School of Engineering and Applied Science Building

Miami University, Oxford, OH

Technical Assignment 2

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AE 481W – Senior Thesis

The Pennsylvania State University

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Executive Summary

The purpose of this report is to research possible alternative floor framing systems that may have been used in lieu of the current composite concrete on steel frame system of Miami University's School of Engineering and Applied Science Building.

When analyzing the alternative floor systems, criteria such as weight of the system, depth of the structure, constructability, lead time, vibration, fireproofing, and relative cost were considered, and compared to the criteria performance of the existing structural system. The following four floor systems were analyzed as possible options:

1. Non-composite lightweight concrete slab on steel beams
2. Two-way slab with drop panels
3. Two-way post-tensioned flat slab with shear caps
4. Precast hollowcore plank on steel beams

After close review of each possible system, it would appear that precast hollowcore floor plank on steel girders is the best possible system for the building. The fact that it requires a long lead time is more than made up for in schedule time, as erection of the plank is by far the fastest option available, saving valuable labor cost. The plank will give superior acoustical performance in comparison to other systems, and vibration is improved over the existing system. The weight of this floor system is more than the current composite steel system, which will increase seismic loads, and will require further analysis to find the effects of this impact on the lateral resisting system. Most importantly though, the overall cost of this system is the lowest of all floors under review. This fact, combined with the other benefits makes hollowcore flooring the best option for this building.

Existing Structural System

- ## Foundation

The lower level of the parking garage is a 5” slab on grade with a minimum 28-day compressive strength of 4500 psi, over 6” of granular subbase. It is reinforced with WWF 6x6 – W4.0xW4.0 wire mesh. The concrete columns, which carry the load from the main building above are supported by spread footings which range in size from 4’-0”x4’-0”x24” reinforced with (7)#5 bars each way to 9’-0”x9’-0”x42” reinforced with (15)#8 bars each way. The garage walls around the exterior are supported by 2’-0”x2’0” footings reinforced with (3)#9 top and bottom steel, while the wall footing running through the center of the garage is only 1’6” deep and reinforced with (2)#7 bottom bars. The School of Engineering and Applied Science Building’s entrance plaza is a slab on grade with a minimum 28 day compressive strength of 4000 psi which varies by location from 5” thick reinforced with WWF 6x6 W4.0xW4.0 to 9” thick reinforced with #5 bottom bars at 12” O.C. and top WWF 6x6 W4.0xW4.0. The plaza is supported by drilled piers that range in size from 36” diameter, 12’-8” deep, to 60” diameter, 17’-4” deep. Grade beams run between the drilled piers and are typically 2’-0”x2’0”. All footings, piers, and grade beams have a minimum concrete strength of 5000 psi.

- ## Floor System
  - ### Upper Floors

The first, second and mechanical floor of the School of Engineering and Applied Science Building utilizes a composite floor system with a typical concrete slab of 3½” on 3” 18 gage composite metal deck with normal weight concrete of minimum 28-day strength of 4000 psi, and is reinforced with WWF 6x6 W2.9xW2.9. The most typical bay is 30’-0”x30’-0” where the deck spans over (3) 10’ spans on W16x26 beams with (26) ¾” diameter, 5” headed shear studs, and are cambered 1½”. The beams frame into W21x83 girders at third-points, which have (40) shear studs of equal dimensions, and are not typically cambered. Girders in areas with larger tributary areas, in the north side of the building are W24x84’s. These girders are also part of the lateral resisting system in the East-West direction and are supported with partially restrained moment connections at the columns. The roof is a mansard roof around the perimeter, sloping at a 12-12 pitch until it flattens off through the central part of the building. The roof does not have a composite slab, and is built of 4” rigid insulation on 1½” 20 gage wide rib roof deck, which spans on wide flange beams which are typically W8x10 on the pitched part of the roof, and are W10x12 or W12x16 in the central, flat area. The beams frame into girders which are generally W18x55.

- ### Garage

The middle and the upper levels of the garage, as well as the ground floor of the main building are comprised of a 2-way reinforced concrete slab with a minimum 28-day compressive strength of 5000 psi. The bay layout generally follows that of the columns above, typically 30'-0"x30'-0", from the main building to avoid the need for transfer slabs and girders. The middle and upper levels of the garage use a 9" flat slab with 10'-0"x10'-0"x8" drop panels at the columns. At the east end of the upper level, the slab turns into a 10" flat slab, and continues to turn into a 12" flat slab at ground floor, particularly on the northern half of the building. This is due to the fact that the live load on the ground floor is higher than anywhere else throughout the main building or garage. There are (3) transfer beams in this northern section of the main floor spanning north to south where the garage column layout doesn't exactly match that of the upper floors, which are 50" deep and are 36" or 48" wide. At the easternmost end of the building, there is a small section of slab where it is thickened to 14" to carry the some masonry walls.

Columns

- ### Upper Floors

Columns supporting the first floor through the roof are rolled W12 shapes with a yield strength of 50 ksi. Most of the columns contribute to the moment frame in the East-West direction, which range in size from W12x40 to W12x136. Where the columns continue all the way to the main roof through the mechanical floor, they are spliced just above the mechanical floor level. The base plates of gravity columns typically 1¼" – 1½" thick on 2" of non-shrink grout, with (4) anchor bolts embedded 16" into the ground floor concrete, and are assumed to act as pin connections. Columns acting as part of the moment frames or the vertical braces have heavier 2" – 2¼" thick, much larger in area so that the anchor bolts can be placed outside of the columns' projected area, unlike the gravity columns, and are assumed to act as fixed connections.

- ### Garage

The concrete columns in the garage are typically 24"x24", and have specified concrete strengths of either 4500 psi or 5000 psi depending on the location, and hence load, on the column. Reinforcement in the columns varies from (4)#11 bars to (12)#11 bars and splice at the middle level of the garage. The number of dowels at the base of the columns follows the number of reinforcement bars in the column, and are embedded to the bottom of the spread footing and hooked, creating a fixed connection.

- ## Lateral Resistance System

    - ### North-South Direction

The lateral system in the transverse (short) direction of the building consists of four single bay concentrically braced steel frames from the ground floor to the mechanical floor, of roughly the same size. There is only one cross brace at each of the three levels of the brace, sloping up from south-to-north, and are made of steel tubing, ranging in size from HSS8x8x¼ to HSS10x10x½. Diagrams can be found in Appendix A of this report. For lateral resistance from the mechanical floor to the roof, the mansard roof around the perimeter helps to brace the roof, but is helped by four single-span moment frames, which frame into the column's weak bending axis.

    - ### East-West Direction

The longitudinal (long) direction of the building utilizes an ordinary moment frame system. Two of the frames in the southern half of the building run the full length of the main building, and are the only two lateral resisting elements at the upper floors where the building steps back at the 2nd floor level. The ground and 1st floor also have four additional, shorter moment frames, two on each side of the rear entrance plaza at the center of the building. The moment frames use a partially restrained moment connection that has plates bolted to the flanges, which then are welded with full-penetration welds into the columns supporting the beams.

Design Codes

The School of Engineering and Applied Science Building was designed using the 2002 Ohio Building Code (OBC) with reference to ASCE 7-98 for building load determination procedures. ACI 318-98 was used to design the concrete portions of the structure, and concrete masonry construction was designed using ACI 530.1, Specifications for Masonry Structures, and construction specification section 04810. The 1992 edition of AISC's Code of Standard Practice for Steel Buildings and Bridges, as modified by the construction documents, was used for design of steel members, and ANSI/AWS Structural Welding Code – Steel D1.1 was used for design of welds.

This report will use the more recent IBC 2006 with reference to ASCE 7-05 for building loads. ACI 318-05, Building Code Requirements for Structural Concrete, and the Load Resistance Factored Design procedure from the 13th edition of AISC's Manual of Steel Construction will be used for design of the concrete and steel structural members, respectively.

Design Loads

• Dead Loads on Existing Structural System

Item	Weight
Concrete (Normal Weight)	150 pcf
Typical Floor	62.5 psf
Upper and Middle Garage 9" Slab	112.5 psf
Ground Floor 10" slab	125 psf
Ground Floor 12" slab	150 psf
Metal Deck	2 psf
Steel Framing	8 psf
Ceiling and Mechanical Allowance	
Typical Floor	15 psf
Mechanical Floor	25 psf
Roof	10 psf
Garage	10 psf
Partition Allowance	10 psf
Roof Materials	
4" Rigid Insulation	6 psf
Roof Membrane	1 psf
1/2" Gypsum Board	2 psf

• Live Loads

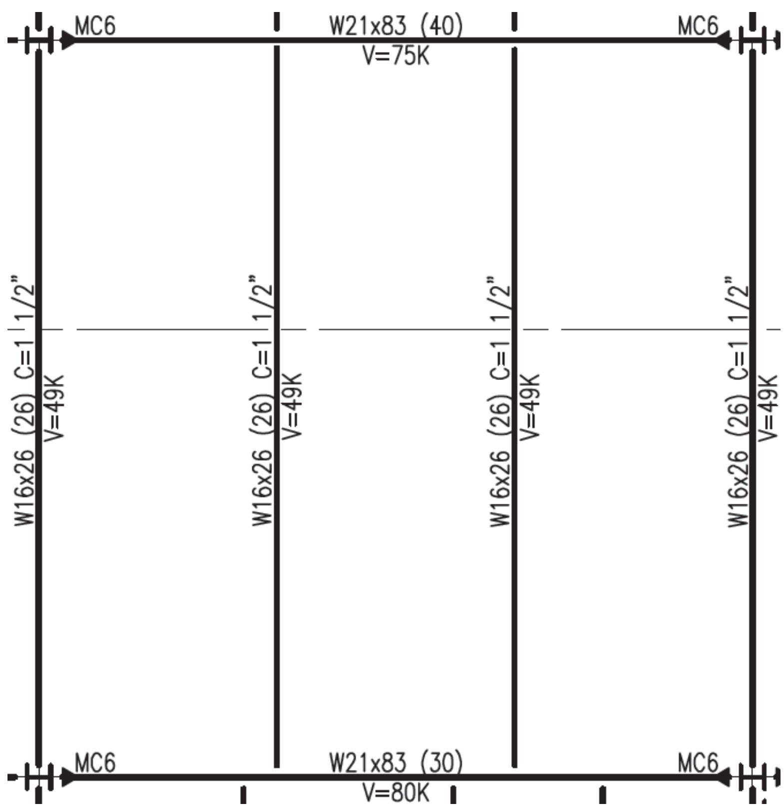
It is worthy to note that ASCE 7-05 does not specify live loads for labs such as the ones within the School of Engineering and Applied Sciences Building, which is what the majority of the space within the building is designated for. The designer chose to use a uniform load of 100 psf for upper level labs and 125 psf for labs at ground floor, which is what this report will use in the analysis.

