Hilton Hotel at BWI Airport

Linthicum Heights, MD

Technical Report 2: Structural Study of Alternate Floor Systems

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EXECUTIVE SUMMARY

The purpose of this technical report is to analyze and compare alternate structural floor systems to the existing post-tensioned floor system.

Existing Floor System

BWI Hilton Hotel's existing structural floor system is a 7-1/2" thick flat plate post-tensioned concrete system transferring load to rectangular reinforced concrete columns.

Alternate Floor Systems

Four alternate structural floor systems were compared to the existing floor system as well as against each other to determine the practicality of their application. The following systems were compared on the basis of the following criteria: depth, weight, fire protection, fire rating, and possibility of vibration, cost, lead time for materials, if form work is necessary and the degree of difficulty of construction.

1. Composite beam with concrete slab
2. *Girder-Slab* with hollow core planks
3. Two-way flat slab with drop panels
4. One-way concrete joists

Conclusions

After analysis of alternate structural floor systems were completed, the various floor systems were compared. The most viable floor system analyzed would be the Girder-Slab with hollow core plank systems. This system maintains the lowest floor thickness of all the systems. This is important due to the assumed height restriction of the building. This system also has an underside that can be used for ceilings by the guest rooms below. This system costs more than the existing, but by engineering construction methods a faster erection of the superstructure could make up for the higher initial cost of the system by earlier occupancy.

EXISTING STRUCTURAL FLOOR SYSTEM

Introduction

The Hilton Hotel at the BWI Airport is an 11-story 280 guest room hotel designed referencing ASCE7-02. The Engineer of Record uses a live load of 40 psf for guest room floors, as well as a superimposed dead load of 10 psf. Calculations for alternative structural floor systems were performed using these loads.

Existing Floor System

Floors 4-11 are typical framing plans for the hotel guest room floors. The existing structural floor system is a two-way post-tensioned reinforced concrete flat plate. Thickness of the slab is 7-1/2" while the concrete is specified to reach a f'c = 4000 psi. Reinforcing the bottom of the slab is a mat of #4 bars 30" o.c. in each direction. The top reinforcement has various sizes of bars placed in each direction. Typical forces applied on tendons are 295^{K} in the East-West direction while 24$^{K/ft}$ in the North-South direction. On the interior of the system, tensioning of tendons was achieved, by two pour strips 4'-0" that were left unpoured so anchors could be set. Strips were then poured at a later time. Columns sizes are 14"x26" and 16"x28" with a specified f'c = 4000psi. Figure 1 shows the typical framing plan with a typical bay highlighted. The highlighted bay of 27'-0" x 22'-0" will be analyzed for alternate floor systems. A larger plan of this layout may be found in Appendix B.

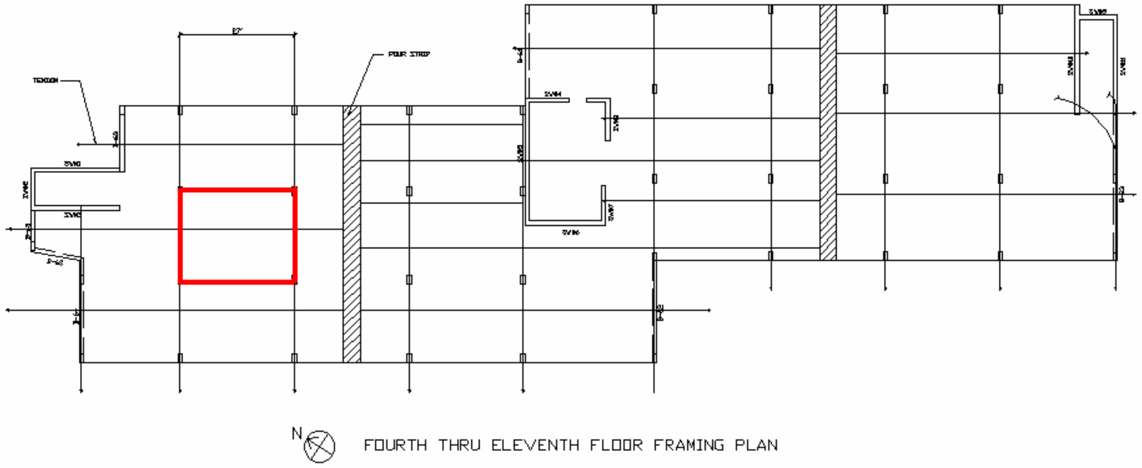


Figure 1: Typical structural floor plan with highlighted bay

The Hilton Hotel has certain architectural restraints with the existing system that must be considered when selecting an alternate system. The restraints are as follows:

1. The bottom of the existing flat plate is the ceiling of the guest room below, with the addition of a coat of plaster to the underside of the flat plate. This can be seen in Figure 2.

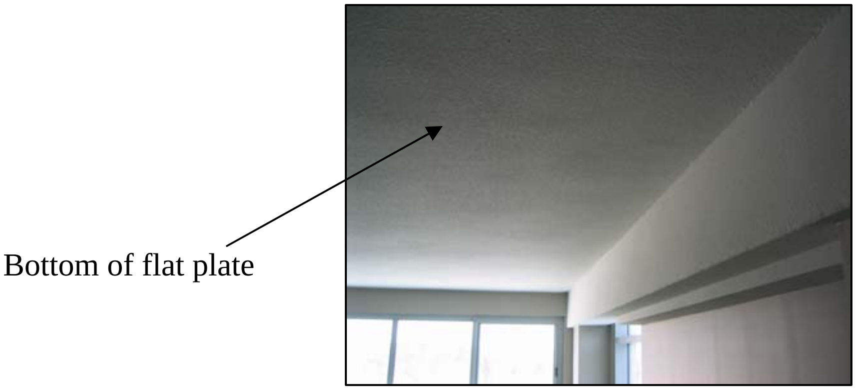


Figure 2: Underside of flat plate with a thin layer of plaster in Guest room

2. An assumed height restriction on the building is imposed due to the close proximity of the BWI Airport. Flight paths of planes may coincide with the hotel. An engineering challenge will be to maintain a similar floor to floor height without raising the total height of the building.

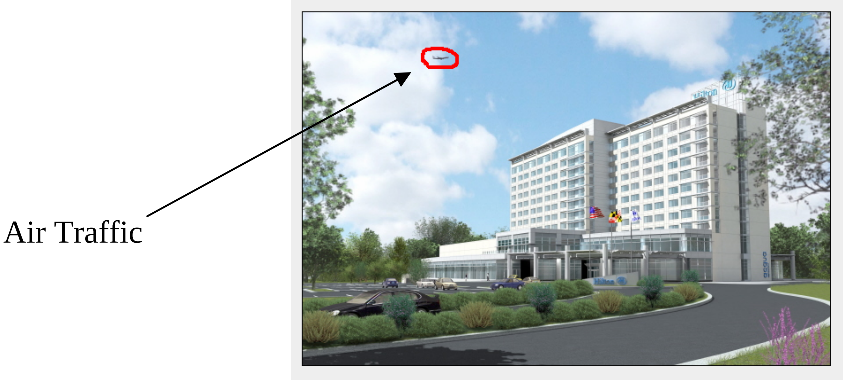


Figure 3: Close proximity to BWI Airport

Both of these constraints make it difficult to engineer an alternate floor system to the existing. There could be possible solutions and ways around these constraints, but a cut back or additional cost will have to be accounted for in some other area.

ALTERNATE STRUCTURAL FLOOR SYSTEMS

Alt. 1: Composite beam with concrete slab

The composite beam and slab system utilizes the construction of a concrete slab and beam to work compositely against flexure. Smaller sized steel beam members can be utilized because of the ability of the concrete slab to carry compressive forces during bending. Shear studs welded to the beam transfer forces to the concrete causing the slab-beam combination to act compositely. Welding the shear studs to the beam is a very labor intensive process which will intern drive construction costs higher. Beams are spaced 9’-0” o.c per bay spanning 22’-0” between girders. A 2” Lok Floor metal deck with a 3” concrete cover was taken from the United Steel Deck Design Manual. Using the composite beam selection tables in the 13th Edition Steel Construction Manual, W10x12 beam was used with 9-3/4” diameter shear studs spanning the beam. For the girders transferring load to the columns a size of a W10x15 member was used with 20-3/4” dia. shear studs spanning the girder. Section and a typical layout can be seen below.

