The Harry and Jeanette Weinberg Center

Mercy Hospital Medical Office Building
Baltimore, MD

Technical Assignment 2

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Executive Summary

This report analyzes alternate floor systems and compares them with the existing floor system. The intent of this report is to determine whether an alternate floor system is a viable alternative to what is currently installed.

Current Floor System

The floor system that is already installed in The Weinberg Center is a traditional composite action concrete slab on metal deck. This system is composed of 3.25" of concrete on 2" metal deck resting on various sized beams. Steel framing works very well in providing an open floor plan for tenants to arrange as they see fit. It also allows for the façade corner (the black glazing shown in the photo above) to be hung relatively easily off of cantilevered beams.

Alternate Floor Systems

Alternate floor systems that have been chosen to be analyzed were chosen for unique characteristics that they bring to the project. Some of these systems are easier to be constructed, some minimize the overall floor depth required, and others utilize their materials more efficiently than others. The alternate floor systems that I have chosen to analyze are as follows:

- Two-way Solid Flat Slab with Drop Panels
- Waffle Slab
- One-way Concrete Joist
- Hollow Core Plank on Steel Beams

Conclusion

Each of the alternative systems has its advantages over the existing composite slab. Most notably is the difference in floor system depths compared to the composite slab. All of the concrete systems reduced the floor depths compared to the composite slab, some more so than others. However concrete floor systems have an increased weight over the existing composite floor. This increased weight, while good at reducing vibrations, leads to larger foundation and seismic loads which increase the overall cost of materials and design. A flat slab is very easy to form and reduces the floor depth considerably compared to the composite slab. Waffle and one-way joist slabs utilize the concrete and steel material a better than the flat slab by increasing the depth of the slab in differing ways to effectively transfer load to the columns. These three concrete systems all have the same drawback of not being able to frame out the façade corner pictured above. This problem leads me to looking at a hollow core plank system using steel framing. Using a precast hollow core plank system leads to a very easy form of construction. This in turn lessens the construction time and reduces the construction costs. In the end a hollow core plank system is the best alternative to the current system because it allows for the strengths inherit in concrete construction, while enabling the corner façade to be framed out relatively easily.

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Existing Conditions

The Weinberg Center is a 6 story medical office building located in downtown Baltimore, B=MD. This building was constructed in 2002 using the 1997 Baltimore City Building Code and 1996 BOCA. This code assigns a 100psf live load to the floors. The design engineer used a 10 psf superimposed dead load for mechanical, electrical, plumbing, and finishes loads. Concrete is designed using The American Concrete Institute (ACI 318). Steel is designed using the "Load and Resistance Factor Design Specification for Structural Steel Buildings, Third Edition"

Existing Floor System – Composite Action Slab on Steel Deck

See Appendix 1 for details of the existing floor system

The existing floor system is a composite action floor slab with simply supported beams and girders. This system is very good for this buildings construction for several reasons; however it does have its flaws.

The obvious advantage to this system is that it is relatively light and does not add much to foundation and seismic loading compared to a concrete system. This type of framing allows for a very open floor plan that can be arranged to the tenants liking. A not so obvious advantage to this system is that it makes framing out the glass/aluminum corner relatively easy by cantilevering beams out to the façade without much trouble. Also framing the curving drive through façade is easier in the same way because infill beams can be added as needed to the main framing members.

Disadvantages to this system include something that was probably done to reduce the depth of the floor system. The beams and girders are relatively small and have very large cambers, as much as 1-7/8", in order to meet deflection criteria during construction and service. Aside from the deflection criteria the beams would have to be shored in order to be constructed as many of the members I checked in Technical Assignment 1 failed deflection criteria under construction loads. Fire proofing has to be added to the steel shapes and metal deck in order to meet fire code, while a concrete system has an inherit fire rating. While fireproofing is not of a huge concern, it is an advantage that some of my alternate systems have over this existing one

Overall this system is very good for this building. Framing of the cantilevered corner is done relatively easily with steel shapes. Also steel framing allows for large clear spans without columns and room for more windows in the façade, both of which is a huge plus when a tenant is looking for space to work in.

Alternate System 1 - Two-Way Solid Flat Slab with Drop Panels

See Appendix 2 for details of Alternate System 1

Using the 2002 CRSI Design Handbook I obtained the following for the design of a flat plate with drop panel floor system:

Total Slab Depth (h)	10"
Drop Panel Depth	8"
Drop Panel Width	10'-0"x10'-0"
Column Size	18"x18"
Floor Weight	125 psf

The flat slab with drop panel design is a decent alternate to the composite beam-slab system already installed. By slightly altering the column grid I was able to design a typical bay that is excellent for two-way slab design. While this layout is ideal for a two-way slab and has other advantages, I believe it is not a good alternative for this building for several reasons illustrated below. I did not include a design for a two-way flat slab without drop panels because to do so with the column layout I have chosen would mean using an initial column size of 47"x47". To me this column size is too high to justify not using the same system with drop panels that will require 18"x18" columns.

Advantages to the existing system include higher initial fire protection which would not require additional fire retardant material. This system is much shallower than the composite floor system and would make Mechanical, Electrical, and Plumbing design and installation much easier. The dead weight of this system is enough to dampen any vibration effects from equipment and live loads as compared to the existing composite system which used lightweight concrete. Column sizes are not dramatically large as compared to the same system with no drop panels.

Disadvantages of a flat-plate system include several things that relate directly to the added weight of a concrete system. Higher dead loads would lead to both higher foundation and seismic loads on the building. A higher foundation load may require the installation of caissons where there are spread footings. Also the existing caissons may have to be increased in size to handle the added load, more caissons per column may have to be added, or the caissons may have to be drilled to a deeper depth to reach the required bearing capacities. An increase in the seismic loads would require a beefier lateral force resisting system. Switching to a concrete system would also require a change in the lateral force resisting system. Concrete systems use shear walls to resist lateral loads. While this is workable, it would probably not go over well because the existing system allows for very open floor plans that the tenants may arrange as the please. Installing shear walls would segment floors or erase windows from what exists. Another disadvantage is the added complexity of supporting the corner façade. To do this a couple columns would have to be added in order to properly frame out any support for the corner.

Alternate System 2 – Waffle Flat Slab

See Appendix 3 for details of Alternate System 2

Using the 2002 CRSI Design Handbook I obtained the following for the design of a waffle slab system:

Total Depth	13"
Rib Depth	10"
Slab Depth	3"
Column Size	18"x18" with additional shear requirements
Voids	30"x30"
Ribs	6" @ 36" on center
Solid Head	12'-6" square
Floor Weight	93.6 psf

The waffle slab design is a better alternate to the flat plate with drop panels. One reason is that you get the same load carrying capacity with a slightly thicker, but much lighter floor system. This particular waffle slab system is 31.4 lb/ft^2 lighter than the previous flat plate construction, not including drop panels. This system naturally lends itself to larger spans and thus works well with 30'x30' sized bays. The need for additional shear capacity is not a problem as this can be solved by using larger columns, using shearheads or another type of reinforcement that will not affect the total slab depth around the columns.

This system has several advantages over the traditional flat plate with drop panel construction. A waffle slab system is lighter than a flat plate system and there for would not impact the foundation or seismic design as much. A waffle slab is friendlier towards the installation of mechanical, electrical, and plumbing systems in the building. Similar to the flat plate, vibrations are mitigated by the self weight of the waffle slab and column sizes are not too large to cause problems with architectural plans.