Area	Design Live Load
Typical Floor	100 psf
Labs at Ground Level	125 psf
Mechanical Equipment Rooms	150 psf
Plaza	100 psf
Roof	25 psf
Parking Decks	50 psf
PSE Basement at Upper Garage Level	125 psf
Utility Tunnel	250 psf + 360 psf overburden

Alternative Floor Framing Systems

The structural layout of The School of Engineering and Applied Science Building is primarily comprised of 30'x30' bays, which gives multiple different floor systems a chance of being considered for the final design. Since the building is only four stories high and there are no height limitations to conform to, the structure depth is not nearly as important as it may be with very tall high rises. The current system meets many of the primary goals of the design and gives a very open, unrestricted floor plan that allows for some of the large laboratories. A typical bay of the composite system is shown below, which was the basic bay used in design of the alternative systems. An overall floor plan for the first floor can be viewed in Appendix A.

This section will summarize the results of the design and compare the advantages and disadvantages of each alternative floor system under consideration. A graph comparing all factors and conclusions may be found at the end of the individual analyses.



Typical Bay Framing Diagram

• Non-composite Lightweight Concrete Slab on Steel Frame

This system is nearly identical to the existing floor system in the building, with the same basic bay layout, but does not utilize the possible composite action that the concrete slab provides in the composite system. In an effort to actually reduce the weight of the structure, an analysis using lightweight (115 pcf) concrete was used. Using the United Steel Deck design manual, the slab depth can actually be reduced from the current 6.5" with the more recent load factors from the original building design. An even more effective system may be to use the lightweight concrete compositely with the steel frame.

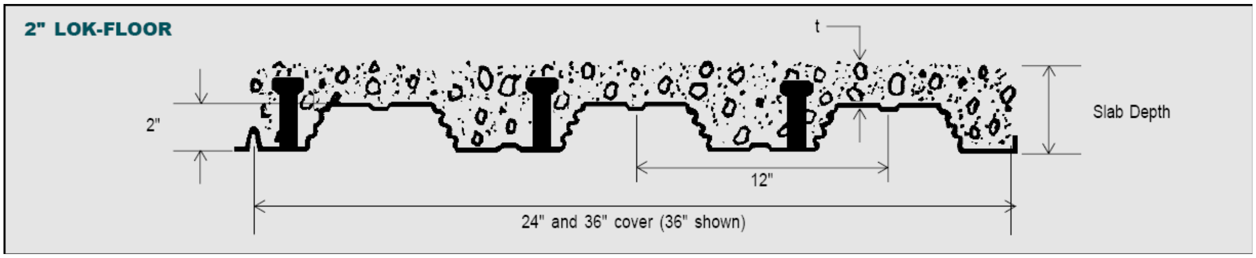


Diagram of 2" LOK-Floor (Note: shear studs shown here are not used)

• Advantages

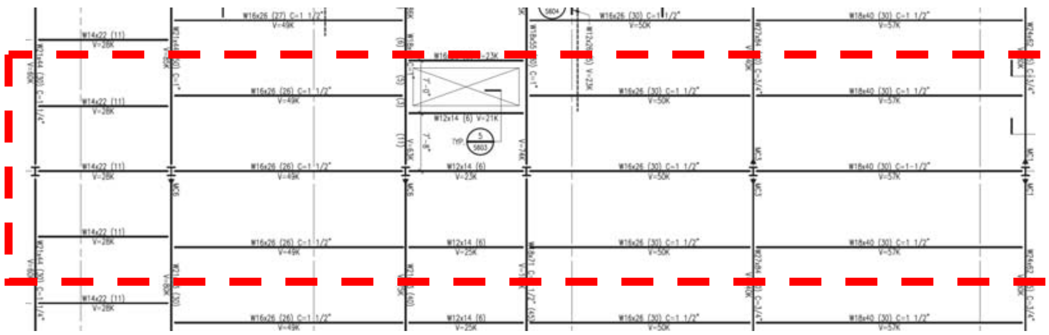
Construction and fabrication of steel members of a non-composite slab is simpler than a composite system since steel shear studs are not needed. Using lightweight concrete and the thinner slab, the dead load of the structure is significantly reduced, and will lower seismic loads, and hence lateral resisting elements' sizes.

• Disadvantages

Steel beam systems in general offer the least vibration resistance, especially when not used compositely with concrete, which may be a deterrent in lab spaces. Steel fabricators need a long lead in time. Fireproofing of steel members is required. Lightweight concrete is also more expensive than normal weight concrete.

• 2-Way Slab with Drop Panels

This is the base structural system used in the below ground parking garage and the ground floor. It utilizes mild reinforcing in both directions with drop panels only rather than having beams run in between each column as in a waffle slab system. It was determined that the irregularity of the bay layout did not lend itself well to the direct design method, so an equivalent frame analysis of a frame in the north-south direction was analyzed to find maximum design moments. It is worthy to note that the 36' exterior bay required a 3" thicker slab than the rest of the building. If the exterior columns were moved in 6', a cantilever slab may be used for the end span, but supporting the wall may prove to be difficult. A rotated diagram of frame under investigation along column line 2 is shown here.



• Advantages

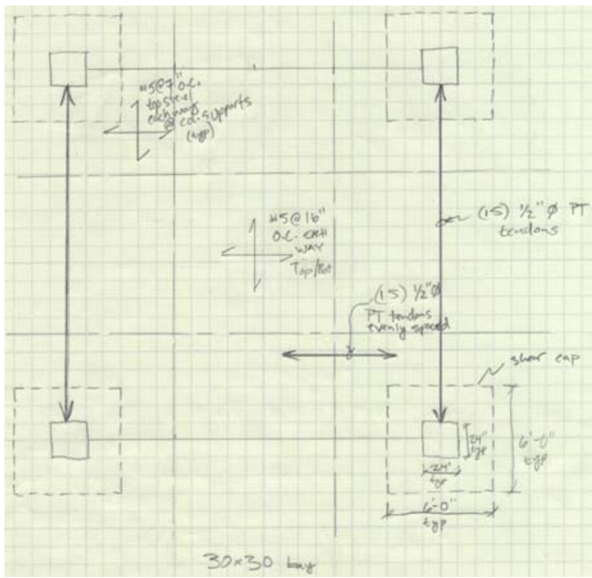
A concrete floor has a much shallower structure depth than the existing steel system, which allows more room mechanical equipment. A significant reduction in vibration is achieved, which may be beneficial for the laboratories. Also, having the same structural system as the garage and ground floor simplifies the construction of the building and provides for an easier transfer of loads to the foundation.

• Disadvantages

The drop panels make routing mechanical chases more difficult than a flat plate slab, which can be an issue. They are also rather unsightly and have potential to disrupt interior designs and partition walls. The increased column size will also take away from usable floor space, and may change the clear width in hallways. The cost of the floor system is relatively high because of the very complicated formwork of the system, and will also add time to the overall schedule of the project. However, the biggest disadvantage of this system is its weight, over 130 psf. This will dramatically increase the magnitude of seismic loads on the building, which would need to be resisted by a new lateral system, either with shear walls or a moment frame within the slab and columns.

• 2-Way Post-Tensioned Flat Slab with Shear Caps

A post-tensioned slab is typically used in an effort to have the thinnest slab depth possible, while reducing deflections of concrete floors on long spans due to the upward camber caused by the prestressing force. Banded tendons run in the north-south direction, where span length frequently changes as seen in the design of the two-way mild reinforced slab, and uniformly spaced tendons are run in the east-west direction along each span. Due to the way these tendons are placed, it is desirable to have the same depth of slab throughout the building, unlike the mild reinforced system, where it is feasible to have an increased slab depth where necessary in larger spans.



• Advantages

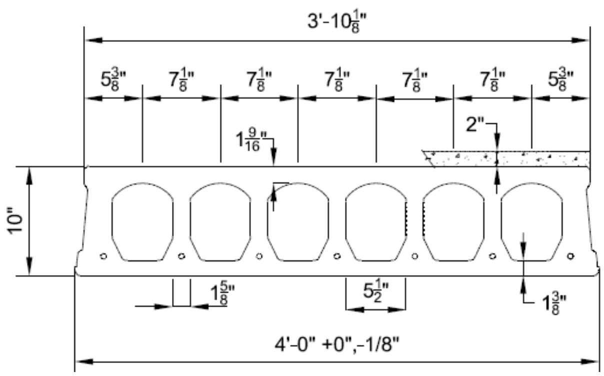
A post-tensioned slab has the thinnest structural depth of all alternative floor systems under consideration, which decreases the overall building height. Also, superior serviceability performance is obtained due to the effect of the prestressing tendons in the slab.

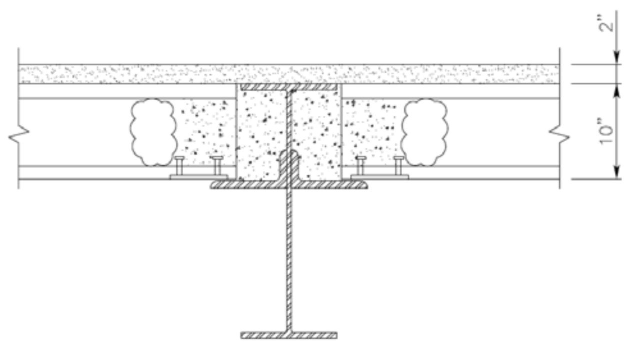
• Disadvantages

The disadvantages of this system are very similar to those of the two-way mild reinforced slab. For these large spans, a flat plate slab with no drop panel or shear caps is uneconomical due to the added cost of shear reinforcing around the column to protect against punching shear. Shear caps, or column capitals, may have adverse effects on routing mechanical equipment and can interrupt partition walls, making framing of them difficult. Construction of a PT slab can be difficult and time consuming depending on the contractor's familiarity with the system.

