CHRISTIANA HOSPITAL 2010 PROJECT

NEWARK, DE

Technical Report #2

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Structural Option
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Executive Summary

This paper is designed to investigate alternative solutions for the floor system of the Christiana Hospital project. After completing this investigation I will compare these alternate systems to each other and to the original system to see in what areas each performs best. In the end I will explain which system I feel best suits the Christiana Hospital and potentially find other systems that warrant further research.

The five alternate systems that will be analyzed are as follow:

- Non-Composite Steel Frame
- Composite Steel Frame
- Prestressed Hollow Core Plank
- Waffle Slab
- Shear Reinforcement in Slab Immediately Surrounding Columns (Replaces Drop Panels)

Conclusion:
After analyzing and gaining a better understanding of the five alternative solutions it was obvious that the current floor system is the best system for this application. The existing floor system works well for the large spans and somewhat varying column placement in the building.

Looking at the five alternatives it is obvious that both the non-composite and the hollow core plank systems do not work well in this situation. Due to the varying layout these designs are not at all advantageous. They also create large member sizes that cause the floor thickness to be much deeper than the original floor system produced. On the other hand, three alternatives that will be further researched are the composite floor system, the waffle slab, and replacing the drop panels with shear reinforcement. While these three alternatives performed well enough to be researched further, I feel that the current two-way flat slab is the best solution for this structure.

Gravity Loading

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Assembly Space	100
Typical Hospital Floor	60
Corridor	80
Mechanical Rooms	150
Stair	100
Roof	15
Partition	20

Floor Dead Loads	
Occupancy or Use	**Dead Load**
Reinforced Concrete	150 pcf
Steel Members	Varies
Floor Superimposed	15 psf
Roof Superimposed	15 psf

Snow Loading	
Item	**Value**
Ground Snow Load (P_g)	25 psf
Exposure Category	B
Roof Exposure	Partially Exposed
Exposure Factor (C_e)	1.0
Thermal Factor (C_t)	1.0
Occupancy Category	IV
Importance Factor (I_s)	1.2
Flat-Roof Snow Load $P_f = 0.7C_eC_tI_sP_g$	21 psf

Existing Floor Framing

The framing of the Christiana Hospital project is currently a combination of both concrete and steel. The main portion of the hospital is concrete with a stand alone adjacent steel framed conference wing. The concrete portion of the building stands 8 stories with one level underground and a penthouse roof. The structure contains varying spans, a majority being in the range of 30', which are created using a typical 9½ inch thick two-way flat slab with 5½ inch drops or shear caps. This slab transfers load to 24 inch square columns which in turn take the load down to a mat foundation. To prevent rotation and lateral displacement due to wind or seismic loading shear walls are strategically placed perpendicular to the buildings perimeter.

The conference wing is a 3 story structural steel frame with a majority of beams having pinned connections and spanning around 30 feet. In the center of this area is a larger span of over 60 feet. The buildings loads are transferred to the beams using a 3¼ inch, light weight concrete, structural slab over a 2 inch deep by 18 gage galvanized composite metal deck creating a total slab thickness of 5¼ inches. The load in the beams is transferred to steel girders which are attached using a pinned connection to W-shaped columns. These columns continue down to 4000 psi concrete spread footings. The wind and seismic loading in this area is distributed using concentrically braced frames.

This paper will focus on the main building comprised of concrete framing. One of the larger more typical bays will be looked at in order to gain a better overall view of my framing alternatives. The bay size that will be looked at is 30' x 28'-6". Loads, as stated in the previous section, will be used for the member sizing and building plans can be viewed in the Appendix.

Framing Alternative #1
Non-Composite Steel Frame

The first framing alternative taken into consideration was a non-composite steel framing system. This system consists of a 2" lightweight concrete slab placed on 22 gage 2" high x 6-1/8" pitch x 24-1/2" wide Versa-Deck S (see Appendix pages 13-16). The metal deck then spans to W16x89 joists that are simply supported by W21x166 girders. Although these beam and girder sizes are not the most economical they were chosen based on their size in an attempt to keep ceiling to floor heights to a minimum for architectural reasons.

Pros:

- Quick erection time after the fabrication is complete.
- Less room for error in the erection process.
- Lighter than the concrete frame creating lower seismic loads allowing for the foundation to be redesigned.

Cons:

- Most likely more costly than concrete due to fact that the shapes were chosen to be smaller in depth to try and keep the floor thickness small making them less economical.
- Steel members will require additional fireproofing that will add both labor and material costs.
- The floor thickness has been increased to 26½" or 2'-2½". While this new thickness may not directly effect the building aesthetically, due to the fact that there is a 3'-4" allowance for structural use including a drop ceiling, it will most likely have a negative affect on the way the MEP is designed and installed.

Framing Alternative #2
Composite Steel Frame

The second framing alternative taken into consideration was a composite steel frame. The frame consists of a USD 2" Lok-Floor with 3¼ inches of concrete placed on top. The concrete and decking work in composite action with the beams below using ¾" diameter shear studs spaced evenly. The load is transferred from the slab to the W14x22 beams below and the composite action is formed using 22 studs. This load is then transferred to W18x35 girders which are directly attached to the columns (See Appendix page 17).

Pros:

- The floor thickness is not nearly as deep as when the non-composite system was looked at. This system has a depth of only 23". While this may merely be a 3½" difference from the non-composite floor, the cost of the composite floor will be lower due to the fact that more economical shapes were allowed to be used.
- Erection time for this frame will also be quicker than that of concrete once the members are fabricated.
- The composite action will work well with vibrations.
- Since the conference wing is also constructed using a composite floor system, it may cut down on the amount of sub contractors needed for the job making the job slightly easier to manage.
- Lighter than concrete frame creating lower seismic loads and allowing for the foundation to be redesigned.

Cons:

- More difficult to fireproof than the original concrete system. Spray on fireproofing or extra layers of gypsum will be required to be added around the members.
- While the floor depth is thinner than both the non-composite and the hollow core plank systems, it is still deeper than the original floor system which may cause trouble for the MEP engineers.

Framing Alternative #3
Prestressed Hollow Core Plank

The third framing alternative taken into consideration was a prestressed concrete hollow core system on non-composite steel girders. This system is composed of 8” x 4’ Spandeck with a 2” cast-in-place concrete topping. The deck spans 28’-6” to W24x162 steel girders. These girders were controlled by the deflection criterion of L/360 and the size with the smallest depth was chosen in an effort to keep the floor thickness as small as possible. Calculations and tables can be viewed on pages 18-20 of the Appendix.

Pros:

- Quick erection time after fabrication is complete which will cut down on labor costs.
- Because the concrete and prestressing for these panels is done in a controlled environment the quality and strength of the panels can be higher than that of concrete formed in the field.