Disadvantages of a waffle slab as compared to the existing composite floor include many of the same that weigh against the flat plate system, but generally to a lesser degree. The lighter weight of the waffle slab, 93.6 psf, is not nearly as dramatic as the 125 psf weight of the flat plate. It is however heaver than the composite floor slab and would require some sort of investigation into the added weight on the foundations and seismic loadings as previously discussed. Again looking at the existing cantilevered façade corner, a waffle slab would not be an ideal system to frame this architectural feature with. The sloping façade corner of the drive through also would cause more problems with a waffle slab system since it is set up on a very rectilinear pattern. Additional fireproofing would be needed under the 3" slab in order to meet the 1-1/2 hour rating required.

Alternate System 3 – One-Way Concrete Joist

See Appendix 4 for details of Alternate System 3

Using the 2002 CRSI Design Handbook I obtained the following for the design of a one-way concrete joist floor system:

Rib	6" Wide x 14" Deep
	36" Center to Center
Slab	4.5" Top Slab
Total Depth	18.5"
Floor Weight	91.5 psf

A one-way concrete joist floor system is not a traditional floor system used in my building type. However I wanted to try something different and decided that this system could have some potential. In order for this system to work I divided up the column grid into a more practical and uniform layout. I kept the North-South spacing at 30'-0" and changed the East-West spacing to 24'-0". Doing this allows me to run the joists 30' which are supported on beams that will run in the 24' dimension. Overall this has the potential to be a decent choice for a floor system. The joist slab weighs 91.5 lb/ft^2, basically the same as a waffle slab system. I did not look at using a precast double-T floor system because the double-T systems typically come in much larger rib depths than what would be practical for this building floor system.

Advantages of using a one-way concrete joist floor are similar to that of a waffle slab, except that you have a slightly lower self weight of the system. While this self weight is lighter than that of the waffle slab, it would still be sufficient to dampen any vibration concerns. This system is also good for installing mechanical, electrical, and plumbing systems as there is adequate room between the ribs and below. The 4.5 inches of concrete is more than adequate to reach the required fire rating of 1.5 hours. The rearrangement of the columns keeps the building relatively open for tenants to arrange floor plans as needed.

Mainly this system is very labor intensive, as with any system that requires setting up formwork to pour the concrete. The higher weight compared to the existing composite floor will require checking foundation and seismic designs. Shear walls would be the best way to handle lateral loads and doing so would divide up the floor areas along column lines or along the façade, taking space away from tenants. And again the issue of dealing with the existing cantilevered corner and sloping façade persists. This system would require lots of added detailing of these areas and probably the addition of several columns in order for the entrance to work correctly.

Alternate System 4 – Hollow Core Plank on Steel Beams

See Appendix 5 for details of Alternate System 4

Using design aids obtained from Nitterhouse Concrete Products, and the AISC LRFD Manual of Steel Construction, Third Ed., I obtained the following for a design of a hollow core plank sytem:

12"x4' Prestressed Concrete SpanDeck with 2" topping slab
(meets U.L. J952)
6 Prestressed Strand Pattern

Steel Support Beams	W24x55 spans 21'-0"
	W24x131 spans 35'-0"
	W27x161 spans 40'-0"
Floor Weight	109.5 psf

Using a prestressed concrete plank system in place of the original composite floor is a decent alternative. Some beams increase in size due to the added weight of the concrete plank, but overall it could save a lot of construction costs. A hollow core plank system allows for the existing column grids to remain. A main reason I looked at this system was to determine if it could be feasible to keep a steel frame construction because framing out the existing cantilevered corner will be easier if a steel system is kept.

Additional advantages of this system include things that border the advantages of the existing steel design and the concrete systems I have investigated so far. Using a concrete system increases the weight which reduces vibration of the floors. Since the hollow core planks rest on a steel frame the overall system still keeps its flexibility toward framing out the unusual features of this building, including the corner façade. A hollow core plank system is much quicker to construct over a cast-in-place concrete slab and could save costs on the project in this regard. Lateral loads can still be transferred to the ground in the same way that they are in the existing system because the steel frame is still present. Fire protection is adequate for the 1-1/2 hour rating required because the planks are built to specification.

Disadvantages to this system are similar to that of a flat plate system. The increase in weight leads to beefier foundation and seismic systems. The self weight of a hollow core plank system (102.5 psf) is not quite that of the flat plate, but it is more than a waffle slab or a one-way joist system.

Comparison between the systems

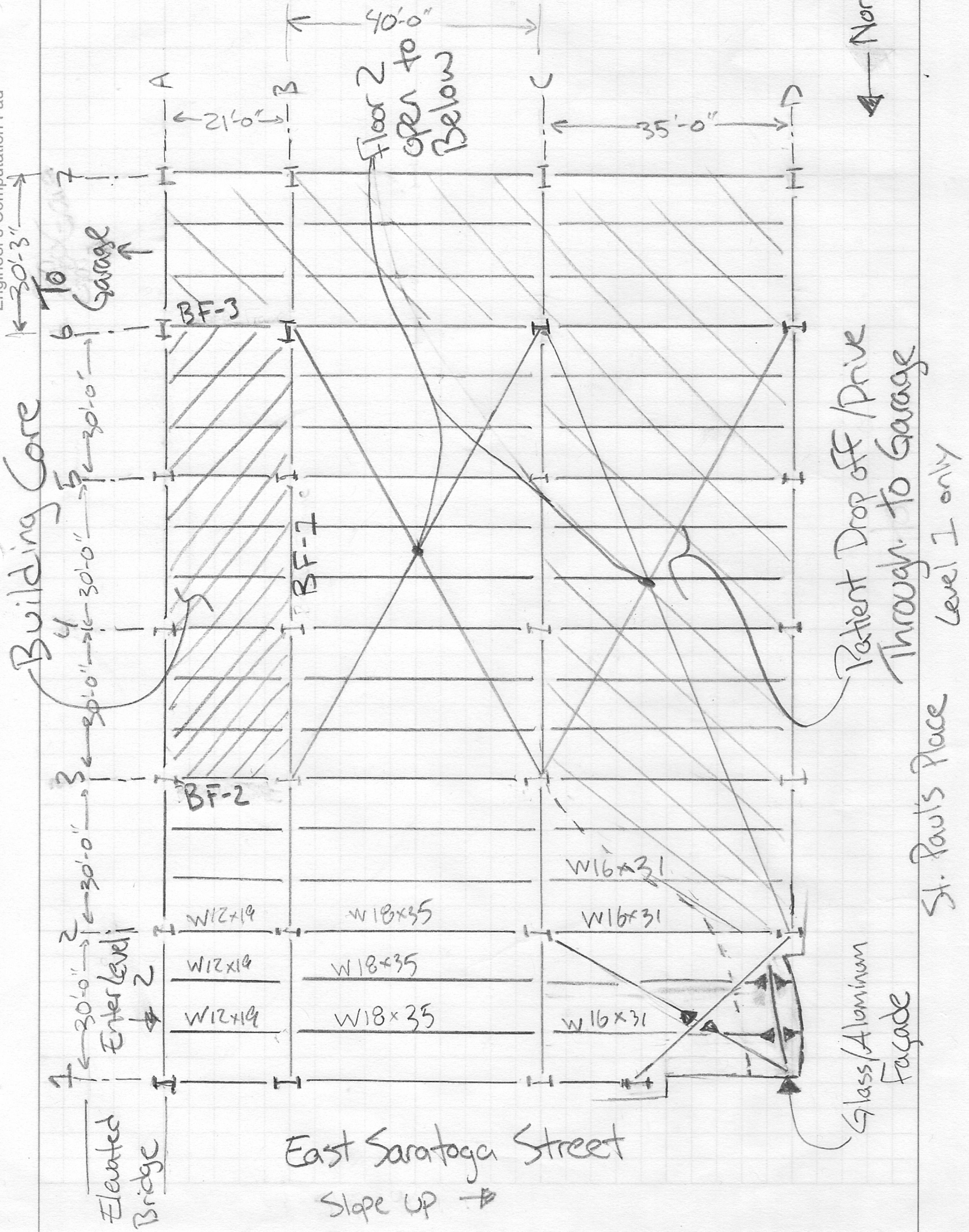
	Steel with Compost Slab	Flat Slab with Drop Panels	Waffle Slab	One-Way Joist	Hollow Core Plank on Steel
Weight (psf)	48	125	93.6	91.5	109.5
Depth (in)	5.25 + 18"	10 + 8.5 Drop	13	18.5	14
Vibration	Possible	No	No	No	No
Fire Rating (hr)	1 1/2	> 2	1 1/2*	> 2	> 2
Column Size	W14	18"x18"	18"x18" **	18"x18"	W14
Form Work Needed	No	Yes	Yes	Yes	No
Construction Difficulty	Medium	Medium/Hard	Hard	Hard	Easy
Acceptable Alternative	--	Yes	Maybe	No	Yes