• Precast Hollowcore Plank on Steel Frame

Hollowcore floor plank cross sections vary from one precast manufacturer to another, so design of these members is not an industry standard, and is typically performed by the precaster's engineering department. For the purpose of this report, Nitterhouse Concrete Products, Inc.'s published load tables have been used to select the appropriate members within their product line. A cast-in-place concrete topping was also chosen to be used in the design, which will be thinner at midspan of the planks, acting as a leveling coat to hide the inherent camber in the members. When bearing on steel beams, plank typically placed on the top flange and grouted at the joint, but this unnecessarily increases the depth of the floor. A girder slab system can be used where a custom steel beam with a narrow top flange so that the plank may be dropped in and set on the bottom flange, but then the beam cannot be used in a moment frame. To use achieve this same effect with standard wide flange shapes, a steel angle can be welded to each side of the beam web, where the long leg sticks out past the edge of the beam's flange, and the hollowcore floor plank can bear on the angle. A diagram is shown here.





• Advantages

The biggest advantage to using precast floor planks is the ease and speed of erection of the floor system. No formwork is required and concrete is already fully cured when they arrive. The same base structural moment frame can be used as in the existing system. The planks eliminate the need for intermediate beams within the bay, which allows greater freedom for the placement of mechanical chases. The cores lend superior acoustical properties to the floor in comparison with other floor systems.

• Disadvantages

A much longer lead time is needed to manufacture the members, but the time saved in on site construction more than makes up for this. This system has the thickest "slab" of any of the systems under consideration.

• Comparison Chart

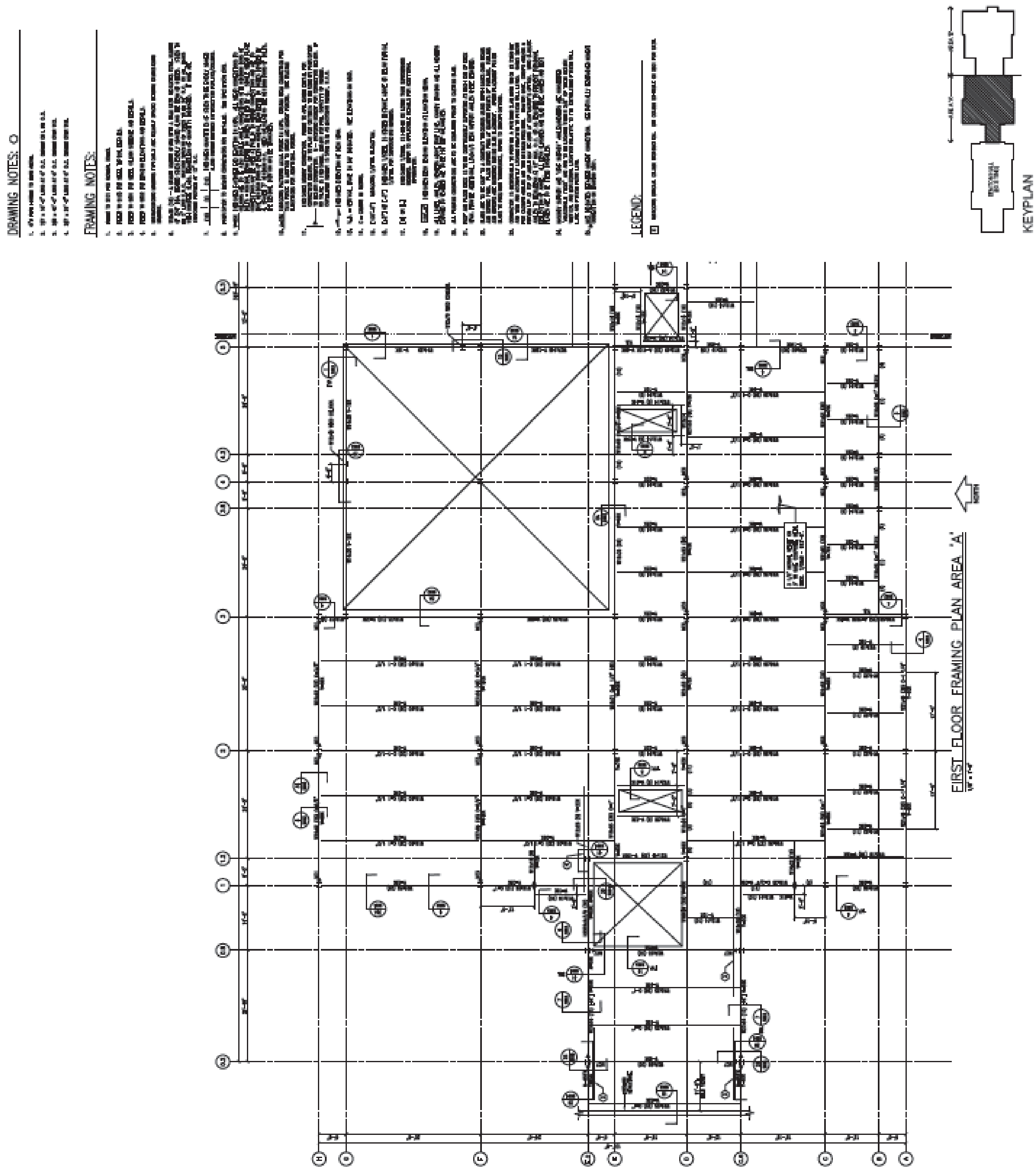
System	Composite Steel (Existing)	Non-composite LWC Slab on Steel Frame	2-Way Slab w/ Drop Panels	2-Way Post-Tensioned Flat Plate Slab w/ Shear Caps	Hollowcore Plank on Steel Frame
Weight (psf)	71	54	131	128	96
Slab Depth (in)	6.5	5.25	9.5	10.25	12
Largest Depth (in)	33.5	26.25	18.5	10.25	29
Column Size (in)	W12	W12	24x24	24x24	W12
Construction Difficulty	Medium	Medium	Hard	Hard	Easy
Lead Time	Medium	Medium	Short	Short	Long
Formwork	Little	Little	Yes	Yes	None
Additional Fireproofing	Yes	Yes	No	No	Some
Lateral System Effects	N/A	None	Concrete Moment Frame or Shear Walls	Shear Walls	Little
Relative Vibration	High	High	Low	Low	Medium
Foundation Impact	-	None	Little	Little	Little
Cost per square foot					
Materials	$14.40	$16.61	$7.64	$10.62	$10.72
Labor	$6.26	$7.73	$8.10	$8.01	$3.15
Total	$20.66	$24.34	$15.74	$18.63	$14.87
Viable Alternative	-	Yes	No	No	Yes
Further Study	-	Yes	No	No	Yes

• Conclusions

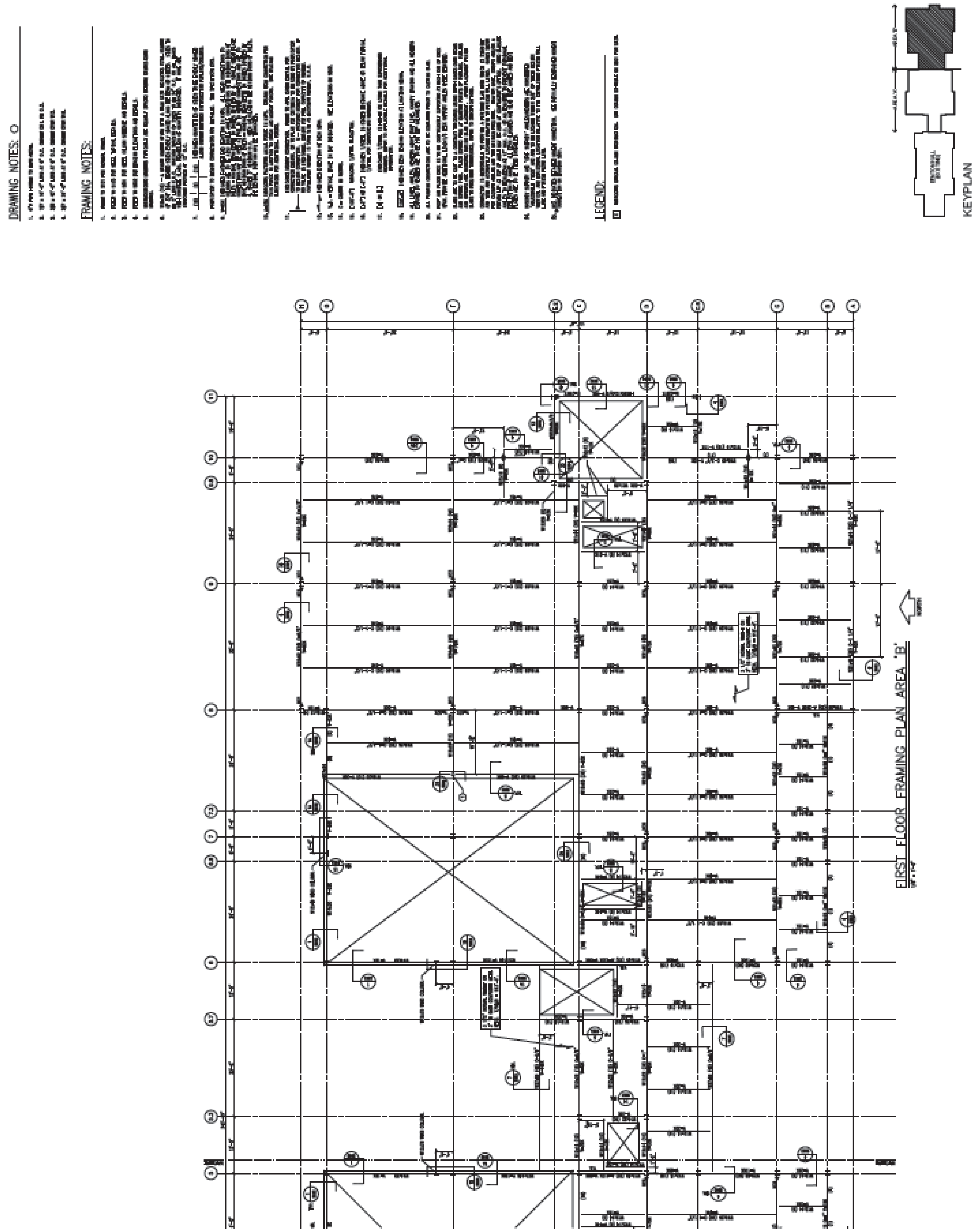
Each alternative floor framing system clearly has its own unique advantages and disadvantages. The column grid was able to remain unchanged, and the foundation system should simply need to be redesigned for the different loads on each system, making those effects have a negligible impact on the selection of the optimal system. Based on the system selected, vibration was clearly not a governing criterion, though increased serviceability in other systems can be achieved. Changing to a concrete slab system would require a different lateral force resisting system, which may have a severely adverse effect on the architectural freedom in floor plan design if shear walls are used. Also, the increased column size may cause problems near the hallways where minimum clear widths are required by the ADA. The most important factor in floor system selection is cost and length of schedule and a precast hollowcore floor is the best system for both of these criteria.