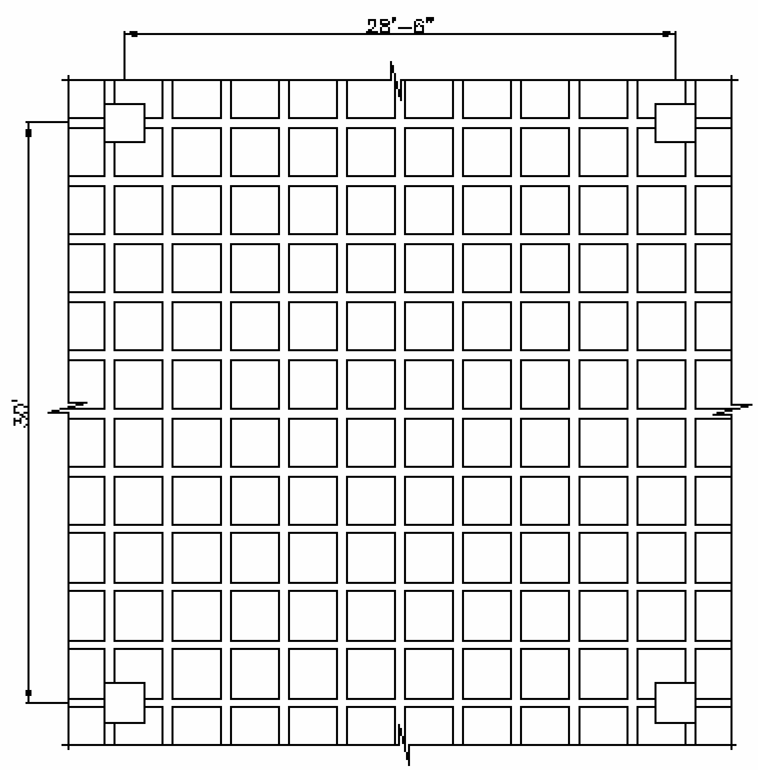
Cons:

- Due to the weight of the panels they cause the girders supporting them to be quite large. These large girders in addition to the 8 inch deck and 2 inches of concrete topping add up to a 35” floor thickness. This is much larger than the current construction using a 9½” two-way slab. As in alternative #1 this may not interfere architecturally due to the drop ceiling but will definitely interfere with the MEP design and installation.
- While the hollow core planking acts great in the event of a fire the steel members that this planking is resting on will require some additional form of fireproofing whether it be sprayed on or additional gypsum or drywall.

Framing Alternative #4
Waffle Slab

The fourth floor alternative taken into account was a waffle slab. The waffle slab is composed of standard 30”x30” domes. Using CRSI to design the slab, column strip and middle strip reinforcement can be seen below (See pages 21-22 of the Appendix.

Column Strip				Middle Strip			
# of Ribs	Short Bars	Long Bars	Top Bars	# of Ribs	Short Bars	Long Bars	Top Bars
5	#6	#6	21#6	5	#5	#5	15#4



Pros:
- Considerable reduction in dead load as compared to conventional solid flat slab construction.
- Use of drip panels or support beams not needed.
- Easily accommodates electrical and mechanical utilities.
- Has inherent fire resistance.
- Only 13” thick.

Cons:
- Difficult to form and construct due to non uniformity in building.

Framing Alternative #5
Shear Reinforcement (No Drop Panels)

The final floor system that was taken into consideration was a 9½" two-way flat slab utilizing shear reinforcement in the slab immediately surrounding the columns. This system is merely a modification of the original two-way flat slab and is an attempt to reduce floor thickness by adding shear reinforcement where the 5½" drops originally were. By calculating the amount of shear reinforcement needed I found that it is possible to replace the 5½" drops with #3 double u-stirrups. Refer to pages 23-24 of the Appendix for calculations.

Pros:

- Thinner floor depth around columns.
- Existing two-way slab works well for the building geometry.
- No additional fireproofing need.
- Carpenters do not need to form the 5½" drops.

Cons:

- More expensive for stirrup placement as opposed to forming a concrete drop panel.
- Will take longer to place stirrups as opposed to formwork.
- May not be noticed due to interior hung ceiling.

Comparison & Conclusions

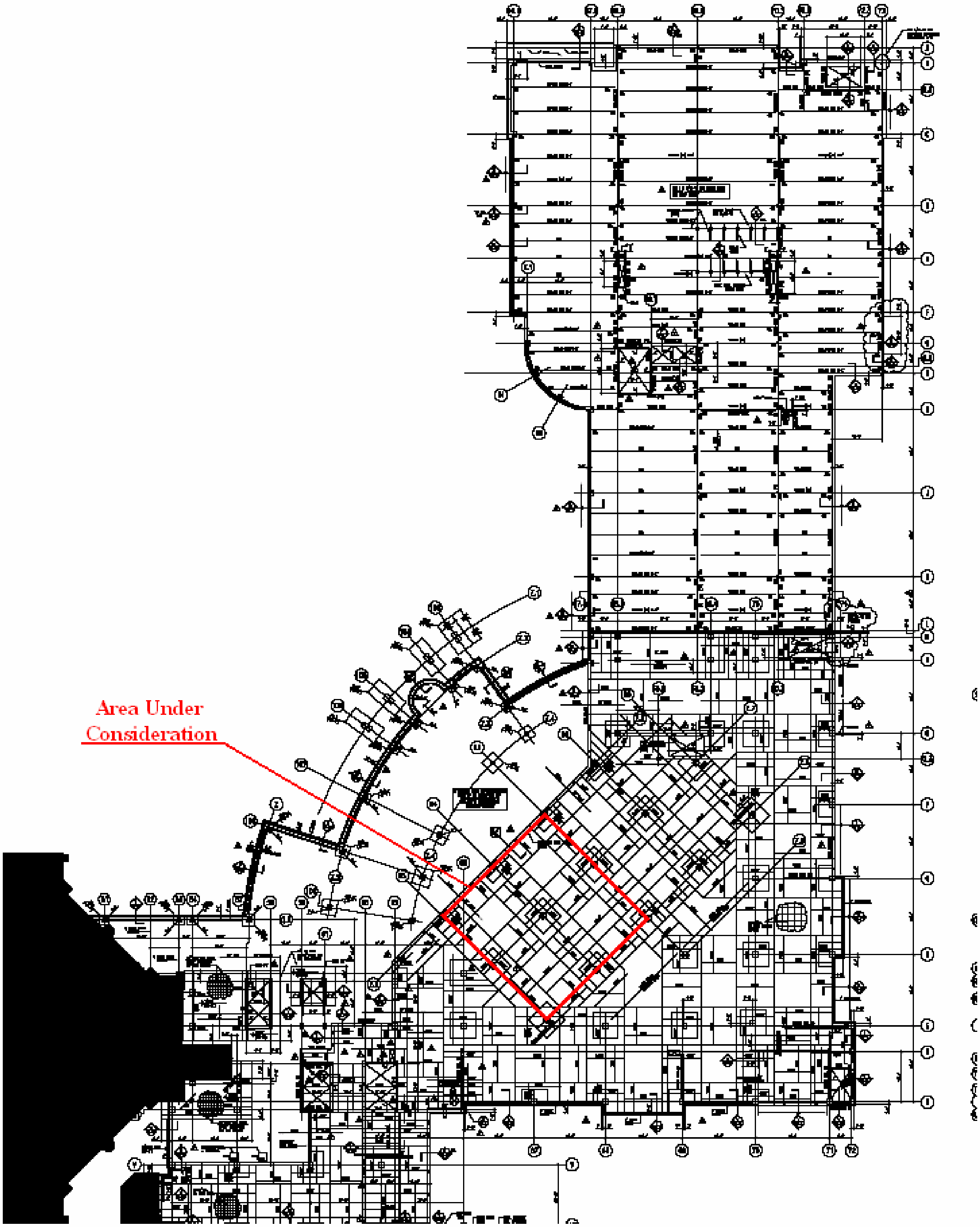
	Existing	Non-Comp. Steel	Comp. Steel	Hollow Core Plank	Waffle Slab	Shear Reinf.
Floor Thickness	9 ½”	26 ½”	23”	35”	13”	9 ½” *
Add. Fire Protection Required	No	Yes	Yes	Yes	No	No
Prefab Time	No	Yes	Yes	Yes	No	No
Formwork	Yes	No	No	No	Yes	Yes
Fast erection time	No	Yes	Yes	Yes	No	No
Foundation Redesign	No	Yes	Yes	No	Possibly	No
Possible Solution	Yes	No	Yes	No	Yes	Yes