* Additional Fireproofing will be required for 3" slab to meet displayed rating.
** Some form of additional punching shear reinforcement is required for this column size.

Conclusion

Each of the alternative systems has its advantages over the existing composite slab. Most notably is the difference in floor system depths compared to the composite slab. All of the concrete systems reduced the floor depths compared to the composite slab, some more so than others. The one-way joist system is a good system with a reduced weight compared to the flat slab. However a one-way joist system is a formwork intensive construction method and this leads to increased construction costs and could even lead to a longer construction period. Waffle slabs are very efficient systems because they provide the depth required for a concrete floor to effectively take advantage of steel reinforcing. However this waffle slab construction has some drawbacks because it would require additional fire protection and design for punching shear. There is enough capacity left in the waffle slab to add another layer of a topping concrete to provide the required fire rating and the design for punching shear would require more research. Hollow core plank is a viable alternative because it provides both a easy means of construction and the required fire protection. A flat plate floor with drop panels is also a good alternative because of the significant reduction is floor system depth. However a flat plat construction requires formwork and this could increase the construction period. With all of the concrete systems the problem of the corner façade that is cantilevered off of the existing framing persists. Framing this façade out with a concrete system would be more difficult that doing so with a steel system. In my opinion a hollow core plank system is the best alternative because it provides a shallower floor system, the required fire rating, and is a very easy construction method. But on top of these a hollow core plank system still utilizes a steel frame construction and would make framing out that corner façade relatively easy.

A-1 Floor Plan



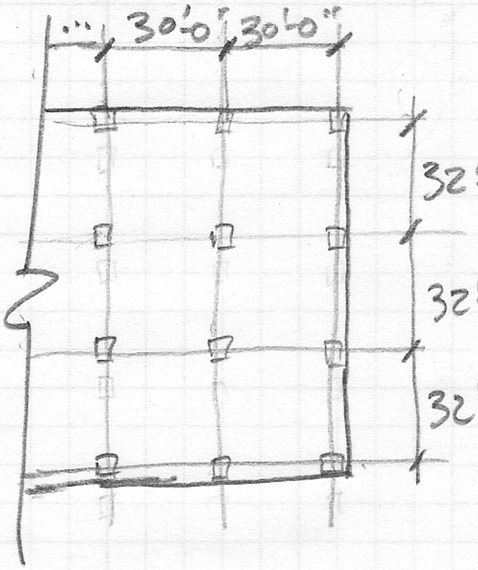
Concrete Systems Using the CRSI Design Handbook

Design Superimposed Loads: 100psf Live

10psf MEP, etc.

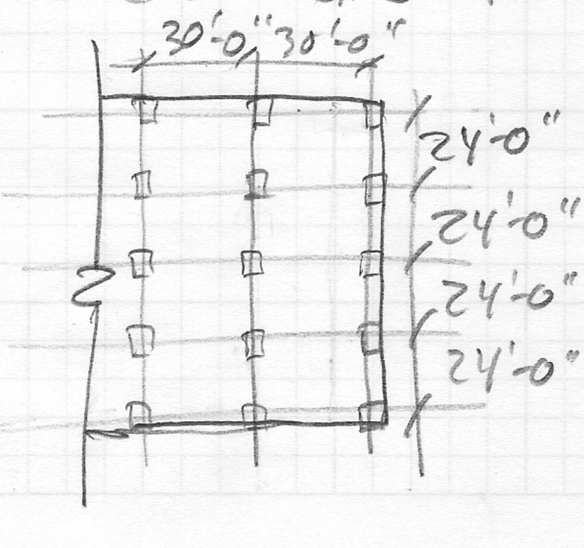
$$1.2(10) + 1.6(100) = \underline{\underline{172\ psf}}$$

Altered Column Grid: 2-way systems (Flat plate w/drops & waffle slab)

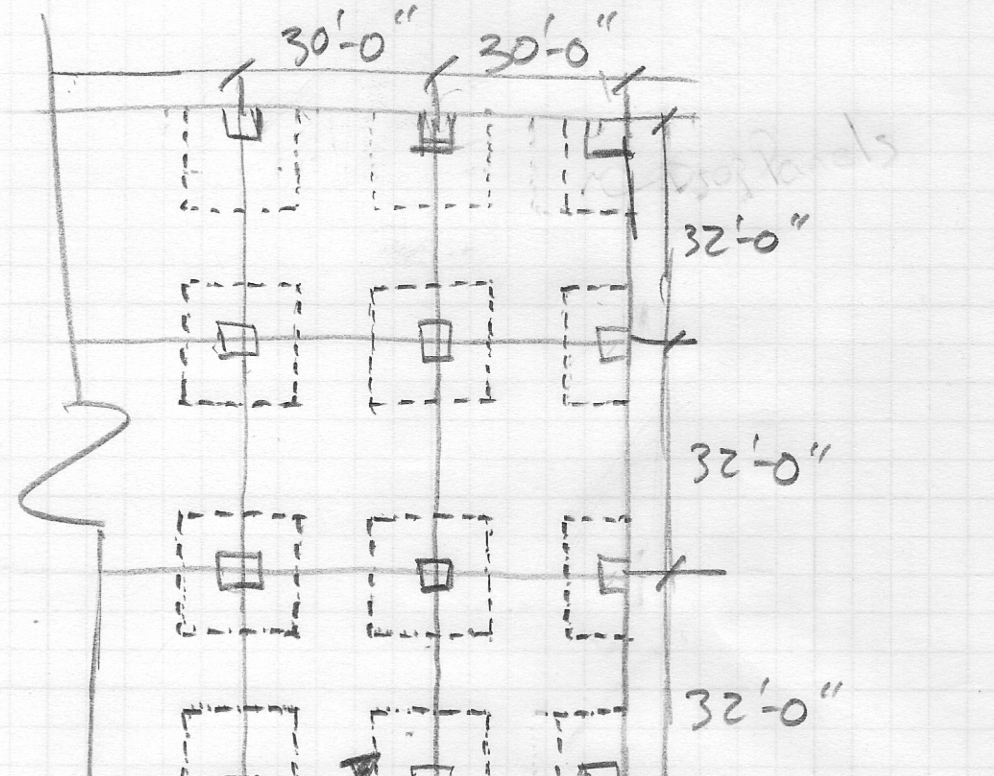


This system keeps the same overall building dimentions while providing a decent 2-way setup.