Appendix A – Plans and Diagrams

First Floor Framing Plan – Area 'A' (West half of building)

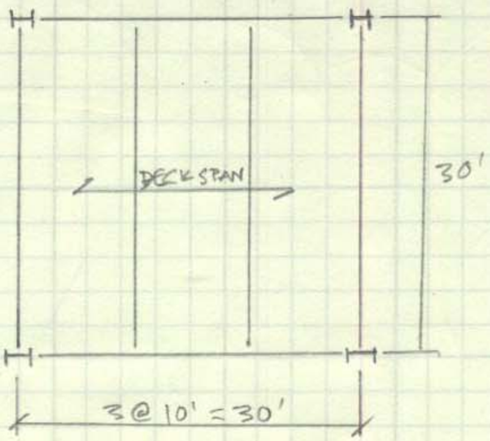


First Floor Framing Plan – Area 'B' (East half of building)



Appendix B – Non-composite Lightweight Concrete Slab on Steel Frame Calculations

NON-COMPOSITE LIGHTWEIGHT CONCRETE SLAB ON STEEL FRAME



REF: UNITED STEEL DECK (USD) DESIGN MANUAL

$f'_c = 3000psi$ L.W.C. (115 pcf

$f_{y,deck} = 33$ ksi

Include SI DL in w_{LL} for deck design tables

Design criteria:

1) Use COMPOSITE DECK
2) TO ACHIEVE A 2-HR. FIRE RATING WITH LWC, A MINIMUM OF 3 1/4" SLAB ABOVE THE RIBS MUST BE USED
3) INCLUDE SUPERIMPOSED DL IN W_{LL} FOR DECK DESIGN TABLES

$$W_{LL} = \underset{LL}{100\ PSF} + \underset{CEIL/MECH}{15\ PSF} + \underset{PARTITIONS}{10\ PSF} = \underline{\underline{125\ PSF}}$$

SELECT 19 GAGE 2" x 12" LOK-FLOOR w/ 5.25" SLAB

MAX UNSHORED SPAN = 10.91' > 10' ok✓

MAX L, LIVE LOAD @ 10' SPAN = 175 PSF > 125 PSF ok✓

DESIGN BEAM

$$W_{DL} = 10'\left[\underset{SLAB}{\left(\frac{3.25" + \frac{2"}{2}}{12}\right)(115\ PCF)} + \underset{DECK}{4\ PSF} + \underset{CEIL/MECH.}{15\ PSF} + \underset{PARTITION}{10}\right] + \underset{BM.WT. GUESS}{75\ PLF}$$

$$= 772\ PLF$$

$$W_U = 1.2(.772) + 1.6(.100 \times 10') = 2.527\ klf$$

$$M_U = \frac{W_U L^2}{8} = \frac{(2.527)(30')^2}{8} = 284.3'^k$$

$L_U = 30'$ (UNBRACED LENGTH

- DEFLECTION CRITERIA
  - LIVE LOAD Δ

$\Delta_L \leq L/360 = 1.0''$

$1.0'' \geq \frac{5(1\text{ klf})(30')^4}{384(29{,}000)\,I}(1728)$

$\Rightarrow I \geq 628.4\text{ in}^4$

  - TOTAL LOAD Δ

$w_T \leq L/240 = 1.5''$

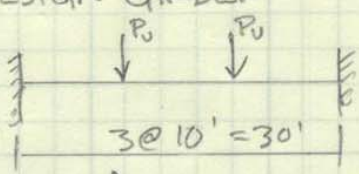
$1.5'' \geq \frac{5(1.772\text{ klf})(30)^4}{384(29000)\,I}(1728)$

$\Rightarrow I \geq 742.4\text{ in}^4 \leftarrow$ controls

$\boxed{\text{SELECT W}14\times74\text{ @ }10'\text{ O.C.}}$

@ 30' : $\phi M_n = 302^{'k} > 284.3^{'k}$ ok✓
$I = 795\text{in}^4 > 742.4\text{ in}^4$ ok✓

DESIGN GIRDER



DESIGN OF LARGEST GIRDER:
30' BAY ON SIDE, 36' BAY ON OTHER SIDE

$P_U^{30'} = 15'\left[1.2(.771\text{ KLF}) + 1.6(1.00\text{ KLF})\right] = 37.88^k$

$P_U^{36'} = \frac{18'}{15'}(37.88^k) = 45.45^k$

$P_U = 37.88 + 45.45 = \underline{83.33^k}$

$M_U^- = \frac{2}{3}P_U a = \frac{2}{3}(83.33^k)(10') = \underline{555.5^{'k}}$

$M_U^+ = \frac{1}{2}M_U^- = \frac{1}{2}(555.5) = \underline{277.8^{'k}}$

$L_U = 10'$ (UNBRACED LENGTH

- DEFLECTION CRITERIA
  - TOTAL LOAD Δ

$\Delta_T \leq L/240 = 1.5''$ cont.

$P^{30'} = 15'\left[.771\,klf + 1.00\,klf\right] = 26.57^k$

$P^{36'} = \frac{18}{15}(26.57) = 31.88^k$

$P = 26.57 + 31.88 = 58.45^k$

$$1.5' \geq 2\frac{(58.45^k)(20)^2(15)^2}{6(29000)I(30)^2}\left(3(10)(30) - 3(10)(15) - (20)(15)\right)\left(144\tfrac{in^2}{ft^2}\right)$$

$\Rightarrow I \geq \underline{967.4\,in^4}$ ← controls

LIVE LOAD Δ

$\Delta_L \leq L/360 = 1.0''$

$$1.0'' \geq 2\frac{(33^k)(20)^2(15)^2}{6(29000)I(30)^2}\left(3(10)(30) - 3(10)(15) - (20)(15)\right)\left(144\tfrac{in^2}{ft^2}\right)$$

$\Rightarrow I \geq \underline{819.3\,in^4}$

$\boxed{\text{SELECT W21} \times 68}$

$\phi M_n^- = 600'^k > 555.5'^k$ ok ✓

@L_b=10': $\phi M_n^+ = 531'^k > 277.8'^k$ ok ✓

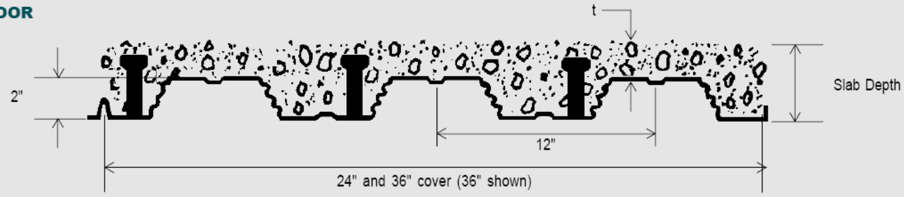
$I = 1480\,in^4 > 967.4\,in^4$ ok ✓

NOTE: THIS GIRDER IS DESIGNED FOR GRAVITY LOADS ONLY, BUT IS USED IN A MOMENT FRAME TO RESIST LATERAL LOADS IN THE EAST-WEST DIRECTION. DESIGN OF GIRDER MAY BE CONTROLLED BY THESE LATERAL LOADS, WHICH WOULD REQUIRE FURTHER ANALYSIS

USD United Steel Deck, Inc.

LRFD

2" LOK-FLOOR



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.[4]); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.[3]); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

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	0.630	0.490	0.445	0.458	1330	2810	0.61
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.69
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3990	0.87

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.[2] per foot of width. **Vol.** is the volume of concrete in ft.[3] per ft.[2] needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.[2]. S_c is the section modulus of the "cracked" concrete composite slab; in.[3] per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.[4] per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are <u>no studs</u> on the beams (the deck <u>is attached</u> to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

COMPOSITE PROPERTIES													
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	34	1.00	4.4	28.13	4270	6.32	8.46	8.56	0.023
	5.00	46.44	37.5	0.333	38	1.18	6.0	33.12	4610	6.03	8.09	8.19	0.027
	5.25	49.53	40.0	0.354	41	1.27	6.9	35.69	4790	5.90	7.93	8.02	0.029
	5.50	52.61	42.6	0.375	43	1.36	7.9	38.29	4970	5.77	7.77	7.86	0.032
	6.00	58.78	48.0	0.417	48	1.55	10.1	43.58	5340	5.55	7.49	7.58	0.036
	6.25	61.87	50.8	0.438	50	1.65	11.3	46.26	5540	5.45	7.36	7.45	0.038
	6.50	64.95	53.6	0.458	53	1.75	12.7	48.97	5730	5.36	7.24	7.32	0.041
	7.00	71.12	59.5	0.500	58	1.94	15.7	54.44	6150	5.18	7.01	7.10	0.045
	7.25	74.21	61.9	0.521	60	2.04	17.4	57.20	6310	5.10	6.91	6.99	0.047
	7.50	77.29	64.3	0.542	62	2.14	19.2	59.97	6480	5.05	6.81	6.89	0.050
20 gage	4.50	48.60	32.6	0.292	34	1.20	4.8	33.77	4560	7.42	9.71	10.03	0.023
	5.00	56.18	37.5	0.333	38	1.42	6.5	39.80	5030	7.07	9.28	9.59	0.027
	5.25	59.96	40.0	0.354	41	1.53	7.4	42.91	5210	6.91	9.09	9.39	0.029
	5.50	63.75	42.6	0.375	43	1.64	8.5	46.05	5390	6.76	8.91	9.20	0.032
	6.00	71.32	48.0	0.417	48	1.87	10.9	52.47	5760	6.49	8.57	8.86	0.036
	6.25	75.11	50.8	0.438	50	1.99	12.2	55.73	5960	6.37	8.42	8.70	0.038
	6.50	78.90	53.6	0.458	53	2.10	13.7	59.02	6150	6.26	8.27	8.55	0.041
	7.00	86.47	59.5	0.500	58	2.34	16.9	65.67	6570	6.05	8.00	8.27	0.045
	7.25	90.26	61.9	0.521	60	2.46	18.7	69.03	6730	5.95	7.87	8.14	0.047
	7.50	94.05	64.3	0.542	62	2.58	20.6	72.41	6900	5.89	7.75	8.01	0.050
19gage	4.50	55.85	32.6	0.292	34	1.38	5.1	38.67	4560	8.35	10.55	10.91	0.023
	5.00	64.68	37.5	0.333	38	1.63	6.9	45.61	5240	7.94	10.10	10.43	0.027
	5.25	69.10	40.0	0.354	41	1.75	7.9	49.19	5590	7.76	9.89	10.22	0.029
	5.50	73.52	42.6	0.375	43	1.88	9.0	52.83	5790	7.59	9.69	10.01	0.032
	6.00	82.35	48.0	0.417	48	2.15	11.6	60.25	6160	7.29	9.33	9.64	0.036
	6.25	86.77	50.8	0.438	50	2.28	13.0	64.02	6360	7.15	9.16	9.47	0.038
	6.50	91.19	53.6	0.458	53	2.42	14.5	67.83	6550	7.02	9.00	9.30	0.041
	7.00	100.03	59.5	0.500	58	2.69	17.9	75.53	6970	6.78	8.71	9.00	0.045
	7.25	104.44	61.9	0.521	60	2.83	19.8	79.42	7130	6.67	8.57	8.86	0.047
	7.50	108.86	64.3	0.542	62	2.97	21.8	83.33	7300	6.59	8.44	8.72	0.050
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
	5.00	72.04	37.5	0.333	38	1.81	7.3	50.72	5240	8.75	10.84	11.20	0.027
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	5.50	82.00	42.6	0.375	43	2.10	9.5	58.78	5950	8.35	10.41	10.76	0.032
	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
16 gage	4.50	62.08	32.6	0.292	34	1.88	6.0	42.99	4560	10.49	12.57	12.99	0.023
	5.00	72.04	37.5	0.333	38	2.22	8.0	50.72	5240	9.96	12.03	12.43	0.027
	5.25	77.02	40.0	0.354	41	2.40	9.2	54.72	5590	9.72	11.78	12.18	0.029
	5.50	82.00	42.6	0.375	43	2.58	10.5	58.78	5950	9.50	11.55	11.94	0.032
	6.00	91.95	48.0	0.417	48	2.94	13.4	67.07	6700	9.11	11.13	11.50	0.036
	6.25	96.93	50.8	0.438	50	3.13	15.0	71.29	7090	8.93	10.94	11.30	0.038
	6.50	101.91	53.6	0.458	53	3.32	16.8	75.55	7490	8.76	10.75	11.11	0.041
	7.00	111.87	59.5	0.500	58	3.71	20.6	84.17	8150	8.45	10.40	10.75	0.045
	7.25	116.85	61.9	0.521	60	3.90	22.8	88.52	8310	8.31	10.24	10.59	0.047
	7.50	121.83	64.3	0.542	62	4.10	25.1	92.91	8480	8.22	10.09	10.43	0.050

