Technical Report #2

8^{th} Street Office Building | Richmond, VA

Carol Gaertner | Structural Option

AE Consultant: Dr. Andres Lepage

November 4, 2009

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Acknowledgements

I am very grateful to the firm of Rathgeber/Goss Associates for their sponsorship of the 8th Street Office Building for AE Senior Thesis. Also, this thesis would not be possible without the assistance of Commonwealth Architects, which provided the renderings.

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Executive Summary

In the second technical report regarding the 8th Street Office Building, the existing floor system is analyzed in depth and three alternative floor systems are investigated. The comparison of floor systems was based on a variety of criteria including self weight, depth, constructability, cost, and fire ratings. A typical exterior bay of 20’-0” by 40’-6” was utilized since it represents the worst case scenario for the building. The existing floor system consists of composite metal decking and slab on composite steel beams. It is an ideal system due to its light self weight and efficient constructability, but it is very costly. The alternative floor systems that are studied are the following:

- One-Way Solid Slab
- One-Way Joist Slab
- Pre-Cast Hollow Core Planks on Steel Beams

The design of the one-way solid slab system resulted in a 6” thick slab with 27” deep girders. This system has a self weight of 75 pounds per square foot, which is the largest self weight of the four floor systems. It also has a relatively high cost of $24.35 per square foot. Additionally, the costs of other components, such as the columns and foundation, may increase due to the large self weight. Therefore, it was determined that the one-way solid slab floor system should not be considered any further.

The design of the one-way joist slab system produced more promising results than the one-way solid slab system. The slab was designed to be 3 $^1/_2$" thick with 6” wide by 13” deep joists spaced 36” on center spanning the 20’-0” direction. The self weight of this system is lighter at approximately 64 pounds per square foot, and the cost is lower at $19.95 per square foot compared to the one-way solid slab. Furthermore, it may be beneficial to design the joists in the 40’-6” direction. Therefore, the one-way joist slab floor system should not be eliminated as an alternative in the future.

Nitterhouse Concrete Products were used for the pre-cast hollow core planks on steel beams floor system. It was determined that 6” thick by 4’-0” wide planks with a 2” topping is sufficient to carry the required service loads in the 20’-0” direction. Unfortunately, none of the planks had a large enough capacity to span the 40’-6” direction. Therefore, large W27x161 girders are required to span between columns in the long direction. The main advantages of this system are its extremely low cost of $13.09 per square foot and its efficient constructability; however, the main disadvantage is the system’s large self weight. It was determined that the advantages may potentially outweigh the disadvantages, so the pre-cast hollow core plank system may require additional study as an alternative.

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Introduction

The new 8th Street Office Building will be located in the bustling Richmond, VA commercial district near the Virginia State Capitol Building. It is intended to be a legacy building that will serve both the needs of the state government and the general public. Initially, the Virginia General Assembly will occupy the 8th Street Office Building for approximately five years while renovations to the Capitol Building are being completed. After that time, it is expected that various Virginia government agencies will move into the new office building.

The 8th Street Office Building will be comprised of 3 $^1/_2$ underground parking garage levels with spaces for 201 cars, ten floors above and a mechanical penthouse. The completed building will stand 176'-5" tall and will enclose approximately 307,000 square feet. Rooftop terraces with planters will be an integral part of the construction on the 3rd, 7th and 10th floors.

A secure main lobby on the first floor will efficiently handle high volume traffic to the large assembly areas. Ground level retail will be located on the corner of East Broad Street and 9th Street. The remainder of the floors will be open office spaces with meeting areas that can be flexibly rearranged to meet the needs of the various tenants. Finally, a six story atrium will connect the building along its southern edge to the existing 9th Street Office Building. The 9th Street Office Building is another Virginia government office building, and the atrium is expected to provide seamless passage between the two buildings. See Figure 1 on the next page for a general site plan.

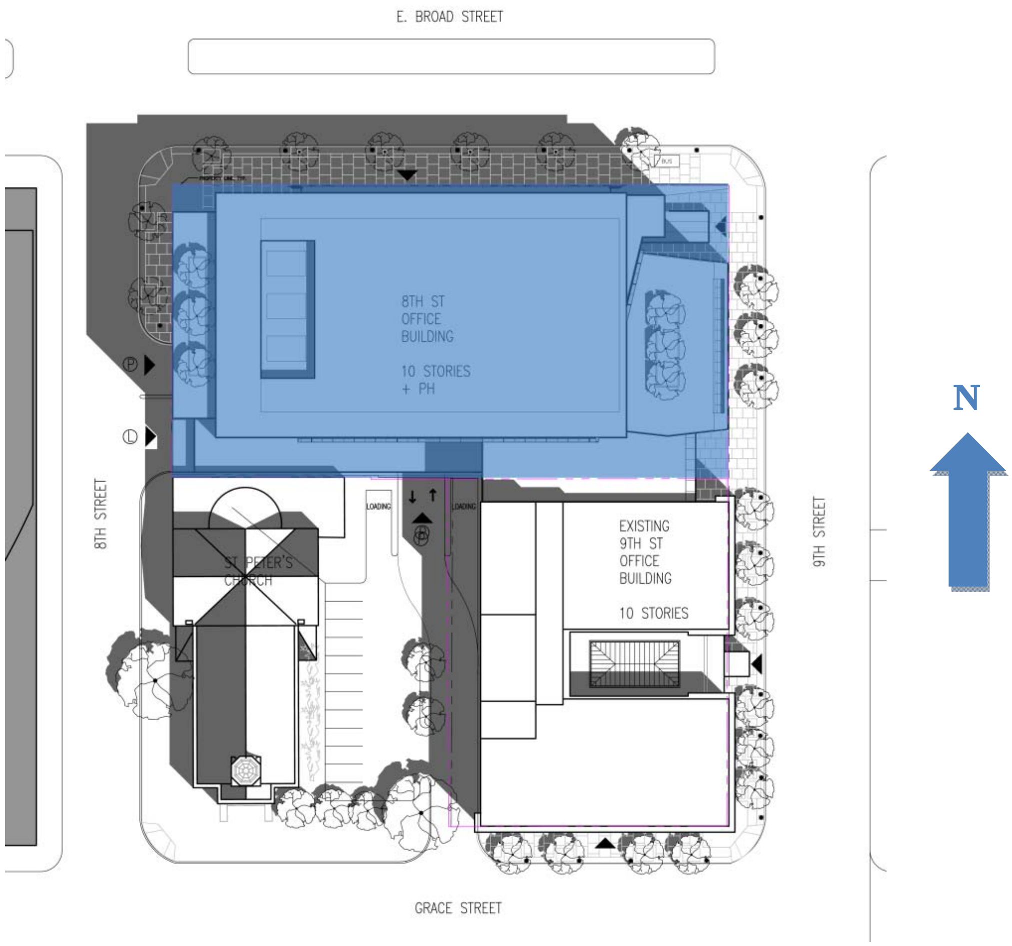

Figure 1 – Site plan

The 8th Street Office Building is designed as a primarily steel structure. However, concrete will play a major role in the construction of the underground parking garage and the shear walls around cores within the building. The façade will consist of several different glass curtain walls and precast concrete panels. Aluminum will be used to frame individual windows and doorways. Finally, a standing seam stainless steel roof will cantilever dramatically over 30'-0" off of the mechanical penthouse. See Figures 2 and 3 for elevations that display façade materials and the cantilevered roof. For a more detailed discussion of the 8th Street Office Building's structural system, please continue to the next section.

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Figure 2 – Broad Street Elevation

Figure 3 – 9th Street Elevation

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Structural System

Foundation

The geotechnical engineering study was conducted by Froehling & Robertson, Inc. of Richmond, VA. A total of nine test borings ranging from 50 to 100 feet were performed in September, 2006 and June-July, 2007. Based on the data from the borings and experience with other buildings located in Richmond, it was recommended in the geotechnical report that the 8th Street Office Building be supported on a mat foundation system. The mat foundation is located at elevations of 130'-0" and 140'-0" since the fourth level of the underground parking garage is only located on the western half of the site. Based on the elevations, it was recommended that the 4000 pounds per square inch concrete mat foundation be designed for a maximum allowable bearing pressure of 3,500 pounds per square foot. Ultimately, the mat foundation was designed to be 48" thick reinforced with #10 at 12" each way on the top and the bottom throughout the entire foundation.

According to the geotechnical report, the mat foundation system at the proposed elevations will be above the permanent groundwater table. However, the permanent perched water system may cause a substantial flow of water. Therefore, it was recommended that the 12" thick foundation walls be constructed with a minimum of 6" of free-draining granular filter material. Furthermore, the 48" thick mat should be placed on a 12" layer of free-draining aggregate for drainage and to provide uniform bearing pressure.

Parking Garage

The 8th Street Office Building's underground parking garage is comprised of 3 ½ levels and can accommodate 201 vehicles. The concrete columns are sized to be 30"x30" and tend to be reinforced with 16 #10 bars. Typical bay sizes are either 20'-0" by 40'-6" or 20'-0" by 30'-0". The concrete beams are typically sized to be 30"x30" although there are several exceptions. Primary reinforcement for the beams ranges anywhere from #7 to #11 bars. The one way concrete slabs span in the 20'-0" direction, and the majority of the slabs are 8" thick and reinforced with #5 bars spaced at 12".

Superstructure

The most typical bay sizes for the 8th Street Office Building are either 20'-0" by 40'-6" around the perimeter or 20'0" by 30'-0" through the middle portion of the building. However, there are several variations due to the shape of the building from floor to floor. The composite floor system consists of 3 ¼" of lightweight concrete and 2" deep, 18 gage metal deck for a total depth of 5 ¼". The deck spans W-shape infill beams spaced at 10'-0" on center. The beams tend to be W16x31, W18x35, or W18x40 depending on the length of their span. Composite action is achieved between the floor system and the beams through ¾" diameter, 4" long headed shear studs. The beams then transfer their loads to W-shape girders whose sizes vary greatly. The girders are connected to W14 columns that range in size from W14x43 to W14x283. The columns are typically spliced every three floors. See Appendix A for typical floor framing plans.

Lateral System

The primary lateral load resisting system for the 8th Street Office Building consists of reinforced concrete shear walls surrounding four cores within the building. The cores are the locations of the main elevators and stairwells for the building. Therefore, openings are provided in the walls for doorways. See Figure 4 for the exact locations of the shear walls. The shear walls are 12" thick and reinforced horizontally with #6 bars spaced at 12" on each face and vertically with #8 bars spaced at 12" on each face. There are a total of 16 shear walls. All of the shear walls are located on the 3rd level of the parking garage through the 10th floor. However, only 8 shear walls extend downwards to the 4th level of the parking garage, only 12 shear walls extend upwards to the Penthouse level, and only 4 shear walls extend upwards to the Penthouse Mezzanine level. It is assumed that the floor system of the 8th Street Office Building acts as a rigid diaphragm and transfers the lateral loads due to wind and seismic activity completely to the shear walls. The shear walls then carry those loads down to the mat foundation.