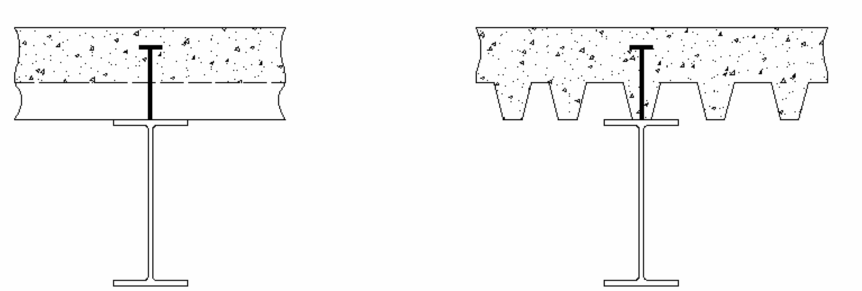


Figure 4: Left: Composite beam with concrete slab, Right: Composite girder and slab

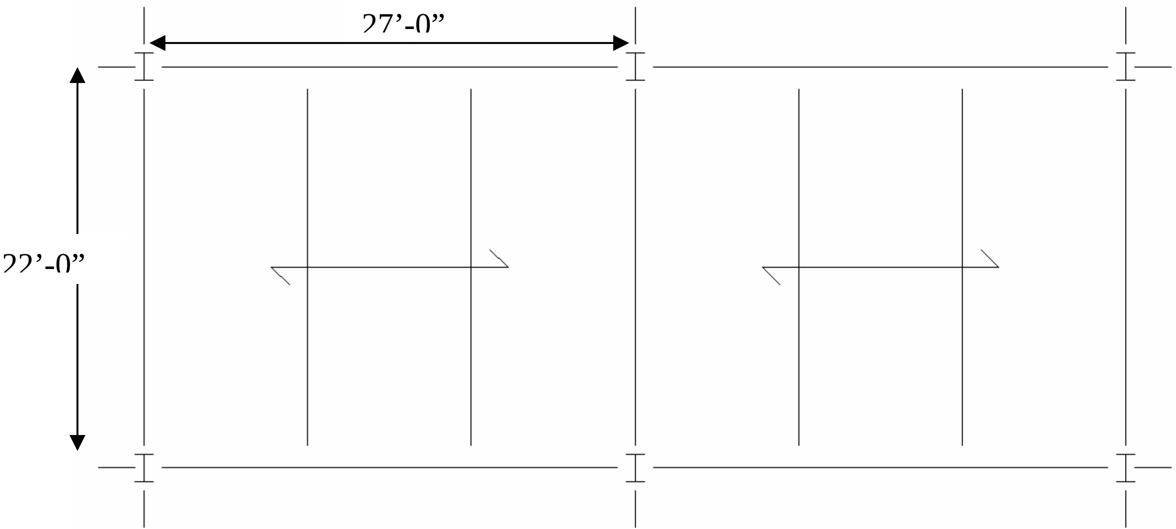
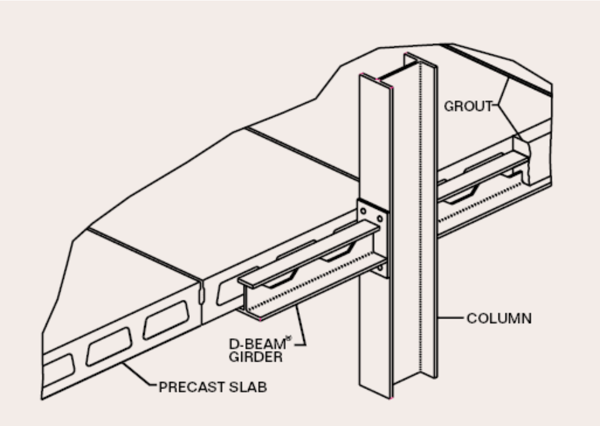


Figure 5: Composite beam with concrete slab plan

Alt 2: ***Girder-Slab*** with hollow core planks

Figure 6: Section of Girder-Slab



Girder-slab is a fairly new structural floor system being used in the industry. Utilizing a steel shape member and pre-cast hollow core planks, the combination creates a monolithic floor slab assembly. Construction of this system is fairly easy with planks being brought to the site in pieces and placed onto the steel D-shape members. D-beams and planks are grouted upon placing of planks. Construction time is significantly shorter than that of a cast-in-place system due to the lack of a cure time for the pre-cast planks. This system's overall cost may be considerably higher than the others, due to the confined pool of manufacturers and contractors to choose from. Plank sizes where taken from the Nitterhouse Concrete Products catalog. Planks will span 27'-0". D-beams will span 20'-0" in the opposite directions. The typical bay size of 27'-0" x 22'-0" was changed to 27'-0"x 20'-0" for this system, so hollow core planks could fit bays evenly. Loading on this bay required the largest D-beam size of DB9x46 to be used. Calculations were performed using ASD and procedures outlined in the Girder-Slab design guide. A typical layout can be seen below.

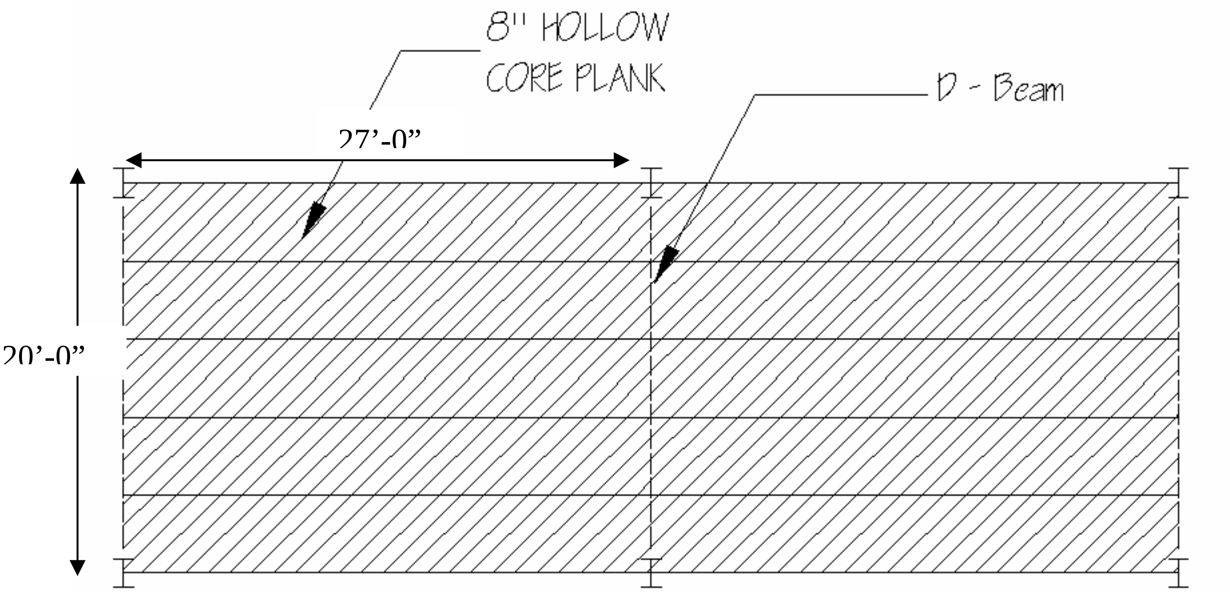


Figure 7: Girder-Slab plan

Alt 3: Two-way flat slab with drop panels

The two-way flat slab is similar in layout to the existing framing system, the two-way flat plate, except for the drop panels. The drop panels are necessary to carry forces due to punching shear, where the existing system accounts for punching shear with the added compressive forces on the concrete due to post tension. This system can utilize smaller columns than the existing flat plate system. The construction of the two-way flat slab is fairly easy. Obtaining a required fire resistance rating is done simply by construction of the system with no additional proofing needed. The two-way flat slab system was sized using the CRSI Handbook 2002. Results from the handbook yield a slab thickness of 9” with a drop panel size of 9’x9’x7” and a column size of 12”x12”. A typical layout and section can be seen below.