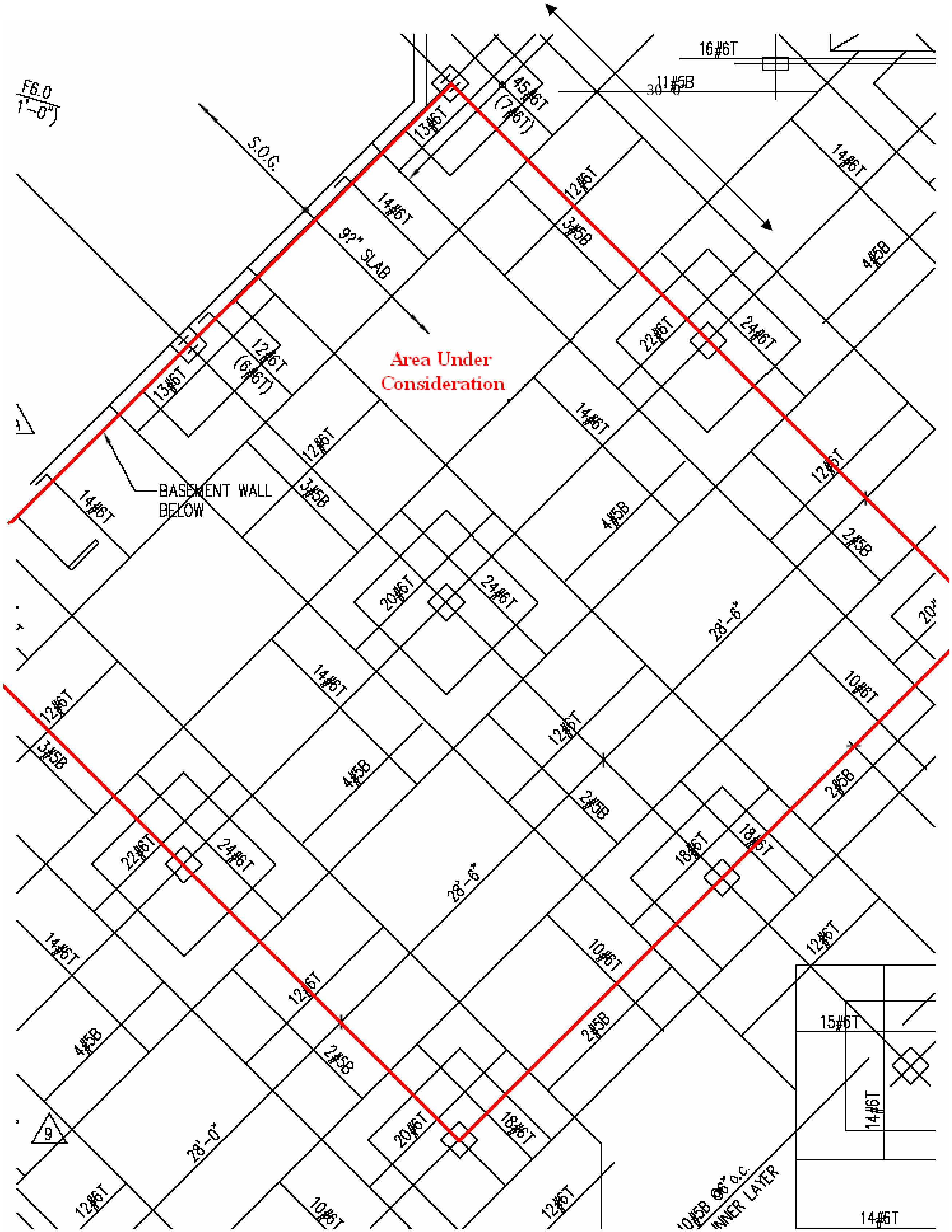
* Requires no drop panels around columns.

Conclusion:
After analyzing and gaining a better understanding of the five alternative solutions it was obvious that the current floor system is the best system for this application. The existing floor system works well for the large spans and somewhat varying column placement in the building.

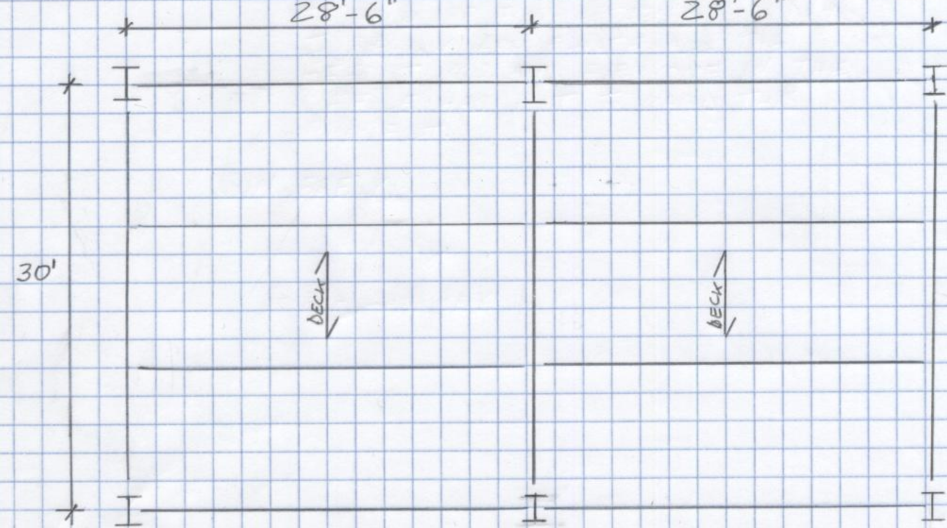
Looking at the five alternatives it is obvious that both the non-composite and the hollow core plank systems do not work well in this situation. Due to the varying layout these designs are not at all advantageous. They also create large member sizes that cause the floor thickness to be much deeper than the original floor system produced. On the other hand, three alternatives that will be further researched are the composite floor system, the waffle slab, and replacing the drop panels with shear reinforcement. While these three alternatives performed well enough to be researched further, I feel that the current two-way flat slab is the best solution for this structure.

Appendix




Area Under
Consideration
BASEMENT WALL
BELOW
S.O.G.
9 1/2" SLAB
28'-6"
28'-0"
16#6T
11#5B
30'-0"
15#6T
14#6T
10#5B @6" o.c.
INNER LAYER

NON-COMPOSITE STEEL FRAME

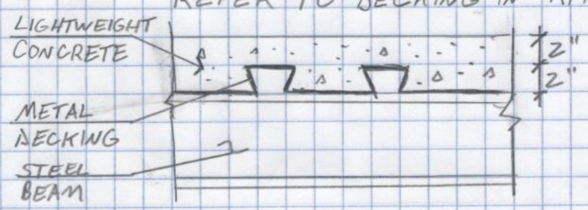


LOADING

LIVE: HOSPITAL 1st FLOOR = 100 psf

DEAD: SUPERIMPOSED FLOOR = 15 psf

SPACE JOISTS @ 10' O.C. FOR USE WITH
22 GAGE 2" HIGH x 6-1/8" PITCH x 24-1/2" WIDE VERSA-DECK S w/2" CONC.
REFER TO DECKING IN APPENDIX



DESIGN STEEL JOIST

$LOAD = 1.2D + 1.6L$

LIVE

HOSPITAL FLOOR = 100 psf

100 psf × 10' = 1000 plf

DEAD

CONC. SLAB = 115 pcf (4") = 38 psf × 10' = 380 plf

STEEL JOIST = ASSUME 30 plf

SUPERIMPOSED DEAD LOAD = 15 psf × 10' = 150 plf

$W_u = 1.2(380+30+150) + 1.6(1000) = 2272 \text{ plf}$

$M_u = \frac{w\ell^2}{8} = \frac{2272(28.5)^2}{8} = 230.7 \text{ ft-k}$

USE W14x61 $\phi M_n = 231$ ft-k @ 28.5' FROM BEAM TABLES

$\phi V_n = 141^k$

$V_u = \frac{w\ell}{2} = \frac{2272(28.5)}{2} = 32.4^k < 141^k \therefore OK$

CHECK DEFLECTION

$I = 640 \text{ in}^4$

$\Delta = \frac{5w\ell^4}{384EI} = \frac{5(2272 \times \frac{1}{12000})(28.5 \times 12)^4}{384(29000)(640)} = 1.82 \text{ in}$

$\Delta_{max} = \frac{28.5 \times 12}{360} = 0.95 \text{ in} < 1.82 \therefore$ NO GOOD

$0.95 = \frac{5(2272 \times \frac{1}{12000})(28.5 \times 12)^4}{384(29000)I} \Rightarrow I = 1224 \text{ in}^4$

USE W16x89 $I = 1310 \text{ in}^4$ ⇒ NOT MOST EFFICIENT BUT USE FOR SMALLER SIZE.