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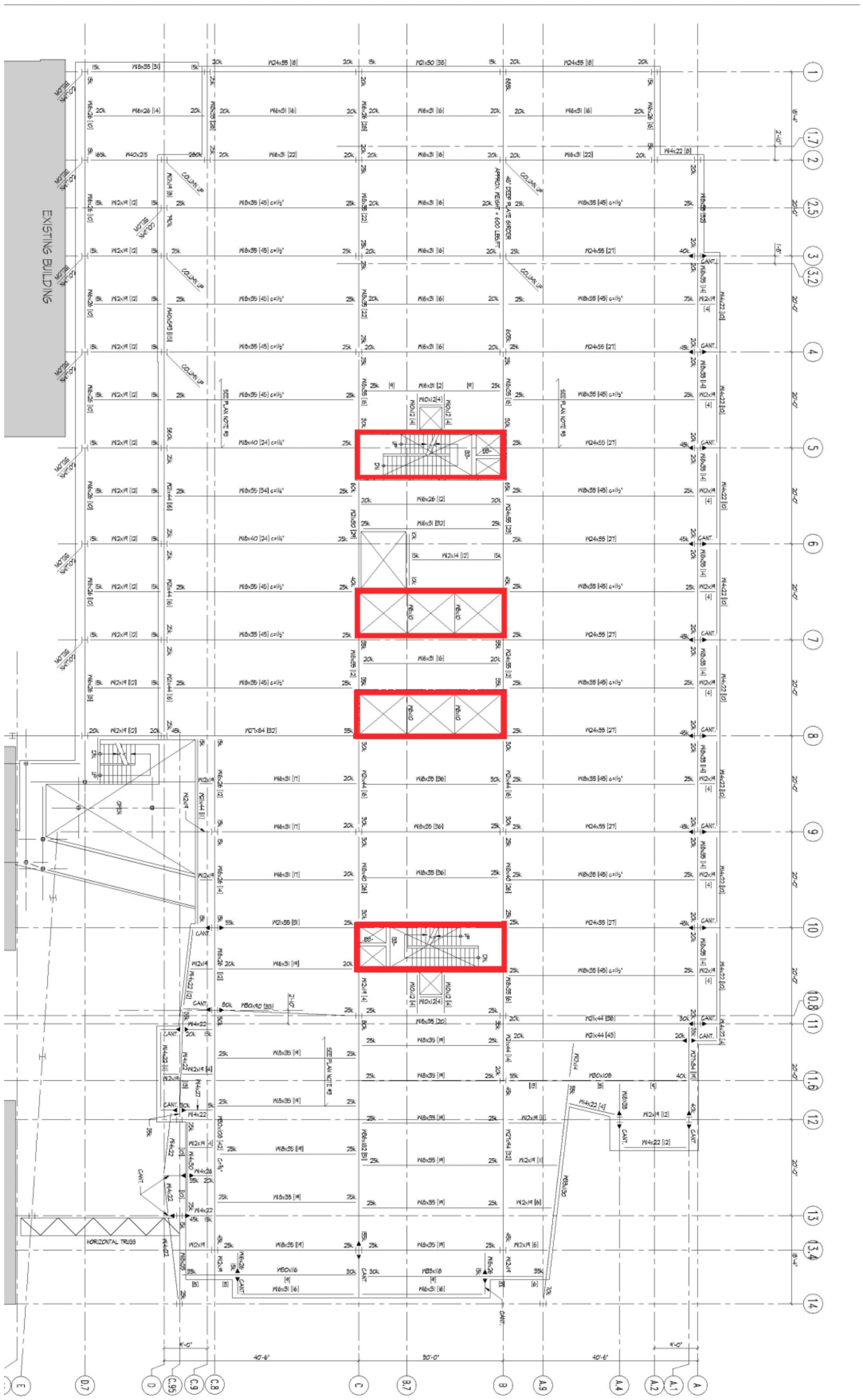

Figure 4 – Locations of Reinforced Concrete Shear Walls

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Materials

Structural Steel:

Rolled Shapes……ASTM A992, Grade 5
Channels, Angles and Plates……ASTM A36
Pipes……ASTM A53, Grade B, F_y=35 ksi
Tubes (Square and Rectangular HSS)……ASTM A500, Grade B, F_y=46 ksi

Metal Decking:

3 1/4" Lightweight Concrete over 2" Composite Deck (5 1/4" total depth)……ASTM A653, 18 Gage
1 1/2" Roof Deck……ASTM A653, 20 Gage

Headed Shear Studs:

3/4" diameter……ASTM A108

High Strength Bolts:

3/4" Bolts……ASTM A-325N

Welding Electrodes:

E70XX……Tensile Strength = 70 ksi

Cast-in-Place Concrete:

Slabs on Grade (Interior)……f'_c=3000 psi
Slabs on Grade (Exterior)……f'_c=3500 psi
Reinforced Slabs……f'_c=5000 psi
Reinforced Beams……f'_c=5000 psi
Fill on Metal Deck……f'_c=3500 psi
Columns……f'_c=5000/7000 psi
Walls……f'_c=4000 psi
Mat Foundation……f'_c=4000 psi

Reinforcement:

Deformed Reinforcing Bars……………………………………………………………………………ASTM A615, Grade 60
Welded Wire Fabric………………………………………………………………………………………………ASTM A185

Codes

Applicable Design Codes:

Model Codes:

Virginia Uniform Statewide Building Code 2003

International Building Code 2003

Structural Standards:

ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Design Codes:

ACI 318-02, Building Code Requirements for Structural Concrete

AISC Manual of Steel Construction – Allowable Stress Design, 9th Edition

AISC Manual of Steel Construction – Volume II, Connections – ASD, 9th Edition/LRFD, 3rd Edition

Applicable Thesis Codes:

Model Codes:

International Building Code 2006

Structural Standards:

ASCE 7-05, Building Code Requirements for Structural Concrete

Design Codes:

ACI 318-05, Building Code Requirements for Structural Concrete

AISC Steel Construction Manual, 13th Edition

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Gravity Loads

Gravity loads were determined using ASCE 7-05.

Dead Loads:

Typical Floor:

2" Composite Metal Deck, 18 Gage	2 psf
3 $^{1}/_{4}$" Lightweight Concrete Slab (115 pcf)	41 psf
Approximated Self Weight of Steel Framing	7 psf
Curtain Walls and Precast Concrete Panels	25 psf
Total for Floor System Design	(2 + 41 + 25) → 68 psf
Total	(2 + 41 + 7 + 25) → 75 psf

Superimposed Dead Loads:

Typical Floor:

Fireproofing	2 psf
Finishes	10 psf
Partitions	20 psf
Ceiling	5 psf
MEP	5 psf
Total SDL	42 psf

Atrium:
To account for finishes and catwalks, 20 psf is assumed for each level that the atrium extends upwards. Structural slabs, partitions and ceiling loads are not included.

Penthouse and Penthouse Mezzanine:
Due to large mechanical spaces, a dead load of 100 psf is assumed to account for concrete pads, sloped floors and other miscellaneous loads. This load replaces the superimposed MEP load. Furthermore, partitions are not included.

Terraces/Roofs: A load of 125 psf is assumed to account for self weights of system components and planters and finishes.

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Live Loads:

Typical Spaces:

	ASCE 7-05	Design Loads
Lobbies & First Floor Corridors	100 psf	100 psf
Corridors above First Floor	80 psf	100 psf
Stairs	100 psf	100 psf
Walkways & Elevated Platforms	60 psf	not available
Retail – First Floor	100 psf	not available
Assembly Areas with Movable Seats	100 psf	not available
Offices	50 psf	50 psf + 20 psf for partitions
Ordinary Roof	20 psf	30 psf minimum
Roofs used for Roof Gardens or Assembly Purposes	100 psf	not available

A comparison between the live loads from Table 4-1 in ASCE 7-05 and the live loads from Table 4-1 in ASCE 7-02 shows no differences. Thus, only the loads from ASCE 7-05 are tabulated above. The design loads that have been provided by the engineers of record are slightly more conservative than the minimum loads from ASCE 7-05. In addition, the engineers classified the partitions as a live load as opposed to a superimposed dead load, which is not unusual. Finally, a design load of 150 psf was specified for mechanical rooms. Since ASCE 7-05 does not provide a live load value for mechanical rooms, a live load of 150 psf will be used in future analyses.

Analysis of Floor Systems

A typical 20'-0" by 40'-6" exterior bay was utilized in the analysis of the existing floor system and three proposed alternative systems. An exterior bay was chosen, as opposed to an interior bay, since it represents the worst case scenario due to its long span. See Figure 5 for a plan view of the typical bay that was utilized.

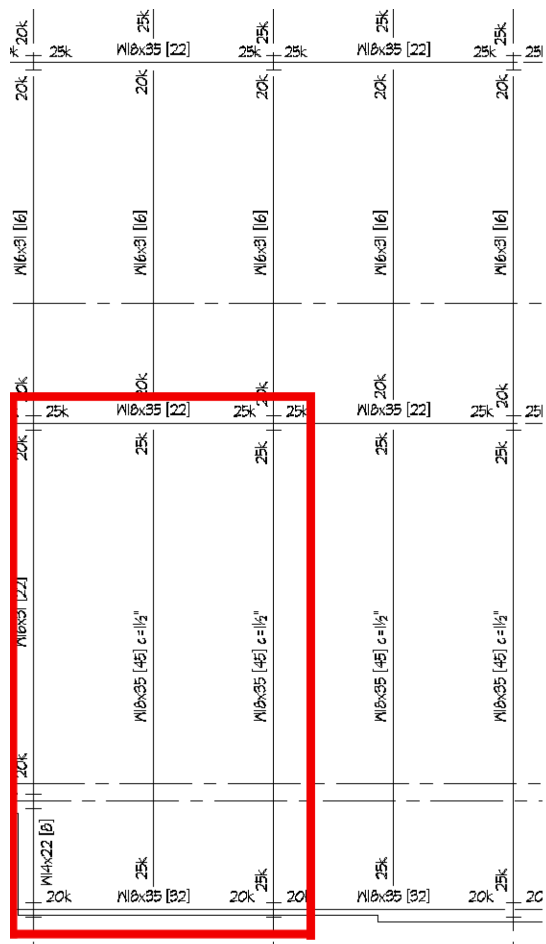

Figure 5 – Typical Exterior Bay

In order to provide flexibility for future tenants who may want to move partitions to rearrange office spaces and corridors, a live load of 80 psf has been used in the floor system analyses and reduced whenever possible. As indicated earlier, the total superimposed dead load is 42 psf. Finally, see the appropriate appendices for the self-weight of each system.

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Existing Floor System: Composite Metal Deck with Composite Steel Beams

The existing floor system for the 8th Street Office Building is a composite steel system. It was determined from the structural general notes and the framing plan notes that the composite metal deck is 2" deep with a minimum thickness of 18 gage. The slab is of lightweight concrete and has a total depth of 5 ¼". Furthermore, it was stipulated that the deck be provided by United Steel Deck with the following properties:

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.43
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.52
19	0.0418	2.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.69
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 1 – United Steel Deck 2" Lok-Floor Properties

The maximum unshored span of 10.97 feet was obtained from Table 2 below. In the 8th Street Office Building, beams are typically spaced 10 feet on center, so the clear span must be less than 10.97 feet. Therefore, the decking is adequate to span the beams.

	COMPOSITE PROPERTIES											
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050

Table 2 – United Steel Deck 2" Lok-Floor Composite Properties

Finally, the maximum uniform live service load was obtained from Table 3 below. The metal deck and slab can support 235 pounds per square foot for an 11'-0" span and a total depth of 5 ¼". This is greater than the total service load of 190 pounds per square foot, so the metal deck and slab are sufficient. In fact, the load provided by United Steel Deck already takes into account the self weight of the deck and slab, so it was conservative to use 190 pounds per square foot.

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	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
			L, Uniform Live Service Loads, psf *												
18 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	6.00	91.95	400	400	400	400	400	400	400	385	345	310	280	250	230
	6.25	96.93	400	400	400	400	400	400	400	400	365	325	295	265	240
	6.50	101.91	400	400	400	400	400	400	400	400	385	345	310	280	255
	7.00	111.87	400	400	400	400	400	400	400	400	400	380	340	310	280

Table 3 – United Steel Deck Uniform Live Service Loads for 2” Lok-Floor

The composite beam that was checked in this report was designed by the engineers of record to be a W18x35 [45] with a camber of 1 ½”. Likewise, the composite girder that was checked in this report was designed by the engineers to be a W18x35 [22]. See Figure 6.