LRFD

Light Weight

	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 Loads, psf *												
22 gage	4.50	40.27	400	370	315	270	235	205	180	160	140	125	110	100	90
	5.00	46.44	400	400	365	315	270	240	210	185	165	145	130	115	105
	5.25	49.53	400	400	390	335	290	255	225	195	175	155	140	125	110
	5.50	52.61	400	400	400	355	310	270	235	210	185	165	150	130	120
	6.00	58.78	400	400	400	400	345	300	265	235	210	185	165	150	135
	6.25	61.87	400	400	400	400	365	320	280	245	220	195	175	155	140
	6.50	64.95	400	400	400	400	380	335	295	260	230	205	185	165	145
	7.00	71.12	400	400	400	400	400	365	320	285	250	225	200	180	160
20 gage	4.50	48.60	400	400	385	335	290	255	225	200	175	155	140	125	115
	5.00	56.18	400	400	400	385	335	295	260	230	205	180	165	145	130
	5.25	59.96	400	400	400	400	360	315	275	245	220	195	175	155	140
	5.50	63.75	400	400	400	400	380	335	295	260	230	205	185	165	150
	6.00	71.32	400	400	400	400	400	375	330	290	260	230	210	185	170
	6.25	75.11	400	400	400	400	400	395	345	310	275	245	220	200	180
	6.50	78.90	400	400	400	400	400	400	365	325	290	255	230	210	185
	7.00	86.47	400	400	400	400	400	400	400	355	315	280	255	230	205
19 gage	4.50	55.85	400	400	400	385	335	295	260	230	205	185	165	150	130
	5.00	64.68	400	400	400	400	390	345	300	270	240	215	190	175	155
	5.25	69.10	400	400	400	400	400	365	325	285	255	230	205	185	170
	5.50	73.52	400	400	400	400	400	390	345	305	270	245	220	200	180
	6.00	82.35	400	400	400	400	400	400	385	345	305	275	245	220	200
	6.25	86.77	400	400	400	400	400	400	400	360	320	290	260	235	210
	6.50	91.19	400	400	400	400	400	400	400	380	340	305	275	245	225
	7.00	100.03	400	400	400	400	400	400	400	400	370	335	300	270	245
18 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	5.00	72.04	400	400	400	400	400	385	340	300	270	240	220	195	180
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	5.50	82.00	400	400	400	400	400	400	390	345	305	275	250	225	205
	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
16 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	5.00	72.04	400	400	400	400	400	385	340	300	270	240	220	195	180
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	5.50	82.00	400	400	400	400	400	400	390	345	305	275	250	225	205
	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
22 gage	4.50	28.13	300	250	215	180	155	135	120	105	90	80	70	60	55
	5.00	33.12	355	295	250	215	185	160	140	125	110	95	85	75	65
	5.25	35.69	380	320	270	235	200	175	150	135	115	105	90	80	70
	5.50	38.29	400	345	290	250	215	185	165	145	125	110	100	85	75
	6.00	43.58	400	395	335	285	245	215	185	165	145	130	115	100	90
	6.25	46.26	400	400	355	305	260	230	200	175	155	135	120	105	95
	6.50	48.97	400	400	375	320	280	240	210	185	165	145	130	115	100
	7.00	54.44	400	400	400	360	310	270	235	205	185	160	145	125	115
20 gage	4.50	33.77	365	305	260	225	195	170	145	130	115	100	90	80	70
	5.00	39.80	400	360	310	265	230	200	175	155	135	120	105	95	85
	5.25	42.91	400	390	335	285	245	215	190	165	145	130	115	105	90
	5.50	46.05	400	400	360	305	265	230	205	180	160	140	125	110	100
	6.00	52.47	400	400	400	350	305	265	235	205	180	160	145	130	115
	6.25	55.73	400	400	400	375	325	280	250	220	195	170	155	135	120
	6.50	59.02	400	400	400	395	345	300	265	230	205	180	160	145	130
	7.00	65.67	400	400	400	400	385	335	295	260	230	205	180	160	145
19 gage	4.50	38.67	400	355	300	260	225	195	170	150	135	120	105	95	85
	5.00	45.61	400	400	360	310	265	235	205	180	160	140	125	115	100
	5.25	49.19	400	400	385	330	290	250	220	195	175	155	135	125	110
	5.50	52.83	400	400	400	355	310	270	240	210	185	165	150	130	120
	6.00	60.25	400	400	400	400	355	310	270	240	215	190	170	150	135
	6.25	64.02	400	400	400	400	375	330	290	255	225	205	180	160	145
	6.50	67.83	400	400	400	400	400	350	310	270	240	215	190	175	155
	7.00	75.53	400	400	400	400	400	390	345	305	270	240	215	195	175
18 gage	4.50	42.99	400	395	340	290	255	220	195	170	150	135	120	110	95
	5.00	50.72	400	400	400	345	300	260	230	205	180	160	145	130	115
	5.25	54.72	400	400	400	375	325	285	250	220	195	175	155	140	125
	5.50	58.78	400	400	400	400	350	305	270	235	210	190	170	150	135
	6.00	67.07	400	400	400	400	400	350	305	270	240	215	195	175	155
	6.25	71.29	400	400	400	400	400	370	325	290	255	230	205	185	165
	6.50	75.55	400	400	400	400	400	395	345	305	275	245	220	195	175
	7.00	84.17	400	400	400	400	400	400	390	345	305	275	245	220	200
16 gage	4.50	42.99	400	395	340	290	255	220	195	170	150	135	120	110	95
	5.00	50.72	400	400	400	345	300	260	230	205	180	160	145	130	115
	5.25	54.72	400	400	400	375	325	285	250	220	195	175	155	140	125
	5.50	58.78	400	400	400	400	350	305	270	235	210	190	170	150	135
	6.00	67.07	400	400	400	400	400	350	305	270	240	215	195	175	155
	6.25	71.29	400	400	400	400	400	370	325	290	255	230	205	185	165
	6.50	75.55	400	400	400	400	400	395	345	305	275	245	220	195	175
	7.00	84.17	400	400	400	400	400	400	390	345	305	275	245	220	200

1 STUD/FT.

NO STUDS

* The **Uniform Live Loads** are based on the LRFD equation $\phi M_n = (1.6L + 1.2D)l^2/8$. Although there are other load combinations that may require investigation, this will control most of the time. The equation assumes there is no negative bending reinforcement over the beams and therefore each composite slab is a single span. Two sets of values are shown; ϕM_{nf} is used to calculate the uniform load when the full required number of studs is present; ϕM_{no} is used to calculate the load when no studs are present. A straight line interpolation can be done if the average number of studs is between zero and the required number needed to develop the "full" factored moment. The tabulated loads are checked for shear controlling (it seldom does), and also limited to a live load deflection of 1/360 of the span.

An upper limit of 400 psf has been applied to the tabulated loads. This has been done to guard against equating large concentrated to uniform loads. Concentrated loads may require special analysis and design to take care of serviceability requirements not covered by simply using a uniform load value. On the other hand, for any load combination the values provided by the composite properties can be used in the calculations.

Welded wire fabric in the required amount is assumed for the table values. If welded wire fabric is not present, deduct 10% from the listed loads.

Refer to the example problems for the use of the tables.