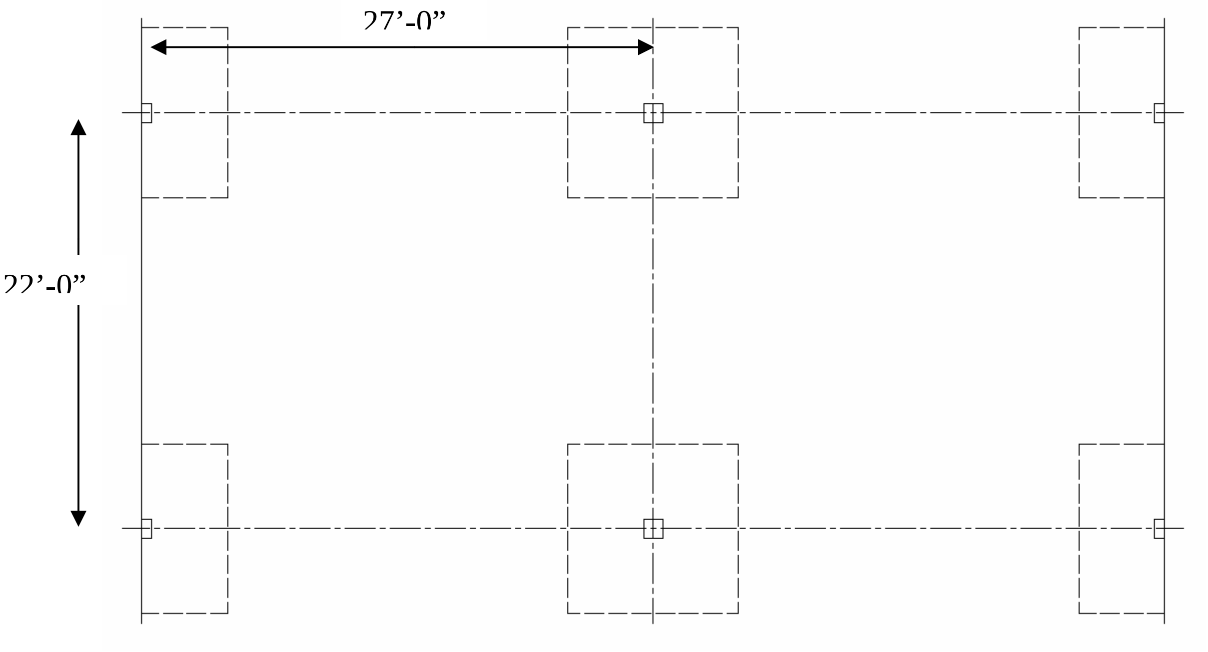


Figure 8: Plan of flat slab with drop panels

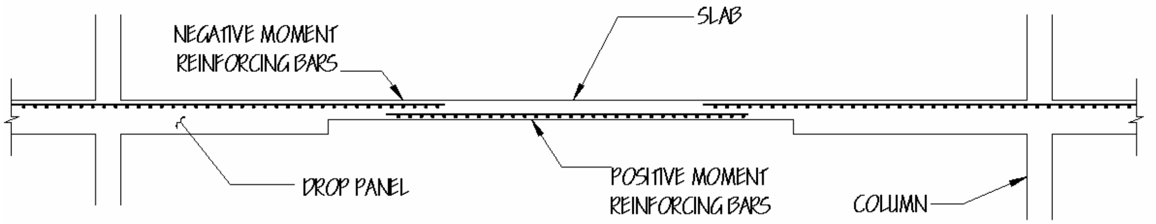


Figure 9: Section of flat slab with drop panels

Alt 4: One-way slab with concrete joists

The one-way concrete slab and joist is a monolithic system that frames perpendicular to joist band beams which transfer load to concrete columns. Fire proofing will not be necessary due to the properties of concrete. Problems concerning vibration should not be an issue due to the stiffness of the structure. Constructing the slab joist system will be more difficult than a flat plate or slab system due to the added formwork. The required dimensions of this system came from the CRSI Handbook 2002. For the given span of 27'-0" concrete joists spaced 36" o.c. will be utilized. A 3" thick slab with 10" deep by 6" wide ribs using a 30" pan are specified. The joist band beams should be 36" wide with a depth equivalent to that of the joist system for ease of construction. A typical layout and section can be seen below.

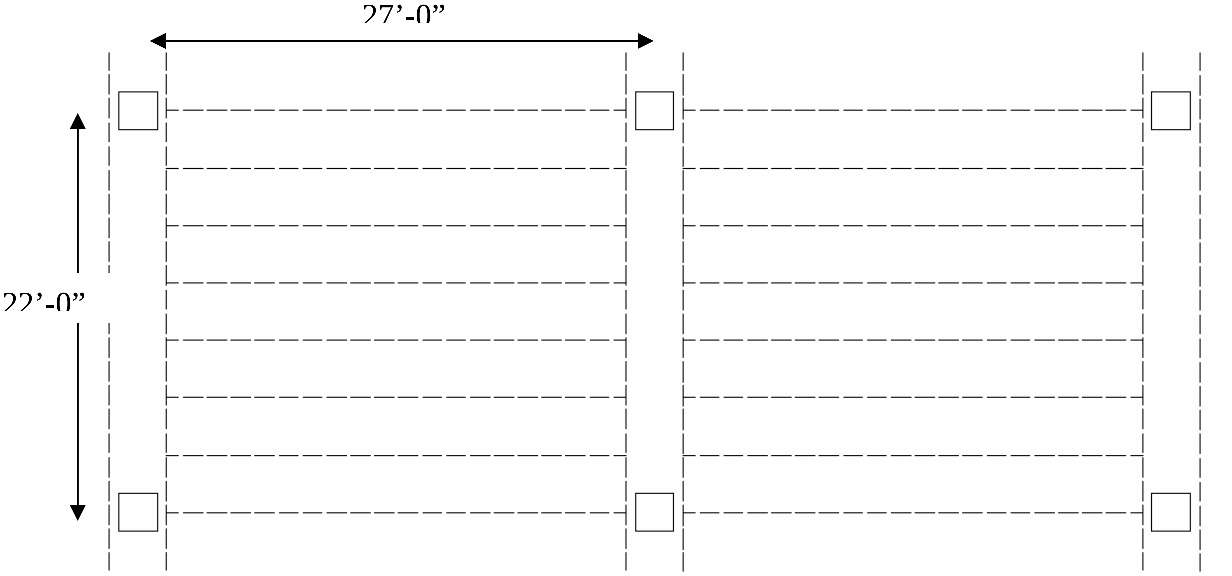


Figure 10: One-way slab with concrete joists

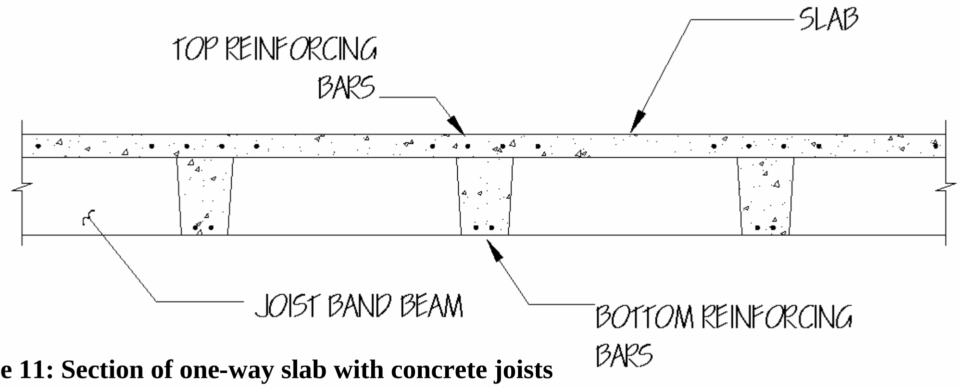


Figure 11: Section of one-way slab with concrete joists

COMPARISON

	Two-way Post Tension Flat Plate	Composite Beam w/ Concrete slab	Girder-Slab w/ Hollow Core Planks	Two-way Flat Slab w/ Drop Panels	One-way Slab w/ Concrete Joist
Depth (in)	7.5	15	10	9+7	13
Weight (psf)	93.75	50.2	103.3	122.25	61
Column Size (in)	16x28	W14	W14	12x12	24x24
Fire Protection	N	Yes	Yes	N	N
Fire Rating (hr)	> 2	1.5 - 2	2 - 3	> 2	1 - 2
Vibration	No	Possible	Possible	No	No
Cost (USD/ft^2)					
Material	$6.12	$9.15	$10.13	$5.75	$5.75
Labor	$7.74	$4.68	$4.23	$7.55	$8.15
Total	$13.86	$13.83	$14.36	$13.30	$13.90
Lead Time	N	Yes	Yes	N	N
Form work	Y	N	N	Y	Y
Constructability	Medium/ Difficult	Medium/ Difficult	Easy	Medium	Difficult
Practical Alternative	XX	No	Yes	No	No

