0	1.00	1.67	31.21
	Init DL	DL	22.4	9.8	---	---		
	Max +	DL+LL	48.1	9.8	---	---	1.67	53.39
Controlling		DL+LL	48.1	9.8	---	---	1.67	53.39

REACTIONS (kips):

	Left	Right
Initial reaction	4.58	4.58
DL reaction	6.33	6.33
Max +LL reaction	3.50	3.50
Max +total reaction (factored)	9.83	9.83

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	9.79 ft =	-0.992	L/D	=	237

Gravity Beam Design

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Live load (in)	at	9.79 ft =	-0.227	L/D	=	1037
Post Comp load (in)	at	9.79 ft =	-0.340	L/D	=	691
Net Total load (in)	at	9.79 ft =	-0.582	L/D	=	404

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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Steel Code: AISC360-05 ASD

Floor Type: Floor 15 **Beam Number = 13**

SPAN INFORMATION (ft): I-End (53.67,18.67) J-End (53.67,38.25)

Beam Size (Optimum) = W8X10 Fy = 60.0 ksi
Total Beam Length (ft) = 19.58

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.00	3.00
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in) =	58.75	Y bar(in) =	10.51
Mnf (kip-ft) =	125.81	Mn (kip-ft) =	85.01
C (kips) =	68.92	PNA (in) =	7.32
Ieff (in4) =	131.30	Itr (in4) =	192.14
Stud length (in) =	3.50	Stud diam (in) =	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
# of studs: Max = 19 Partial = 8 Actual = 8
Number of Stud Rows = 1 Percent of Full Composite Action = 38.80

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.456	0.456	0.000	---	NonR	0.000
	19.583	0.456	0.456	0.000			0.000
2	0.000	0.179	0.000	0.358	---	NonR	0.000
	19.583	0.179	0.000	0.358			0.000
3	0.000	0.010	0.010	0.000	---	NonR	0.000
	19.583	0.010	0.010	0.000			0.000

SHEAR: Max Va (DL+LL) = 9.81 kips Vn/1.50 = 32.19 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	PreCmp+	DL	22.3	9.8	0.0	1.00	1.67	25.62
	Init DL	DL	22.3	9.8	---	---		
	Max +	DL+LL	48.1	9.8	---	---	1.67	50.90
Controlling		DL+LL	48.1	9.8	---	---	1.67	50.90

REACTIONS (kips):

	Left	Right
Initial reaction	4.56	4.56
DL reaction	6.31	6.31
Max +LL reaction	3.50	3.50
Max +total reaction (factored)	9.81	9.81

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	9.79 ft =	-1.726	L/D =	136

Gravity Beam Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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Steel Code: AISC360-05 ASD

Live load (in)	at	9.79 ft =	-0.311	L/D =	756
Post Comp load (in)	at	9.79 ft =	-0.466	L/D =	504
Net Total load (in)	at	9.79 ft =	-0.942	L/D =	249

Gravity Column Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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10/25/09 18:32:30
Steel Code: AISC360-05 ASD

Story level Level 15, Column Line 4 - B

Fy (ksi)	=	60.00	Column Size	=	W10X33
Orientation (deg.)	=	0.0			

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	9.00	9.00
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	7.36	6.48
Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Load Case 10:

	Dead	Live	Roof
Axial (kip)	38.02	17.19	0.00
Moments Top Mx (kip-ft)	-0.00	-0.00	0.00
My (kip-ft)	-0.16	-1.89	0.00
Bot Mx (kip-ft)	0.00	0.00	0.00
My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + LL)

Pa (kip)	=	55.21	Pn/1.67 (kip)	=	265.91
Max (kip-ft)	=	0.00	Mnx/1.67 (kip-ft)	=	107.03
May (kip-ft)	=	2.05	Mny/1.67 (kip-ft)	=	40.60
Rm	=	1.00			
Cbx	=	1.00			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.208
Eq H1-1a: 0.208 + 0.000 + 0.045 = 0.253

Gravity Column Design

RAM Steel v12.1 Page 7/16
DataBase: Composite Slab 10/25/09 18:32:30
Building Code: IBC Steel Code: AISC360-05 ASD

