University Health Building

Located in the Mid-Atlantic Region

Technical Report II

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Executive Summary

The objective of the following Technical Report was to design and analyze three prospective floor systems as well as analyze the existing floor system of the University Health Building. After the analysis, the findings were compared in order to determine which floor system would be the best fit for the building considering factors of slab depth, system depth, system weight, deflection, system cost, fire protection, formwork, lateral system alterations, foundation alterations, feasibility, and advantages/disadvantages of each system.

To start, the main elements of the structural system were analyzed to determine how the load gets transferred throughout the building. This was completed by looking at the foundation, slabs, lateral, and roofing systems used in the project. The report includes details about the materials used as well as reference to codes, standards, and loads that were used for the design

Prestressed hollow core planks on steel girders, non-composite steel deck on steel beams and girders, and a one-way reinforced concrete slab were analyzed and compared to the existing two-way post tensioned slab. It was determined that the existing floor system is the most suitable for this building. This was based on the findings that the other systems were much too deep for the story height of only 12 ft.

Building Introduction

This new 9 story 161000 square foot building will be a great addition to the university's campus. It is being built to house leaders in the public and private health policy sectors. The building is a mesh between office space and student classrooms nestled around a central sky lit atrium. The architect hopes that this mesh will help to bridge the gap between faculty and students. The classroom area appears as if the classrooms are floating on clouds in a glass enclosure. The concrete structure is enclosed by a curtain wall which is the building's main architectural feature. The curved saw blade-like curtain wall system encompasses one quarter of the building's façade and gives the building an edgy appearance.

The building façade is constructed of many different types of materials, ranging from stone to metal. The building's first floor is covered by a stone veneer giving the building a very stereotomic base. The rest of the building is clad in a mixture of glazing, metal panels, and terracotta. The West and Southeast facades are relatively similar to one another. They both have a pattern of terracotta, metal paneling, and glazing above the first floor with the majority material being covered with the terracotta. The south and north facades are also very similar except the south facade has an aluminum sunscreen system in place. Otherwise, these ends of the building are almost fully glazed. Lastly, the curved curtain wall with reveals located on the northeast side of the building is composed of mainly glazing with the reveals clad in terracotta. Some of these features can be seen in Figure 1.

Figure 1: Photo of Northwest corner of building showing façade materials. Rendering by Payette Architecture.

The majority of the roof is a garden roofing system. The system used on this project is the Sika Sarnafil Extensive Greenroof system. It uses 3in. of growing medium as well as pavers for maintenance. The rooftop penthouse will be coveredd with a fully adhered white, 60mm thick PVC membrane with a layer of 8in. thick tapered polyisocyanurate insulation boards underneath.

Lastly, the University Health Building is registered as a LEED – NC 2.2 Silver building. This rating includes many different LEED credits involving the façade, roof, and internal systems. The main points came from the heat island effect roof system, the building's proximity to transit, and use of efficient plumbing and lighting fixtures.

Structural Overview

Foundation

The foundation of University Health Building (UHB) consists of spread footings at the base of each column. On the western block of the building, the engineers utilized a grade beam and spread footing combination to help with the bracing of the basement wall shown in the Figure 2 below. This was not used on the east side of the building due to the absence of any underground levels. The spread footings are to be set on soils suitable to hold about 5000psf according to the Geotechnical report.

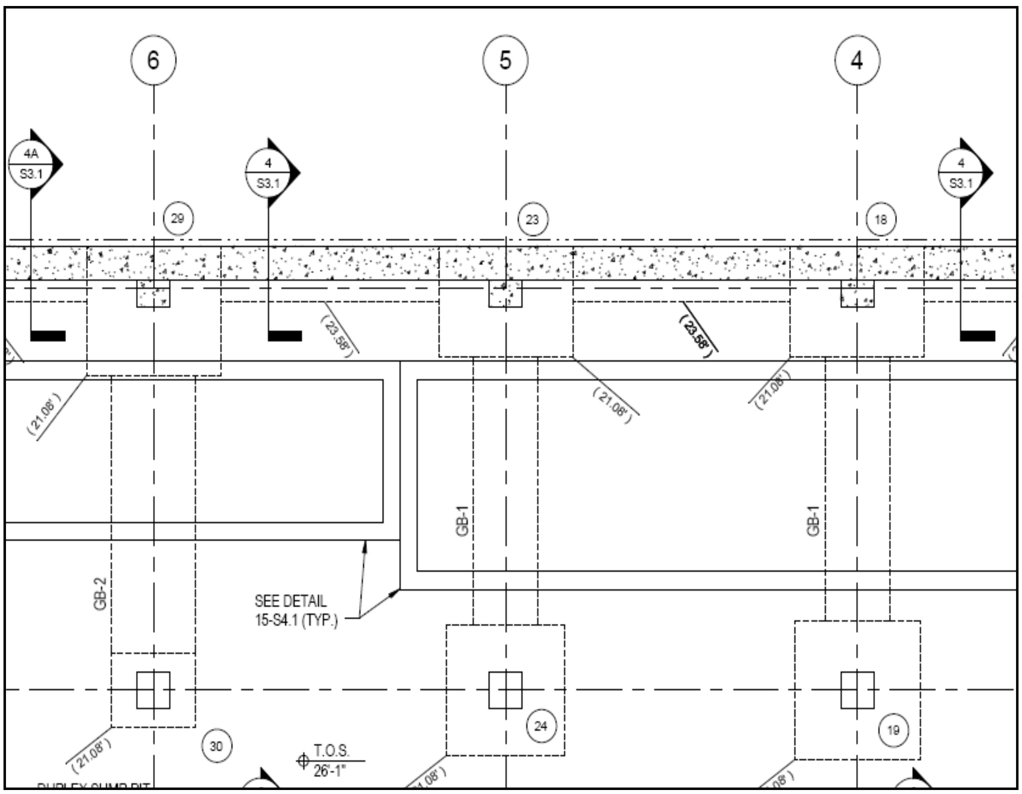


Figure 2: Grade beam and spread footing combination, taken from drawing S1.1

Floor Slabs

The basement level and ground level floor slabs are similar in the fact that they both have a relatively thick floor slab and drop panels comprised of high strength concrete in order to minimize the amount of beams necessary to handle the 21 ft. spans. Once you leave the ground floor, you will find that the slabs change from what was mentioned above to a post tensioned slab system. Also, above the ground floor on the east half of the building, the slabs have large continuous drop panels running between select columns. This type of system extends all the way to the penthouse slab with variations in slab and drop panel thicknesses.