Table 1: Comparison of structural floor systems

CONCLUSIONS

Engineering an alternate system that works as well as the existing post-tensioned system will be a challenge. Though investigation of alternate systems yielded some advantages compared to the existing system. Composite beam and slab system is considerably lighter than the existing system which would reduce seismic design loads on the structure. The depth of the beams causes the total floor thickness to increase which would intern cause an increase in building height if floor to floor heights remained the same. The goal of keeping a similar floor to floor height with the same number of floors is difficult when the building is in an assumed restricted height area having close proximity to the BWI airport. The alternative that would maintain a similar floor thickness to the existing system would be the Girder-slab system which is only 2.5” thicker. Girder-slab system is approximately 10 psf more than the existing system. Increase in the weight of the structure would make it necessary to conduct a structural capacity check of the foundation, which could result in a redesign. Of all the alternate systems, the least viable alternative is the two-way flat slab with drop panels. This system is the heaviest system of all considered in this investigation as well as having the deepest floor thickness. A redesign of the foundations would be inevitable. Cost of this system is the cheapest among all other systems though. The last system considered was the one-way slab with concrete joists. Problems concerning this system are the floor thickness and intensity of material and labor during construction. System advantage of the one-way slab is the low cost. The vibration of each floor system was just briefly considered. An in dept study would have to be done when an alternate system is chosen.

The most viable alternative system would be the composite the Girder-Slab system. Floor thickness is a concern due to the reality of an assumed height restriction. The other architectural constraint previously mentioned is the underside of the slab acting as the ceiling in the hotel guest rooms for each floor below. Increased cost may be a major factor when engineering the Girder-Slab, but schedule and speed of construction might be able make up for the added cost with a superstructure assembled in less time and hotels rooms being occupied earlier.

APPENDIX A

REFERENCES

CRSI Handbook, 2002

Girder-Slab Design Guide v1.4

NitterHouse Concrete Products, Inc. (PCI Certified) Product Data

RS Means Assemblies Cost, 2006

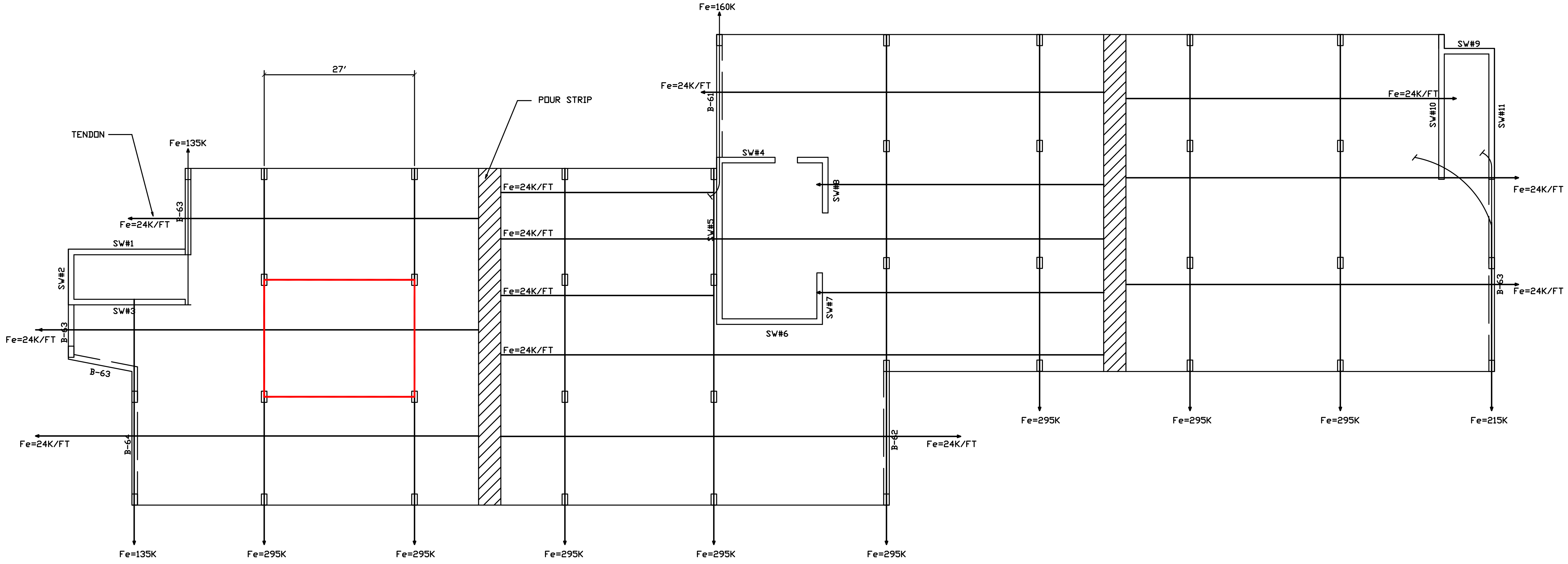
RS Means Building Construction Cost Data, 2006

Steel Construction Manual, 13th Edition, 2005

Underwriters Laboratories Fire Resistance – Volume 1, 2002

United Steel Deck Design Manual and Catalog of Products

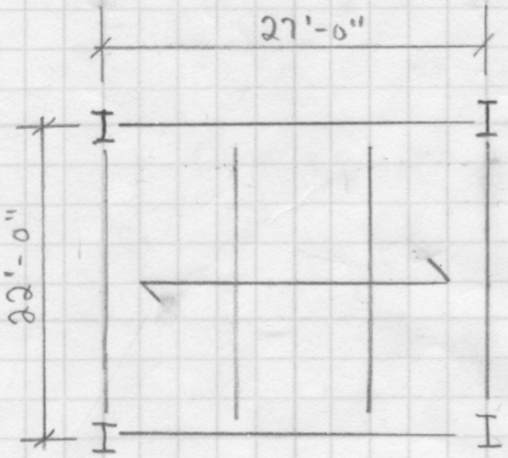
APPENDIX B



FOURTH THRU ELEVENTH FLOOR FRAMING PLAN

APPENDIX C

Composite Beam | Thomas Sabol



Live Load: Private rooms & Corridors

40 psf

Dead Loads

superimposed	10 psf
Deck	2.1 psf
concrete slab	48 psf
Beam Assume:	50 plf

Deck

2" Lok-Floor 19 gage

3" concrete Normal wt. $f'_c = 3^{ksi}$

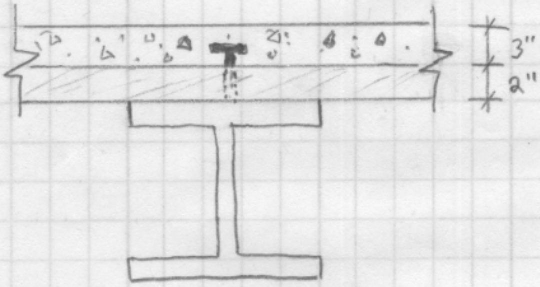
Max unshored 3 spans => 9.61' > 9' ∴ ok

@ 9' capacity of 295 psf

Factored Loads

$$1.2(10+2.1+48) + 1.6(40) = 136.1\,psf$$

Beam Self Wt => 1.2(50) = 60plf



A992 steel

$F_y = 50^{ksi}$

$$w = (136.1_{psf})(9') + \overset{\text{Beam self wt.}}{60\,plf} = 1285\,plf = 1.29^{K/ft}$$

$$M_u = \frac{1.29(22)^2}{8} = 78.1^{'K}$$

$b_{eff} \le$ 1/4 span $= \frac{22 \times 12}{4} = 66"$ ✓ controls

$b_{eff} \le$ spacing $= 9 \times 12 = 108"$

Assume $a = 1''$

$Y_2 = 5 - \frac{1}{2} = 4.5''$

AISC Manual Table 3-19 $\Rightarrow$ Try W 10×12 PNA 6 $\phi M_n = 86.8^{1k}$

$Y_1 = 1.31''$ $\Sigma Q_n = 68.9^k$

$A_g = 3.54 \text{ in}^2$ $d = 9.87''$

$C_c = 0.85 f'_c b_{eff} t = 0.85(4)(66)(3)$
$= 673.2^k$

$T_s = A_s F_y = (3.54)(50) = 177^K$

$\Sigma Q_n = 68.9^k \leftarrow$ controls

$68.9 = 0.85 f'_c b a$

$\Rightarrow a = \frac{68.9}{(0.85)(3)(66)}$

$= 0.41'' < 1''$

$Y_2 = 5 - \frac{0.41}{2} = 4.80$

@ $Y_2 = 5''$ PNA 6 $\phi M_n = 89.4^{1K} > 78.1^{1K}$ ∴ ok

Studs - Table 3-21

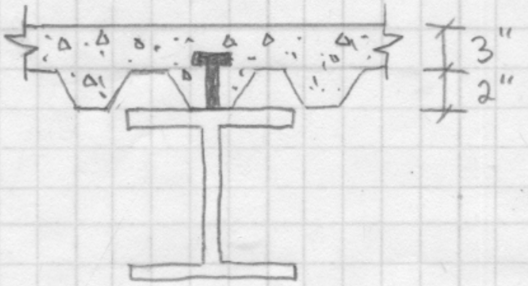
Weak studs per rib
$1 - \phi = 3/4'' \Rightarrow 17.2^K$