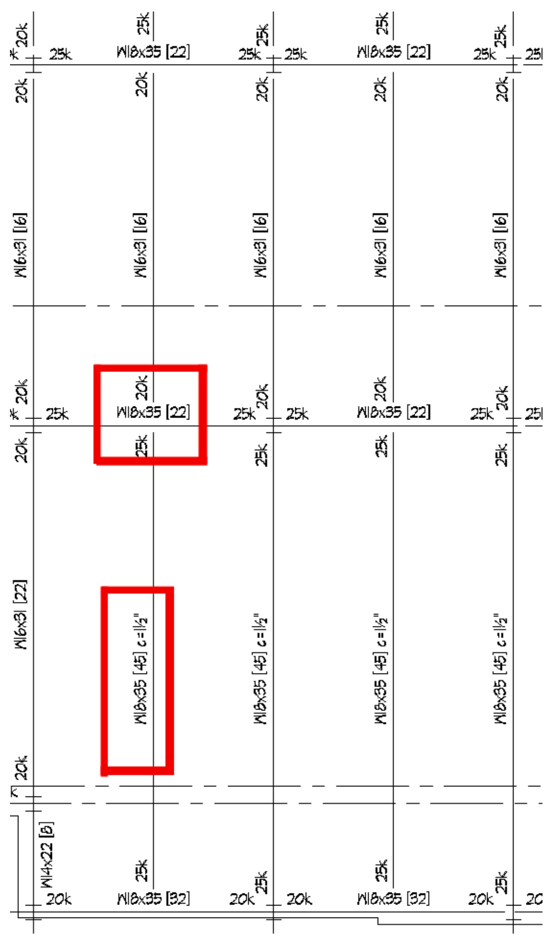

Figure 6 – Existing Composite Beams and Girders with Composite Metal Deck

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It was found in this study that a W18x35 [54] with a camber of 2" is needed for the composite beam and a W16x36 [54] is needed for the composite girder. These members are similar in size to those designed by the engineers, and it is assumed that the slight discrepancies are due to load variations. Therefore, the existing floor system is adequate for the 8th Street Office Building. See Appendix B for all supporting calculations.

Pro-Con Analysis: Composite Metal Deck with Composite Steel Beams

Advantages of the existing floor system include its low self weight and constructability. The system's self weight of 41 psf is significantly lower than the self weights of the alternative floor systems investigated in this report. Therefore, the amount of gravity load that will need to be handled by the foundation will be significantly lower. The construction of the existing system will be simple compared to the alternative systems since the 10 foot spans are achieved without shoring. In general, steel erection takes less time than forming, placing, and curing concrete. In this particular system, the metal decking will act as formwork for the concrete slab, which will allow for a very efficient construction process. Additional advantages include the fire rating of 2 hours and the relatively small total depth of 23 inches. This will leave more space for mechanical ducts and pipes in the ceiling.

Although the existing system is the lightest as well as very efficient to construct, it is also the most expensive system due to the large bay size and costly materials. However, the costs associated with the columns and foundation should be lower due to the low self weight. Also, the steel beams will need to be protected with fireproofing, and the associated cost and labor were not investigated.

In summary, the existing floor system of composite metal deck and composite steel beams was a good choice for the 8th Street Office Building. The long spans and loads did not present a challenge, and the self weight of the system should minimize the weight of the total building. Despite the high cost of the floor system, overall costs may be kept reasonable through smaller columns and foundations.

Alternative Floor System #1: One-Way Solid Slab

The one-way solid slab was designed to span the 20'-0" direction of the typical bay. 24" by 24" columns were chosen as a reasonable size, and a 6" thick slab was found to be reasonable to limit deflections. Flexural reinforcement for the positive moment was designed to be #5 bars spaced at 12" on center, and flexural reinforcement for the negative moment was designed to be #5 bars spaced at 8" on center. Temperature and shrinkage reinforcement was checked, and no shear reinforcement was necessary.

A beam spanning 40'-6" was designed as the worst case beam for the floor system. A 24" wide beam was designed to match the assumed columns in order to provide for better constructability. For deflection control, a total depth of 27" was used for the beam. Eight #9 bars are needed as flexural reinforcement for the negative moment, and seven #8 bars are needed for the positive moment. See Appendix C for all supporting calculations.

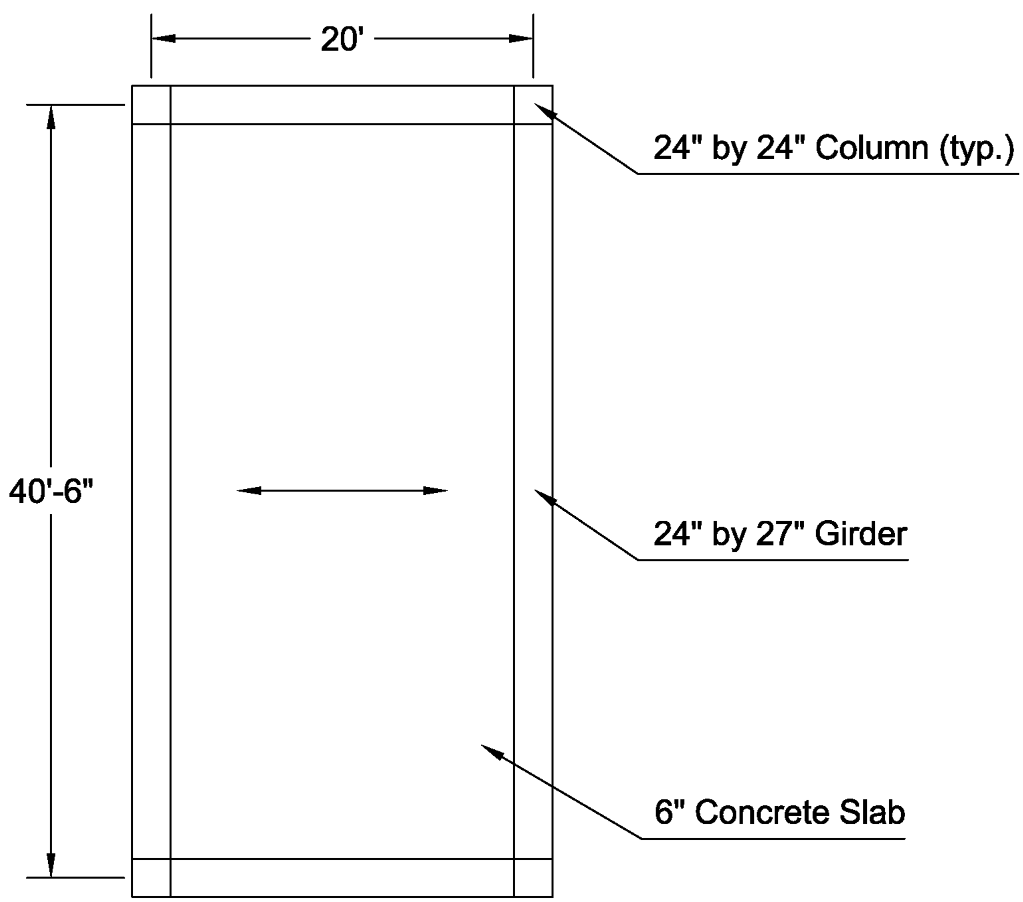


Figure 7 – One-Way Solid Slab Floor System

Pro-Con Analysis: One-Way Solid Slab

A one-way solid slab floor system was chosen as the first alternative system due to the high aspect ratio of the typical exterior bay. Initially, several two-way concrete floor systems were investigated since the aspect ratio is only slightly higher than the limit of 2. However, preliminary calculations showed that the required slab thicknesses for the various two-way systems were prohibitive due to the 40'-6" span. Therefore, one of the main advantages of the one-way solid slab is its relatively small thickness of 6". Other advantages include the slab's fire rating of 2 hours without the need for additional fireproofing as well as the slab's ability to limit vibrations.

Unfortunately, the disadvantages of the one-way solid slab system outweigh the advantages. Even though the slab has a relatively small thickness of 6", the self weight of the system is the largest of all of the systems considered in this report since normal weight concrete was used. Furthermore, the cost of the one-way solid slab system is the second largest, only behind the existing system. It must, therefore, be considered that the total cost of the building may actually surpass the total cost associated with the existing system because the large slab self weight will require larger columns and foundations. Other disadvantages include increased time for construction as well as a slightly larger total depth of the floor system.

In summary, a one-way solid slab will not be considered in any future reports primarily due to the large self weight of the slab and the high cost.

Alternative Floor System #2: One-Way Joist Slab

The one-way joist slab was designed to span the 20'-0" direction of the typical bay, and 24" by 24" columns were chosen. A 3 ½" slab was used in conjunction with 6" wide by 13" deep joists spaced 36" on center. The 13" depth is adequate for deflection control. The flexural reinforcement required for the positive moment is two #4 bars in each joist. The flexural reinforcement required for the negative moment is #3 bars spaced at 8" on center. The reinforcement in the slab perpendicular to the ribs is #3 bars at 16" on center and was dictated by temperature and shrinkage.

A beam spanning 40'-6" was designed as the worst case beam for the floor system. A 24" wide beam was designed to match the assumed columns in order to provide for better constructability. For deflection control, a total depth of 27" was used for the beam. Eight #9 bars are needed as flexural reinforcement for the negative moment, and seven #8 bars are needed for the positive moment. See Appendix D for all supporting calculations.

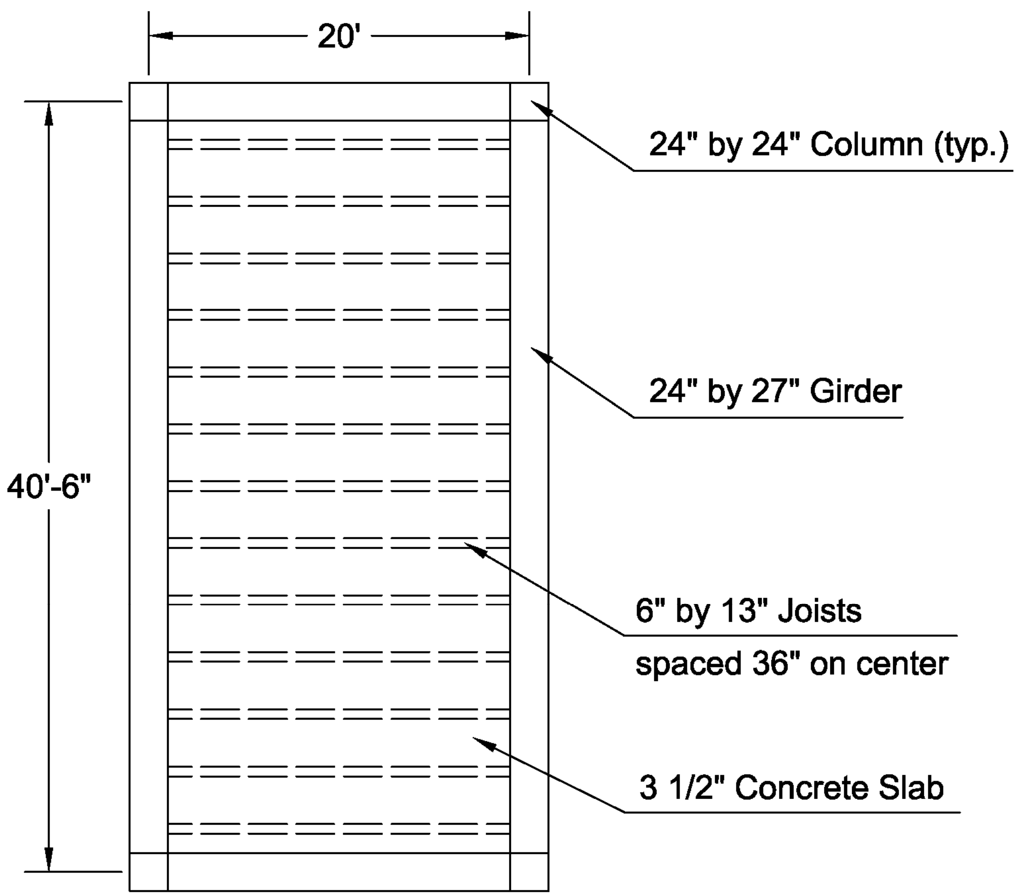


Figure 8 – One-Way Joist Slab Floor System

Pro-Con Analysis: One-Way Joist Slab

A one-way joist slab system was chosen to investigate after the one-way solid slab in order to determine whether any material or cost savings could be achieved. It was discovered that the one-way joist slab does indeed have a smaller self weight than the one-way solid slab; there was an approximately 15% decrease. Similarly, the cost of the one-way joist slab decreased by approximately 18% compared to the one-way solid slab. Other advantages of the one-way joist slab system are identical to those of the one-way solid slab. It is capable of a 2 hour fire rating without additional fireproofing, and vibration issues should be negligible.