Lateral System

Since the walls of the UHB building are non-load bearing, the lateral loads, due to wind and seismic, must be resolved by the columns and slabs of the building. The dominant lateral system of the UHB is concrete moment frames consisting of the post-tensioned slab and interior/exterior column system. In the case of wind, the load is transferred from the cladding to the exterior columns and slab edge. Then, it is distributed to the interior columns through the slab, and finally, its transferred to the foundation through the columns. The lateral system also utilizes one shear wall located beside the elevator shaft. The shear wall is called out in Figure 2.1.

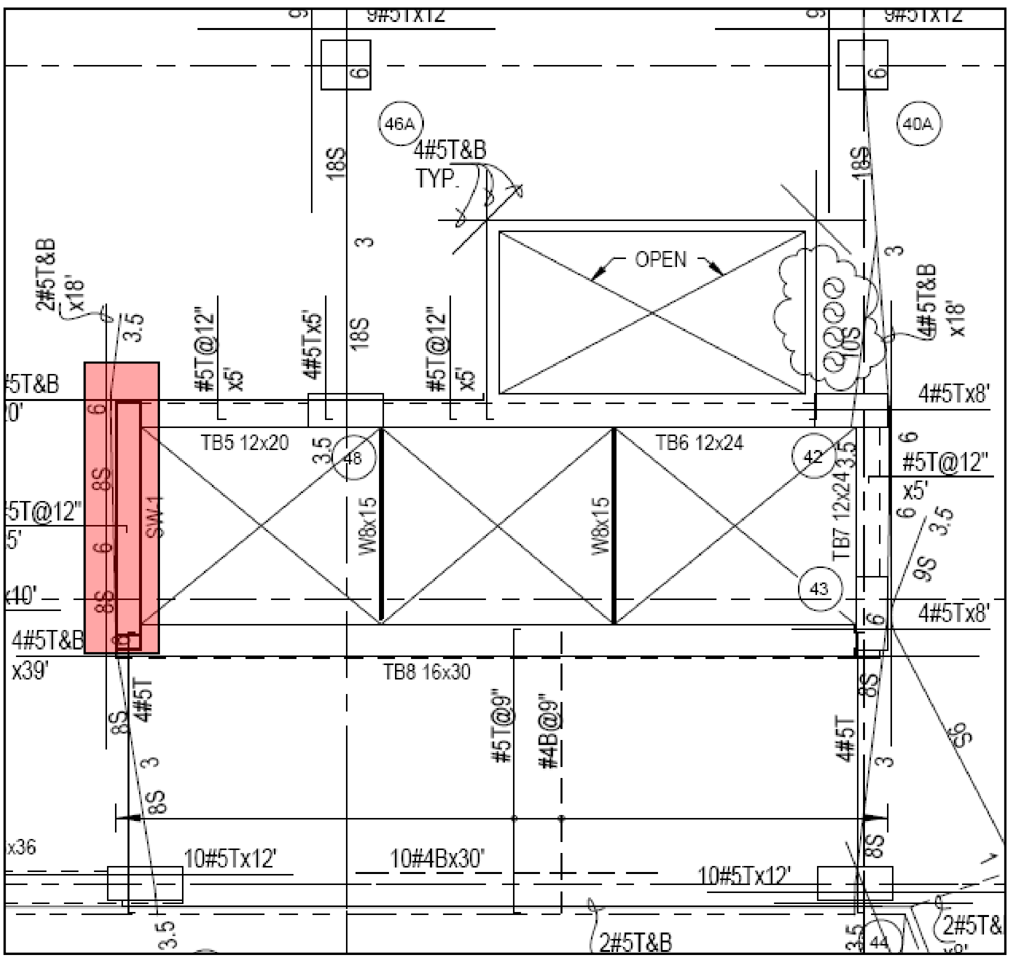


Figure 2.1: Location of shear wall, taken from S1.8

Roof System

The roof system is comprised of two different levels. The first being the lower roof where the green roof is located, and the second is the upper roof that covers the penthouse. The lower roof is a 12-14in. thick post tensioned slab and topped with a green roof system where exposed to the outside. The upper roof is supported by an 8in. post tensioned slab. Also, a portion of the penthouse roof is spanned with steel beams with a glazing system overtop to serve are the skylight for the central stair tower. Figure 3 below shows a partial roof plan showing the integration of the post tensioned concrete slab and central skylight area.