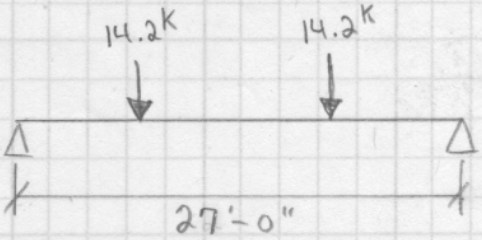
Total studs per beam
$\frac{2\Sigma Q_n}{Q_n} = \frac{2(68.9)}{17.2} = 8.01$ studs

$\boxed{\text{Use W 10×12 with 9 - 3/4''}\phi \text{ studs}}$

Girder

Trib width 22'





$b_{eff} \leq 1/4\,(27 \times 12) = 81"$ ✓ controls

$b_{eff} \leq 22 \times 12 = 264"$

$M_u = 14.2^K (9') = 127.8^{\prime K}$

Assume $a = 1"$

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

Try 10×15 PNA 3 $\phi M_n = 139^{\prime K}$

$A_s = 4.44\,in^2$ $\Sigma Q_n = 167^K$ ✓ controls

$C_c = 0.85(3)(81)(3) = 620^K$

$T_s = (9.99)(50) = 499.5^K$

$$a = \frac{167}{(0.85)(3)(81)} = 0.81" < 1"$$

$$Y_2 = 5 - \frac{0.81}{2} = 4.6"$$

Studs - Table 3-21

weak studs per rib
1 - 3/4" ø = 17.2^K

Total studs per girder

$$\frac{2\,\Sigma Q_n}{q_n} = \frac{2(167)}{17.2} = 19.4 \text{ studs}$$

$\boxed{\text{Use W10×15 with 20 - 3/4" ø studs}}$

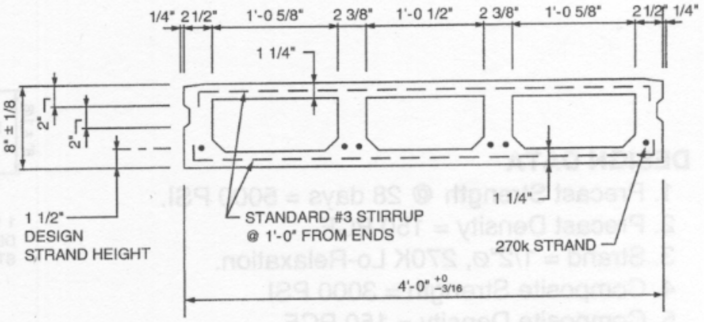
Prestressed Concrete

8" x 4' SpanDeck – U.L. – J917

(NO TOPPING)

PHYSICAL PROPERTIES
Precast

$A = 180 \text{ in.}^2$ $S_b = 397 \text{ in.}^3$
$I = 1543 \text{ in.}^4$ $S_t = 375 \text{ in.}^3$
$Y_b = 3.89 \text{ in.}$ Wt. = 230 PLF
$Y_t = 4.11 \text{ in.}$ Wt. = 57.5 PSF
$e = 2.39 \text{ in.}$



8" SPANDECK CROSS SECTION

UL FIRE RATED J917

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF
3. Strand = 1/2"ø, 270 K Lo-Relaxation.
4. Strand Height = 1.50 in.
5. Ultimate moment capacities (when fully developed) . . .
   - 4 – 1/2"ø, 270K = 74.3'K
   - 6 – 1/2"ø, 270K = 105.6'K
6. Maximum bottom tensile stress is $6\sqrt{f'c} = 424$ PSI.
7. All superimposed load is treated as live load in the strength analysis of flexure and shear.
8. Flexural strength capacity is based on stress/strain strand relationships.
9. All values in this table are based on ultimate strength and are not governed by service stress.
10. Shear values are the maximum allowable before shear reinforcement is required.
11. Deflection limits were not considered when determining allowable loads in this table.

8" SPANDECK W/O TOPPING			ALLOWABLE SUPERIMPOSED LOAD (PSF)																					
STRAND PATTERN			SPAN (FEET)																					
			10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure	4 –	1/2"ø	610	550	499	457	399	341	294	255	222	195	171	151	133	117	103	92	82	72	66	56	49	43	
Shear	4 –	1/2"ø	441	393	354	321	294	270	249	231	215	201	188	177	160	145	132	120	110	101	95	90	82	75	
Flexure	6 –	1/2"ø	885	800	726	667	586	509	437	382	334	296	263	234	208	187	168	151	136	122	111	100	90	81	73
Shear	6 –	1/2"ø	459	411	370	337	308	283	262	243	226	211	197	185	174	164	155	147	139	131	120	111	102	94	87

NITTERHOUSE
CONCRETE PRODUCTS

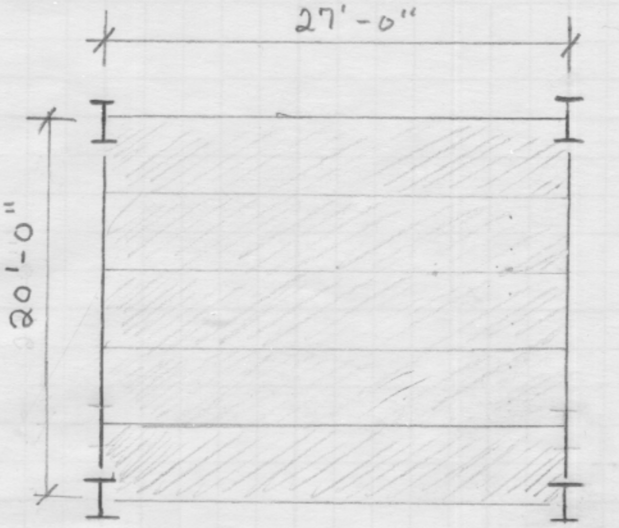
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

Prestressed Hollow Core Plank w/ Girder Slab

Thomas Sabol



2" Light weight Concrete Topping (Non-Structural) => 12psf

LL = 40psf

DL => superimposed = 10psf

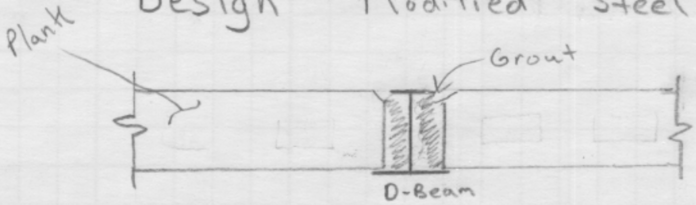
$\omega = 10 + 40 + 12 = 62\ psf$

Bay size changed to 27' x 20' to fit planks evenly

J917 8" x 4' Span Deck w/o 2" Structural Topping

Strand Pattern 4 - 1/2 ⌀
Span 27'
Allowable loads: Flexure - 72psf > 62psf
Shear - 101psf

Design Modified Steel D-Beam



Grout = 5 ksi

Shape is modified to fit flush with hollow core planks "Girder-slab"