Disadvantages include the fact that the self weight is still substantially larger than the self weight of the existing system, so the impact on the foundation will need to be taken into account. Also, the construction will not be as efficient as the existing system due to the necessary formwork. Another disadvantage is that the total depth of the one-way joist floor system is slightly larger; however, there is the possibility to run raceways between the joists, so the depth may not be a problem. Furthermore, it is possible to design the joists to span in the long direction, so that the concrete beams would only be needed to span 20'-0". The beams would then not have to be nearly as deep, and the total depth of the floor system could be reduced.

In summary, it may be worthwhile in the future to compare the total cost of the building associated with the one-way joist slab against the total cost of the building using the existing floor system. The potential exists for the comparatively low floor system cost to outweigh the effects of the larger self weight. On that basis alone, it was determined that the one-way joist slab is a feasible alternative that may require additional study. Furthermore, it would be interesting to design the joists with a span of 40'-6" and ascertain whether any more advantages could be obtained.

Alternative Floor System #3: Pre-Cast Hollow Core Planks on Steel Beams

The pre-cast hollow core planks were sized according to Nitterhouse Concrete Products. It was determined that a 6" thick by 4'-0" wide plank spanning 20'-0" has a capacity of 127 pounds per square foot and is sufficient to carry the required services loads. See Table 4. Additionally, a 2" topping of lightweight concrete was assumed to level floors from camber of the planks and to create a more rigid floor system for lateral loads. See Appendix E for more specific information.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

Table 4 – Load Capacity for Nitterhouse Prestressed Concrete 6" by 4'-0" Hollow Core Plank

A girder was designed to span 40'-6" between columns perpendicular to the hollow core planks. It was determined that a W27x161 is the most economical shape that will carry the loads. See Appendix E for calculations. See Figure 9 for the hollow core planks bearing on the girder.

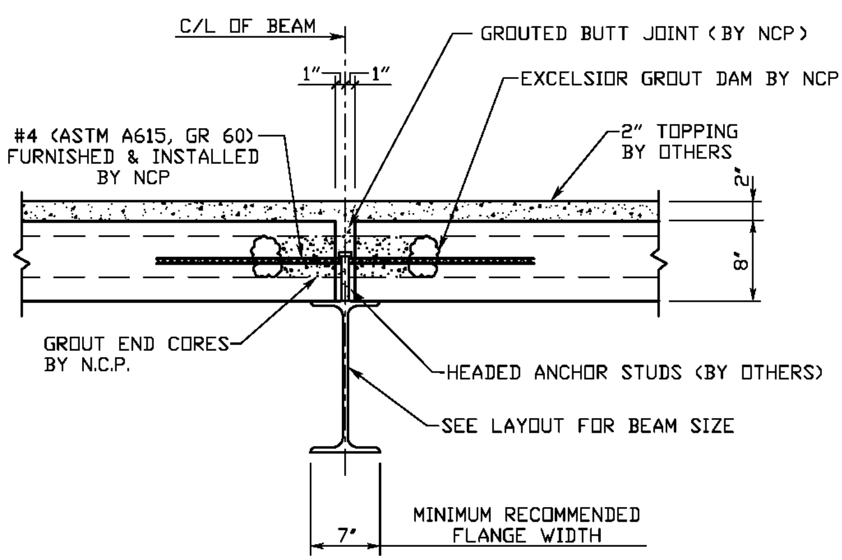


Figure 9 – Nitterhouse Detail of Hollow Core Plank Bearing on Steel Beam

Technical Report #2

Pro-Con Analysis: Pre-Cast Hollow Core Planks on Steel Beams

The main advantages of the pre-cast hollow core planks are the low cost and efficient construction process. The hollow core plank floor system has the lowest cost compared to all of the floor systems investigated in this report. In fact, the cost is approximately 67% lower than the cost of the existing floor system. The pre-cast members are constructed in a plant where curing can take place year round under controlled conditions. Therefore, the planks will be completely up to strength when it is time for erection, thus accelerating the construction process. Another advantage is the 2 hour fire rating without the need for additional fireproofing.

Unfortunately, a couple of rather large disadvantages to the pre-cast hollow core plank floor system argue against its use. The self weight of the system is almost as large as the self weight of the one-way solid slab system, which will affect the sizes of the columns and foundation and possibly increase costs. The hollow core plank system also has the largest total depth due to the large girder that is necessary to span 40'-6". It was attempted to span the pre-cast hollow core planks 40'-6", but none of the Nitterhouse Concrete Products were capable of carrying the required service loads over that length. Finally, the planks may not perform well when introduced to vibrations.

Despite the significant disadvantages, the pre-cast hollow core planks on steel beams may warrant further study due to the very low cost and good constructability. The planks may be an ideal solution if the bay sizes of the 8th Street Office Building can be modified slightly.

Comparison of Floor Systems

Criterion	Floor System Comparison - Typical Exterior Bay			
	Existing Composite Steel	One-Way Solid Slab	One-Way Joist Slab	Pre-Cast Hollow Core Planks on Steel Beams
Self Weight of Slab (psf)	41	75	64	73.75
Slab Depth (in.)	$5\ ^1/_4$	6	$3\ ^1/_2$	6
Total Depth (in.)	23	27	27	$33\ ^5/_8$
Fire Rating (hrs.)	2	2	2	2
Constructability	Good	Below Average	Below Average	Good
Foundation Impact	not applicable	High	Medium	High
Architectural Impact	not applicable	Medium	Medium	High
Vibration	Average	Good	Good	Below Average
Total Cost per SF ($)	40.10	24.35	19.95	13.09
Possible Alternative	not applicable	No	Yes	Yes
Additional Study	not applicable	No	Yes	Yes

Table 5 – Floor System Comparison of Typical Bay

Conclusion

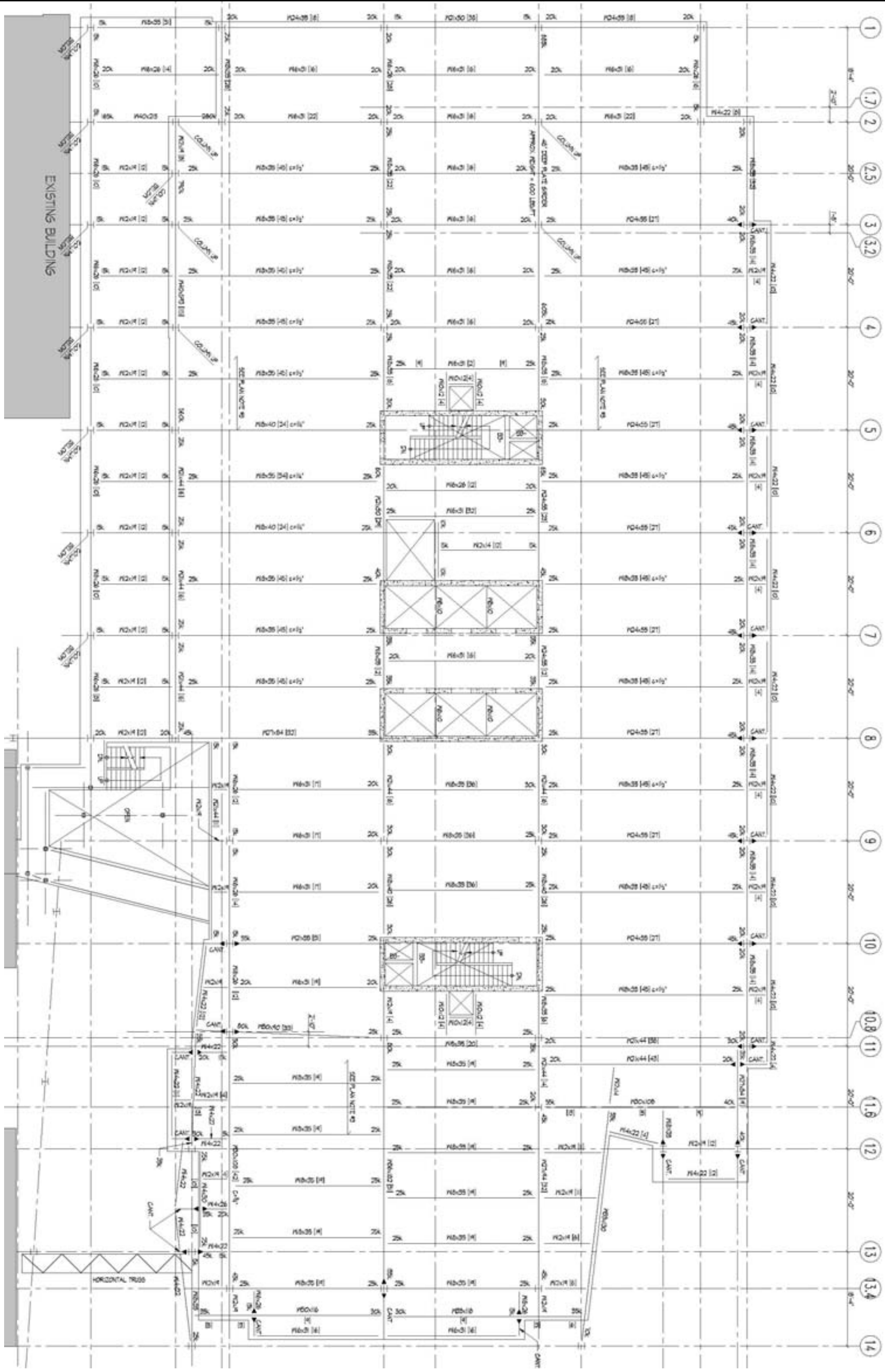
In the second technical report regarding the construction of the 8th Street Office Building, alternative floor systems were designed for a typical exterior bay of 20'-0" by 40'-6". The alternative systems were compared to the existing floor system as well as to each other. The existing floor system consists of United Steel Deck 2" Lok-floor supporting lightweight concrete for a total depth of 5 $^1/_4$". The major comparison factors were self weight, cost, and constructability.

The first alternative floor system that was investigated was a one-way solid slab system. A 6" slab was found to be sufficient in flexure, shear, and deflection. It was immediately concluded that this system is not a viable alternative due to its prohibitive self weight and cost, which would result in a large total building cost. The foundation and column sizes would be severely impacted by a one-way solid slab system.

Although the one-way solid slab system had a few major disadvantages, some advantages indicated that a one-way joist slab floor system should be designed. It was determined that a 3 ½" thick slab with 6" wide by 13" deep joists spaced 36" on center was sufficient to carry the loads and limit deflections. The system had the same advantages as the one-way solid slab system, such as an inherent 2 hour fire rating and negligible vibration issues. It was also found that the self weight and cost of the one-way joist slab system decreased compared to the one-way solid slab. Furthermore, it is anticipated that more advantages may be obtained by designing the joists in the 40'-6" direction rather than the 20'-0" direction. Therefore, the one-way joist slab system is an alternative that should be considered in the future.