Altered Column Grid: 1-way Concrete Joist



Flat Plate w/ Drop Panels



A-2.2

f'_c = 4,000 psi Grade 60 Bars

FLAT SLAB SYSTEM

SQUARE EDGE PANEL — With Drop Panels — No Beams

h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	Square Column[3] Size (in.)	Square Column[3] γ_f	Column Strip[1] Top Ext.	Column Strip[1] +	Column Strip[1] Bottom	Column Strip[1] Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (−) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (−) (ft-k)
25	100	5.50	8.33	12	0.776	12-#5	2	10-#6	14-#5	9-#5	9-#5	2.39	130.1	260.2	350.3
25	200	5.50	8.33	15	0.809	12-#5	4	13-#6	13-#6	12-#5	10-#5	2.95	171.3	342.6	461.2
25	300	7.00	8.33	18	0.664	12-#5	1	17-#6	15-#6	15-#5	9-#6	3.59	212.4	424.7	571.8
25	400	8.50	8.33	19	0.632	12-#5	1	15-#7	12-#7	10-#7	15-#5	4.25	254.3	508.6	684.6
25	500	8.50	10.00	21	0.744	13-#5	3	11-#9	26-#5	15-#6	10-#7	4.97	295.4	590.8	795.3
26	100	5.50	8.67	12	0.810	12-#5	3	11-#6	16-#5	11-#5	10-#5	2.60	146.8	293.7	395.3
26	200	7.00	8.67	15	0.704	12-#5	1	11-#7	14-#6	10-#6	12-#5	3.17	194.0	388.0	522.3
26	300	8.50	8.67	18	0.633	12-#5	1	11-#8	15-#6	9-#7	15-#5	3.88	240.6	481.1	647.6
26	400	8.50	8.67	19	0.745	13-#5	3	13-#8	18-#6	11-#7	9-#7	4.73	287.7	575.5	774.7
26	500	8.50	10.40	24	0.745	15-#5	4	13-#9	12-#8	10-#8	14-#6	5.49	330.9	661.8	890.9
27	100	7.00	9.00	12	0.746	12-#5	2	18-#5	16-#5	12-#5	10-#5	2.63	165.4	330.8	445.4
27	200	7.00	9.00	15	0.804	12-#5	5	17-#6	15-#6	11-#6	13-#5	3.37	218.2	436.3	587.4
27	300	8.50	9.00	18	0.674	12-#5	2	16-#7	13-#7	19-#5	16-#5	4.12	270.7	541.5	728.9
27	400	8.50	10.80	22	0.756	14-#5	5	12-#9	12-#8	10-#8	19-#5	5.09	321.6	643.2	865.8
27	500	8.50	10.80	27	0.682	16-#5	3	17-#8	13-#8	9-#9	9-#8	5.78	366.6	733.3	987.1
28	100	7.00	9.33	12	0.784	13-#5	2	14-#6	18-#5	13-#5	11-#5	2.76	185.0	370.0	498.1
28	200	8.50	9.33	16	0.714	13-#5	3	11-#8	15-#6	17-#5	15-#5	3.56	243.2	486.4	654.8
28	300	8.50	9.33	19	0.757	13-#5	5	11-#9	14-#7	12-#7	10-#7	4.56	302.4	604.8	814.1
28	400	8.50	11.20	25	0.692	16-#5	3	17-#8	13-#8	11-#8	12-#7	5.47	357.1	714.3	961.5
29	100	8.50	9.67	12	0.737	13-#5	2	22-#5	18-#5	15-#5	12-#5	2.91	206.7	413.4	556.5
29	200	8.50	9.67	16	0.758	13-#5	4	12-#8	13-#7	19-#5	16-#5	3.81	271.2	542.5	730.3
29	300	8.50	9.67	22	0.718	15-#5	4	20-#7	16-#7	10-#8	20-#5	4.92	334.3	668.6	900.1
29	400	8.50	11.60	28	0.639	17-#5	2	15-#9	14-#8	12-#8	10-#8	5.83	392.7	785.4	1057.3
30	100	8.50	10.00	12	0.774	14-#5	2	10-#8	20-#5	16-#5	10-#6	3.16	229.4	458.8	617.6
30	200	8.50	10.00	18	0.744	14-#5	4	11-#9	14-#7	21-#5	10-#7	4.16	299.6	599.1	806.5
30	300	8.50	10.00	24	0.675	16-#5	3	17-#8	14-#8	11-#8	12-#7	5.24	369.5	739.1	994.9

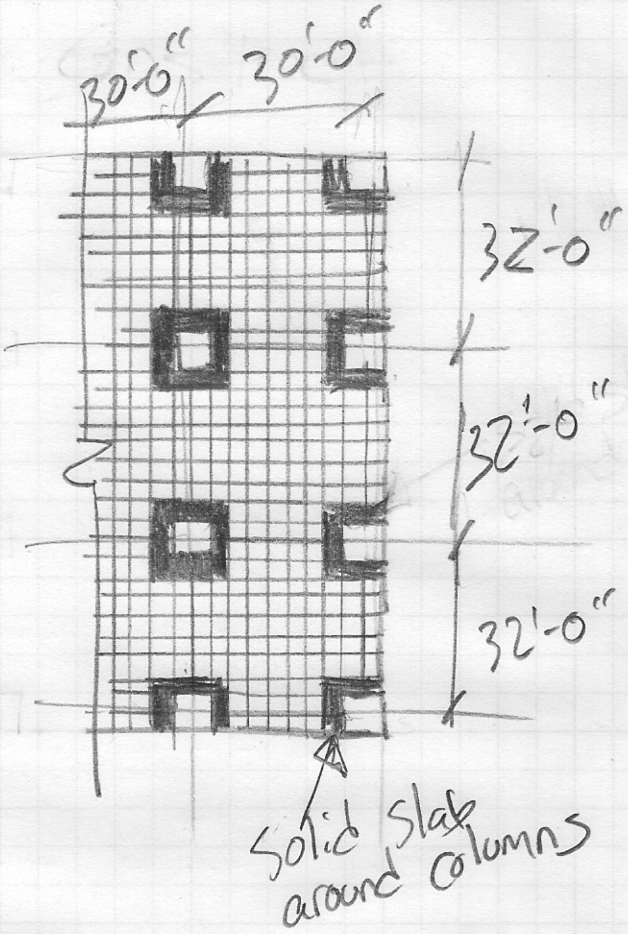
SQUARE INTERIOR PANEL — With Drop Panels[2] — No Beams