Wt. of hollow core planks => 57.5 psf

Intial Load - Pre composite (construction)

$$M = (0.0575psf)(27')(20)^2/8 = 77.6^{1K}$$

<u>Use a DB9x46</u> => Allowable Moment = 84^1k

Total Load composite Action

$$M_{sup} = (0.01+0.04+0.012)(27')(20)^2/8 = 83.7^{1K}$$

$$M_{Total} = 77.6^{1K} + 83.7^{1K} = 161.3^{1K}$$

$$S_{req} = (163.1^{1K})(12in/ft)/(0.6)(50Ksi) = 65.2in^3 < 68.6in^3$$

∴ ok ✓

Girder Slab | Thomas Sabol

Allowable $\Delta_{LL} = L/360 = (20')(12^{in/ft})/360 = 0.67''$

$$\Delta_{LL} = \frac{(5)(27')(0.04 + 0.01 + 0.012)(20')^4(1728^{in^3/ft})}{384\,(356 in^4)\,(29\,000 Ksi)}$$

$= 0.58'' < 0.67''$ ✓ ∴ ok

Compressive Stress on concrete Grout ⇒ $f'_c = 4000psi$

$$N = \frac{E_{steel}}{E_{concrete}} = \frac{29000 \text{ Ksi}}{57000\sqrt{4000}} = 8.04$$

$S_{tc} = N \cdot S = (8.04)(68.6) = 551.5 in^3$

Allowable $F_c = (0.45)(5 Ksi) = 2.25 Ksi$

$$f_c = \frac{M_{sup}}{S_{tc}} = \frac{83.7^{1k} \times 12^{in/ft}}{551.5 in^3} = 1.82 \text{ Ksi} < 2.25 \text{ ksi} \therefore ok$$

Check Bottom Flange Tension stress (Total Load)

Allowable $F_b = 0.9(50 Ksi) = 45 Ksi$

$$f_b = \frac{(77.6^{1k})(12^{in/ft})}{50.8 in^3} + \frac{(83.7^{1k})(12^{in/ft})}{80.6 in^3}$$

$= 18.3 + 12.5 = 30.8^{Ksi} < 45^{ksi} \therefore ok$

Check Shear

Total Load $= 40 + 10 + 12 + 57.5 = 119.5 psf$

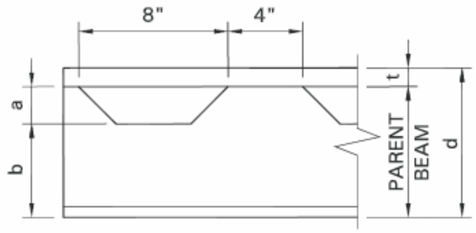
$w = (0.120 Ksf)(27') = 3.24 Klf$

$$R = \frac{(3.24^{K/ft})(20')}{2} = 32.4^{K}$$

$$f_v = \frac{32.4}{(0.375 \times 5.75)} = 15.5 \text{ Ksi}$$

$F_v = 0.4(50 Ksi) = 20^{Ksi} > 15^{ksi} \therefore ok$

D-BEAM® DIMENSIONS TABLE

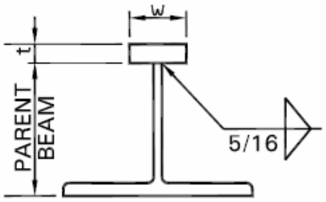


D-BEAM® REFERENCE CALCULATOR IS AVAILABLE ON WEBSITE, TECHNICAL BULLETIN

Designation	Web Included		Depth	Web	Parent Beam			
	Weight	AVG AREA	d	Thickness t_w	Size	a	b	Top Bar w x t
	lb./ft.	In.^{2}	In.	In.		In.	In.	In. x In.
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5

D-BEAM® PROPERTIES TABLE

Designation	Steel Only Web Ignored						Transformed Section Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b= 0.6Fy	Ix	C bot	C top	S bot	S top
	In.^{4}	In.	In.	In.^{3}	In.^{3}	kft	In.^{4}	In.	In.	In.^{3}	In.^{3}
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6



Flat - Slab | Thomas Sabol

Live Load => 40 psf

Superimposed DL => 10 psf

Factored Loads

↳ To use CRSI Handbook 2002
=> 1.4 D + 1.7 L

$$1.4(10) + 1.7(40) = 82\ \text{psf}$$

Typical bay 27' × 22' => use 27' × 27' for CRSI Handbook

<u>Flat Slab System</u> Pg. 10-19

slab thickness = 9"

Drop Panel = 9' × 9' × 7"

column size = 12 × 12"

* sizes are same for 10" thick slab

Self weight

$$\left(\frac{\text{Cu. ft}}{\text{Sq. ft}}\right) = 0.815$$

$$0.815(150\ \text{pcf}) = 122.25$$

Reinforcing Bars:

	Edge Panel	Interior Panel
<u>Column Strip</u>		
Top Ext.	14 - #4	
Bottom	19 - #5	19 - #4
Top Int	16 - #5	15 - #5
<u>Middle strip</u>		
Top	16 - #4	10 - #5
Bottom	14 - #4	9 - #5

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM

SQUARE EDGE PANEL With Drop Panels
No Beams

SQUARE INTERIOR PANEL

With Drop Panels[2]
No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel		Square Column[3]		REINFORCING BARS (E. W.)						MOMENTS			Factored Superimposed Load (psf)	Square Column[3]	REINFORCING BARS (E. W.)					Concrete $\left(\frac{\text{cu. ft}}{\text{sq. ft}}\right)$
		Depth (in.)	Width (ft)	Size (in.)	γ_f	Column Strip[1]			Middle Strip		Total Steel (psf)	Edge (−) (ft-k)	Bot. (+) (ft-k)	Int. (−) (ft-k)		Size (in.)	Column Strip		Middle Strip		Total Steel (psf)	
						Top Ext. +	Bottom	Top Int.	Bottom	Top Int.							Top	Bottom	Top	Bottom		
h = 9 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS															h = 9 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
23	100	4.00	7.67	12	0.771	12-#4 4	17-#4	19-#4	8-#5	8-#5	2.16	93.9	187.9	252.9	100	12	12-#5	12-#4	8-#5	8-#5	2.09	0.787
23	200	5.50	7.67	15	0.631	12-#4 1	11-#6	22-#4	10-#5	13-#4	2.61	125.9	251.8	339.0	200	18	14-#5	15-#4	8-#5	8-#5	2.34	0.801
23	300	7.00	7.67	17	0.631	13-#4 1	8-#8	25-#4	9-#6	16-#4	3.19	158.6	317.2	427.0	300	20	15-#5	19-#4	10-#5	13-#4	2.71	0.815
23	400	7.00	7.67	19	0.664	15-#4 3	17-#6	14-#6	11-#6	13-#5	3.89	190.0	380.0	511.5	400	22	18-#5	11-#6	12-#5	10-#5	3.38	0.815
23	500	8.50	9.20	21	0.629	16-#4 2	16-#7	11-#7	19-#5	8-#7	4.59	222.6	467.3	599.3	500	23	14-#6	19-#5	10-#6	19-#4	3.93	0.863
24	100	5.50	8.00	12	0.689	13-#4 2	13-#5	19-#4	13-#4	8-#5	2.20	107.5	215.0	289.4	100	12	12-#5	13-#4	8-#5	8-#5	2.04	0.801
24	200	5.50	8.00	15	0.746	13-#4 5	18-#5	12-#6	12-#5	10-#5	2.86	143.8	287.7	387.3	200	18	11-#6	18-#4	9-#5	8-#5	2.51	0.801
24	300	7.00	8.00	17	0.684	14-#4 4	12-#7	14-#6	8-#7	12-#5	3.58	181.3	362.6	488.2	300	20	17-#5	15-#5	8-#6	10-#5	3.07	0.815
24	400	8.50	8.00	19	0.631	16-#4 2	15-#7	11-#7	8-#8	8-#7	4.39	217.6	435.2	585.9	400	22	14-#6	18-#5	10-#6	12-#5	3.70	0.829
24	500	8.50	9.60	21	0.684	18-#4 3	14-#8	13-#7	11-#7	8-#8	5.08	253.8	507.6	683.4	500	24	12-#7	11-#7	16-#5	10-#6	4.32	0.863
25	100	5.50	8.33	12	0.735	13-#4 3	15-#5	14-#5	10-#5	13-#4	2.36	121.9	243.8	328.2	100	12	13-#5	15-#4	13-#4	13-#4	2.13	0.801
25	200	7.00	8.33	15	0.666	13-#4 4	20-#5	12-#6	13-#5	11-#5	2.95	163.7	327.5	440.8	200	18	16-#5	13-#5	10-#5	13-#4	2.58	0.815
25	300	8.50	8.33	17	0.633	15-#4 3	11-#8	14-#6	9-#7	10-#6	3.92	205.7	411.3	553.7	300	21	18-#5	9-#7	19-#4	11-#5	3.32	0.829
25	400	8.50	10.00	20	0.702	18-#4 5	13-#8	13-#7	20-#5	9-#7	4.55	247.2	494.5	665.6	400	23	15-#6	20-#5	11-#6	13-#5	3.87	0.863
25	500	8.50	10.00	24	0.669	13-#5 2	20-#7	19-#6	10-#8	14-#6	5.49	285.7	571.4	769.2	500	24	13-#7	10-#8	10-#7	11-#6	4.80	0.863
26	100	7.00	8.67	12	0.646	13-#4 2	9-#7	14-#5	11-#5	9-#5	2.47	138.0	276.0	371.6	100	12	13-#5	11-#5	13-#4	13-#4	2.15	0.815
26	200	7.00	8.67	15	0.720	15-#4 4	23-#5	14-#6	15-#5	19-#4	3.27	185.0	370.0	498.1	200	18	18-#5	15-#5	12-#5	10-#5	2.87	0.815
26	300	8.50	8.67	17	0.715	17-#4 5	12-#8	12-#7	10-#7	16-#5	4.13	232.6	465.2	626.2	300	21	14-#6	19-#5	22-#4	19-#4	3.46	0.829
26	400	8.50	10.40	22	0.687	13-#5 2	15-#8	26-#5	23-#5	10-#7	5.02	277.8	555.7	748.0	400	23	13-#7	23-#5	10-#7	15-#5	4.36	0.863
27	100	7.00	9.00	12	0.716	14-#4 3	19-#5	16-#5	19-#4	16-#4	2.61	155.0	310.1	417.4	100	12	15-#5	19-#4	10-#5	9-#5	2.31	0.815
27	200	8.50	9.00	15	0.658	15-#4 3	11-#8	14-#6	9-#7	10-#6	3.62	208.5	417.0	561.4	200	18	18-#5	9-#7	13-#5	11-#5	3.07	0.829
27	300	8.50	9.00	19	0.701	12-#5 3	14-#8	13-#7	15-#6	10-#7	4.52	260.0	520.0	700.0	300	21	12-#7	21-#5	16-#5	10-#6	3.79	0.829
27	400	8.50	10.80	24	0.717	22-#4 6	17-#8	12-#8	11-#8	9-#8	5.62	310.9	621.7	836.9	400	23	12-#8	11-#8	14-#6	9-#7	4.91	0.863