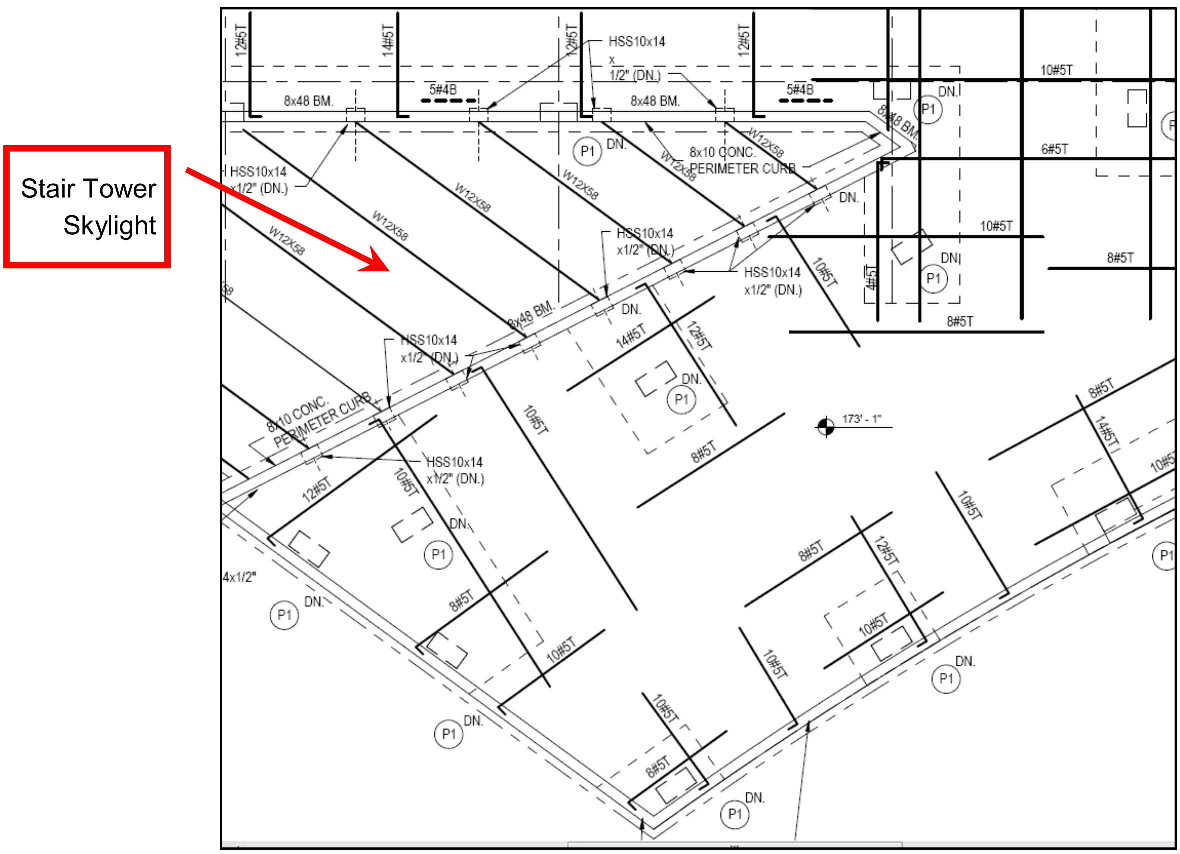


Figure 3: Integrations of both steel and concrete systems on roof, taken from drawing S1.11

Codes & References

Design Codes

Building Code

International Building Code - IBC 2006 system

Reference Codes

American Society of Civil Engineers - ASCE 7-05

American Concrete Institute Building Code - ACI 318-05, ACI 530-05, ACI 530.1-05

American Institute of Steel Construction - AISC 360-05

Thesis Codes

Building Code

International Building Code - IBC 2009

Reference Codes

American Society of Civil Engineers - ASCE 7-05

American Concrete Institute Building Code - ACI 318-08

American Institute of Steel Construction - AISC 14th Edition

Material Strengths

General material strengths were found on S4.9 and are displayed in Figure 5. The general types and strengths can be overridden per special callouts on the floor plans. On many floors, slab strengths are a combination of 6000psi and 8000psi. See Figure 6 and 7 for good examples of the drawings superseding the general strengths. The figures show variations in concrete strength as the building elevation increases and slab thickness increases.

Item	Type	Strength
Steel Beams	ASTM-A992	Fy= 50
Post tensioning Tendons	ASTM A-416	Fu= 270
Reinforcement	ASTM-A615	Fy= 60
Masonry	ASTM C-90	f'c=1.5
Grade Beams	NW Conc.	f'c= 4
Column Footings	NW Conc.	f'c= 5
Slab on grade	NW Conc.	f'c= 5
Floor slabs	NW Conc.	f'c= 6
Columns	NW Conc.	See Fig.

Figure 5: Material strength table

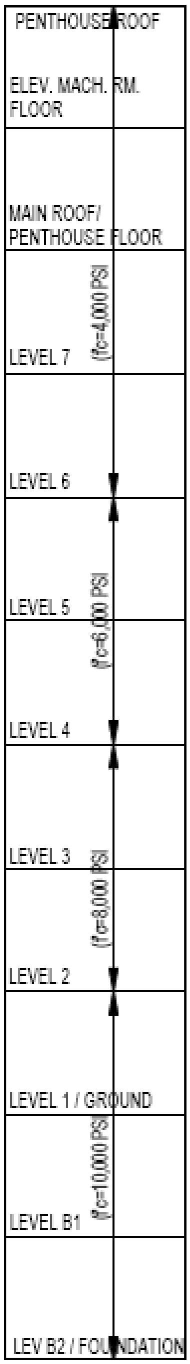


Figure 6: Variations in column concrete strengths per level

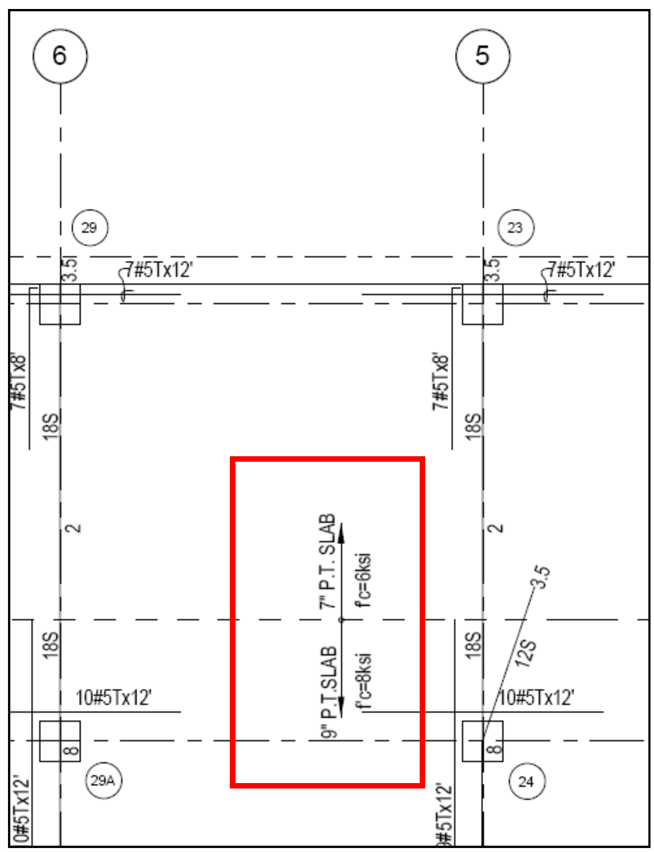


Figure 7: Variations in slab concrete strength

Design Loads

This thesis project will be conducted using the Load and Resistance Factor Design (LRFD) method as it is quickly becoming the industry standard. Thesis loads were determined using ASCE 7-05 unless a category were not listed specifically. Then, design loads were used in its place. At the time this report was written, it was undetermined what the design engineer used for dead loads. See Figure 4 below to see the comparison between design and thesis loads.