Story level Level 15, Column Line 4 - C

Fy (ksi)	= 60.00	Column Size	=	W10X33
Orientation (deg.)	= 0.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Load Case 6:

			Dead	Live	Roof
Axial (kip)			38.02	17.19	0.00
Moments	Top	Mx (kip-ft)	-0.00	-0.00	0.00
		My (kip-ft)	0.16	1.89	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + LL)

Pa (kip)	=	55.21	Pn/1.67 (kip)	=	265.91
Max (kip-ft)	=	0.00	Mnx/1.67 (kip-ft)	=	107.03
May (kip-ft)	=	2.05	Mny/1.67 (kip-ft)	=	40.60
Rm	=	1.00			
Cbx	=	1.00			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.208
Eq H1-1a: 0.208 + 0.000 + 0.045 = 0.253

Gravity Column Design

RAM Steel v12.1 Page 10/16
DataBase: Composite Slab 10/25/09 18:32:30
Building Code: IBC Steel Code: AISC360-05 ASD

Story level Level 15, Column Line 5 - B

Fy (ksi)	= 60.00	Column Size	= W10X33
Orientation (deg.)	= 0.0		

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Load Case 10:

		Dead	Live	Roof
Axial (kip)		38.02	17.19	0.00
Moments	Top Mx (kip-ft)	-0.00	-0.00	0.00
	My (kip-ft)	-0.16	-1.89	0.00
	Bot Mx (kip-ft)	0.00	0.00	0.00
	My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + LL)

Pa (kip)	=	55.21	Pn/1.67 (kip)	=	265.91
Max (kip-ft)	=	0.00	Mnx/1.67 (kip-ft)	=	107.03
May (kip-ft)	=	2.05	Mny/1.67 (kip-ft)	=	40.60
Rm	=	1.00			
Cbx	=	1.00			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.208
Eq H1-1a: 0.208 + 0.000 + 0.045 = 0.253

Gravity Column Design

RAM Steel v12.1
DataBase: Composite Slab
Building Code: IBC

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10/25/09 18:32:30
Steel Code: AISC360-05 ASD

Story level Level 15, Column Line 5 - C

Fy (ksi)	=	60.00	Column Size	=	W10X33
Orientation (deg.)	=	0.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Load Case 6:

		Dead	Live	Roof
Axial (kip)		38.02	17.19	0.00
Moments	Top Mx (kip-ft)	-0.00	-0.00	0.00
	My (kip-ft)	0.16	1.89	0.00
	Bot Mx (kip-ft)	0.00	0.00	0.00
	My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

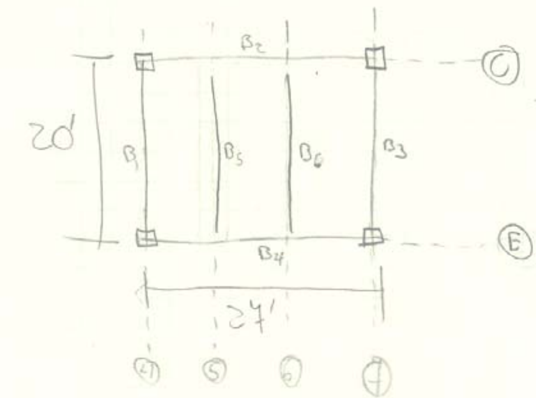
CALCULATED PARAMETERS: (DL + LL)

Pa (kip)	=	55.21	Pn/1.67 (kip)	=	265.91
Max (kip-ft)	=	0.00	Mnx/1.67 (kip-ft)	=	107.03
May (kip-ft)	=	2.05	Mny/1.67 (kip-ft)	=	40.60
Rm	=	1.00			
Cbx	=	1.00			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.208
Eq H1-1a: 0.208 + 0.000 + 0.045 = 0.253

① Composite Steel



Beams B_1 + B_3 W8x10
Beams B_2 + B_4 W16x26
Beams B_5 + B_6 W10x12
Columns are W10x33

All designs are from RAM Structural System

Metal Deck Type 2VLI18 5.00 (t=3.00) 51 psf

clear span 9'-0"

Superimposed Live Load = 228 psf

$w_L = 40$ psf < 256 psf $\therefore$ ok