RECHECK FOR NEW BEAM WEIGHT

$W_u = 1.2(380+89+150) + 1.6(1000) = 2343 \text{ plf}$

$\Delta = \frac{5(2343 \times \frac{1}{12000})(28.5 \times 12)^4}{384(29000)(1310)} = 0.92 \text{ in} < 0.95 \text{ in} \therefore OK$

$M_u = \frac{2343(28.5)^2}{8} = 238 \text{ ft-k} < 451 \text{ ft-k} \therefore OK$

$V_u = \frac{2339(28.5)}{2} = 33.3 K < 238 K \therefore OK$

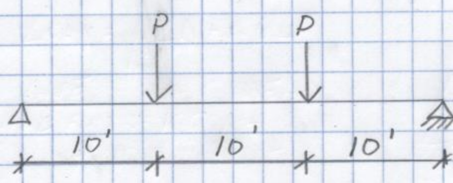
THIS IS ONE OF THE SMALLEST SIZES THAT WILL WORK FOR ALL CRITERION.

DESIGN STEEL GIRDER

FROM JOIST FRAMING IN ON EACH SIDE.

$P = 2\left(\frac{wl}{2}\right) = wl = 2343(28.5)$

$= 66776.16 = 66.8^{K}$



$M_u = Pa = 66.8(10') = 668'^{K}$

$V_u = P = 66.8K$

$\Delta_{max} = \frac{30 \times 12}{360} = 1''$

$\Delta = 1'' = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{66.8(10 \times 12)}{24(29000)I}(3(30 \times 12)^2 - 4(10 \times 12)^2)$

$\Rightarrow I = 3809 \text{ in}^4$

TRY W21x166 (ALTHOUGH THIS IS NOT MOST EFFICIENT SHAPE STILL ATTEMPT TO USE TO KEEP CEILING TO FLOOR HEIGHT SMALL)

ADD IN BEAM WEIGHT

$w_u = 1.6(166) = 266 \text{ plf}$

$\Delta = \frac{66.8(10 \times 12)(3(30 \times 12)^2 - 4(10 \times 12)^2)}{24(29000)(4280)} + \frac{5(266/12000)(30 \times 12)^4}{384(29000)(4280)} = 0.94'' < 1 \therefore OK$

$M_u = 668'^{K} + \frac{226/1000(30)^2}{8} = 693'^{K} < 1242'^{K} \therefore OK$

$V_u = 66.8 + \frac{226/1000}{2} = 66.9^{K} < 456^{K} \therefore OK$

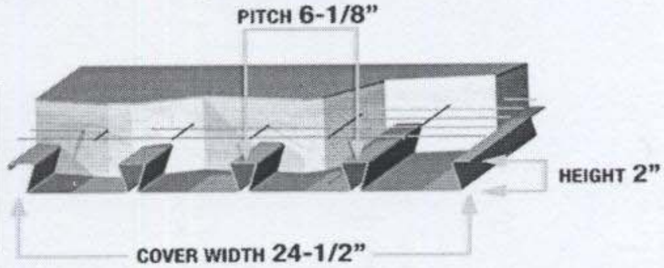
MAX DEPTH OF CONSTRUCTION

2" SLAB + 2" DECK + 22.5" GIRDER = 26.5"

Metal Dek Group™
a unit of CSi

VERSA-DEK® S

2" high x 6-1/8" pitch x 24-1/2" wide



SECTION PROPERTIES fy = 40 ksi

GAGE	t (in)	Ip (in^4)	In (in^4)	Sp (in^3)	Sn (in^3)
22	0.0295	0.4027	0.3266	0.2895	0.2692
20	0.0358	0.4918	0.4251	0.3620	0.3354
18	0.0474	0.6578	0.6166	0.4852	0.4616
16	0.0598	0.8372	0.8185	0.6192	0.6000

115 PCF LIGHTWEIGHT CONCRETE

h	4"				4.25"				4.5"				4.75"				5"				5.25"			
GAGE	22	20	18	16	22	20	18	16	22	20	18	16	22	20	18	16	22	20	18	16	22	20	18	16
Wc	35.2	35.2	35.2	35.2	37.6	37.6	37.6	37.6	40.0	40.0	40.0	40.0	42.4	42.4	42.4	42.4	44.8	44.8	44.8	44.8	47.2	47.2	47.2	47.2
Ac	39.7	39.7	39.7	39.7	42.1	42.1	42.1	42.1	44.4	44.4	44.4	44.4	46.8	46.8	46.8	46.8	49.2	49.2	49.2	49.2	51.5	51.5	51.5	51.5
Iav	4.6	5.0	5.6	6.2	5.5	5.9	6.6	7.3	6.4	6.9	7.7	8.5	7.4	8.0	8.9	9.8	8.5	9.2	10.2	11.3	9.7	10.5	11.7	12.9
Sb	1.48	1.75	2.24	2.75	1.61	1.91	2.44	2.99	1.74	2.07	2.65	3.25	1.87	2.23	2.86	3.50	2.01	2.39	3.07	3.77	2.15	2.56	3.29	4.03
St	30.5	32.5	35.4	38.0	34.0	36.1	39.3	42.1	37.6	40.0	43.5	46.5	41.4	44.0	47.9	51.2	45.4	48.2	52.5	56.1	49.5	52.6	57.3	61.2
L	MAXIMUM ALLOWABLE UNIFORM LIVE LOADS, (psf) - ASD/LRFD - NO STUDS ON BEAMS																							
9'-0"	227	301	338	373	245	354	397	400	263	389	400	400	281	400	400	400	300	400	400	400	318	400	400	400
	230	278	338	373	251	304	397	400	273	330	400	400	294	356	400	400	317	383	400	400	339	400	400	400
10'-0"	170	186	246	272	204	223	289	319	235	246	337	372	251	263	390	400	267	280	400	400	284	297	400	400
	181	219	246	272	198	240	289	319	215	261	337	372	232	282	370	400	250	303	399	400	268	325	400	400
11'-0"	119	131	185	204	145	158	217	240	173	189	254	279	204	222	253	323	237	253	274	371	253	268	291	400
	145	165	185	204	158	193	217	240	172	210	254	279	186	227	293	323	200	245	323	371	215	262	347	400
12'-0"	84	93	108	157	103	114	131	185	124	137	157	215	148	162	185	249	174	190	217	285	203	221	252	280
	117	127	142	157	128	149	167	185	139	171	195	215	151	186	226	249	163	200	260	285	175	214	285	326
13'-0"	59	66	78	124	74	82	95	108	90	99	115	130	108	119	137	154	128	141	162	181	150	165	188	211
	92	100	112	124	105	117	132	145	114	137	154	169	124	153	178	196	133	165	204	225	143	177	233	256
14'-0"	41	46	56	64	52	59	69	79	65	72	85	96	79	87	102	115	94	104	121	136	112	123	142	160
	74	80	90	99	86	94	105	116	94	110	123	135	102	127	142	157	110	137	163	180	118	148	187	205
15'-0"				46		41	49	57	46	52	62	71	57	64	75	86	69	77	90	103	83	92	107	121
				80		76	86	95	78	89	100	110	85	103	116	127	91	115	133	146	98	124	152	167
16'-0"								41			44	52	40	45	55	64	49	56	67	77	60	68	80	92
								78			82	91	70	85	95	105	76	97	110	120	82	104	125	137
17'-0"																46		40	49	57	43	49	59	69
																87		82	91	100	68	88	104	115
18'-0"																				41			43	51
																				85			88	97
19'-0"																								
20'-0"																								
	MAXIMUM UNSHORED CONSTRUCTION CLEAR SPANS																							
1span	6'-10"	7'-10"	9'-4"	10'-9"	6'-8"	7'-8"	9'-1"	10'-6"	6'-6"	7'-6"	8'-11"	10'-3"	6'-4"	7'-4"	8'-8"	10'-0"	6'-3"	7'-2"	8'-6"	9'-10"	6'-2"	7'-0"	8'-4"	9'-7"
2span	8'-7"	9'-7"	11'-2"	12'-7"	8'-5"	9'-4"	10'-11"	12'-4"	8'-3"	9'-2"	10'-9"	12'-2"	8'-1"	9'-0"	10'-6"	11'-11"	7'-11"	8'-10"	10'-4"	11'-9"	7'-10"	8'-8"	10'-2"	11'-6"
3span	8'-11"	9'-11"	11'-6"	13'-1"	8'-8"	9'-8"	11'-4"	12'-9"	8'-6"	9'-6"	11'-1"	12'-7"	8'-4"	9'-4"	10'-10"	12'-4"	8'-3"	9'-2"	10'-8"	12'-1"	8'-1"	9'-0"	10'-6"	11'-11"
cantilever	2'-8"	3'-2"	4'-0"	4'-10"	2'-7"	3'-1"	3'-11"	4'-9"	2'-7"	3'-1"	3'-11"	4'-8"	2'-7"	3'-0"	3'-10"	4'-7"	2'-6"	3'-0"	3'-9"	4'-6"	2'-6"	2'-11"	3'-9"	4'-6"
cy/100sf	1.13				1.21				1.29				1.37				1.44				1.52			