	(psf)	
Live Loads	Design	Thesis
Roof	30	20
Mechanical Penthouse	150	150
Green Roof	35	35
Stairways	100	100
Corridors	100	100
Loading Dock	450	450
Light Storage	125	125
Retail	100	100
Office	80	80
Partitions	20	20

	(psf)	
Snow	Design	Thesis
Ground Snow	30	30
Flat Roof	21	21
Snow Exposure Factor	0.7	0.7
Snow Importance Factor	1	1

	(psf)	
Dead Load	Design	Thesis
MEP Allowance	-	5
Roof material	-	5
Green Roof	-	50
	(pcf)	
NW Concrete	150	150

Figure 4: Summary of Live Snow and Dead loads

Alternate Floor Systems

Note: Floor systems were designed considering gravity loads only. Lateral loading effects were not in the scope of this project and were neglected in all calculations.

This technical report will explore three alternative floor systems and then compare them to each other as well as the existing floor system. The factors for comparison will be slab depth, system depth, system weight, deflection, system cost, fire protection, formwork, lateral system alterations, foundation alterations, and feasibility. The typical bay chosen for this comparison is shown and highlighted below in Figure 5. The bay is 21x21.5 ft. with 24x24 in. columns at the corners.

All systems will be described and their advantages and disadvantages will be discussed. Following this discussion a figure to compile all of the results, and a conclusion will be drawn.

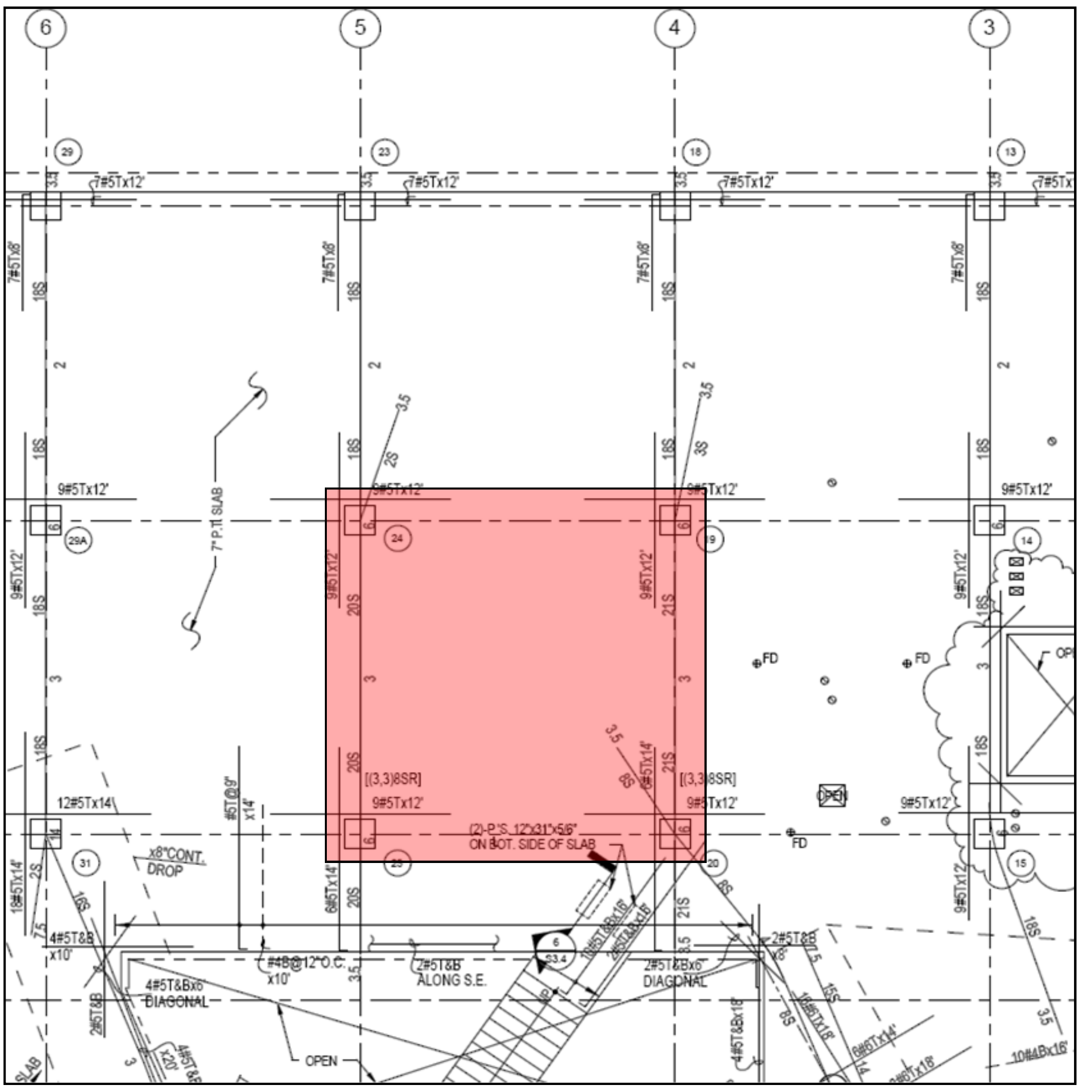


Figure 5: Showing the bay of analysis. Bay is 21'x21.5' with 24"x24" columns.

Two-way Post Tensioned Slab

The designers of the existing structure chose to use an unbounded two-way post tensioned slab system. The particular bay of interest in this report was constructed with 6 ksi concrete. The use of higher strength concrete in post tension systems is very common. The compressive force generated by the post tensioned tendons allows designers to take full advantage of the higher strength concrete because they are able to work with the entire concrete cross-section. The system is comprised of 1/2” tendons with parabolic profiles draped at span length divided by 10. The slabs two-way action is created by having tendons in both the transverse and lateral directions as well as other standard reinforcement. Figure 6 shows a section cut of the typical slab. Calculations for this system can be found in Appendix A.