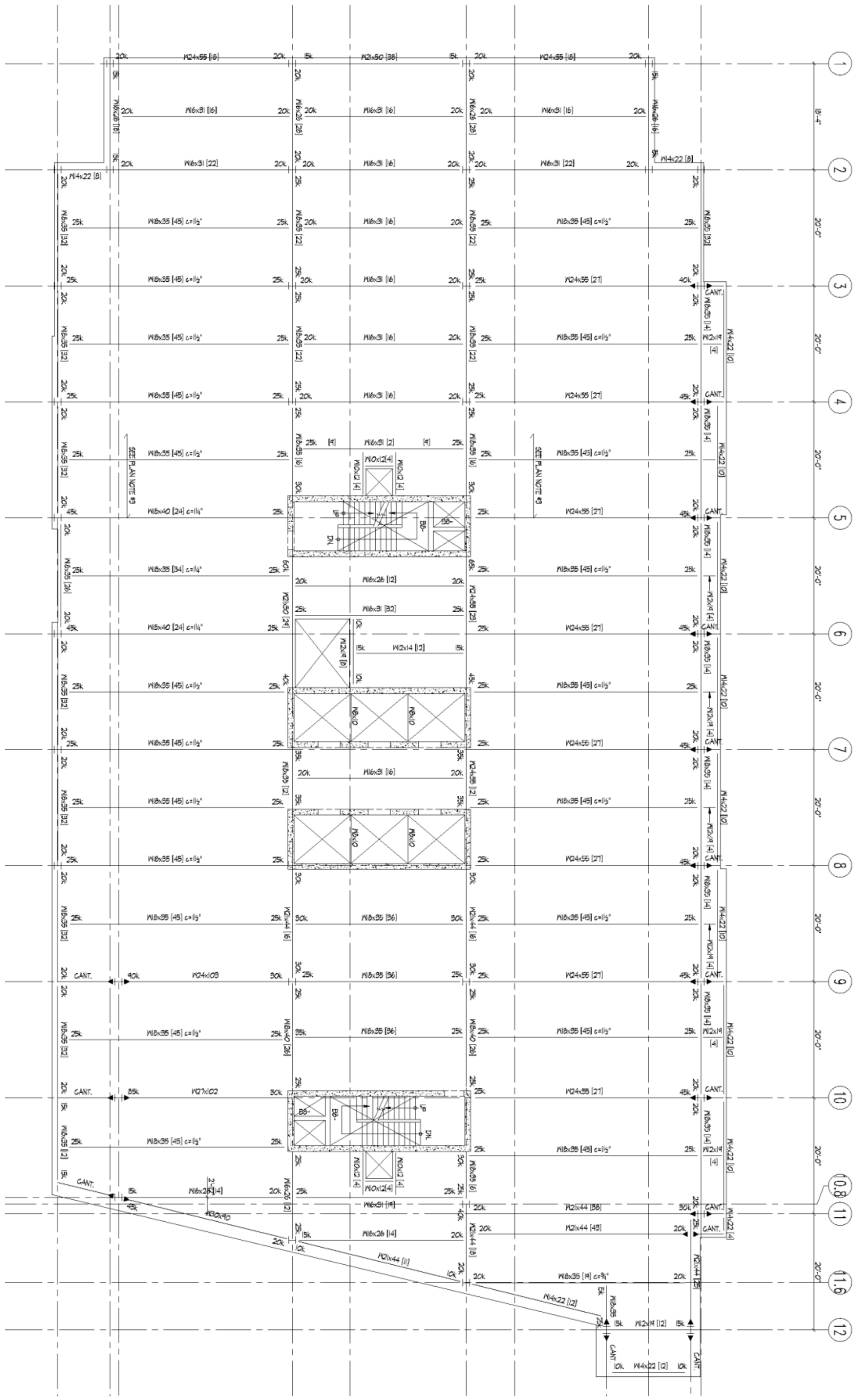
The last alternative floor system that was investigated was the pre-cast hollow core planks on steel beams. Nitterhouse Concrete Products were used, and 6" thick by 4'-0" wide planks were chosen to carry the required service loads in the 20'-0" direction. This system also warrants further study because of its low cost per square foot and efficient constructability. However, the rather large self weight associated with the planks cannot be ignored.

Appendix A – Typical Framing Plans



3rd Floor Framing Plan

Technical Report #2



8th Floor Framing Plan

Technical Report #2

Appendix B – Composite Metal Deck with Composite Steel Beams

Typical Slab/Metal Deck:

2" lightweight concrete fill on 3 1/4" 18 gage composite metal deck

Deck properties specified by engineer:

$I_p = 0.560\ in^4$; $S_p = 0.523\ in^3$; $S_n = 0.529\ in^3$

match 2×12" deck w/ 115 pcf concrete in the United Steel Deck Design Manual & Catalog
2" LOK-FLOOR

Max. unshored span:

5 1/4", 3 span ⇒ $\boxed{10.97\ ft}$ ← considered clear span

beams typically spaced 10 ft on center

∴ typical clear span < 10.97 ft okay

Uniform Live Service Load:

Studs spaced 1 per ft., 5 1/4" slab, 11 ft span to be conservative

⇒ $\boxed{235\ psf}$

use unfactored loads to check

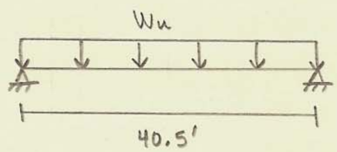
DL = 68 psf
SDL = 42 psf
LL = 80 psf ← corridors above 1st floor

Total = 190 psf* < 235 psf okay

* includes selfweight of slab/metal deck even though United Steel Deck accounts for it so was conservative

Technical Report #2

Typical Beam:



Trib. width = 10'

Loads: DL = 68 psf × 10' = 0.68 k/ft
SDL = 42 psf × 10' = 0.42 k/ft
LL = 80 psf

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{2(40.5)(10)}}\right)$$

$L = 80(0.78) = 62.2$ psf

LL = 62.2 psf × 10' = 0.622 k/ft

$W_u = 1.2D + 1.6L = 1.2(0.68 + 0.42) + 1.6(0.622) = 2.32$ k/ft

$$M_u = \frac{W_u l^2}{8} = \frac{2.32(40.5)^2}{8} = 475.7 \text{ ft-k}$$

$$V_u = \frac{W_u l}{2} = \frac{2.32(40.5)}{2} = 47.0 \text{ k}$$

Assume $a = 1.5$ in then $Y2 = 5\frac{1}{4}'' - \frac{1.5''}{2} = 4.5''$

Table 3-19 in Steel Manual:

Try a $\boxed{W\ 18 \times 35}$ → $\phi M_p = 249$ ft-k
$\phi M = 494$ ft-k
$Y1 = 2$
$\Sigma Q_n = 451$ k

Table 3-21
$Q_n = 17.2$ k

$$\#\text{ studs} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{451}{17.2} \times 2$$

$\boxed{\#\text{studs} = 54}$

Equivalent weight = (40.5')(35 lb/ft) + (54 studs)(10 lb/stud)
= 1.96 k
assumed

check assumptions: b_{eff} = 10 ft. or $\frac{40.5'}{4} = 10.125'$

$\therefore b_{eff} = 10' \times 12 = 120''$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{451}{0.85(3.5)(120)} = 1.26'' < 1.5''$$ so conservative ✓

Technical Report #2

check deflection:

construction Δ : $DL = 0.68\ k/ft + \underset{beam}{0.035\ k/ft} + \underset{studs}{0.013\ k/ft}$
$= 0.728\ k/ft$

$$\Delta_{\substack{construction \\ DL}} = \frac{5}{384}\frac{(0.728)(40.5)^4(1728)}{(29000)(\underset{I}{510})} = 2.98''$$

$\frac{L}{360}$ or 1" is criteria $\Rightarrow$ 2.98" > 1" so NOT okay

however, can $\boxed{\text{camber}}$ the beam $\boxed{2''}$ which is slightly more than by the design engineer

Alternatively, could use a larger beam size like W18×40 if concerned and want to reduce the camber.

live Δ : $\Delta_{LL} = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\frac{(0.622)(40.5)^4(1728)}{(29000)(1370)}$ Table 3-20

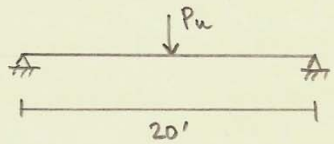
$= 0.95''$

$\frac{L}{360} = \frac{40.5 \times 12}{360} = 1.35'' > 0.95''$ so okay

Finally, $\phi V_n = 159\ k > 47.0\ k$ so okay (Table 3-2

Technical Report #2

Typical Girder:



Beam 1: $W_{u1} = 1.2(\underset{DL}{0.75} + \underset{SDL}{0.42}) + 1.6(\underset{LL}{0.622}) = 2.40\ k/ft$ ← reduced from 80 psf

10' trib width $V_{u1} = \dfrac{2.40(40.5')}{2} = 48.6\ k$

Beam 2: $LL = 80\left(0.25 + \dfrac{15}{\sqrt{2(30)(10)}}\right) = 69.0\ psf$

10' trib width

$W_{u2} = 1.2(0.75 + 0.42) + 1.6(0.69) = 2.51\ k/ft$

$V_{u2} = \dfrac{2.51(30')}{2} = 37.7\ k$

$\therefore\ P_u = 48.6 + 37.7 = 86.3\ k$

$M_u = \dfrac{P_u l}{4} = \dfrac{86.3(20)}{4} = 431.5\ ft\text{-}k$

$V_u = \dfrac{P_u}{2} = \dfrac{86.3}{2} = 43.2\ k$

Assume $a = 2.5''$ so $Y2 = 5.25'' - \dfrac{2.5''}{2} = 4''$

Table 3-19: Try a $\boxed{W\,16 \times 36}$ → $\phi M_p = 240\ ft\text{-}k$
$\phi M = 450\ ft\text{-}k$
$Y1 = 2$
$\Sigma Q_n = 453\ k$

Table 3-21: $Q_n = 17.1\ k$ ← based on $f'_c = 3\ ksi$ so slightly conservative

$\#studs = \dfrac{\Sigma Q_n}{Q_n} \times 2 = \dfrac{453}{17.1} \times 2 = \boxed{54\ studs}$

Equivalent weight $= (20')(36\ lb/ft) + (54\ studs)(10\ lb/stud)$
$= 1.26\ k$

Check assumptions: $b_{eff} = 10'$ or $\dfrac{20'}{4} = 5'$

$\therefore\ b_{eff} = 5(12) = 60''$

Technical Report #2

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{453}{0.85(3.5)(60)} = 2.54'' \approx 2.5'' \text{ okay}$$

Check deflection: $P_{DL} = \frac{0.75(40.5')}{2} + \frac{0.75(30')}{2} = 26.44\text{ k}$

$$W_{DL} = 0.036\text{ k/ft} + 0.027\text{ k/ft} = 0.063\text{ k/ft}$$

$$\Delta_{construction} = \frac{26.44(20)^3(144)}{48(29000)(448)} + \frac{5}{384}\frac{(0.063)(20)^4(1728)}{(29000)(448)}$$

$$= 0.049'' + 0.017'' = 0.066''$$

$$\frac{L}{360} = \frac{20 \times 12}{360} = 0.67'' > 0.066'' \text{ so okay}$$

$$P_{LL} = \frac{0.622(40.5)}{2} + \frac{0.69(30)}{2} = 22.95\text{ k}$$

$$\Delta_{live} = \frac{22.95(20)^3(144)}{48(29000)(1140)} = 0.017'' < \frac{L}{360} \text{ so okay}$$