Appendix C – Two-Way Slab with Drop Panels Calculations

TWO-WAY SLAB WITH DROP PANELS

Design Drop Panel for punching shear:

@ Col C2:

12.5" slab: $w_u = 1.2\left[\left(\frac{12.5}{12}\right)(150) + 25\right] + 1.6(100) = 377.5 \text{ psf}$

9.5" slab: $w_u = 1.2\left[\left(\frac{9.5}{12}\right)(150) + 25\right] + 1.6(100) = 332.5 \text{ psf}$

$$V_u = (30')\left(\frac{36'}{2}\right)\left(\frac{377.5}{1000}\right) + (30')\left(\frac{30'}{2}\right)\left(\frac{332.5}{1000}\right) = \underline{353.5^k}$$

$$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d = \phi 4\sqrt{f'_c}\,(4(\overset{\text{sq. col dim}}{c} + d))(d) \geq V_u$$

$$353.5^k \leq 0.75(4)\sqrt{5000}\,(4(24)d + 4d^2)$$

$$\Rightarrow d \geq 11.7''$$

If #8 bars used:

$$h_{min} = 11.7'' + \underset{clr}{.75''} + 1'' + \frac{1}{2}'' = 13.95'' \Rightarrow \underline{\text{use } 14''}$$

Design Drop Panel for panel shear: Assume 10' × 10' drop panels as in garage

$$V_u = (30')\left(\frac{36'}{2}\right)\left(\frac{377.5}{1000}\right) = 203.9^k$$

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

$$203.9 \leq 0.75(2)\sqrt{5000}\,(120'')d$$

$$\Rightarrow d \geq 16.0''$$

If #8 bars used:

$$h_{min} = 16.0'' + .75'' + 1'' + \frac{1}{2}'' = 18.25'' \Rightarrow \underline{\text{use } 18.5''}$$

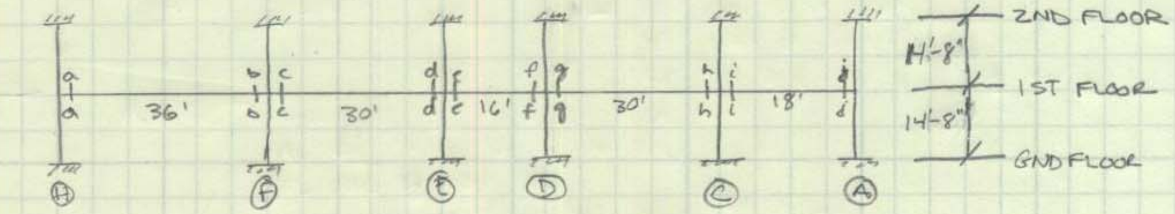
controls

Drop panel depth $= 18.5'' - 9.5'' = \boxed{9''}$ @ 9.5" slab

$= 18.5'' - 12.5'' = \boxed{6''}$ @ 12.5" slab

TWO-WAY SLAB WITH DROP ~~DROP~~ PANELS

- FRAME IN N-S DIRECTION: ALONG GRID LINE 2 ON 1ST FLOOR



FRAME 2 LOOKING EAST

Assume: 24 × 24 col's (CURRENTLY USED IN GARAGE)

$f'_c = 5000$ psi

$f_y = 60{,}000$ psi

FRAME WIDTH = 30'-0"

- DETERMINE SLAB THICKNESS:

ACI TABLE 9.5(c): Exterior panels with drop slabs and no edge beams

$$t_{min} = \frac{l_n}{33} = \frac{(36)(12) - 24}{33} = \frac{408''}{33} = 12.4''$$

ACI TABLE 9.5(c):

$$t_{min} = \frac{l_n}{36} = \frac{(30)(12) - 24}{36} = \frac{336''}{36} = 9.3''$$

→ TRY SLAB THICKNESS = 12.5" IN END SPAN & 9.5" IN INTERIOR SPANS

EQUIVALENT FRAME METHOD

$$12.5''\text{ slab}: \quad I_s = \frac{bh^3}{12} = \frac{(30(12))(12.5'')^3}{12} = 58{,}594 \text{ in}^4$$

$$9.5''\text{ slab}: \quad I_s = \frac{bh^3}{12} = \frac{(30(12))(9.5'')^3}{12} = 25{,}721 \text{ in}^4$$

$$24\times24\text{ col}: \quad I_c = \frac{bh^3}{12} = \frac{(24)(24)^3}{12} = 27{,}648 \text{ in}^4$$

- Stiffnesses:

$$12.5''\text{ slab}: \quad K_s = \frac{4E_{cs}I_s}{l_n - c_1/2} = \frac{4(58{,}594)E_c}{(36)(12) - \frac{24}{2}} = 558.0\,E_c = K_s^{a\text{-}a} = K_s^{b\text{-}b}$$

$$9.5''\text{ slab}: \; 30'\text{ span}: \quad K_s = \frac{4(25{,}721)E_c}{(30)(12) - \frac{24}{2}} = 295.6\,E_c = K_s^{c\text{-}c} = K_s^{d\text{-}d} = K_s^{g\text{-}g} = K_s^{h\text{-}h}$$

$$16'\text{ span}: \quad K_s = \frac{4(25{,}721)E_c}{(16)(12) - \frac{24}{2}} = 571.6\,E_c = K_s^{e\text{-}e} = K_s^{f\text{-}f}$$

$$18'\text{ span}: \quad K_s = \frac{4(25{,}721)E_c}{18(12) - \frac{24}{2}} = 504.3\,E_c = K_s^{i\text{-}i} = K_s^{j\text{-}j}$$

$$24\times24\text{ col}: \quad K_c = \frac{4E_{cc}I_c}{L - 2t} = \frac{4(27{,}648)E_c}{(14.67')(12) - 2(18.5)} = 795.6\,E_c$$

↳ slab thickness @ drop panel

cont →

FRAME IN N-S DIRECTION (CONT.)

24x24 col. torsional stiffness:

$$K_t = \frac{9E_{cs}C}{l_2(1-c_2/l_2)^3}$$

total height of drop panel

$$C = \left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3y}{3}\right) = \left(1 - 0.63\frac{18.5}{24}\right)\left(\frac{(18.5)^3(24)}{3}\right) = 26{,}055 \text{ in}^4$$

$$K_t = \frac{9(26{,}055)E_{cs}}{30(12)\left(1-\frac{24}{30(12)}\right)} = 697.9\ E_c$$

Equiv. Col. stiffness:

$$\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t} = \frac{1}{795.6E_c} + \frac{1}{697.9E_c}$$

$$K_{ec} = 371.8\ E_c$$

CALCULATE DISTRIBUTION FACTORS:

$$DF^{a-a} = \frac{558.0E_c}{558.0E_c + 371.8E_c} = 0.600$$

$$DF^{b-b} = \frac{558.0E_c}{(558.0+295.6+371.8)E_c} = 0.455$$

$$DF^{c-c} = \frac{295.6E_c}{(295.6+558.0+371.8)E_c} = 0.241$$

$$DF^{d-d} = DF^{g-g} = \frac{295.6E_c}{(295.6+571.6+371.8)E_c} = 0.238$$

$$DF^{e-e} = DF^{f-f} = \frac{571.6\ E_c}{(571.6+295.6+371.8)E_c} = 0.461$$

$$DF^{h-h} = \frac{295.6E_c}{(295.6+504.3+371.9)E_c} = 0.252$$

$$DF^{i-i} = \frac{504.3E_c}{(504.3+295.6+371.8)E_c} = 0.430$$

$$DF^{j-j} = \frac{504.3E_c}{(504.3+371.8)E_c} = 0.576$$

FIXED END MOMENTS

36' span: $w_u = 377.5\text{ psf}(30') = 11.325\text{ klf}$

$$M_u^- = \frac{w_uL^2}{12} = \frac{(11.325)(36')^2}{12} = 1223.1'^k$$

30' span: $w_u = 332.5(30') = 9.975\text{ klf}$

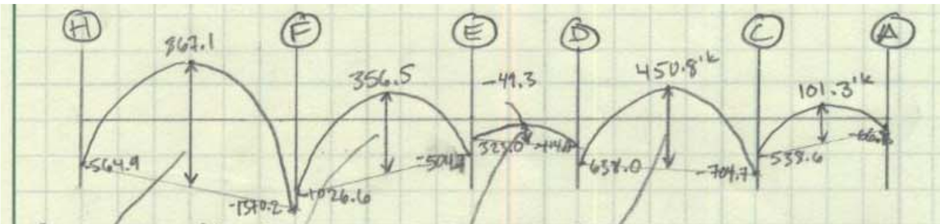
$$M_u^- = \frac{(9.975)(30)^2}{12} = 748.1'^k$$

16' span: $M_u^- = \frac{(9.975)(16)^2}{12} = 212.8'^k$

18' span: $M_u^- = \frac{(9.975)(18')^2}{12} = 269.3'^k$

MOMENT DISTRIBUTION:

Col. Line	Ⓐ	Ⓕ		Ⓔ		Ⓓ		Ⓒ		Ⓐ
DF	.600	.455	.241	.238	.461	.461	.238	.252	.430	.576
FEM	-1223.1	+1223.1	-748.1	+748.1	-212.8	+212.8	-748.1	+748.1	-269.3	+269.3
Distr.	+733.9									
Carry over		+366.9								
Distr.		-383.1	-202.9							
Carry over	-191.5			-101.4						
Distr.				-103.3	-200.0					
Carry over			-51.6			-100.0				
Distr.						+292.9	+151.2			
Carry over					+146.4			+75.6		
Distr.								-139.7	-238.4	
Carry over							-69.9			-119.2
Distr										-86.5
Carry over									-43.2	
D								+10.9	+18.6	
CO							+5.4			+9.3
D						+29.7	+15.4			
CO					+14.9			+7.7		
D				-38.4	-74.4					
CO			-19.2			-37.2				
D		+32.2	+17.1							
CO	+16.1			+8.5						
D	+105.2									
CO		+52.6								
D		-23.9	-12.7							
CO	-12.0			-6.3						
D				-0.5	-1.0					
CO			-0.3			-0.5				
D						+17.4	+9.0			
CO					+6.9			+4.5		
D								-3.1	-5.2	
CO							-1.5			-2.6
D										-3.9
CO									-1.9	
D								+0.5	+0.8	
CO							+0.2			+0.4
D						+0.6	+0.3			
CO					+0.3			+0.2		
D				-1.7	-3.3					
CO			-0.9			-1.7				
D		+0.5	+0.3							
CO	+0.3			+0.1						
D	+7.0									
CO		+3.5								
Distr		-1.6	-0.8							
Carryover	-0.8			-0.4						
FINAL MOMENTS	-564.9	+1370.2	-1026.6	+504.7	-323.0	+414.0	-638.0	+704.7	-538.6	+66.8



$\frac{wl^2}{8}$ $\frac{(11.325)(36)^2}{8} = 1834.7'^k$ $\frac{9.975(30)^2}{8} = 1122.2'^k$ $\frac{9.975(16)^2}{8} = 319.2'^k$ $= 1122.2'^k$ $\frac{(9.975)(18)^2}{8} = 404.0'^k$

DESIGN MAX M_u^- ($1370.2'^k$) AND MAX M_u^+ IN 9.5" SLAB ($450.8'^k$)