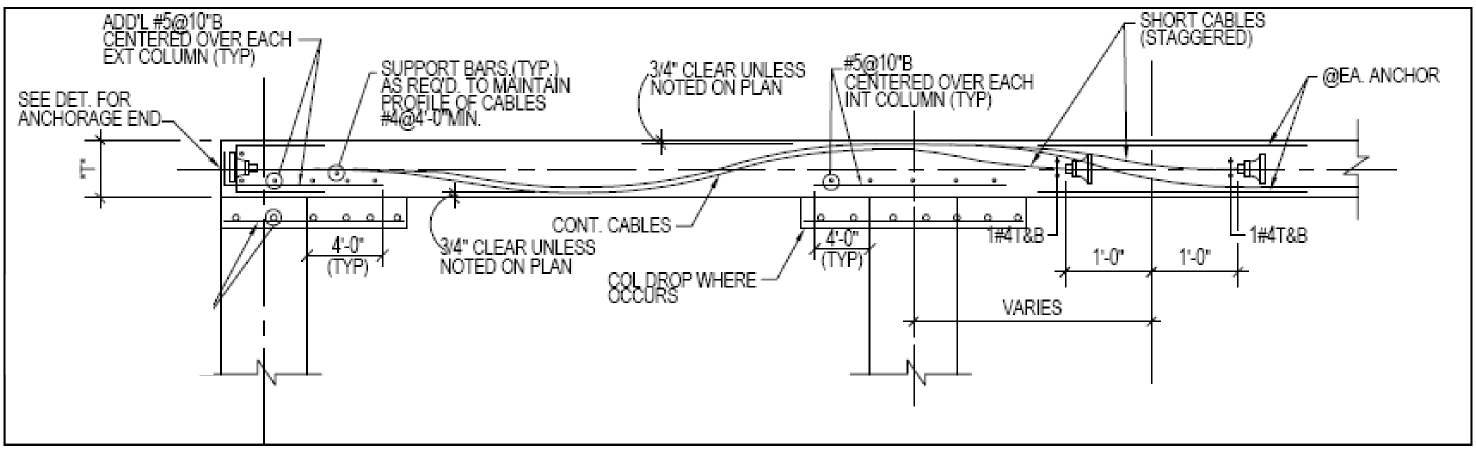


Figure 6: Typical section of the UHB’s post tensioned slab.

Advantages

One of the greatest advantages to a post tensioned slab is its ability to span large distances with minimal slab thickness. This helps to keep the story to story heights smaller allowing for higher ceiling heights. Post tensioned concrete is also capable of having very small defections due to its stiffness and the ability to easily induce calculated camber into the slabs. Crack control is very good with post tensioned concrete. Which is both esthetically pleasing and good for protection of internal steel members from corrosion.

Disadvantages

One disadvantage of post tensioned concrete is that the placement of tendon profiles has to be accurate. Not only for structural stability but also so that when future renovations are made to the building, the owner and occupants know where the tendons are located. If they were to rupture one of the tendons while cutting or drilling, the damage would be costly to reverse. Another disadvantage is the cost associated with the time taken to jack and place the tendons as well as the time for building and removing formwork.

Prestressed Hollow Core planks on Steel Beams

Hollow core planks are a very good way to manage long spans with relatively heavy loading. The cross section of a hollow core plank can be seen in Figure 7. The removal of unnecessary material helps to lighten the planks helping them to span greater distances. In this report, 8” x 4’ Nitterhouse Concrete planks with a 2 in. topping were chosen for this alternative design. The addition of the 2 in. topping and six 1/2 in. diameter tendons made this plank more than capable of handling the building loads and span of 21 ft. The planks are rested on top of W21x55 girders that run perpendicular to the planks and transfer the load to the columns. Calculations for this slab can be found in Appendix B

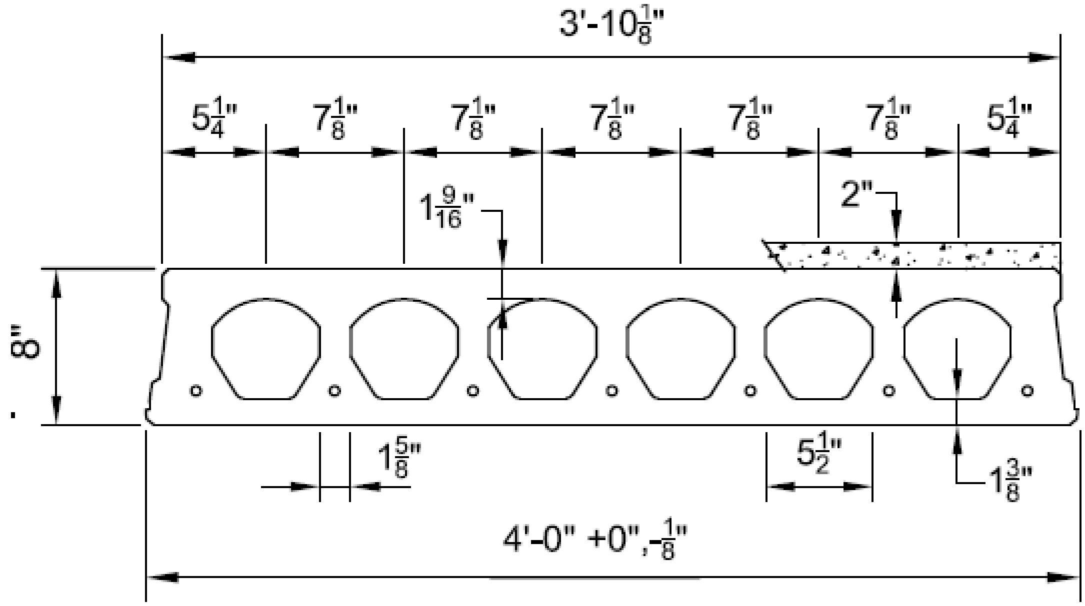


Figure 7: A dimensioned cross section of the hollow core plank used in this report. Photo by Nitterhouse.com

Advantages

An advantage to hollow core planks is their ease of constructability. They do not require any form-work, shoring, or formwork removal. They are transported precast ready to be placed. Also, because they are precast they can be placed under any weather condition. Another advantage is the planks built in camber due to the prestressing helps to limit deflections.