Table 3-20

Finally, $\phi V_n = 140\text{ k} > 43.2\text{ k}$ okay

Table 3-2

Appendix C – One-Way Solid Slab

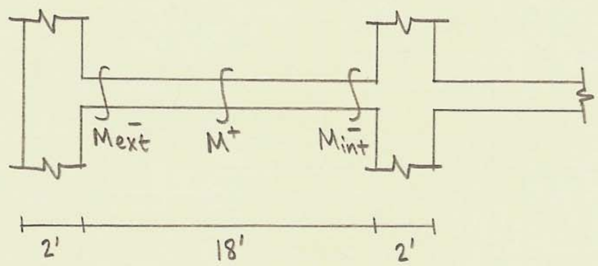
One-Way Solid Slab Design

Assume 24" x 24" columns
LL = 80 psf ← no LL reduction because negligible for unit strip
SDL = 42 psf
Assume h = 6" so $DL = \frac{6"}{12}(150\ lb/ft^3) = 75\ psf$

$f_y = 60{,}000\ psi \qquad f'_c = 5000\ psi$

span center-to-center of columns = 20 ft ⇒ $\ell_n = 18\ ft$

Will look at exterior span → discontinuous end integral with support since worst case moments from Moment Coefficient Method



$W_u = 1.2D + 1.6L = 1.2(0.042 + 0.075) + 1.6(0.080) = 0.2684\ ksf$

$M^-_{ext} = \frac{W_u \ell_n^2}{16} = \frac{0.2684(18)^2}{16} = 5.44\ ft\text{-}k/ft$

$M^+ = \frac{W_u \ell_n^2}{14} = \frac{0.2684(18)^2}{14} = 6.21\ ft\text{-}k/ft$

$M^-_{int} = \frac{W_u \ell_n^2}{10} = \frac{0.2684(18)^2}{10} = 8.70\ ft\text{-}k/ft$

Use this neg. moment because larger and close to interior span $M^- = \frac{W_u \ell_n^2}{11}$

Estimate Reinf. for pos. Moment:

$A_s = \frac{M_u}{4d}$ assume #5 bars so $d = 6" - 0.75" - \frac{0.625"}{2}$

$d = 4.94"$

$A_s = \frac{6.21}{4(4.94)} = 0.31\ in^2/ft$ so $\boxed{\text{use \#5 @ 12"}}$

Check $\phi M_n > M_u$:

assume $f_s > f_y$ $\quad a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.31(60)}{0.85(5)(12)} = 0.365''$

$c = \frac{a}{\beta_1} = \frac{0.365}{0.80} = 0.456''$

$f_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{0.456}(4.94-0.456) = 0.0295 > 0.005$

$\Rightarrow \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = 0.9(0.31)(60)(4.94 - \frac{0.365}{2})/12$

$\phi M_n = 6.64$ ft-k $> M^+ = 6.21$ ft-k so okay

Estimate Reinf. for neg. Moment:

$A_s = \frac{M_u}{4d} = \frac{8.70}{4(4.94)} = 0.44 \text{ in}^2/\text{ft}$

so $\boxed{\text{use \#5 @ 8''}}$

$A_s = 0.31 \text{ in}^2 \times \frac{12}{8} = 0.465 \text{ in}^2/\text{ft}$

Check $\phi M_n > M_u$:

assume $f_s > f_y$ $\quad a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.465(60)}{0.85(5)(12)} = 0.547''$

$c = \frac{a}{\beta_1} = \frac{0.547}{0.80} = 0.684''$

$f_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{0.684}(4.94-0.684) = 0.0187 > 0.005$

$\Rightarrow \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = 0.9(0.465)(60)(4.94 - \frac{0.547}{2})/12$

$\phi M_n = 9.76$ ft-k $> M^- = 8.70$ ft-k so okay

Temperature & Shrinkage Reinf:

$A_T = 0.0018\, b h = 0.0018(12)(6) = 0.13 \text{ in}^2/\text{ft}$

$\boxed{\text{use \#4 @ 18''}} = 0.133 \text{ in}^2/\text{ft}$

Crack Control: $s = 15\left(\frac{40000}{f_s}\right) - 2.5 c_c = 13\ 1/8$

$\Rightarrow s = 12''$ and $8''$ is okay

Spacing & ρ okay by inspection.

Technical Report #2

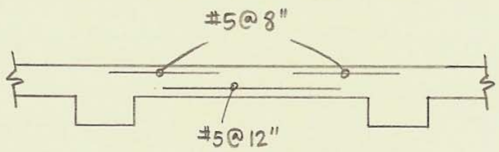
Check Shear in 12" slab strip : (Reference Example 5-7 of W & M textbook)

from shear coefficients

$$V_{u_{max}} = \frac{1.15\, w_u l_n}{2} = 1.15\,\frac{(268.4)(18)}{2} = 2778 \text{ lb/ft}$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}\,(12)(4.94) = 8383 \text{ lb}$$

$$\phi V_c = 0.75\,(8383) = 6288 \text{ lb/ft} > 2778 \text{ lb/ft so okay}$$



temp & shrinkage bars not shown for clarity

Technical Report #2

Must calculate deflection since Table 9.5 (a) could not be used due to partitions:

According to Section 9.5.2.4, calculate deflection @ midspan. Compare with $\frac{l}{480}$ according to Table 9.5 (b).

Reference PCA Example 10.1 and AE 402 notes:

$$W_D = 0.042 + 0.075 = 0.117 \text{ ksf}$$

$$M_D = \frac{W_D l_n^2}{14} = \frac{0.117(18)^2}{14} = 2.71 \text{ ft-k}$$

$$W_L = 0.080 \text{ ksf}$$

$$M_L = \frac{W_L l_n^2}{14} = \frac{0.080(18)^2}{14} = 1.85 \text{ ft-k}$$

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530 \text{ psi}$$

$$E_c = w_c^{1.5}\, 33\sqrt{f'_c} = 150^{1.5}(33)\sqrt{5000} = 4.29 \times 10^6 \text{ psi}$$

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{4.29 \times 10^6} = 6.76$$

$$I_g = \frac{bh^3}{12} = \frac{12(6)^3}{12} = \boxed{216 \text{ in}^4}$$

$$B = \frac{b}{nA_s} = \frac{12}{6.76(0.31)} = 5.73 \text{ in}$$

$$kd = (\sqrt{2dB+1} - 1)/B = (\sqrt{2(4.94)(5.73)+1} - 1)/5.73$$

$$= 1.15 \text{ in}$$

$$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d-kd)^2 = \frac{12(1.15)^3}{3} + 6.76(0.31)(4.94-1.15)^2$$

$$= \boxed{36.18 \text{ in}^4}$$

$$\frac{I_g}{I_{cr}} = \frac{216}{36.18} = 5.97$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{530(216)}{3} / 12000 = \boxed{3.18 \text{ ft-k}}$$

$$M_{D+L} = 2.71 + 1.85 = \boxed{4.56 \text{ ft-k}}$$

Technical Report #2

$$(I_e)_{D+L} = \left(\frac{M_{cr}}{M_{D+L}}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_{D+L}}\right)^3\right] I_{cr}$$

$$= \left(\frac{3.18}{4.56}\right)^3 (216) + \left[1 - \left(\frac{3.18}{4.56}\right)^3\right](36.18) = \boxed{97.17 \text{ in}^4}$$

$$\Delta_{i,D+L} = \frac{K\,(5/48)\,M_{D+L}\,l^2}{E_c\,(I_e)_{D+L}}$$

$$K = 1.2 - 0.2\frac{M_o}{M_a} \quad \text{where } M_o = \frac{wl^2}{8} = \frac{(0.117+0.08)(20)^2}{8}$$

$$= 9.85 \text{ ft-k}$$

$$\Rightarrow K = 1.2 - 0.2\left(\frac{9.85}{4.56}\right) = 0.77$$

$$\Delta_{i,D+L} = \frac{0.77\,(5/48)(4.56)(20)^2}{4.29\times10^6\,(97.17)} \times 12^3 = 0.00061 \text{ in}$$

$$\Delta_{i,D} = \left(\frac{M_D}{M_{D+L}}\right)\Delta_{i,D+L} = \frac{2.71}{4.56}(0.00061) = 0.00036 \text{ in}$$

$$\Delta_{i,L} = \Delta_{i,D+L} - \Delta_{i,D} = 0.00061 - 0.00036 = 0.00025 \text{ in}$$

$$\Delta_{i,L\,sus} = 0.5\,\Delta_{i,L} = 0.5(0.00025) = 0.00013 \text{ in}$$

$\Delta_o = \Delta_{i,D} = 0.00036$ in ← removal of shoring

$\Delta_\infty = \Delta_{i,D} + \Delta_{i,L} + \Delta_{\infty,D} + \Delta_{\infty,L\,sus}$ ← long term deflection

$$\Delta_{\infty,D} = 2\,\Delta_{i,D} = 2(0.00036) = 0.00072 \text{ in}$$

$$\Delta_{\infty,L\,sus} = 2\,\Delta_{i,L\,sus} = 2(0.00013) = 0.00026 \text{ in}$$

$$\Delta_\infty = 0.00036 + 0.00025 + 0.00072 + 0.00026 = 0.00159 \text{ in}$$

$$\Delta_a = \Delta_\infty - \Delta_o = 0.00159 - 0.00036 = \boxed{0.00123 \text{ in}}$$

↑after attachment of nonstructural elements

$\dfrac{l}{480} = \dfrac{20\times12}{480} = 0.5'' > 0.00123''$ so okay

Design beam spanning from column to column perpendicular to one-way solid slab:

Use span = 40'-6" for worst case $\quad f_y = 60$ ksi, $f'_c = 5000$ psi

Use 24" wide beam to match columns

For deflection control from Table 9.5(a) use $h = \frac{l}{18.5}$

for worst case one end continuous $\Rightarrow h = \frac{40.5 \times 12}{18.5} = 26.3"$

use $h = 27"$
$d = 24.5"$

from one way slab strip DL = 75 psf, SDL = 42 psf
LL = 80 psf but reduce with $L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

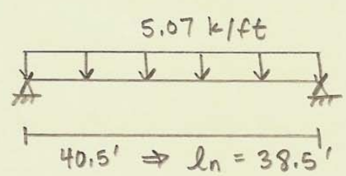
$$LL = 80\left(0.25 + \frac{15}{\sqrt{2(40.5)(20)}}\right) = 80(0.62) = 50 \text{ psf}$$

$$W_{u\,slab} = 1.2(0.075 + 0.042) + 1.6(0.050) = 0.2204 \text{ k/ft}$$

$$\text{Use } V_u = \frac{1.15(0.2204)(18')}{2} + \frac{0.2204(18')}{2} = 4.26 \text{ k/ft}$$

$$W_{sw\,beam} = \frac{24"(27")}{144} \times 150\frac{\text{lb}}{\text{ft}^3} = 0.675 \text{ k/ft}$$

$$W_{u\,beam} = 4.26 \text{ k/ft} + 1.2(0.675) = 5.07 \text{ k/ft}$$



Use moment coefficients because continuous and do end span since worst case.