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.

Typical Guest room Floors 4-11

Live Load => 40 psf

Superimposed DL => 10 psf

Factored Loads

↳ To use CRSI Design Handbook 2002

$$\Rightarrow 1.4D + 1.7L$$

$$1.4(10) + 1.7(40) = 82 \text{ psf}$$

@ 48" Spacing

$$82 \text{ psf} (4') = 328 \text{ plf}$$

@ 60" Spacing

$$82 \text{ psf} (5') = 410 \text{ plf}$$

Multiple Spans - Use a Clear Span of 27'-0"

Spacing	36" o.c	48" o.c	60" o.c
Pan size	30"	40"	53"
Overall Depth	13"	18.5"	20.5"
Rib Size	6"	8"	7"
Weight	61 psf	359 psf ✓	420 psf ✓
Top Bars Ext. Span	#5 @ 12" o.c.	#4 @ 9" o.c.	#4 @ 11.5" o.c.
Top Bars Int span	#4 @ 11.5" o.c	#4 @ 10.5" o.c.	#4 @ 12" o.c.
Bot Bars Ext span	1-#5 & 1-#6	2 #6 bars	2 # 6 bars
Bot Bar Int span	2-#4 bars	3 - #4 bars	3 - #4 bars
Joists per Bay	$7\frac{1}{3}$	$5\frac{1}{2}$	$4\frac{2}{5}$
	Pg. 8-21 Use	Pg. 8-59	Pg. 8-68

STANDARD ONE-WAY JOISTS [1] MULTIPLE SPANS

30" Forms + 5" Rib @ 35" c.-c. [2] FACTORED USABLE SUPERIMPOSED LOAD (PSF)

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

10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth

TOP BARS	Size @	# 4 12	# 4 12	# 4 11.5	# 4 9	# 5 11.5	End Span Defl. Coeff. (3)	# 4 12	# 4 12	# 4 11	# 4 8	# 5 10	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6		# 3 # 3	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	
Steel (psf)		.50	.60	.72	.89	1.08		.56	.63	.77	1.00	1.25	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
17'-0"		131	192	264*	274*	282*	.978	139	227	305*	315*	322*	.602
		0	0	0	338	417*		0	0	316	422	521*	
18'-0"		108	162	227	249*	255*	1.229	115	194	273	288*	293*	.756
		0	0	0	293	368		0	0	0	367	463	
19'-0"		88	137	195	227*	232*	1.525	95	166	237	265*	269*	.939
		0	0	0	254	322		0	0	0	321	407	
20'-0"		72	116	168	207*	212*	1.873	78	142	206	244*	248*	1.153
		0	0	0	222	282		0	0	0	282	360	
21'-0"		58	98	145	190*	194*	2.276	63	121	179	225*	229*	1.401
		0	0	0	193	248		0	0	0	248	318	
22'-0"		45	82	125	169	179*	2.742	50	103	156	209*	212*	1.687
		0	0	0	0	219		0	0	0	219	283	
23'-0"			68	107	148	165*	3.276		87	136	193	196*	2.016
			0	0	0	193			0	0	0	252	
24'-0"			56	92	129	152*	3.884		73	118	171	183*	2.390
			0	0	0	171			0	0	0	225	
25'-0"			45	78	112	141*	4.572		61	102	151	170*	2.814
			0	0	0	151			0	0	0	201	
26'-0"				66	98	131*	5.349		50	88	134	159*	3.292
				0	0	134			0	0	0	179	
27'-0"				56	85	118	6.221		41	76	118	149*	3.828
				0	0	0			0	0	0	160	
28'-0"				46	73	104	7.195			65	104	139*	4.428
				0	0	0				0	0	143	
29'-0"					63	91	8.279			55	91	128	5.095
					0	0				0	0	0	
30'-0"					53	80	9.481			46	80	114	5.835
					0	0				0	0	0	

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

PROPERTIES FOR DESIGN (CONCRETE .39 CF/SF) [4]

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.58	.58	.61	.78	.94		.58	.58	.64	.88	1.09	
STEEL % (UNIFORM)	.83	.83	.87	1.11	1.35		.83	.83	.91	1.25	1.55	
(TAPERED)	.45	.45	.47	.60	.74		.45	.45	.49	.68	.85	
EFF. DEPTH, IN.	11.8	11.8	11.8	11.8	11.7		11.8	11.8	11.8	11.8	11.7	
– ICR/IGR	.191	.191	.197	.237	.268		.191	.191	.204	.258	.296	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.31	.40	.51	.62	.75		.22	.31	.40	.51	.62	
STEEL %	.08	.10	.12	.15	.18		.05	.08	.10	.12	.15	
EFF. DEPTH, IN.	11.8	11.8	11.7	11.7	11.6		11.8	11.8	11.8	11.7	11.7	
+ICR/IGR	.145	.184	.227	.270	.317		.106	.145	.184	.227	.270	

STANDARD ONE-WAY JOISTS [1] MULTIPLE SPANS

30" Forms + 6" Rib @ 36" c.-c. [2] FACTORED USABLE SUPERIMPOSED LOAD (PSF)