Disadvantages

A disadvantage to hollow core planks is the cost to transport the planks from the precast plant to the jobsite. If the plant is not in close proximity this cost can quickly escalate. The biggest drawback is that they come in widths of 4 ft. The bay size of 21x21 ft. is not a multiple of four. The column layout of the building would have to change in order to accommodate the planks.

Non-composite Steel Deck with Steel Beams and Girders

This floor system was constructed using Volcraft 3C22 decking with a 6 in. concrete topping. This is a non-composite deck, which means it is entirely capable if supporting its own weight as well as the additional loading of the building with or without the concrete slab. A photo of the system can be seen in Figure 8. The concrete slab is placed to give a smooth even finish to the floor as well as help with noise and vibration control. The system is supported by W12x22 beams and the beams are supported by W16x40 girders which transfer the load to the columns. Calculation for this floor system can be found in Appendix C.

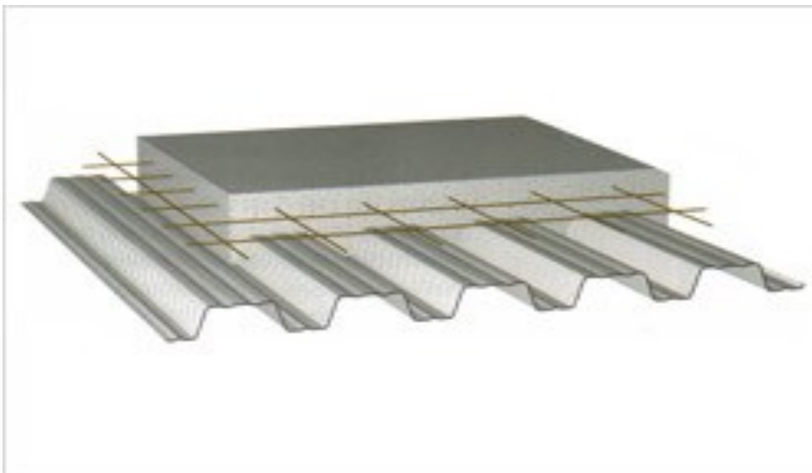


Figure 8: Photo of steel deck supporting a concrete slab. Courtesy of prefabjbn.com

Advantages

Non-composite steel decking requires very little formwork for the casting for the concrete. This saves both time and money. The ability of non-composite deck to support its own weight and the weight of the concrete eliminates the need for shoring for the deck system until the concrete reaches required strength like is sometimes required for its brother the composite deck. This system is also relatively lightweight. This system is also easy to design and construct making it a favorite for both designers and contractors.

Disadvantages

The steel beams and girders will require the application of fireproofing. This system will see an increase in labor and cost due to the steel beams and decking requiring welding. Also, if the building were to keep its moment resisting frames for a lateral system, the amount of necessary welding would greatly increase.

One-way Reinforced Concrete Slab

This system is a cast-in-place slab, beam, and girder one-way slab. The supports and slab reinforcement work together to induce load travel in one direction. This helps to reduce the complexity of the reinforcement system. One-way slabs are sufficient for vibration control, long spans, and large loads. Figure 9 below shows a three dimensional view of a typical one-way slab and beam system. Calculations for this system can be found in Appendix D.

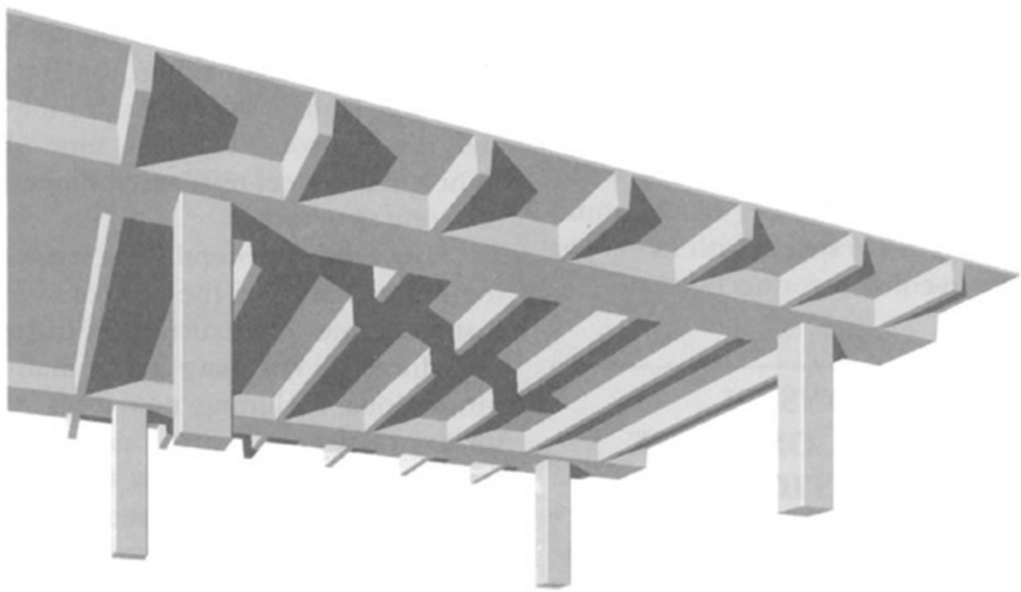


Figure 9: Typical one-way slab design. Courtesy of studyblue.com.

Advantages

Utilization of a thin slab with intermittent beams make this system one of the thinnest. This is great for story to story clearances and placement of MEP equipment. Also, the monolithic columns and beams are a great lateral forces resisting system. This system will also disperse vibrations in the building.

Disadvantages

Concrete requires time to cure which can be troublesome for fast track projects. Also, finishing the concrete can be very labor intensive, which can increse the cost of the system. Concrete placement also requires constant supervision while it is being placed to ensure that the quality that is expected is received. Many things can happen during the vibrating, pouring, and finishing of the concrete that can cause the need for costly repairs in the future.

Comparison of Floor Systems

The most current R.S. Means 2012 was used for the cost comparison of the four systems. This building is currently under construction, so this will give the most accurate pricing. See Appendix E for the values and assemblies used for each system as well as reasoning and assumptions made. Figure 10 breaks down the comparison into various categories that were used. The values were determined after the hand calculations were completed for each system.