$$M_u^- = \frac{W_u l_n^2}{10} = \frac{5.07(38.5)^2}{10} = 752 \text{ ft-k}$$

$$M_u^+ = \frac{W_u l_n^2}{14} = \frac{5.07(38.5)^2}{14} = 537 \text{ ft-k}$$

Estimate Neg. Reinf.:

$$\frac{\phi f_y^2 A_s^2}{1.7 f'_c b} - \phi f_y d A_s + M_u = 0$$

$$\left[\frac{0.9(60)^2}{1.7(5)(24)}\right] A_s^2 - 0.9(60)(24.5)A_s + 752(12) = 0$$

$$A_s = 7.5 \text{ in}^2$$

$\boxed{\text{Use (8) \#9 bars}}$ $A_s = 8 \text{ in}^2$

$$d = \underbrace{27''}{} - \underbrace{1.5''}_{cover} - \underbrace{\tfrac{4}{8}''}_{stirrup} - \underbrace{\tfrac{1.128''}{2}}_{d_b/2} = 24.44''$$

$$s_c = \max(1'', 1.128'', \tfrac{4}{3}(1'')) = 1.33''$$

$$b - 2d_{tr} - nd_b - (n-1)s_c = 24 - 2\left(\tfrac{4}{8}\right) - 8(1.128) - 7(1.33)$$

$$= 4.67'' > 2(1.5'')$$

$\Rightarrow$ cover and spacing okay

$A_{s,min}$ okay by inspection, also crack control

$$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85(0.80)\left(\frac{5}{60}\right)\left(\frac{0.003}{0.007}\right)$$

$$= 0.0243$$

$$A_{s,max} = \rho_{max} bd = 0.0243(24)(24.44) = 14.25 \text{ in}^2 > A_s \text{ okay}$$

Check $\phi M_n > M_u^-$:

assume $\varepsilon_s > \varepsilon_y$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8(60)}{0.85(5)(24)} = 4.71''$$

$$c = \frac{a}{\beta_1} = \frac{4.71}{0.80} = 5.88''$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{5.88}(24.44 - 5.88) = 0.0095 > \varepsilon_y$$

ε_y // 0.00207 so okay

also, $\varepsilon_t > 0.005 \Rightarrow \phi = 0.9$

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right) = 0.9(8)(60)\left(24.44 - \frac{4.71}{2}\right)/12$$

$$= 795 \text{ ft-k} > 752 \text{ ft-k} \quad \text{okay}$$

Estimate Pos. Reinf :

$$\left[\frac{0.9(60)^2}{1.7(5)(24)}\right] A_s^2 - 0.9(60)(24.5)A_s + 537(12) = 0$$

$$A_s = 5.2 \text{ in}^2$$

$\boxed{\text{Use (7) \# 8 bars}}$ $A_s = 5.53\ in^2$

$d = 27'' - 1.5'' - \frac{4}{8}'' - \frac{1}{2}'' = 24.5''$

cover, spacing, $A_{s,min}$, $A_{s,max}$, crack control all okay by inspection

Check $\phi M_n > M_u^+$:

assume $\varepsilon_s > \varepsilon_y$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.53(60)}{0.85(5)(24)} = 3.25''$$

$$c = \frac{a}{\beta_1} = \frac{3.25''}{0.80} = 4.07''$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{4.07}(24.5 - 4.07) = 0.015 > \underset{0.00207}{\varepsilon_y}$$

okay

also, $\varepsilon_t > 0.005 \Rightarrow \phi = 0.9$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = 0.9(5.53)(60)\left(24.5 - \frac{3.25}{2}\right)/12$$

$$= 569\ ft\text{-}k > 537\ ft\text{-}k \text{ okay}$$

Shear: $V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}(24)(24.5)/1000$

$$= 83.2\ k$$

$$\phi V_n = 0.5\,\phi V_c = 0.5(0.75)(83.2) = 31.2\ k$$

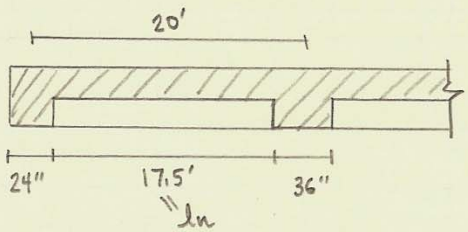
$$V_{u\,max} = \frac{1.15(5.07\ k/ft)(38.5')}{2} = 112.2\ k$$

$\therefore$ will need stirrups

Appendix D – One-Way Joist Slab

One-Way Joist Design   Reference PCA Example 7.6

Use 6" wide joists spaced 36" o.c.
Assume spandrel beam width = 24"
and interior beam width = 36"
$f_y = 60{,}000$ psi
$f'_c = 5000$ psi



From Table 9.5(a), preliminarily begin with $h = \frac{l}{18.5} = \frac{20 \times 12}{18.5}$

$h = 13''$

Also, use a 3.5" thick slab

LL = 80 psf ← no reduction since only 3' trib width
SDL = 42 psf
use $DL = \left[\frac{(13'' \times 6'' + 3.5'' \times 30'')}{144}\right](150\ \text{lb/ft}^3) = 191\ \text{lb/ft}$

$W_u = 1.2(0.042)(3') + 1.2(0.191) + 1.6(0.08)(3') = 0.7644\ \text{k/ft}$

Look at end span - discontinuous end integral w/ support - M^+ and M^-_{int} since worst case moments from Moment Coefficient Method and very close to interior span moments

$M^+ = \frac{W_u l_n^2}{14} = \frac{0.7644(17.5)^2}{14} = 16.72$ ft-k

$M^-_{int} = \frac{W_u l_n^2}{10} = \frac{0.7644(17.5)^2}{10} = 23.41$ ft-k

for $f'_c = 5000$ psi & $f_y = 60$ ksi $\Rightarrow \rho_t = 0.02125$

set $\rho = 0.5\rho_t = 0.5(0.02125) = 0.010625$

req'd depth of joist : $\omega = \frac{\rho f_y}{f'_c} = \frac{0.010625(60)}{5} = 0.1275$

then $\frac{M_u}{\Phi f'_c b d^2} = 0.1179$

$$d = \sqrt{\frac{M_u}{\phi f'_c b (0.1179)}} = \sqrt{\frac{23.41 \times 12}{0.9(5)(6)(0.1179)}} = 9.4 \text{ in}$$

$\therefore$ just continue using $h = 13''$ ← should be a bit conservative

$$d = 13'' - 1.25'' = 11.75''$$

Reinf. for pos. Moment:

$$\frac{M_u}{\phi f'_c b d^2} = \frac{16.72 \times 12}{0.9(5)(6)(11.75)^2} = 0.0538$$

$$\omega \approx 0.056$$

$$A_s = \omega b d \frac{f'_c}{f_y} = 0.056(6)(11.75)\left(\frac{5}{60}\right) = 0.329 \text{ in}^2$$

Check rectangular section behavior:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.329(60)}{0.85(5)(6)} = 0.774'' < 3.5'' \text{ so okay}$$

Use $\boxed{(2)\ \#4 \text{ bars}}$ for $A_s = 0.40 \text{ in}^2$

Reinf. for neg. Moment:

$$\frac{M_u}{\phi f'_c b d^2} = \frac{23.41 \times 12}{0.9(5)(6)(11.75)^2} = 0.0754$$

$$\omega \approx 0.080$$

$$A_s = \omega b d \frac{f'_c}{f_y} = 0.08(6)(11.75)\left(\frac{5}{60}\right) = 0.47 \text{ in}^2$$

Distribute uniformly in slab:

$$\frac{0.47 \text{ in}^2}{3} = 0.157 \text{ in}^2/\text{ft} \Rightarrow \text{use } \boxed{\#3 \text{ @ } 8''}$$

$$A_s = 0.165 \text{ in}^2/\text{ft}$$

Reinf. for Slab normal to ribs:
Locate at mid-depth of slab for both pos. & neg. moments

Use $M_u = \dfrac{w_u l_n^2}{12}$ where $DL = \dfrac{3.5''}{12}(150 \text{ lb/ft}^3) = 44 \text{ psf}$

so $W_u = 1.2(0.044 + 0.042) + 1.6(0.080) = 0.2312 \text{ k/ft}^2$

$$M_u = \frac{0.2312\,(2.5)^2}{12} = 0.120 \text{ ft-k}$$

$$\frac{M_u}{\phi f'_c b d^2} = \frac{0.120 \times 12}{0.9\,(5)(12)(1.75)^2} = 0.00874$$

$w \approx 0.009$

$$A_s = \frac{w b d f'_c}{f_y} = \frac{0.009\,(12)(1.75)(5)}{60} = 0.016 \text{ in}^2/\text{ft}$$

for slabs, $A_{s,min}$ governed by temperature & shrinkage

$$A_{s,min} = 0.0018\, b h = 0.0018\,(12)(3.5)$$
$$= 0.076 \text{ in}^2/\text{ft}$$

spacing $s_{max} = 5h = 5(3.5'') = 17.5'' > 18''$ ← governs

use $\boxed{\#3 \text{ bars @ } 16''}$ $A_s = 0.08 \text{ in}^2/\text{ft}$

Crack control, ρ, spacing okay by inspection for pos. and neg. reinf. design.

Check Shear : Reference PCA Example 12.4

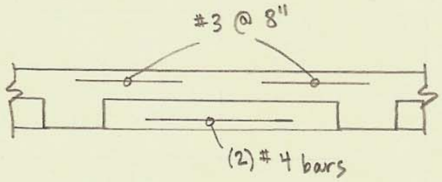
$w_u = 0.7644$ k/ft

$$V_{u\,max} = \frac{1.15\, w_u l_n}{2} = \frac{1.15\,(0.7644)(17.5)}{2} = 7.69 \text{ k}$$

By Section 8.13.8 V_c can be increased by 10%

$$\phi V_c = 1.1\, \phi\, 2\sqrt{f'_c}\, b\, d = 1.1(0.75)(2)\sqrt{5000}\,(6)(11.75)/1000$$

$\phi V_c = 8.23$ k > 7.69 k so okay



temp & shrinkage bars not shown for clarity

Technical Report #2

Check deflection since Table 9.5(a) cannot be used due to partitions:

Calculate deflection @ midspan and compare to $\frac{l}{480}$ from Table 9.5(b).

Reference PCA Example 10.2 and AE 402 notes:

$W_D = 0.042(3') + 0.191 = 0.317$ k/ft

$W_L = 0.080(3') = 0.240$ k/ft

$M_D = \frac{W_D l_n^2}{14} = \frac{0.317(17.5)^2}{14} = 6.93$ ft-k

$M_L = \frac{W_L l_n^2}{14} = \frac{0.240(17.5)^2}{14} = 5.25$ ft-k

$M_{D+L} = 6.93 + 5.25 = \boxed{12.18 \text{ ft-k}}$

$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530$ psi

$E_c = w_c^{1.5}\, 33\sqrt{f'_c} = 150^{1.5}(33)\sqrt{5000} = 4.29 \times 10^6$ psi

$n = \frac{E_s}{E_c} = \frac{29\times10^6}{4.29\times10^6} = 6.76$

$$y_t = h - 0.5\,\frac{\left((b-b_w)h_f^2 + b_w h^2\right)}{\left((b-b_w)h_f + b_w h\right)}$$

$h = 13''$
$h_f = 3.5''$
$b_w = 6''$
$b = 36''$

$$y_t = 13 - 0.5\left[\frac{(36-6)(3.5)^2 + 6(13)^2}{(36-6)(3.5) + 6(13)}\right] = 9.23''$$

$$I_g = \frac{(b-b_w)h_f^3}{12} + \frac{b_w h^3}{12} + (b-b_w)h_f\left(h - \frac{h_f}{2} - y_t\right)^2 + b_w h\left(y_t - \frac{h}{2}\right)^2$$

$$I_g = \frac{30(3.5)^3}{12} + \frac{6(13)^3}{12} + 30(3.5)\left(13 - \frac{3.5}{2} - 9.23\right)^2 + 6(13)\left(9.23 - \frac{13}{2}\right)^2 = \boxed{2215 \text{ in}^4}$$

$$B = \frac{b}{nA_s} = \frac{36}{6.76(0.40)} = 13.31$$

Technical Report #2

Check deflection since Table 9.5(a) cannot be used due to partitions:

Calculate deflection @ midspan and compare to $\frac{l}{480}$ from Table 9.5(b).