$M_u^- = 1370.2'^k$

$C.S. M_u^- = 0.75 M_u^- = .75(1370.2) = 1027.7'^k$

$M.S. M_u^- = .25 M_u^- = .25(1370.2) \quad 342.6'^k$

$M_u^+ = 450.8'^k$

$C.S. M_u^+ = 0.60 M_u^+ = .60(450.8) = 270.5'^k$

$M.S. M_u^+ = .40 M_u^+ = .40(450.8) = 180.3'^k$

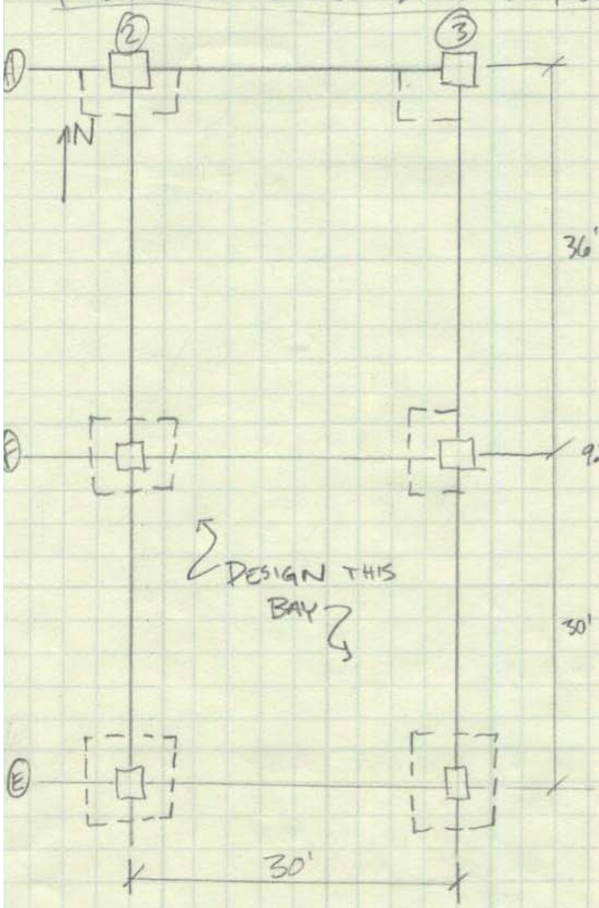
REINFORCEMENT DESIGN

Step	Description	M_u^-		M_u^+	
		C.S.	M.S.	C.S.	M.S
1	MOMENTS (M_u)	$1027.7'^k$	$342.6'^k$	$270.5'^k$	$180.3'^k$
2	EFFECTIVE STEEL DEPTH, d (3/4" cover) Assume #7 top, #5 bot	$18.5" - .75" - \frac{7/8}{2}$ 17.3125"	$9.5 - .75 - \frac{7/8}{2}$ 8.3125"	$9.5 - .75 - \frac{5/8}{2}$ 8.4375"	8.4375"
3	DESIGN MOMENTS $M_n = M_u/\phi$ ($\phi = .90$)	$1141.9'^k$	$380.7'^k$	$300.6'^k$	$200.3'^k$
4	$R = \frac{M_n}{bd^2}$ (b = 180")	254.0 psi	367.3 psi	281.5 psi	187.6 psi
5	Reinf. ratio, ρ (From Table A.5 of NDD)	.00438	.00641	.00487	.00320
6	Steel Area $A_s = \rho bd$	13.65 in²	9.59 in²	7.40 in²	4.86
7	Min Steel Area $A_{s,min} = .0018bt$	5.99 in²	3.08 in²	3.08 in²	3.08 in²
8	Number of bars $N = A_s/A_{s,bar}$	#7 23	#7 16	#6 17	#6 12
9	Min. num. of bars $N_{min} = b/2t$	5	10	10	10
10	Spacing of bars $s = b/N$	7.5"	11"	10.5"	15"
11	FINAL REINFORCEMENT	#7 @ 7.5" o.c.	#7 @ 11" o.c.	#6 @ 10.5" o.c.	#6 @ 15" o.c.

Ref.: "Design of Concrete Structures," Nilson, Darwin, Dolan. 2004.

Appendix D – Two-Way Post-Tensioned Flat Slab with Shear Caps Calculations

POST-TENSIONED 2-WAY FLAT PLATE WITH SHEAR CAPS



24x24 cols + Shear caps

$f'_c = 5000$ psi

$f_y = 60$ ksi

$f_{pu} = 270$ ksi $\frac{1}{2}''\phi$ tendons

$A_T = 0.153$ in^2

SPAN/DEPTH RATIO = 45

$\frac{36' \times 12''/'}{45} = 9.6'' \Rightarrow$ try $t = 9.75$

1" clear top & bottom for 2 hr. fire

$a = 9.75 - 2(1.25) = 7.25''$

LOADS

DL – 9.75" slab = 122 psf
ceil/mech = 15 psf
Part. = 10 psf
147 psf

LL = 100 psf

Banded Tendons in N-S dir.
Uniform " " E-W dir

Note: IDEAL B/C OTHER N-S BAYS ARE SHORTER

$W_{pre} = 0.9(122\text{ psf}) = 110$ psf

$W_{net} = 247 - 110 = 137$ psf

$A = (9.75'')(12'') = 117$ in^2

BOTH SPANS

$M_{pre} = \frac{(.110)(30')^2}{8} = 12.38'^k$

$F = \frac{M_{pre}}{a} = \frac{12.88(12)}{7.25} = 20.48^k$

$F/A = \frac{20.48^k/(1000)}{117\text{ in}^2} = 175.1$ psi

Bundles @ strips in N-S dir: $F = (20.48^k/\text{ft})(30') = \underline{614.5^k}$

$M_o = \frac{(.137)(30')^2}{8} = 15.41'^k$

$M^- = .65(15.41) = \underline{10.02}'^k$

$M^+ = .35(15.41) = \underline{5.39}'^k$

Check avg. Stresses

$$f = \frac{F}{A} \pm \frac{M_n}{S}$$

$$S = 2(9.75")^2 = 190.13\ in^3$$

Stress limits : Span ends : $f_t = 6\sqrt{f'_c} = 424.3$ psi

Midspan : $f_t = 3\sqrt{f'_c} = 212.1$ psi

$f_c = .45f'_c = 2250$ psi

$$M_u^- : \ f = -175.1_{psi} \pm \frac{10.02(12)}{190.13} = 457.3 > 424.3 \text{ No Good!}$$

Try $t_{slab} = 10.25"$

$DL_{slab} = 128$ psf $\qquad a = 10.25 - 2(1.25) = 7.75"$

$w_{pre} = 0.9(128) = 115$ psf

$w_{net} = 253 - 115 = 138$ psf

$A = 10.25(12) = 123\ in^2$

$$M_{pre} = \frac{(.115)(30)^2}{8} = 12.94'^k$$

$$F = \frac{M_{pre}}{a} = \frac{12.94(12)}{7.75} = 20.03^k$$

Bundles @ strips:

$F = (20.03\ {}^k/_{ft})(30') = 600.9^k$

$$F/A = \frac{20.03^k(1000)}{123\ in^2} = 162.9\ psi$$

$$M_o = \frac{.138(30)^2}{8} = 15.53'^k$$

$M^- = .65(15.53) = 10.09'^k$

$M^+ = .35(15.53) = 5.43'^k$

Check stresses:

$S = 2(10.25)^2 = 210.13\ in^3$

$$M^- : \ f = -162.9 \pm \frac{10.09(12)}{210.13} = \begin{matrix} +413.3\text{ psi} < 424.3\text{ psi ok} \checkmark \\ -739.1\text{ psi} < 2250\text{ psi ok} \checkmark \end{matrix}$$

$$M^+ : \ f = -162.9 \pm \frac{5.43(12)}{210.13} = \begin{matrix} +147.2\text{ psi} < 212.1\text{ psi ok} \checkmark \\ -473.0\text{ psi} < 2250\text{ psi ok} \checkmark \end{matrix}$$

$$N_t = \frac{600.9^k}{(270\text{ ksi})(0.153\ in^2)} = 14.5 \rightarrow \boxed{\text{Use 15 tendons each way}}$$

RIGID STEEL DESIGN

LOADS

DL - $^{slab}0.1(128) = 12.8$ psf

SuperImp. = 25 psf

37.8 psf

LL - 100 psf

$w_u = 1.2(37.8) + 1.6(100) = 205.4$ psf

$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{.2054(30)(30-2)^2}{8} = 603.9'^k$

$M_u^- = .65(603.9) = 392.5'^k$

$M_u^+ = .35(603.9) = 211.4'^k$

$C.S. M_u^- = .75(392.5) = 294.4'^k$

$M.S. M_u^- = .25(392.5) = 98.1'^k$

$C.S. M_u^+ = .60(211.4) = 126.8'^k$

$M.S. M_u^+ = .40(211.4) = 84.6'^k$

Assume #4 bars each way

Step	Description	M_u^- C.S.	M_u^- M.S	M_u^+ C.S.	M_u^+ M.S.
1	Moments, M_u	$294.4'^k$	$98.1'^k$	$126.8'^k$	$84.6'^k$
2	Depth, d (Use avg. d each way)	9	9	9	9
3	Design Moment $M_n = M_u/\phi$	$327.1'^k$	$109.0'^k$	$140.9'^k$	$94.0'^k$
4	Flex. Resist. Factor $R = M_n/bd^2$	269.2 psi	89.7 psi	104.4 psi	77.4 psi
5	Reinf. ratio, ρ From Table A.5	.00464	.00151	.00177	.00130
6	Steel area, As $A_s = \rho bd$	7.52 in²	2.45 in²	2.87 in²	2.11 in²
7	Min Steel Area $A_{s,min}$ $A_{s,min} = .0018bt$	3.32 in²	3.32 in²	3.32 in²	3.32 in²
8	Num of bars, N $N = A_s/A_{s\#5}$	25	11	11	11
9	Min num of bars $N_{min} = b/2t$	9	9	9	9
10	Spacing, s $s = b/N$	7"	16"	16"	16"
11	FINAL REINF.	#5 @ 7" o.c.	#5 @ 16"	#5 @ 16"	#5 @ 16"