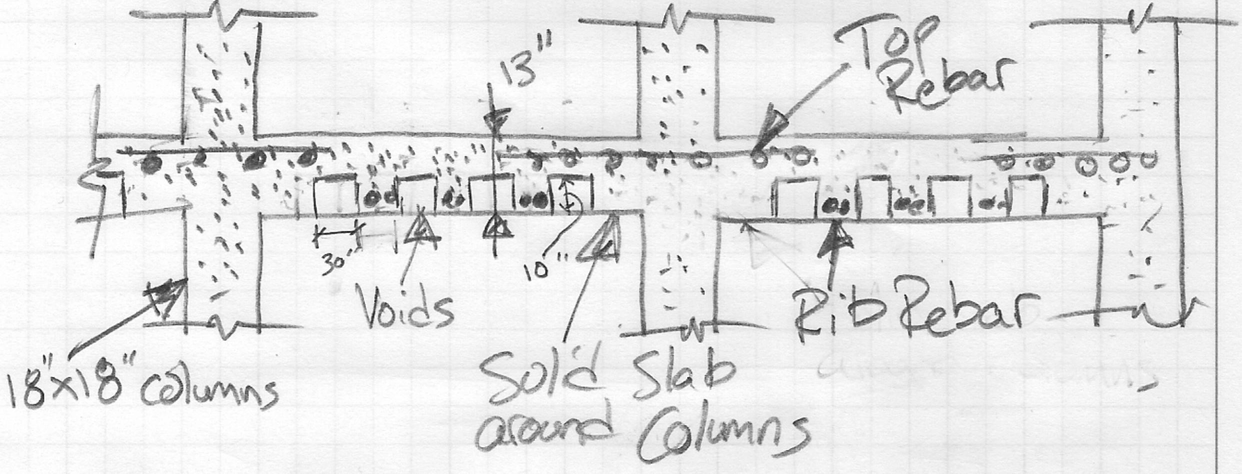
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Column[3] Size (in.)	Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
25	100	12	13-#5	9-#5	9-#5	9-#5	2.19	0.884
25	200	18	12-#6	12-#5	10-#5	9-#5	2.63	0.884
25	300	21	14-#6	15-#5	12-#5	10-#5	3.10	0.898
25	400	23	15-#6	18-#5	10-#6	12-#5	3.63	0.912
25	500	25	13-#7	15-#6	16-#5	10-#6	4.26	0.947
26	100	12	15-#5	11-#5	10-#5	10-#5	2.40	0.884
26	200	18	17-#5	14-#5	11-#5	10-#5	2.73	0.898
26	300	21	14-#6	9-#7	13-#5	11-#5	3.31	0.912
26	400	23	13-#7	11-#7	16-#5	10-#6	4.17	0.912
26	500	25	27-#5	10-#8	10-#7	16-#5	4.65	0.947
27	100	12	15-#5	12-#5	10-#5	10-#5	2.37	0.898
27	200	18	14-#6	11-#6	12-#5	10-#5	2.92	0.898
27	300	21	12-#7	19-#5	15-#5	9-#6	3.56	0.912
27	400	24	26-#5	10-#8	10-#7	15-#5	4.35	0.947
27	500	27	16-#7	11-#8	11-#7	18-#5	5.02	0.947
28	100	12	17-#5	13-#5	10-#5	10-#5	2.42	0.898
28	200	19	14-#6	17-#5	13-#5	12-#5	3.02	0.912
28	300	21	13-#7	22-#5	12-#6	10-#6	3.85	0.912
28	400	24	16-#7	11-#8	20-#5	12-#6	4.71	0.947
29	100	12	17-#5	15-#5	12-#5	11-#5	2.58	0.912
29	200	19	16-#6	19-#5	15-#5	13-#5	3.27	0.912
29	300	21	15-#7	10-#8	10-#7	16-#5	4.34	0.912
29	400	26	13-#8	12-#8	12-#7	10-#7	5.06	0.947
30	100	12	14-#6	12-#6	13-#5	11-#5	2.77	0.912
30	200	19	18-#6	22-#5	12-#6	10-#6	3.57	0.912
30	300	21	16-#7	11-#8	11-#7	18-#5	4.56	0.912

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.

Waffle Slab System





A-B.2

WAFFLE FLAT SLAB SYSTEM 30″ X 30″ Voids: 6″ Ribs @ 36″

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANELS

Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Edge Column: $c_1 = c_2$ (in.)	Square Edge Column: γ_f	Square Edge Column: (2) Stirrups	Reinforcing Bars—Each Direction, Column Strip: Top Edge No.-size +	Column Strip: Bottom No. Ribs	Column Strip: Bottom Bars per Rib	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" D= 6.500 RIB ON COLUMN LINE 0.597 CF/SF	50	1.86	12	0.664		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	28	56	76
	100	1.86	12	0.687		13-#5+ 0	3	2-#4	13-#5	3	#4	#4	5-#5	37	77	98
	150	1.92	12	0.711		13-#5+ 0	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5	45	106	121
	200	1.98	12	0.735		13-#5+ 0	3	2-#5	13-#5	3	#4	#4	5-#5	53	132	144
	300	2.16	12	0.782		13-#5+ 0	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5	70	185	189
	400	2.61	12	0.830		13-#5+ 0	3	1-#6 and 1-#7	13-#5	3	#5	#6	5-#5	87	236	235
21'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.637 CF/SF	50	1.84	12	0.735		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	47	93	125
	100	1.84	12	0.763		15-#5+ 0	4	2-#4	15-#5	3	#4	#4	6-#5	60	128	162
	150	2.01	12	0.791		15-#5+ 0	4	2-#5	15-#5	3	#4	#5	6-#5	74	174	199
	200	2.19	12	0.818		15-#5+ 0	4	1-#5 and 1-#6	15-#5	3	#5	#5	6-#5	87	218	236
	300	2.63	12 *	0.874		15-#5+ 0	4	1-#6 and 1-#7	15-#5	3	#6	#6	6-#5	115	302	309
	400	3.13	12 *	0.634		15-#5+ 0	4	2-#7	18-#5	3	#7	#7	6-#5	142	385	383
24'- 0" D= 9.500 RIB NOT ON COLUMN LINE 0.613 CF/SF	50	1.92	12	0.801		18-#5+ 0	4	1-#4 and 1-#5	18-#5	4	#4	#4	7-#5	69	138	186
	100	1.98	12	0.831		18-#5+ 0	4	2-#5	18-#5	4	#4	#4	7-#5	90	190	242
	150	2.32	12	0.862		18-#5+ 0	4	2-#6	18-#5	4	#5	#5	7-#5	111	260	298
	200	2.57	12 *	0.892		18-#5+ 0	4	1-#6 and 1-#7	18-#5	4	#5	#6	7-#5	131	325	353
	300	3.32	12 *	0.634		18-#5+ 0	4	2-#8	16-#6	4	#6	#7	7-#5	172	453	464
	400	4.07	16 *	0.628		18-#5+ 1	4	2-#9	20-#6	4	#7	#7	9-#5	210	564	565
27'- 0" D= 9.500 RIB NOT RIB NOT ON COLUMN LINE 0.597 CF/SF	50	1.96	13	0.824		20-#5+ 0	4	2-#5	20-#5	5	#4	#4	8-#5	98	196	264
	100	2.22	13	0.860		20-#5+ 0	4	2-#6	20-#5	5	#4	#5	8-#5	127	263	343
	150	2.49	13 *	0.896		20-#5+ 0	4	2-#7	20-#5	5	#5	#5	8-#5	157	363	422
	200	3.03	13 *	0.632		20-#5+ 0	4	2-#8	25-#5	5	#5	#6	8-#5	186	455	502
	300	3.99	18 *	0.626		20-#5+ 1	4	2-#9	23-#6	5	#6	#7	8-#6	240	617	647
30'- 0" D=12.500 RIB ON COLUMN LINE 0.624 CF/SF	50	2.12	15	0.829		22-#5+ 0	5	1-#5 and 1-#6	22-#5	5	#4	#5	9-#5	136	272	367
	100	2.31	15 *	0.887		22-#5+ 3	5	2-#6	23-#5	5	#5	#5	9-#5	177	353	475
	150	2.95	15 *	0.933		22-#5+ 6	5	2-#7	28-#5	5	#6	#6	9-#5	217	483	584
	200	3.53	18 *	0.626		22-#5+ 3	5	2-#8	24-#6	5	#6	#7	11-#5	255	604	686
33'- 0" D=12.500 RIB ON COLUMN LINE 0.609 CF/SF	50	2.25	16 *	0.867		25-#5+ 2	5	2-#6	25-#5	6	#4	#5	10-#5	180	360	485
	100	2.71	16 *	0.922		25-#5+ 6	5	2-#7	31-#5	6	#5	#5	10-#5	234	467	629
	150	3.42	19 *	0.931		25-#5+10	5	2-#8	27-#6	6	#6	#6	13-#5	285	614	768
	200	4.05	25 *	0.621	3 S 4 1	25-#5+ 3	5	2-#9	32-#6	6	#6	#7	15-#5	331	711	892
36'- 0" D=12.500 RIB ON COLUMN LINE 0.597 CF/SF	50	2.52	18 *	0.875		27-#5+ 3	5	2-#7	31-#5	7	#5	#5	11-#5	231	463	623
	100	3.15	19 *	0.926		27-#5+ 9	5	2-#8	40-#5	7	#5	#6	13-#5	300	601	808
	150	3.93	27 *	0.620	3 S 4 1	27-#5+ 5	5	2-#9	35-#6	7	#6	#6	11-#6	362	725	976