9'-0"	227	← maximum allowable live load (psf) based on ASD composite design
	230	← maximum allowable live load (psf) based on LRFD composite design

clear span

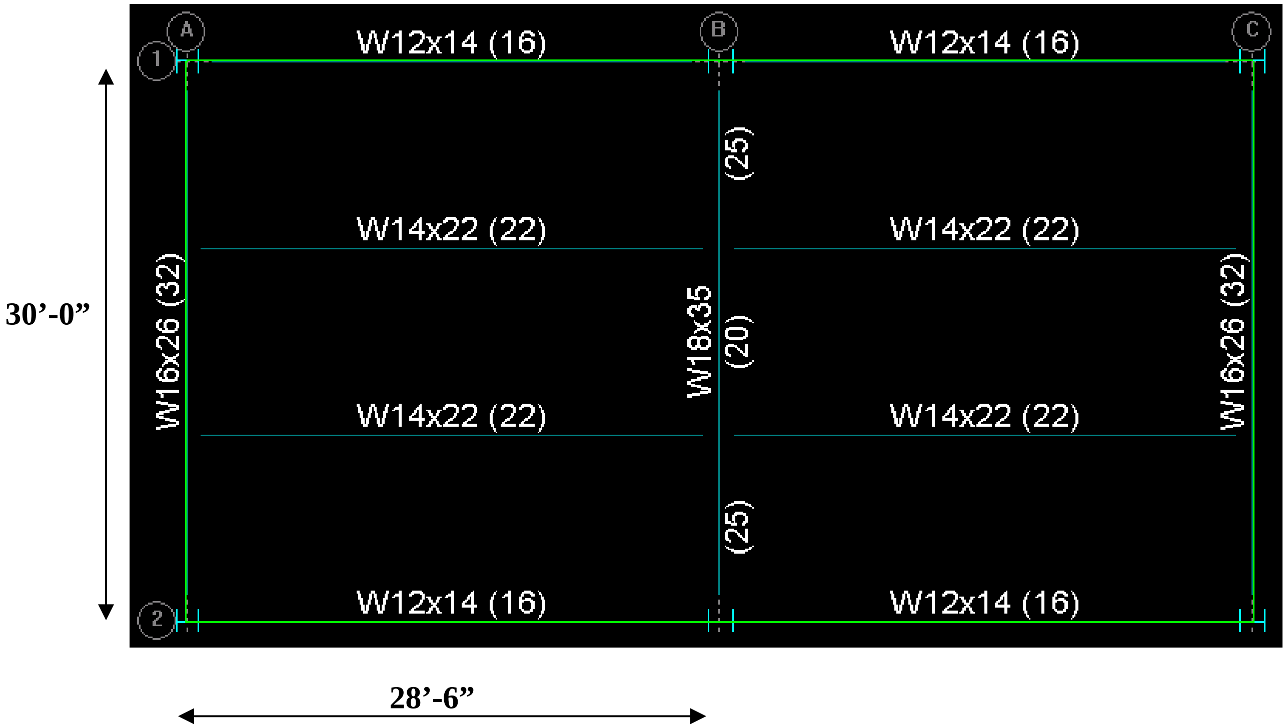
- **t** Design thickness of deck
- **Ip** Moment of inertia of deck for positive bending
- **In** Moment of inertia of deck for negative bending
- **Sp** Section modulus of deck for positive bending
- **Sn** Section modulus of deck for negative bending
- **fy** 40 ksi
- **f'c** 3000 psi
- **h** Total height of concrete slab
- **Wc** Weight of concrete (neglecting deflection)
- **Ac** Effective area of concrete available to resist shear
- **Iav** Average moment of inertia of cracked & uncracked section
- **Sb** Cracked section modulus for positive bending
- **St** Cracked section modulus for negative bending
- **L** Span length; clear distance of deck between supports

Interior bearing of 5" in the above tables. If welded wire fabric is not supplied per ACI requirements (0.00075*Ac), reduce loads by 10%. The section property table is based on AISI's Cold-Formed Steel Design Manual, 2001 Edition. The live loads and unshored construction clear spans are based on the Steel Deck Institute's Composite Deck Design Handbook, March 1997 and Design Manual, Pub. No. 30, and ASCE's Standard for the Structural Design of Composite Slabs. Maximum Unshored Construction Clear Spans are based on ASD design. The loads in these tables are based on a Simple Span Design Analysis.

Rev: 01/27/05

115 PCF LIGHTWEIGHT CONCRETE TABLE

RAM Steel Design
Composite Floor Layout – Shape (# of Shear Studs)



PRESTRESSED CONC.

LOAD
LIVE: HOSPITAL 1st FLOOR = 100 psf
DEAD: SUPERIMPOSED FLOOR = 15 psf
TOTAL LOAD = 115 psf

SPAN = 28.5'

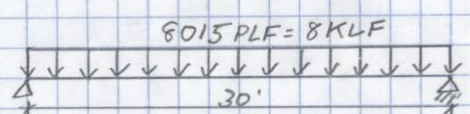
USE 8"x4' SPANDECK - U.L. - J917 w/ A
SHEAR 6 - ½" φ STRAND PATTERN
2" C.I.P. TOPPING

AT 29' THE MAX DIST. LOAD IS 132 PSF > 115 PSF ∴ OK
AS SHOWN IN THE APPENDIX.