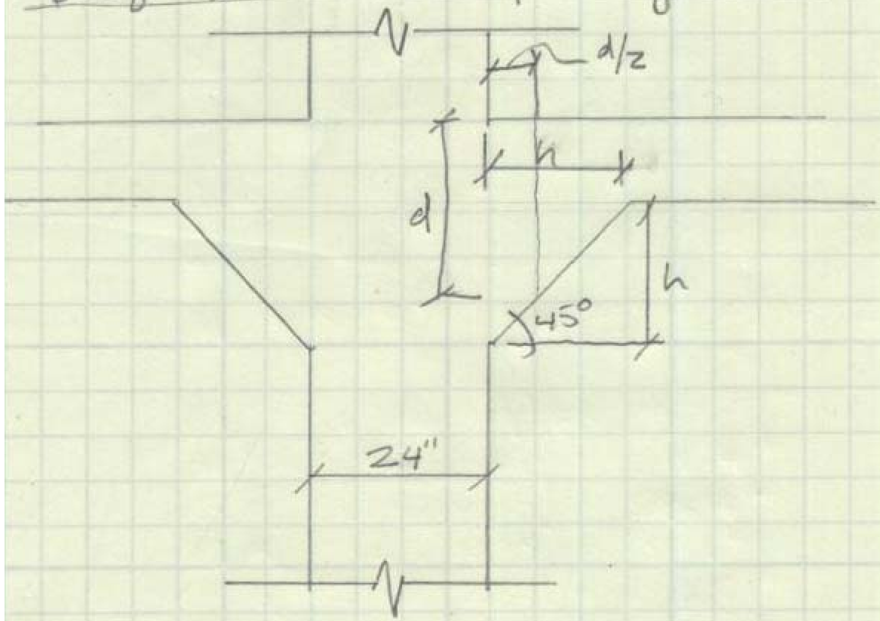
$d = 10.25 - .75 - 0.5$

$b = \frac{30'(12)}{2} = 180"$

Use #5 bars

$A_s(\#5) = .31$

Design Shear cap for punching shear



$$\text{Area} = (36')(30') = 1080\ ft^2$$

$$W_u = 1.2(153\ psf) + 1.6(100\ psf) = 343.6\ psf$$

$$V_u = (W_u)(\text{Area}) = 1080(.3436) = 371.1^k$$

@ Shear cap edge:

$$\phi V_n = \phi 4\sqrt{f'_c}\ b_o d = .75(4)\sqrt{5000}\ b_o (8.875) \geq V_u = 371.1^k$$

$$b_o \geq 197.1"$$

$$b_o = 4(2h+24)$$

$$h \geq 12.6"$$

@ d/2 from col face:

$$\phi V_n = .75(4)\sqrt{5000}\left(4(24+8.875)\right) d \geq 371.1^k$$

$$d \geq 9.98"$$

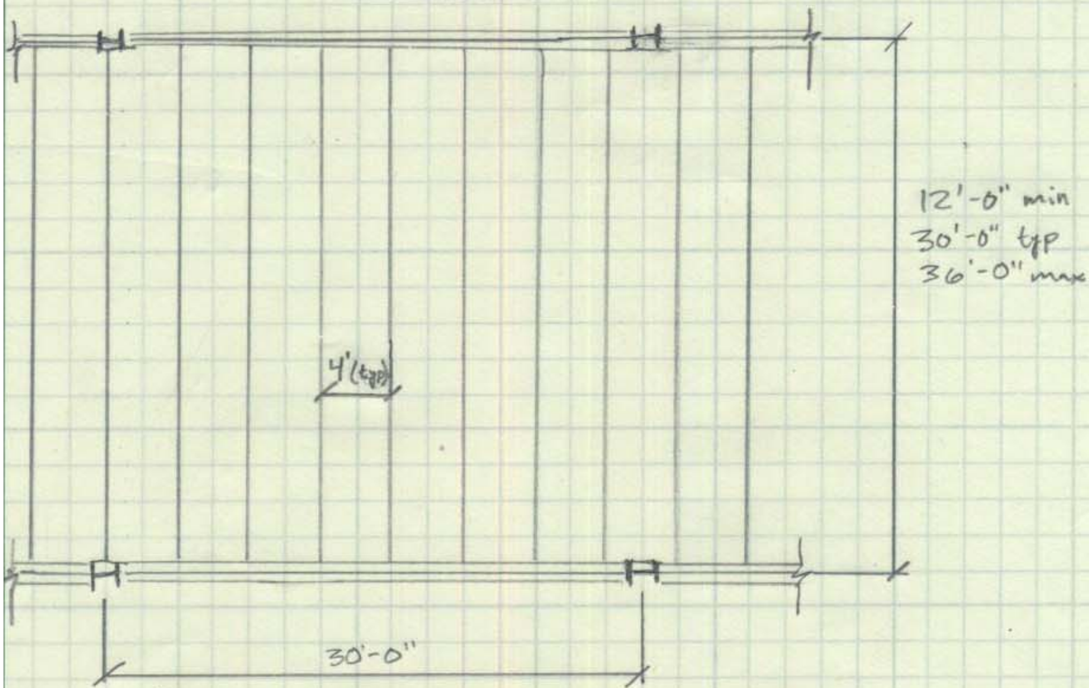
$$\Rightarrow h \geq 9.98" + 8.875" = 18.85"$$

use $h = 24"$

$\therefore$ $\boxed{\text{shear cap is } 72" \times 72" \text{ @ } 45°}$

Appendix E – Precast Hollowcore Plank on Steel Frame Calculations

HOLLOCORE ON STEEL GIRDERS



DESIGN HOLLOWCORE PLANK:

FIRST FLOOR TYP BAY:

W_{LL} = 100 PSF

W_{DL} = 15 PSF CEILING & MECH ALLOWANCE

=+ 10 PSF PARTITION ALLOWANCE

W_{TOTAL} = 125 PSF TOTAL SUPERIMPOSED SERVICE LOAD

FROM LOAD TABLES FROM NITTERHOUSE CONCRETE PRODUCTS, INC.'S PUBLISHED DATA, WHICH ASSUMES ALL ALLOWABLE SERVICE LOAD TO BE LIVE LOAD, WE CAN SIZE THE PLANK.

@ 30' SPAN, USE 10" HOLLOWCORE WITH 2" TOPPING WITH (7) ½" Ø 270k LO-RELAXATION STRANDS

W_{allow} = 153 PSF > 125 PSF OK ✓

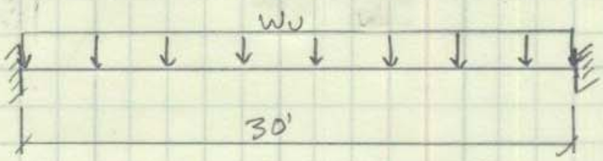
@ 36' SPAN, USE 10" HOLLOWCORE WITH 2" TOPPING WITH (7) 0.6" Ø 270k LO-RELAXATION STRANDS

W_{allow} = 140 PSF > 125 PSF OK ✓

DESIGN STEEL GIRDERS

GIRDER WITH MOST LOAD: F2 - F3 IN DRAWINGS
↳ 36' BAY TO NORTH
30' BAY TO SOUTH
30' SPAN ℄ OF COL TO ℄ OF COL

ASSUME: - BEAM TO BE FULLY BRACE ALONG ITS LENGTH BY HOLLOWCORE PLANK GROUT
- FIXED CONNECTIONS (IN MOMENT FRAME)
- W12 col's



$$w_{LL} = 100\text{ PSF}\left[\left(\frac{36'}{2}\right) + \left(\frac{30'}{2}\right)\right] = 3.30\text{ klf}$$

$$w_{DL} = \underbrace{(\underset{\text{PLANK}}{86\text{PSF}} + \underset{\text{2" TOPPING}}{25\text{PSF}} + \underset{\text{CEIL/MECH}}{15\text{PSF}} + \underset{\text{PARTITION}}{10\text{PSF}})}_{136\text{ PSF}} \cdot \left[\left(\frac{36'}{2}\right) + \left(\frac{30'}{2}\right)\right] = 4.90\text{ klf}$$

$$w_u = 1.2(4.90 + \underset{\text{SELF-WT GUESS}}{0.100}) + 1.6(3.30) = \underline{\underline{11.28\text{ klf}}}$$

$$M_u^- = \frac{w_u L^2}{12} = \frac{(11.28)(30')}{12} = 846'^k$$

L_u (fully braced) = 0', so M_u^+ WILL NOT GOVERN

<u>TRY W27 × 84</u> $(I = 2850\text{ in}^4,\ \phi M_n = 915'^k)$

$$\Delta_L \leq L/360 = \frac{30(12)}{360} = 1.0"$$

$$\Delta_L = \frac{w_L L^4}{384EI} = \frac{(3.30\text{ klf})(30')^4}{384(29{,}000\text{ksi})(2850\text{ in}^4)}(1728) = 0.146" < 1.0" \text{ ok✓}$$

$$\Delta_T \leq L/240 = \frac{30(12)}{240} = 1.5"$$

$$\Delta_T = \frac{w_T L^4}{384EI} = \frac{(8.284)(30')^4}{384(29{,}000)(2850)} = 0.365" < 1.5" \text{ ok✓}$$

$\boxed{\text{USE W27} \times 84}$

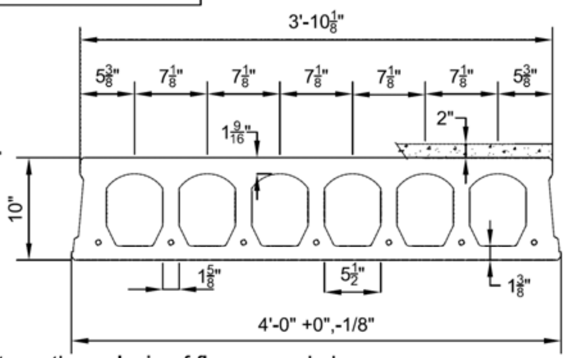
NOTE: WAS ALSO THE SAME GIRDER BEING USED IN CURRENT COMPOSITE SYSTEM

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2} Precast S_{bc} = 824 in.^{3}
I_c = 5102 in.^{4} Topping S_{tc} = 1242 in.^{3}
Y_{bc} = 6.19 in. Precast S_{tc} = 1340 in.^{3}
Y_{tc} = 3.81 in. Wt.= 272 PLF
Wt.= 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI or 4000 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   7-1/2"Ø, 270K = 192.2 k-ft
   7-0.6"Ø, 270K = 256.4 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 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 2003 & ACI 318-02 (1.2 D + 1.6 L)	
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
7 - 1/2"ø	LOAD (PSF)	234	210	189	170	153	137	123	110	98	87	77	68	60	52					
7 - 0.6"ø	LOAD (PSF)			256	244	233	222	202	185	168	154	140	128	116	106	96	87	78	70	63

NITTERHOUSE
CONCRETE N PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
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.

05/14/07

10F2.0T