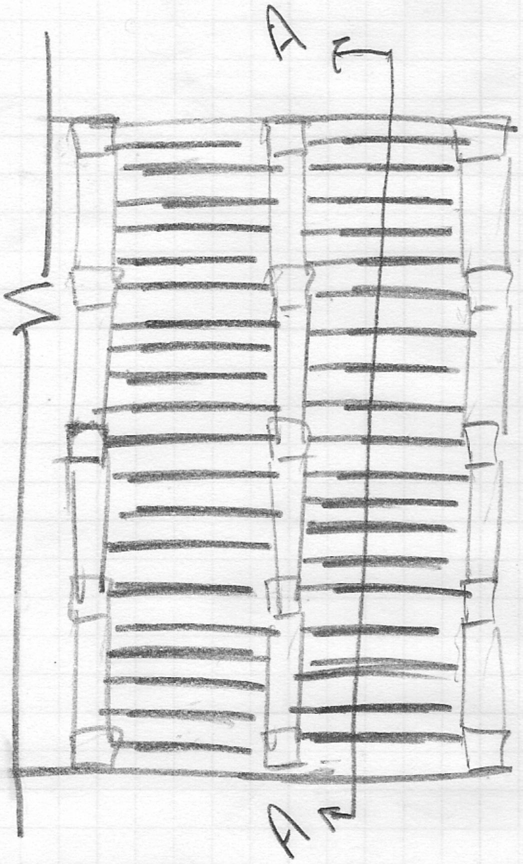
SQUARE INTERIOR PANELS

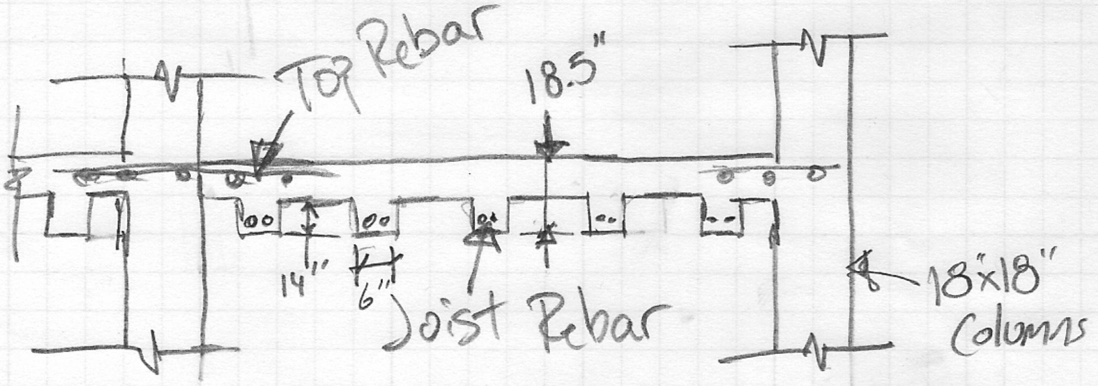
Total Depth = 13 in. Rib Depth = 10 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Interior Column: $c_1 = c_2$ (in.)	Square Interior Column: (2) Stirrups	Reinforcing Bars—Each Direction, Column Strip: Bottom No. Ribs	Column Strip: Bottom Bars per Rib	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	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	100	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	150	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	200	1.84	12		3	2-#4	13-#5	3	#4	#4	5-#5
	300	1.95	12	3 S 4 1	3	1-#4 and 1-#5	13-#5	3	#4	#4	5-#5
	400	2.28	12	3 S 4 1	3	1-#5 and 1-#6	13-#5	3	#4	#5	5-#5
21'- 0"	50	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	100	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	150	1.82	12		4	2-#4	15-#5	3	#4	#4	6-#5
	200	1.93	12		4	1-#4 and 1-#5	15-#5	3	#4	#4	6-#5
	300	2.22	12	3 S 4 1	4	2-#5	15-#5	3	#5	#5	6-#5
	400	2.69	12 *	3 S 4 1	4	2-#6	17-#5	3	#5	#6	6-#5
24'- 0"	50	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	100	1.86	12		4	2-#4	18-#5	4	#4	#4	7-#5
	150	2.05	12		4	2-#5	18-#5	4	#4	#4	7-#5
	200	2.25	12	3 S 4 1	4	1-#5 and 1-#6	18-#5	4	#4	#5	7-#5
	300	2.78	12 *	3 S 4 1	4	1-#6 and 1-#7	21-#5	4	#5	#5	7-#5
	400	3.49	13 *	3 S 4 1	4	2-#7	18-#6	4	#6	#6	9-#5
27'- 0"	50	1.86	13		4	2-#4	20-#5	5	#4	#4	8-#5
	100	2.02	13		4	2-#5	20-#5	5	#4	#4	8-#5
	150	2.13	13	3 S 4 1	4	1-#5 and 1-#6	20-#5	5	#4	#4	8-#5
	200	2.61	13 *	3 S 4 1	4	1-#6 and 1-#7	23-#5	5	#4	#5	8-#5
	300	3.55	14 *	3 S 4 2	4	1-#7 and 1-#8	21-#6	5	#5	#6	10-#5
30'- 0"	50	1.94	15		5	1-#4 and 1-#5	22-#5	5	#4	#4	9-#5
	100	2.03	15		5	2-#5	22-#5	5	#4	#4	9-#5
	150	2.63	15 *	3 S 4 1	5	2-#6	26-#5	5	#5	#5	9-#5
	200	3.12	15 *	3 S 4 1	5	1-#6 and 1-#7	22-#6	5	#5	#6	10-#5
33'- 0"	50	2.06	16		5	2-#5	25-#5	6	#4	#4	10-#5
	100	2.46	16 *	3 S 4 1	5	2-#6	28-#5	6	#4	#5	10-#5
	150	3.07	16 *	3 S 4 1	5	1-#6 and 1-#7	25-#6	6	#5	#5	12-#5
	200	3.63	18 *	3 S 4 2	5	2-#7	30-#6	6	#5	#6	10-#6
36'- 0"	50	2.16	18	3 S 4 1	5	1-#5 and 1-#6	28-#5	7	#4	#4	11-#5
	100	2.79	18 *	3 S 4 1	5	1-#6 and 1-#7	37-#5	7	#4	#5	12-#5
	150	3.62	18 *	3 S 4 2	5	1-#7 and 1-#8	33-#6	7	#5	#5	15-#5

See the notes on page 11-19.