DESIGN GIRDER THIS WILL BE RESTING ON

LOAD:
DECKING = 82.5 psf × 28.5 = 2351 pLF
ASSUME BEAM SELF WEIGHT = 100 PLF

$W_u = 1.2D + 1.6L$
$1.2(2351 + 100 + 428) + 1.6(2850) = 8015\ PLF$



$M_u = \frac{wl^2}{8} = \frac{8(30)^2}{8} = 900^{1k}$

$V_u = \frac{wl}{2} = \frac{8(30)}{2} = 120^k$

$\Delta_{max} = \frac{30 \times 12}{360} = 1'' = \frac{5(8 \cdot \frac{1}{12})(30 \times 12)^4}{384(29000)I} \Rightarrow I = 5028$

TRY W24x131
$\phi M_n = 940^{1k}$
$\phi V_n = 400^k$
$I = 4020 in^4 < 5028 in^4$ ∴ NO GOOD

TRY W24x162 (NOT MOST EFFICIENT IN ATTEMPT TO GET SMALLER FLOOR THICKNESS)
$I = 5170 in^4$
$\phi M_n = 1296$ ft-k
$\phi V_n = 476^k$

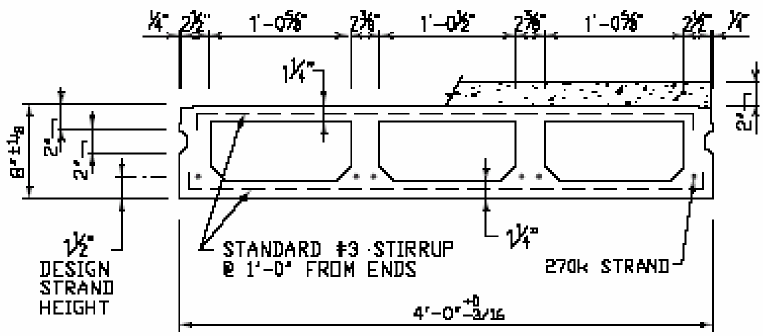
$W_u = 1.2(2351 + 162 + 428) + 1.6(2850) = 8089$ plf = 8.09 KLF

$M_u = \frac{8.09(30)^2}{8} = 910^{1k} < 1296^{1k}$ ∴ OK $V_u = \frac{8.09(30)}{2} = 121^k < 476^k$ ∴ OK

$\Delta = \frac{5(8.09 \times \frac{1}{12})(30 \times 12)^4}{384(29000)(5170)} = 0.98'' < 1''$ ∴ OK

Prestressed Concrete
8"x4' SpanDeck—U.L.—J917
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES Composite	
$A' = 254$ in.^{2}	$S'_b = 547$ in.^{3}
$I' = 2944$ in.^{4}	$S'_t = 1124$ in.^{3} (At Top of SpanDeck)
$Y'_b = 5.38$ in.	$S'_{tt} = 637$ in.^{3} (At Top of Topping)
$Y'_t = 2.62$ in. (To Top of SpanDeck)	Wt. = 330 PLF
$Y'_{tt} = 4.62$ in. (To Top of Topping)	Wt. = 82.5 PSF



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. Composite Strength = 3000 PSI.
5. Composite Density = 150 PCF.
6. Strand Height = 1.5 in.
7. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ø, 270K = 94.6'K
   6 – 1/2"ø, 270K = 133.3'K
8. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
9. All superimposed load is treated as live load in the strength analysis of flexure and shear.
10. Flexural strength capacity is based on stress/strain strand relationships.
11. Load values to the left of the solid line are controlled by ulitimate strength. Load values to the right are controlled by service stress.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. All loads shown refer to allowable loads applied after the topping has hardened.

8" SPANDECK W/2" 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	795	718	650	580	500	426	366	317	275	240	210	184	162	142	125	110	96	84	73	60	49	39	X
Shear	4	–	1/2"ø	571	509	458	415	378	347	320	296	275	257	240	222	199	178	160	145	133	126	115	103	93	84	X
Flexure	6	–	1/2"ø	1155	1040	945	859	732	629	544	474	416	366	324	287	256	228	204	183	164	147	132	118	103	90	77
Shear	6	–	1/2"ø	589	525	472	428	391	360	331	308	286	266	249	235	220	207	195	184	175	160	145	132	120	110	100

Value Used

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.

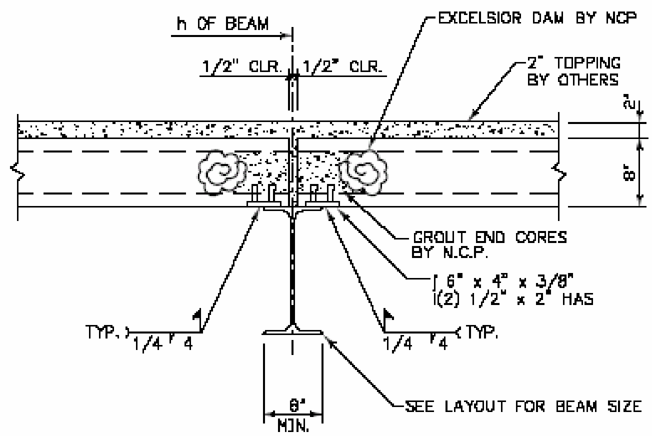
2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

REVISED 12/93

NITTERHOUSE

SPANDECK HOLLOW CORE PLANK CONNECTION

DETAIL OF BEARING ON STEEL BEAM



NOTES:

1. N.C.P. WILL PROVIDE A BROOMED FINISH IN ORDER TO CREATE A COMPOSITE TOPPING. C.I.P. TOPPING BY OTHERS IS TO BE 3,000 PSI. (NORMAL WEIGHT CONCRETE).
2. THE DESIGN OF CONNECTIONS FOR SPANDECK TO OTHER BUILDING COMPONENTS IS THE RESPONSIBILITY OF THE ENGINEER OF RECORD, SINCE THEY ARE PART OF THE GLOBAL DESIGN OF THE STRUCTURE.
3. CONSULT N.C.P. 'S ENGINEERING DEPARTMENT FOR CANTILEVER RECOMMENDATIONS
4. N.C.P. WILL PROVIDE A SMOOTH FINISH FOR INSTALLATION OF ROOFING MATERIALS BY OTHERS.
5. WELD PLATES ARE FOR BRACING THE COMPRESSION FLANGE OF THE STEEL BEAM AND FOR TRANSFERRING DIAPHRAGM FORCES. THEY ARE NOT TO HOLD THE PLANKS ON THE STEEL BEAMS. THE CONTRACT DRAWINGS SHALL INDICATE THE REQUIRED SPACING IN 4'-0" INCREMENTS.

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December 2002

WAFFLE FLAT SLAB (SQUARE PANELS)

HOSPITAL FLOOR LOAD: 100 psf
SUPERIMPOSED DEAD LOAD: 15 psf

- USE CRSI DESIGN MANUAL

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

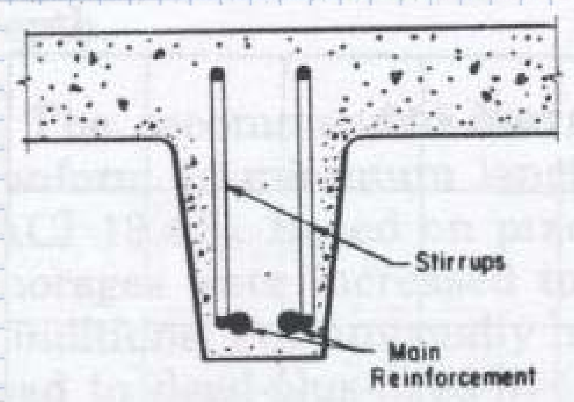
USE STANDARD 30" x 30" SQUARE DOMES

NORMAL WEIGHT CONCRETE: $f'_c = 4000$ psi
REBAR: $F_y = 60$ ksi

30' INTERIOR BAY w/ 200psf FACTORED LOAD > 191 psf

	COLUMN STRIP			MIDDLE STRIP			
# OF RIBS	LONG BARS	SHORT BARS	TOP INT	LONG BARS	SHORT BARS	TOP INT	# OF RIBS
5	#6	#6	21#6	#5	#5	15 #4	5

STIRRUPS REQUIRED
#3 @ 4" FROM FACE OF SOLID HEAD
TO THE FIRST RIB



WAFFLE FLAT SLAB SYSTEM 30" × 30" Voids; 6" Ribs @ 36"

SQUARE EDGE PANELS

Square Edge Column: ℓ_c = 12'-0"; Reinforcing Bars—Each Direction. Total Depth = 13 in. Rib Depth = 10 in. Top Slab Depth = 3 in.