Floor System Comparison				
Category	Two-way Post Tensioned Slab	Prestressed Hollow Core Planks	Non-Composite Steel Deck	One-way Concrete Slab
Slab Depth	7"	8"	6"	5"
System Depth	7"	29"	22"	19"
System Weight	87.5 psf	91.4 psf	61.1 psf	65.3 psf
Beam Deflection	-	0.510"	0.944"	0.765"
System Cost / SF	$19.51	$13.23	$6.22	$17.51
Fire Protection	Inherent	Spray-on	Spay-on	Inherent
Formwork	Yes	No	No	Yes
Lateral System Alterations	No	Yes	Yes	No
Foundation Alterations	No	Yes	No	No
Feasibilty	Yes	No	No	Yes

Figure 10: Beak down of comparison factors

- The foundation system will need altered only if the new floor system exceeds the weight of the existing.
- Lateral alterations come into play when you are changing from a primarily concrete moment frame structure to something else that does not have moment resisting connections.
- The steel structures will need a different lateral force resisting system, such as, shear walls.
- The one-way system is feasible because it will still allow the building to have a 9 ft. ceiling if the MEP is coordinated into the small space.
- Inherent fire protection systems are able to provide the 2-hr. fire rating without any covering. The systems that are not inherent will need some type of fire proofing, such as, spray-on fire protection.

Conclusion

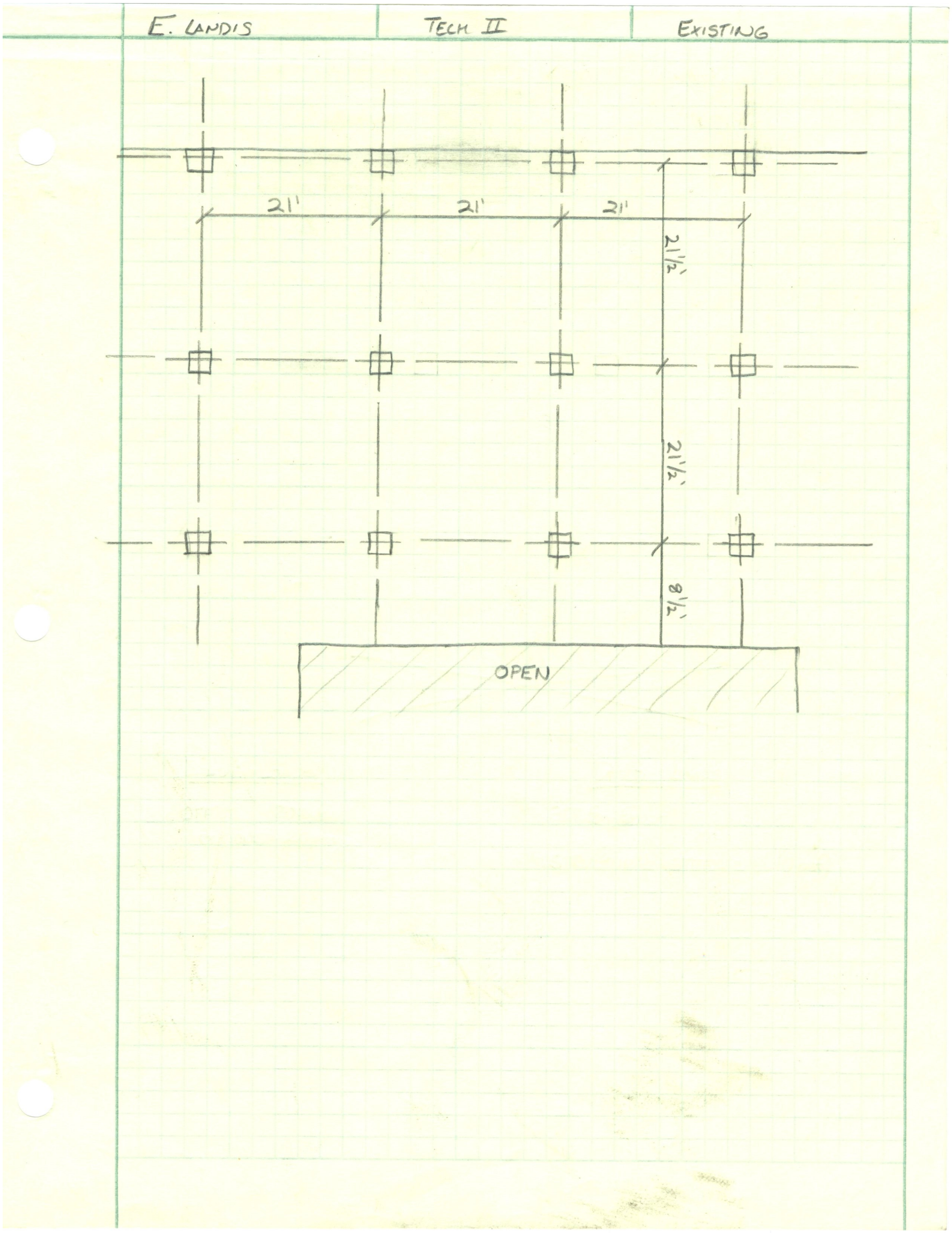
This technical report was prepared in order to investigate three alternative flooring systems and compare them to the existing two-way post tension slab system. The chosen floor systems were prestressed hollow core planks on steel girders, non-composite steel deck on steel beams and girders and finally one-way reinforced concrete slab.

These systems were then compared in Figure 10. It was found that the least expensive and lightest system was the non-composite steel deck on steel beams and girders. The biggest problem with this system is the system depth. It would require a large alteration to the lateral system. This would most likely be solved with shear walls, which will take away from the open floor plan of the building.