One-Way Concret Joist System





Section A-A

A-4.2

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	
		14" Deep Rib + 4.5" Top Slab = 18.5" Total Depth										
TOP BARS	Size @	# 4 11	# 4 9.5	# 5 11	# 5 10	# 5 8	End Span Defl. Coeff. (3)	# 4 11	# 4 8.5	# 4 7	# 5 8.5	# 6 10	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7		# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	
Steel (psf)		.72	.88	1.09	1.28	1.52		.77	.97	1.20	1.48	1.80	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		114 0	164 0	223 0	252* 282	264* 348*	4.572	149 0	221 0	293* 295	299* 379	305* 398*	2.814
26'-0"		96 0	143 0	197 0	234* 251	244* 316	5.349	128 0	195 0	263 0	279* 342	284* 367*	3.292
27'-0"		80 0	124 0	173 0	217* 224	226* 284	6.221	110 0	172 0	235 0	261* 308	266* 340*	3.828
28'-0"		66 0	106 0	153 0	200 0	210* 256	7.195	94 0	152 0	210 0	245* 278	249* 315*	4.428
29'-0"		53 0	91 0	134 0	178 0	196* 230	8.279	79 0	133 0	188 0	229* 251	233* 293*	5.095
30'-0"		42 0	77 0	117 0	158 0	182* 207	9.481	66 0	116 0	167 0	215* 226	219* 274*	5.835
31'-0"			64 0	102 0	140 0	170* 186	10.810	54 0	101 0	149 0	203* 204	206* 256*	6.652
32'-0"			53 0	88 0	124 0	159* 167	12.274	43 0	87 0	132 0	184 0	193* 236	7.553
33'-0"			42 0	76 0	110 0	149* 150	13.882		75 0	117 0	166 0	182* 215	8.543
34'-0"				64 0	96 0	134 0	15.642		63 0	103 0	149 0	171* 195	9.626
35'-0"				54 0	84 0	120 0	17.565		53 0	90 0	134 0	162* 177	10.809
36'-0"				44 0	72 0	107 0	19.661		43 0	79 0	120 0	152* 161	12.099
37'-0"					62 0	94 0	21.938			68 0	107 0	144* 146	13.500
38'-0"					53 0	83 0	24.407			58 0	95 0	132 0	15.020

(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 = $\ell_n/360$.

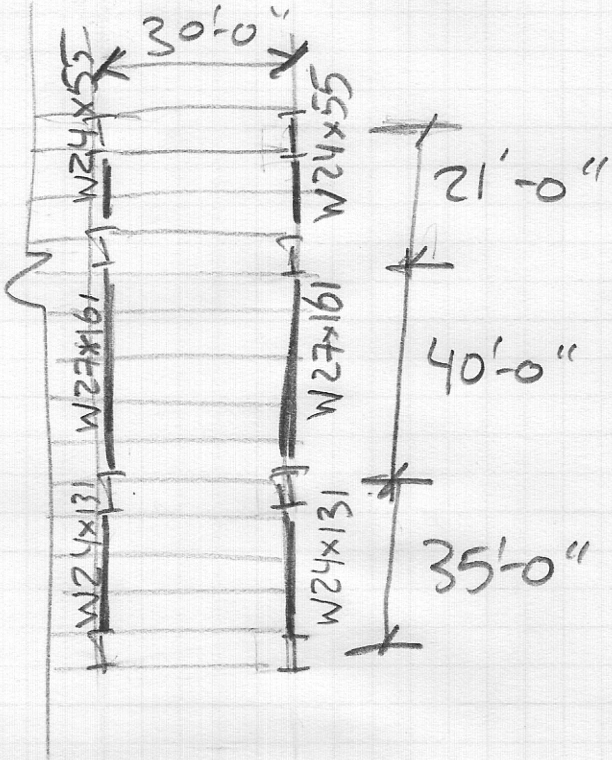
PROPERTIES FOR DESIGN (CONCRETE .58 CF/SF) (4)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.64	.74	.99	1.09	1.36		.64	.82	1.00	1.28	1.54	
STEEL % (UNIFORM)	.57	.66	.89	.98	1.23		.57	.74	.90	1.15	1.40	
(TAPERED)	.32	.37	.50	.55	.69		.32	.42	.51	.65	.79	
EFF. DEPTH, IN.	17.3	17.3	17.2	17.2	17.2		17.3	17.3	17.3	17.2	17.1	
– ICR/IGR	.163	.183	.227	.244	.287		.163	.200	.231	.274	.311	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.51	.62	.75	.88	1.04		.40	.51	.62	.75	.88	
STEEL %	.08	.10	.13	.15	.17		.07	.08	.10	.13	.15	
EFF. DEPTH, IN.	17.2	17.2	17.1	17.1	17.1		17.3	17.2	17.2	17.1	17.1	
+ICR/IGR	.172	.206	.243	.281	.323		.138	.172	.206	.243	.281	

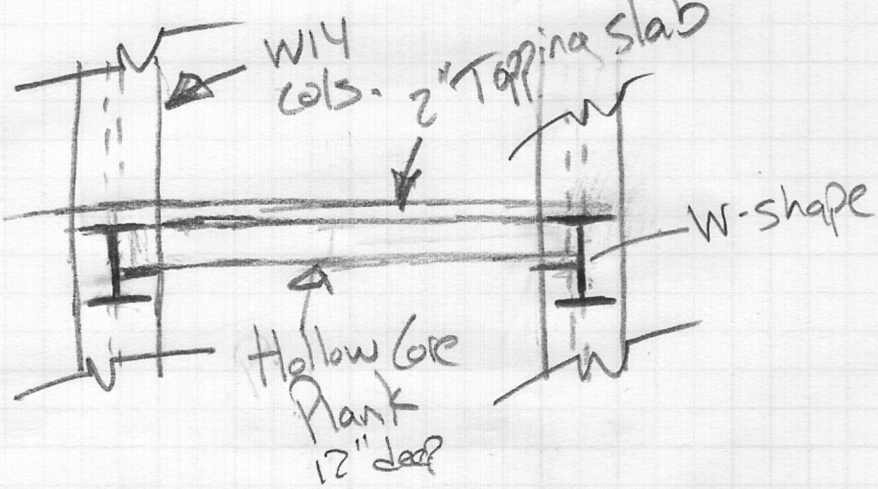
STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS		30" Forms + 6" Rib @ 36" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)									f'_c = 4,000 psi f_y = 60,000 psi	
		14" Deep Rib + 4.5" Top Slab = 18.5" Total Depth										
TOP BARS	Size @	# 4 12	# 4 10	# 4 8	# 5 10.5	# 5 8.5	End Span Defl. Coeff. (3)	# 4 11	# 4 9	# 5 11	# 5 9	# 6 11	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7		# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	
Steel (psf)		.69	.85	1.04	1.24	1.46		.75	.93	1.18	1.42	1.70	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		102 0	151 0	208 0	265 0	295* 334	4.703	136 0	206 0	278 0	339* 360	342* 441*	2.894
26'-0"		84 0	130 0	182 0	235 0	274* 299	5.502	116 0	181 0	247 0	317* 323	320* 400	3.386
27'-0"		69 0	111 0	160 0	209 0	255* 268	6.398	98 0	159 0	220 0	290 0	300* 362	3.938
28'-0"		55 0	94 0	140 0	185 0	238* 240	7.400	82 0	138 0	195 0	261 0	282* 328	4.554
29'-0"		43 0	79 0	121 0	164 0	215 0	8.516	68 0	120 0	174 0	235 0	265* 297	5.240
30'-0"			66 0	105 0	145 0	193 0	9.752	55 0	104 0	154 0	211 0	249* 269	6.001
31'-0"			53 0	90 0	128 0	172 0	11.119	43 0	89 0	136 0	189 0	235* 244	6.842
32'-0"			42 0	77 0	112 0	154 0	12.625		76 0	120 0	170 0	221 0	7.769
33'-0"				65 0	98 0	137 0	14.278		64 0	105 0	152 0	200 0	8.787
34'-0"				53 0	84 0	122 0	16.089		53 0	91 0	136 0	181 0	9.901
35'-0"				43 0	73 0	108 0	18.067		43 0	79 0	121 0	164 0	11.118
36'-0"					62 0	95 0	20.222			68 0	107 0	148 0	12.445
37'-0"					51 0	83 0	22.565			57 0	95 0	133 0	13.886
38'-0"					42 0	72 0	25.105			48 0	83 0	119 0	15.449