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

10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth

TOP BARS	Size @	# 4 12	# 4 12	# 4 9.5	# 5 12	# 5 10	End Span Defl. Coeff. (3)	# 4 12	# 4 11.5	# 4 8.5	# 4 7	# 5 8.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6		# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	
Steel (psf)		.58	.69	.86	1.05	1.25		.63	.74	.95	1.18	1.46	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
17'-0"		180	251	309*	317*	328*	1.006	215	301	358*	365*	374*	.619
		0	0	322	404	478*		0	0	404	508	570*	
18'-0"		151	214	278	288*	297*	1.264	182	259	328*	334*	342*	.778
		0	0	0	351	425		0	0	351	444	525*	
19'-0"		127	184	241	263*	271*	1.569	155	224	301*	307*	314*	.966
		0	0	0	306	373		0	0	306	390	481*	
20'-0"		106	157	209	241*	248*	1.926	131	194	268	284*	289*	1.185
		0	0	0	268	328		0	0	0	343	429	
21'-0"		88	135	182	221*	228*	2.342	111	168	235	263*	267*	1.441
		0	0	0	235	290		0	0	0	303	381	
22'-0"		73	115	158	204*	210*	2.820	94	145	207	244*	248*	1.736
		0	0	0	207	256		0	0	0	269	340	
23'-0"		59	98	137	182	194*	3.369	78	126	182	227*	231*	2.073
		0	0	0	0	227		0	0	0	239	303	
24'-0"		48	83	119	160	180*	3.995	65	108	160	212*	215*	2.458
		0	0	0	0	202		0	0	0	0	272	
25'-0"			70	103	141	167*	4.703	53	93	141	189	201*	2.894
			0	0	0	179		0	0	0	0	244	
26'-0"			58	89	124	156*	5.502	43	80	123	168	188*	3.386
			0	0	0	159		0	0	0	0	219	
27'-0"			48	76	108	141	6.398	68	108	150	176*		3.938
			0	0	0	0		0	0	0	197		
28'-0"				65	95	125	7.400		57	95	133	166*	4.554
				0	0	0			0	0	0	177	
29'-0"				54	82	111	8.516		47	82	118	156*	5.240
				0	0	0			0	0	0	159	
30'-0"				45	71	98	9.752			71	105	143	6.001
				0	0	0				0	0	0	

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

PROPERTIES FOR DESIGN (CONCRETE .41 CF/SF) [4]

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	. 60	.60	.76	.93	1.12		.60	.63	.85	1.03	1.31	
STEEL % (UNIFORM)	.73	.73	.92	1.14	1.37		.73	.76	1.03	1.25	1.61	
(TAPERED)	.43	.43	.54	.66	.80		.43	.44	.60	.73	.94	
EFF. DEPTH, IN.	11.8	11.8	11.8	11.7	11.7		11.8	11.8	11.8	11.8	11.7	
– ICR/IGR	.179	.179	.214	.246	.280		.179	.185	.232	.268	.314	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.40	.51	.62	.75	.88		.31	.40	.51	.62	.75	
STEEL %	.09	.12	.15	.18	.21		.07	.09	.12	.15	.18	
EFF. DEPTH, IN.	11.8	11.7	11.7	11.6	11.6		11.8	11.8	11.7	11.7	11.6	
+ICR/IGR	.162	.200	.239	.280	.323		.128	.162	.200	.239	.280	

One-way Joists

Joist - Band Beams

Wt. of slab

$\hookrightarrow (27')\left(\frac{3}{12}\right)(150\,pcf) = 825\,plf$

Joists

$\hookrightarrow \left(7\frac{1}{3}\right)(27')\left(\frac{6'' \times 10''}{144}\right)(150\,pcf) = 12374.4^{lb}$

$$\frac{12374.4^{lb}}{22'} = 562.5\,plf$$

$DL = 1387\ ^{lb}/ft + (10\,psf)(27) = 1657\ ^{lb}/ft$

$LL = 40\,psf \times 27' = 1080\ ^{lb}/ft$

Factored Loads:

$$1.4(1657) + 1.7(1080) = 4155.8\ ^{lb}/ft$$
$$= 4.2\ ^{k}/ft$$

Joist Band   Pg. 12-95

$h = 12.5'' \Rightarrow$ use 13" to match Joist depth

$b = 36''$

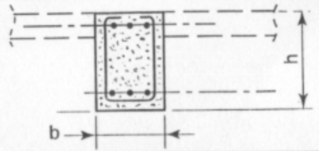
Span 22' can carry $5.3\ ^{k}/ft > 4.2\ ^{k}/ft$

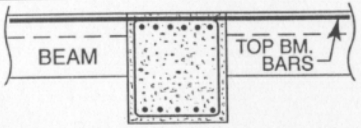
Reinforcing Bars

Bottom	Top
2-#9	5-#10

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

JOIST-BAND BEAMS, INTERIOR SPANS





STEM		BARS(1)				TOTAL CAPACITY $U = 1.4D + 1.7L$(3)																				$+\phi M_n$ $-\phi M_n$	DEFL (C)
h	b	BOTTOM		Lay-ers	TOP	SPAN, ℓ_n = 20 ft					SPAN, ℓ_n = 22 ft					SPAN, ℓ_n = 24 ft					SPAN, ℓ_n = 26 ft					(6)	(7) $\times 10^{-9}$
in.	in.	ℓ_n + 12 in.	0.875 ℓ_n	(2)		LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	ft-kip	in.
		2# 6	1# 6	1	4# 6	2.1	103C	6	-	222	1.7	093C	6	-	233	1.4	083C	6	-	243	1.2	073C	6	-	254	57	1126
				1			243C	24	1.0	351		263C	24	1.0	382		293C	24	1.0	422		313C	24	1.0	453	75	
		2# 7	1# 7	1	4# 7	2.7	143C	6	-	304	2.3	143C	6	-	327	1.9	133C	6	-	343	1.6	133C	6	-	365	76	1550
				1			243C	24	1.0	411		263C	24	1.0	448		293C	24	1.0	493		313C	24	1.0	531	99	
	24	2# 8	1# 8	1	4# 9	3.9	173C	6	-	429	3.2*	183C	6	-	468	2.7*	183C	6	-	501	2.3*	193C	6	-	539	97	1597
				1			243C	24	1.0	519		263C	24	1.0	567		293C	24	1.0	623		313C	24	1.0	671	152	
		2# 9	1# 9	1	4#10	4.8*	233B	6	-	553	3.9*	243B	6	-	600	3.3*	253B	6	-	648	2.8*	263B	6	-	695	119	1464
				1			303B	24	1.0	655		333B	24	1.0	719		363B	24	1.0	784		393B	24	1.0	849	182	
		2# 6	2# 6	1	5# 7	3.1	N/A	10	-	244	2.5	N/A	10	-	269	2.1	N/A	10	-	293	1.8	N/A	10	-	317	76	737
				1			403A	41	1.5	623		443A	41	1.5	685		483A	40	1.5	747		**	40	0.0	317	125	
		2# 7	2# 7	1	5# 8	4.1	N/A	10	-	325	3.4	N/A	10	-	357	2.8	N/A	10	-	389	2.4	N/A	10	-	421	102	1038
				1			403A	41	1.5	704		443A	40	1.5	774		483A	40	1.5	844		523A	40	1.5	914	160	
12.5	36	2# 9	2# 9	1	5#10	6.4*	223B	10	-	678	5.3*	233B	10	-	737	4.5*	243B	10	-	796	3.8*	253B	10	-	855	161	1054
				1			403A	41	1.5	910		443A	40	1.5	1001		483A	40	1.5	1091		523A	40	1.5	1181	237	
		2#10	2#10	1	5#11	7.6*	233B	10	-	817	6.3*	253B	10	-	896	5.3*	263B	10	-	968	4.5*	273B	10	-	1040	197	957
				1			403A	41	1.5	1043		443A	40	1.5	1146		483A	40	1.5	1249		523A	40	1.5	1353	276	
		3# 6	3# 6	1	6# 7	4.2	N/A	15	-	326	3.5	N/A	14	-	358	2.9	N/A	14	-	390	2.5	N/A	14	-	422	114	577
				1			**	58	4.2	326		**	58	4.3	358		**	57	0.0	390		**	57	0.0	422	152	
		3# 7	3# 7	1	6# 9	6.1	153C	15	-	607	5.0	N/A	14	-	541	4.2	N/A	14	-	590	3.6	N/A	14	-	639	152	817
				1			**	58	4.2	493		**	58	4.2	541		**	57	4.1	590		**	57	4.1	639	238	
	48	3# 8	3# 8	1	6#10	7.8	173C	15	-	763	6.4*	183C	14	-	833	5.4*	183C	14	-	895	4.6*	193C	14	-	965	194	818
				1			**	58	2.1	633		**	58	2.0	695		**	57	2.0	758		**	57	4.1	820	290	
		3# 9	3# 9	1	6#11	9.4*	233B	15	-	968	7.7*	243B	14	-	1054	6.5*	253B	14	-	1140	5.5*	263B	14	-	1226	238	757
				1			**	58	2.1	792		**	58	2.0	870		**	57	2.0	949		**	57	2.0	1027	340	

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

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

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

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

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

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

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

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