Span c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Steel (1) (psf)	$c_1 = c_2$ (in.)	α_{ec}	Stirrups (2)	Column Strip: Top Edge No.-size	Column Strip Bottom: No. Ribs	Column Strip Bottom: Long Bars	Column Strip Bottom: Short Bars	Column Strip: Top Interior No.-size	Middle Strip Bottom: No. Ribs	Middle Strip Bottom: Long Bars	Middle Strip Bottom: Short Bars	Middle Strip: Top Interior No.-size	Moments: −M Edge (ft-k)	Moments: +M Bot. (ft-k)	Moments: −M Int. (ft-k)
18'-0"	50	1.39	12	0.249		13-#4	3	#3	#4	13-#4	3	#3	#4	9-#4	14	63	80
D = 6.5	100	1.43	12	0.249		13-#4	3	#4	#4	13-#4	3	#3	#4	9-#4	18	85	104
Rib on	150	1.49	12	0.249		13-#4	3	#4	#5	13-#4	3	#3	#4	9-#4	22	117	127
C. L. cf/sf	200	1.63	12	0.249		13-#4	3	#5	#5	13-#4	3	#4	#4	9-#4	26	146	151
0.611	300	2.00	12	0.249		13-#4	3	#6	#6	14-#4	3	#5	#5	9-#4	35	204	198
	400	2.44	12	0.249		13-#4	3	#7	#7	12-#5	3	#5	#6	9-#4	43	261	246
	500	2.79	12	0.249		13-#4	3	#7	#8	14-#5	3	#6	#6	9-#4	51	313	292
21'-0"	50	1.35	12	0.215		15-#4	4	#3	#4	15-#4	3	#3	#4	10-#4	21	105	133
D = 9.5	100	1.51	12	0.215		15-#4	4	#4	#5	15-#4	3	#4	#4	10-#4	27	143	171
Rib not on	150	1.66	12	0.215		15-#4	4	#5	#5	15-#4	3	#4	#5	10-#4	33	195	210
C. L. cf/sf	200	1.98	12	0.215		15-#4	4	#5	#6	18-#4	3	#5	#6	10-#4	39	243	248
0.649	300	2.58	12	0.215		15-#4	4	#6	#7	15-#5	3	#6	#7	10-#4	51	338	325
	400	3.08	12	0.215		15-#4	4	#7	#8	13-#6	3	#7	#7	10-#4	62	416	395
	500	3.31	16 *	0.581		15-#4	4	#7	#8	15-#6	3	#7	#7	11-#4	150	430	449
24'-0"	50	1.45	12	0.221		17-#4	4	#4	#5	17-#4	4	#3	#4	12-#4	31	157	198
D = 9.5	100	1.65	12	0.221		17-#4	4	#5	#5	18-#4	4	#4	#4	12-#4	41	213	256
Rib not on	150	2.07	12	0.221		17-#4	4	#6	#6	23-#4	4	#5	#5	12-#4	50	291	314
C. L. cf/sf	200	2.50	12	0.221		17-#4	4	#7	#7	17-#5	4	#5	#6	12-#4	60	364	372
0.626	300	2.96	16 *	0.585		17-#4	4	#7	#8	15-#6	4	#6	#6	12-#4	155	423	462
	400	3.31	20 *	1.172		17-#5	4	#7	#8	18-#6	4	#6	#6	13-#4	273	434	542
27'-0"	50	1.63	13	0.287		19-#4	4	#5	#6	20-#4	5	#4	#4	13-#4	55	217	279
D = 9.5	100	1.93	13	0.287		19-#4	4	#6	#6	26-#4	5	#4	#5	13-#4	72	288	361
Rib not on	150	2.40	13	0.287		19-#4	4	#7	#7	21-#5	5	#5	#5	13-#4	88	388	442
C. L. cf/sf	200	2.67	17 *	0.702		19-#4	4	#7	#7	17-#6	5	#5	#6	13-#4	189	399	499
0.611	300	3.18	21 *	1.310		21-#5	4	#7	#8	21-#6	5	#5	#6	15-#4	332	434	625
30'-0"	50	1.84	15	0.389		22-#4	5	#5	#6	28-#4	5	#4	#5	15-#4	97	294	385
D = 12.5	100	2.29	15	0.389		22-#4	5	#6	#7	23-#5	5	#5	#5	15-#4	125	379	497
Rib on	150	2.60	19 *	0.775		22-#4	5	#6	#7	20-#6	5	#5	#6	15-#4	234	419	583
C. L. cf/sf 0.637	200	3.05	23 *	1.283		22-#5	5	#7	#7	23-#6	5	#5	#6	16-#4	348	451	662
33'-0"	50	2.15	16	0.461		24-#4	5	#6	#7	24-#5	6	#4	#5	16-#4	144	382	506
D = 12.5	100	2.57	20 *	0.842		25-#4	5	#7	#7	21-#6	6	#5	#5	16-#4	266	450	632
Rib on C. L. cf/sf 0.622	150	3.13	24 *	1.334		25-#5	5	#7	#8	26-#6	6	#5	#6	18-#4	400	506	746
36'-0"	50	2.32	28	1.752		35-#4	5	#6	#7	29-#5	7	#4	#5	18-#4	366	399	607
D = 12.5	100	3.03	29 *	1.886		31-#5	5	#7	#8	27-#6	7	#5	#5	19-#4	483	508	778
Rib on C. L. cf/sf 0.611	150	3.66	29 *	1.884	S	27-#6	5	#8	#8	33-#6	7	#5	#6	24-#4	587	618	946

SQUARE INTERIOR PANELS

Sq. Interior Column: ℓ_c = 12'-0"; Reinforcing Bars—Each Direction. Total Depth = 13 in. Rib Depth = 10 in. Top Slab Depth = 3 in.