(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 .61 CF/SF) (4)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.60	.72	.90	1.06	1.31		.65	.80	1.01	1.24	1.44	
STEEL % (UNIFORM)	.47	.56	.70	.83	1.03		.51	.62	.79	.97	1.13	
(TAPERED)	.28	.34	.42	.50	.61		.31	.37	.47	.58	.68	
EFF. DEPTH, IN.	17.3	17.3	17.3	17.2	17.2		17.3	17.3	17.2	17.2	17.1	
– ICR/IGR	.142	.165	.197	.222	.259		.153	.179	.214	.249	.275	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.51	.62	.75	.88	1.04		.40	.51	.62	.75	.88	
STEEL %	.08	.10	.12	.14	.17		.06	.08	.10	.12	.14	
EFF. DEPTH, IN.	17.2	17.2	17.1	17.1	17.1		17.3	17.2	17.2	17.1	17.1	
+ICR/IGR	.152	.182	.215	.249	.287		.122	.152	.182	.215	.249	

Precast Plank on Steel Beams System





A-5.2

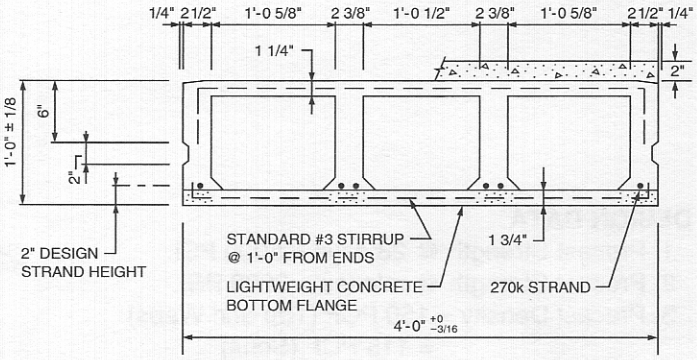
Prestressed Concrete
12" x 4' SpanDeck – U.L. – J952

(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES

Composite

A' = 312 in.^{2}	S'$_b$ = 826 in.^{3}
I' = 6542 in.^{4}	S'$_t$ = 1602 in.^{3} (At Top of SpanDeck)
$Y_{b'}$ = 7.92 in.	S'$_{tt}$ = 1076 in.^{3} (At Top of Topping)
Y_t' = 4.08 in. (To Top of SpanDeck)	Wt.' = 410 PLF
Y_{tt}' = 6.08 in. (To Top of Topping)	Wt.' = 102.5 PSF



12" SPANDECK CROSS SECTION

UL FIRE RATED J952

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PCF.
7. Strand Height = 2.00 in.
8. Ultimate moment capacities (when fully developed) . . .
   4 – 1/2"ø, 270K = 139.7'K
   6 – 1/2"ø, 270K = 198.7'K
9. Maximum bottom tensile stress is $6\sqrt{f'c}$ = 424 PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
11. Flexural strength capacity is based on stress/strain strand relationships.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determining allowable loads in this table.
14. All values in this table are based on ultimate strength and are not governed by service stress.
15. All loads shown refer to allowable loads applied after topping has hardened.

12" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
Flexure 4 – 1/2"ø	422	370	326	288	255	226	200	179	159	140	125	111	98	86	76	66	60						
Shear 4 – 1/2"ø	409	381	357	335	315	294	266	242	221	201	184	171	162	152	139	127	115						
Flexure 6 – 1/2"ø	636	562	499	445	398	357	321	289	261	236	213	193	175	158	144	130	117	106	95	86	77	69	61
Shear 6 – 1/2"ø	423	395	370	348	327	308	292	276	261	248	236	221	202	186	172	158	146	134	124	115	110	105	98

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

A-5.3

Precast Plank System Design

-Can Keep Same Column Layout - run Steel Girders only

HollowCore Plank spans 30' direction

Nitterhouse Concrete Products, Inc.

Prestressed Concrete 12"x4' Spandeck - U.L. - J952 w/ 2" CIP Topping

Live Load: 100psf (1.6) = 160psf $\qquad w_{LL} = .1(30) = 3$ klf

12"x4' spandeck w/ 6 strand pattern

-spanning 30'-0" can support 175psf

weight of SpanDeck: 102.5psf.

Steel Beam Design: Live Load: 100psf

Dead Load: 102.5psf ← spandeck
8. psf ← Misc (MEP, Finishes, etc..)

$W_u = 1.6(100) + 1.2(110.5) = 293$ psf.

$w_u = 292.6(30) = 8778$ plf $= 8.78$ klf

3 beams: 35'-0", 40'-0", 21'-0"

21'-0" $\quad L_b$ = Continuously braced -

$M_u = \dfrac{8.78(21)^2}{8} = 484'k$ plank & Topping slab can brace beam

W24x55 ⇒ 506'k T5.5 LRFD

$\Delta_{max} = \dfrac{5wl^4}{384EI} = \dfrac{5(3\,k/ft)(21\,ft)^4(1728)}{384(29000\,ksi)(1360)} = .33"$

$\dfrac{L}{\Delta} = \dfrac{21(12)}{.33"} = 763$

763 > 360 - ok

use W24x55

A-5.4

8.78 klf

35'-0" $M_u = \dfrac{8.78^{k/f}(35)^2}{8} = 1344.44'^k$

W30x116

W24x131 $\Rightarrow$ 1388'k -ok

$\hookrightarrow$ save depth, sim wt.

$$\Delta_{max} = \frac{5(838)(35)^4(1728)}{384(29000)(4020)} = 0.87''$$

$$\frac{35(12)}{0.87} = 482 > 360 \text{ -ok}$$

use W24x131

40'-0" $M_u = \dfrac{8.78\ (40)^2}{8} = 1756'^k$

W27x161 $\Rightarrow$ 1928'k -ok

$$\Delta_{max} = \frac{5(830)(40)^4(1728)}{384(29000)(6310)} = .94''$$

$$\frac{40 \times 12}{.94} = 510 > 360$$

-ok

use:

W24x55 for 21' span

W24x131 for 35' span

W27x161 for 40' span $\leftarrow$ very heavy & deep beam

HollowCore planks cannot develope strength if try to span planks in long direction.

8" Plank does not have adequate strength to span 30'-0"