Reference PCA Example 10.2 and AE 402 notes:

$$W_D = 0.042(3') + 0.191 = 0.317 \text{ k/ft}$$

$$W_L = 0.080(3') = 0.240 \text{ k/ft}$$

$$M_D = \frac{W_D \ln^2}{14} = \frac{0.317(17.5)^2}{14} = 6.93 \text{ ft-k}$$

$$M_L = \frac{W_L \ln^2}{14} = \frac{0.240(17.5)^2}{14} = 5.25 \text{ ft-k}$$

$$M_{D+L} = 6.93 + 5.25 = \boxed{12.18 \text{ ft-k}}$$

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530 \text{ psi}$$

$$E_c = w_c^{1.5}\, 33\sqrt{f'_c} = 150^{1.5}(33)\sqrt{5000} = 4.29 \times 10^6 \text{ psi}$$

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{4.29 \times 10^6} = 6.76$$

$$y_t = h - 0.5\frac{\left((b-b_w)h_f^2 + b_w h^2\right)}{\left((b-b_w)h_f + b_w h\right)}$$

$h = 13''$
$h_f = 3.5''$
$b_w = 6''$
$b = 36''$

$$y_t = 13 - 0.5\left[\frac{(36-6)(3.5)^2 + 6(13)^2}{(36-6)(3.5) + 6(13)}\right] = 9.23''$$

$$I_g = \frac{(b-b_w)h_f^3}{12} + \frac{b_w h^3}{12} + (b-b_w)h_f\left(h - \frac{h_f}{2} - y_t\right)^2 + b_w h\left(y_t - \frac{h}{2}\right)^2$$

$$I_g = \frac{30(3.5)^3}{12} + \frac{6(13)^3}{12} + 30(3.5)\left(13 - \frac{3.5}{2} - 9.23\right)^2 + 6(13)\left(9.23 - \frac{13}{2}\right)^2 = \boxed{2215 \text{ in}^4}$$

$$B = \frac{b}{nA_s} = \frac{36}{6.76(0.40)} = 13.31$$

Technical Report #2

$$kd = \frac{\sqrt{2\,dB+1} - 1}{B} = \frac{\sqrt{2(11.75)(13.31)+1} - 1}{13.31} = 1.26 \text{ in}$$

$1.26 \text{ in} < h_f = 3.5 \text{ in}$ so okay to treat as rectangular compression area

$$I_{cr} = \frac{b(kd)^3}{3} + nA_s(d-kd)^2$$

$$I_{cr} = \frac{36(1.26)^3}{3} + 6.76(0.40)(11.75-1.26)^2 = \boxed{322 \text{ in}^4}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{530(2215)}{9.23} / 12000 = \boxed{10.60 \text{ ft-k}}$$

$$(I_e)_{D+L} = \left(\frac{M_{cr}}{M_{D+L}}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_{D+L}}\right)^3\right] I_{cr}$$

$$(I_e)_{D+L} = \left(\frac{10.60}{12.18}\right)^3 (2215) + \left[1 - \left(\frac{10.60}{12.18}\right)^3\right](322) = \boxed{1570 \text{ in}^4}$$

$$(\Delta i)_{D+L} = \frac{K(5/48)M_{D+L}\,l^2}{E_c\,(I_e)_{D+L}}$$

where $K = 1.2 - 0.2\dfrac{M_o}{M_a}$; $M_o = \dfrac{wl^2}{8} = \dfrac{(0.317+0.240)(20)^2}{8} = 27.85 \text{ ft-k}$

$$\Rightarrow K = 1.2 - 0.2\left(\frac{27.85}{12.18}\right) = 0.74$$

$$(\Delta i)_{D+L} = \frac{0.74(5/48)(12.18)(20)^2}{4.29\times10^6\,(1570)} \times 12^3 = 0.000096 \text{ in.}$$

$$\Delta_{i,D} = \left(\frac{M_D}{M_{D+L}}\right)\Delta_{i,D+L} = \left(\frac{6.93}{12.18}\right)(0.000096) = 0.000055 \text{ in}$$

$$\Delta_{i,L} = \Delta_{i,D+L} - \Delta_{i,D} = 0.000096 - 0.000055 = 0.000041 \text{ in}$$

$$\Delta_{i,L\,sus} = 0.3\,\Delta_{i,L} = 0.3(0.000041) = 0.000012 \text{ in}$$

$\Delta_o = \Delta_{i,D} = 0.000055$ in $\leftarrow$ removal of shoring

$\Delta_\infty = \Delta_{i,D} + \Delta_{i,L} + \Delta_{\infty,D} + \Delta_{\infty,L\,sus}$ $\leftarrow$ long term deflection

Technical Report #2

$\Delta_{\infty,D} = \frac{2}{1+50\rho'} \Delta_{i,D} = 2(0.000055) = 0.00011$ in

$\Delta_{\infty,L\,sus} = \frac{2}{1+50\rho'} \Delta_{i,L\,sus} = 2(0.000012) = 0.000024$ in

$\Delta_{\infty} = 0.000055 + 0.000041 + 0.00011 + 0.000024 = 0.00023$ in.

$\Delta_a = \Delta_{\infty} - \Delta_0 = 0.00023 - 0.000055 = \boxed{0.000175 \text{ in.}}$

after attachment of nonstructural elements

$\frac{l}{480} = \frac{20 \times 12}{480} = 0.5'' > 0.000175''$ so okay

Technical Report #2

Design beam spanning from column to column perpendicular to one-way joists:

Use span = 40'-6" for worst case

$f_y = 60$ ksi
$f'_c = 5000$ psi

Use 24" wide beam to match columns

again use $h = 27"$ for deflection control according to one end continuous from Table 9.5(a)

then $d = 24.5"$

from one-way joist SDL = 42 psf , DL = 0.191 k/ft

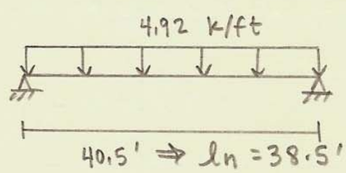
LL = 80 psf reduced to 50 psf (see beam for one-way slab)

$$W_{u\,joist} = 1.2(0.191) + 1.2(0.042)(3') + 1.6(0.050)(3') = 0.6204 \text{ k/ft}$$

$$\text{Use } V_u = \frac{1.15(0.6204)(18')}{2} + \frac{0.6204(18')}{2} = 12.32 \text{ k / 3 ft.}$$

$$W_{sw\,beam} = 0.675 \text{ k/ft} \quad \text{(see beam for one-way slab)}$$

$$W_{u\,beam} = 1.2(0.675) + \frac{12.32}{3} = 4.92 \text{ k/ft}$$



Use moment coefficients because continuous and do end span since worst case.

$$M_u^- = \frac{W_u l_n^2}{10} = \frac{4.92(38.5)^2}{10} = 729 \text{ ft-k}$$

$$M_u^+ = \frac{W_u l_n^2}{14} = \frac{4.92(38.5)^2}{14} = 521 \text{ ft-k}$$

Estimate Neg. Reinf.:

$$\frac{\phi f_y^2 A_s^2}{1.7 f'_c b} - \phi f_y d A_s + M_u = 0$$

$$\left[\frac{0.9(60)^2}{1.7(5)(24)}\right] A_s^2 - 0.9(60)(24.5)A_s + 729(12) = 0$$

$$A_s = 7.24 \text{ in}^2$$

Technical Report #2

$\boxed{\text{Use (8) \#9 bars}}$ $\quad A_s = 8 \text{ in}^2$

see beam design for one-way slab for calculations showing cover, spacing, etc. are okay

$\phi M_n = 795 \text{ ft-k} > 729 \text{ ft-k}$ so okay

Estimate Pos. Reinf:

$$\left[\frac{0.9(60)^2}{1.7(5)(24)}\right] A_s^2 - 0.9(60)(24.5)A_s + 521(12) = 0$$

$$A_s = 5.03 \text{ in}^2$$

$\boxed{\text{Use (7) \# 8 bars}}$ $\quad A_s = 5.53 \text{ in}^2$

Again, see beam design for one-way slab for additional calculations.

$\phi M_n = 569 \text{ ft-k} > 521 \text{ ft-k}$

Shear: $V_c = 2(1)\sqrt{5000}(24)(24.5)/1000 = 83.2 \text{ k}$

$\phi V_n = 0.5\,\phi V_c = 0.5(0.75)(83.2) = 31.2 \text{ k}$

$V_{u\,max} = \dfrac{1.15(4.92)(38.5)}{2} = 108.9 \text{ k}$

∴ will need stirrups

Technical Report #2

Appendix E – Pre-Cast Hollow Core Planks on Steel Beams

Pre Cast Hollow Core Planks on Steel Beams:

Use worst case typical bay where planks span 20' and girders span 40'-6".

Loading for Planks:

Safe superimposed service load = LL + SDL
= 80 + 42 = $\boxed{122 \text{ psf}}$

Assumption is that plank & topping selfweight is already taken into account in table.

⇒ Use Nitterhouse prestressed concrete $\boxed{6'' \times 4'\text{-}0''}$ hollow core plank w/ $\boxed{2'' \text{ topping}}$

at a 20' span with $\boxed{4\text{-}\tfrac{1}{2}''\ \phi}$ strands → can carry $\boxed{127 \text{ psf}}$

Assume within deflection limits.

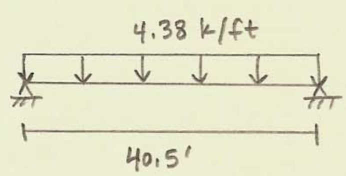
Plank & topping SW = 48.75 + 25 = 73.75 psf

Design girder between columns running perpendicular to planks:

SDL = 42 psf
DL = 73.75 psf
LL = 80 psf ← reduced to $LL = 80\left(0.25 + \dfrac{15}{\sqrt{2(40.5)(20)}}\right)$

$LL = 80(0.62) = 50 \text{ psf}$

$W_u = 1.2(42 + 73.75)(20') + 1.6(50)(20') = 4.38 \text{ k/ft}$



$M_u = \dfrac{W_u l^2}{8} = \dfrac{4.38(40.5)^2}{8} = 898 \text{ ft-k}$

$V_u = \dfrac{W_u l}{2} = \dfrac{4.38(40.5)}{2} = 88.7 \text{ k}$

From Table 3-10: Use a $\boxed{W27 \times 161}$

($\phi M_n = 1035$ ft-k at unbraced length = 38.5')

$\therefore$ infer ϕM_n @ 40.5' > 898 ft-k okay

$\phi V_n = 546$ k > 88.7 k okay (Table 3-2)

Check deflection:

construction Δ: DL = 0.07375 (20') + 0.161
= 1.636 k/ft

$$\Delta = \frac{5}{384} \frac{(1.636)(40.5)^4(1728)}{(29000)(6310)} = 0.54''$$

$\frac{L}{360}$ or 1" is criteria $\Rightarrow$ 0.54" < 1" okay

live Δ: $$\Delta = \frac{5}{384} \frac{(0.05 \times 20')(40.5)^4(1728)}{(29000)(6310)} = 0.33''$$

$$\frac{L}{360} = \frac{40.5 \times 12}{360} = 1.35'' > 0.33'' \text{ okay}$$

Technical Report #2

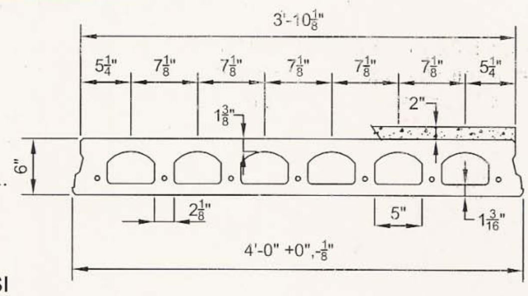
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast b_w = 16.13 in.
I_c = 1519 in.^{4}	Precast S_{bcp} = 370 in.^{3}
Y_{bcp} = 4.10 in.	Topping S_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 3.90 in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 95.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

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. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

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