Span c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Steel (1) (psf)	$c_1 = c_2$ (in.)	α_{ec}	Stirrups (2)	Column Strip Bottom: No. Ribs	Column Strip Bottom: Long Bars	Column Strip Bottom: Short Bars	Column Strip: Top Interior No.-size	Middle Strip Bottom: No. Ribs	Middle Strip Bottom: Long Bars	Middle Strip Bottom: Short Bars	Middle Strip: Top Interior No.-size
18'-0"	50	1.40	12	0.111		3	#3	#4	13-#4	3	#3	#4	9-#4
D = 6.5	100	1.40	12	0.111		3	#3	#4	13-#4	3	#3	#4	9-#4
Rib on	150	1.40	12	0.111		3	#3	#4	13-#4	3	#3	#4	9-#4
C. L. cf/sf	200	1.47	12	0.111		3	#4	#4	13-#4	3	#3	#4	9-#4
0.611	300	1.68	12	0.111	S	3	#4	#5	13-#4	3	#4	#4	9-#4
	400	1.98	12	0.111	S	3	#5	#5	16-#4	3	#4	#5	9-#4
	500	2.24	13	0.140	S	3	#5	#6	12-#5	3	#4	#5	9-#4
21'-0"	50	1.37	12	0.100		4	#3	#4	15-#4	3	#3	#4	10-#4
D = 9.5	100	1.37	12	0.100		4	#3	#4	15-#4	3	#3	#4	10-#4
Rib not on	150	1.45	12	0.100		4	#4	#4	15-#4	3	#3	#4	10-#4
C. L. cf/sf	200	1.67	12	0.100		4	#4	#5	16-#4	3	#4	#4	10-#4
0.649	300	2.17	12	0.100	S	4	#5	#5	14-#5	3	#5	#5	10-#4
	400	2.52	16 *	0.238	S	4	#5	#6	16-#5	3	#5	#6	10-#4
	500	2.89	17 *	0.279	S	4	#6	#6	14-#6	3	#5	#6	10-#4
24'-0"	50	1.38	12	0.097		4	#3	#4	17-#4	4	#3	#4	12-#4
D = 9.5	100	1.45	12	0.097		4	#4	#4	17-#4	4	#3	#4	12-#4
Rib not on	150	1.82	12	0.097		4	#5	#5	20-#4	4	#4	#4	12-#4
C. L. cf/sf	200	2.18	12	0.097	S	4	#5	#6	16-#5	4	#4	#5	12-#4
0.626	300	2.63	16 *	0.213	S	4	#6	#6	20-#5	4	#5	#5	12-#4
	400	3.01	20 *	0.335	S	4	#6	#7	17-#6	4	#5	#5	12-#4
27'-0"	50	1.43	13	0.114		4	#4	#4	19-#4	5	#3	#4	13-#4
D = 9.5	100	1.70	13	0.114		4	#5	#5	23-#4	5	#3	#4	13-#4
Rib not on	150	2.11	13	0.139	S	4	#5	#6	19-#5	5	#4	#4	13-#4
C. L. cf/sf	200	2.44	17	0.212	S	4	#6	#6	22-#5	5	#4	#5	13-#4
0.611	300	2.91	21 *	0.295	S	4	#6	#7	20-#6	5	#4	#5	14-#4
30'-0"	50	1.63	15	0.160		5	#4	#5	25-#4	5	#3	#4	15-#4
D = 12.5	100	2.02	15	0.160		5	#5	#5	21-#5	5	#4	#4	15-#4
Rib on	150	2.38	18	0.241	S	5	#5	#6	25-#5	5	#4	#5	15-#4
C. L. cf/sf 0.637	200	2.85	22	0.342	S	5	#6	#6	21-#6	5	#5	#5	15-#4
33'-0"	50	1.95	16	0.169		5	#5	#5	22-#5	6	#4	#4	16-#4
D = 12.5	100	2.36	18	0.211	S	5	#5	#6	28-#5	6	#4	#5	16-#4
Rib on C. L. cf/sf 0.622	150	2.85	22 *	0.285	S	5	#6	#7	24-#6	6	#4	#5	17-#4
36'-0"	50	2.17	18	0.185	S	5	#5	#6	28-#5	7	#4	#4	18-#4
D = 12.5	100	2.81	20	0.215	S	5	#6	#7	26-#6	7	#4	#5	19-#4
Rib on C. L. cf/sf 0.611	150	3.43	24 *	0.263	SS	5	#7	#7	32-#6	7	#5	#5	23-#4

NOTES * Shear for the column size so designated is greater than $4\sqrt{f_c'}$, but less than $6\sqrt{f_c'}$. Shear reinforcement, steel shearheads, or shear caps must be provided with columns indicated.

(1) Average steel weight (psf) includes twice the flexural steel tabulated (tabulated steel placed in each direction). Temperature steel, integrity steel, and shearhead steel are not included.

(2) "S" indicates recommended stirrups for a length of one module in each joist rib beginning at the face of the solid head; "SS" indicates two modules.

Shear Reinforcement
(No Drop Panels)

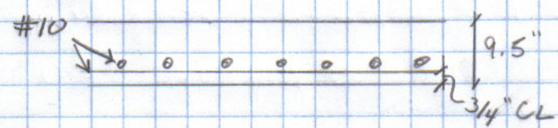
COLUMN SHEAR REINF.

$W_u = 1.2(15 + 9.5/12 \times 150) + 1.6(100) = 320.5\,psf$

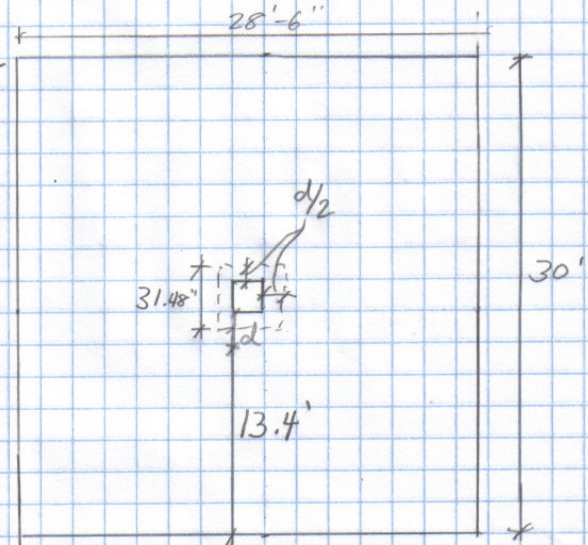
$f'_c = 5000\,psi$

$f_y = 60\,ksi$

$V_u =$



$d = 9.5 - 3/4 - 1.27 = 7.48"$



WIDE BEAM ACTION

$V_u = 13.4'(28.5')(320.5) = 122.4^k$

$V_c = 2\sqrt{f'_c}\, b_w d$

$= 2\sqrt{5000}(28.5 \times 12)(7.48) = 361.8^k$

$V_u = 122.4^k < 361.8^k \therefore OK$

TWO-WAY ACTION

$b_o = 2(31.48) + 2(31.48) = 125.9"$

$\frac{b_o}{d} = \frac{125.9}{7.48} = 16.8 < 20$

$\beta_c = \frac{24}{24} = 1.0 < 2$

$2 + \frac{4}{1} = 6 > 4$

$\frac{\alpha_s \cdot d}{b_o} + 2 = \frac{40 \times 7.48}{125.9} + 2 = 2.38 < 4$

$\therefore V_c = 2.38\sqrt{f'_c}\, b_o d = \sqrt{5000}(125.9)(7.48)(2.38) = 158.5^k$

$.75(158.5) = 118.9^k < 122.4^k \therefore$ NEED SHEAR REINF

LIMIT FOR BAR/WIRE REINF.

$6\sqrt{f'_c}\, b_o d = 6\sqrt{5000}(125.9)(7.48) = 399.5^k$

$\phi 6\sqrt{f'_c}\, b_o d = .75(399.5) = 299.6^k$

$V_u = 122.4^k < 299.6^k \therefore$ USE BAR/WIRE REINF

COLUMN SHEAR REINF. (CONT.)

DESIGN OF REINF.

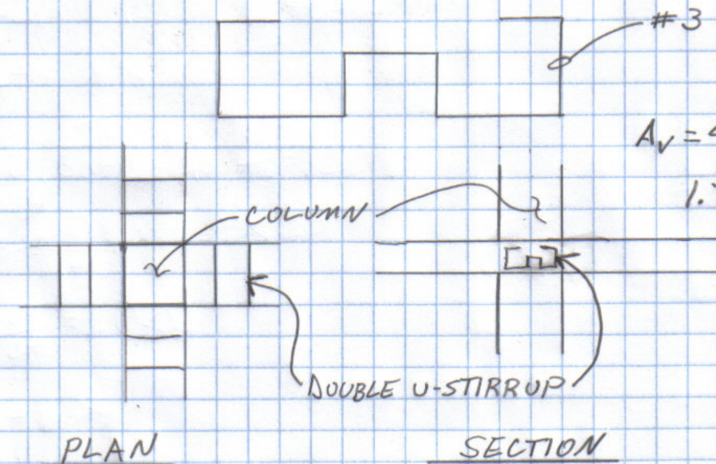
$V_n = \frac{V_u}{\phi} = \frac{122.4}{.75} = 163.2^K$

$V_c = 2\sqrt{f'_c}\, b_o d = 2\sqrt{5000}(125.9)(7.48) = 133.2^K$

$V_n = V_c + V_s \Rightarrow V_s = 163.2^K - 133.2^K = 30^K$

ASSUME $S = d/2 = \frac{7.48}{2} = 3.74$

$A_{v_{req}} = \frac{V_s \cdot S}{f_y d} = \frac{30(3.74)}{60(7.48)} = 0.25\,in^2$



$A_v = 4 \times 4 \times 0.11 = 1.76\,in^2$

$1.76\,in^2 > 0.25\,in^2 \therefore OK$