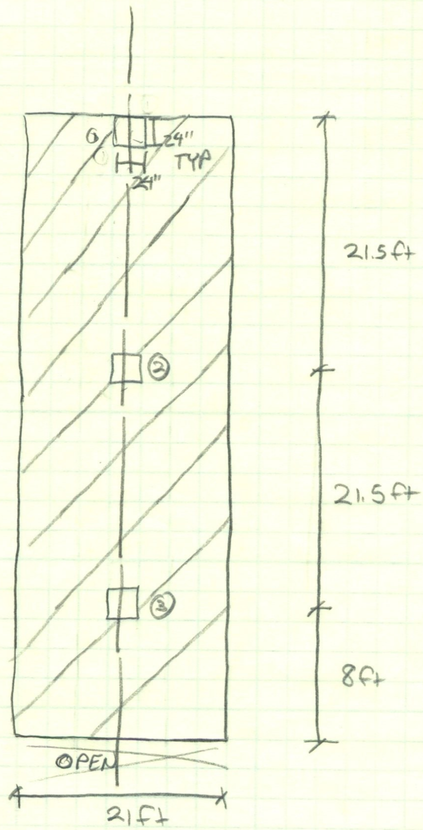
One would begin to think that the non-composite steel deck on steel beams and girders system would be the best system for the building. It is the opinion of the author that this is not the case. The controlling factor in this buildings floor system design was the story to story heights. The UHB building has top of slab to top of slab heights of only 12 ft. This makes it very difficult to fit both your structural system as well as MEP into this space and still have a reasonable ceiling height. The thinnest system is the two-way post tensioned slab, making it the best solution. Also, the bottom of the flat plate is smooth, making it so that all MEP elements can be tucked right up against the slab.

In conclusion, the existing two-way post tension slab system is the perfect system for this building even though it is the most expensive. It has the smallest system depth, making it the only system that can allow for MEP and still keep reasonable ceiling heights.

Appendix A



E. LANDIS | TECH II | EXISTING



$$K_{SLAB} = \frac{4E_cI}{l} = \frac{4E_c(12)(21)(7)^3}{12\times12\times21.5} = 114E_c$$

$$K_{COL} = \frac{4E_c(24)^4}{12\times12\times12} = 768E_c$$

$$C = \left(1.0 - 0.63\,x/y\right)\frac{x^3y}{3} = \left(1-0.63\frac{7}{24}\right)\frac{7^3(24)}{3}$$

$$= 2239.79\ \text{IN}^4$$

$$K_t = \sum \frac{9E_cC}{l_2(1-c_2/l_2)^3} = \frac{9E_c(2239.79)}{21\left(1-24/(21\times12)\right)^3}$$

$$= 1296E_c$$

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_{COL}} + \frac{1}{K_t} = \frac{1}{768E_c} + \frac{1}{1296E_c}$$

STORY HEIGHT = 12ft

$$K_{ec} = 482.23E_c$$

$$K_{CANTI\text{-}SLAB} = \frac{4E_c(12)(21)(7)^3}{12\times12\times8} = 300E_c$$

DEAD LOAD

SW: $150\times 7/12 = 87.5$ psf

SDL: 5psf

LIVE LOAD

100 psf ⇒ REDUCED 75psf

$$W = 1.2(87.5+5) + 1.6(75) = 231\text{psf} \times 21\text{ft} = 4.85\ \text{klf}$$

$$M_u = wl^2/12 = \frac{4.85(21.5)^2}{12} = 186\ \text{k·ft}$$

$$M_u = wl^2/16 = 140.1\ \text{k·ft} \qquad M_u = wl^2/10 = 224.1\ \text{k·ft}$$

$$M_{CANT} = 4.85(7\text{ft})(3.5\text{ft}) = 118.8\ \text{k·ft}$$

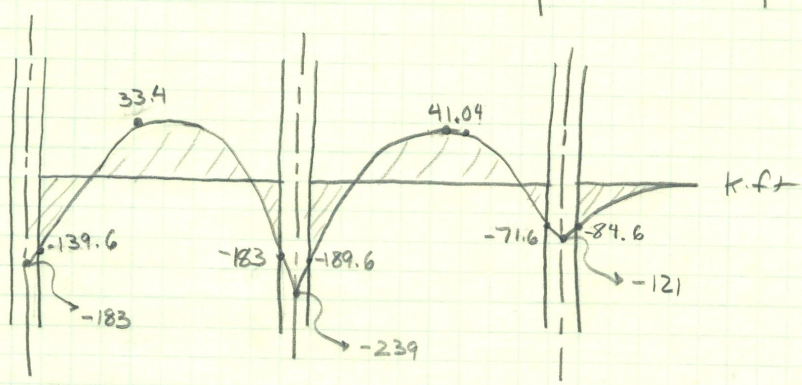
$DF_1 = \frac{482.23}{482.23 + 119} = .809$ $1 - .809 = .191$ $DF_2 = \frac{482.23}{2(119) + 482.23}$

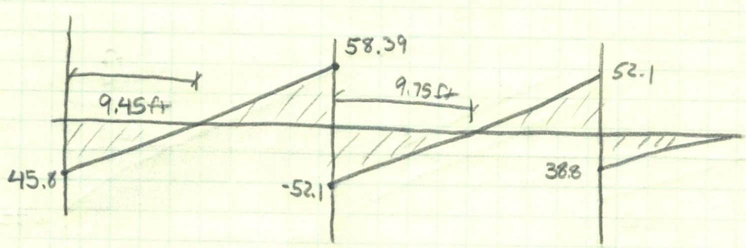
$= .679$

$1 - .679 = .321$

COF = 0.5 (CONSERVATIVE)

DF	.809	.191	.191	.679	.321	.321	.679	0
	COL	SLAB	SLAB	COL	SLAB	SLAB	COL	CANT
FEM		224	-224		186	-186		-118
BAL	-181	-43	7.26	25.8	12.2	60	126.3	
CO		3.63	-21.5		30	6.1		
BAL	-2.94	-.69	-1.62	-5.77	-2.72	-1.95	-4.14	
CO		-.81	-.345		-.975	1.36		
BAL	.655	.154	.252	.896	.424	-.43	-.92	
	-183	183	-239	19	220	121	-121	-118





TECH II EXISTING

Target Load Balance

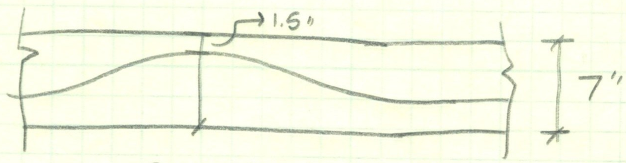
$\frac{3}{4} w_u = \frac{3}{4}(231) = 173.25\ psf$

Forces Required For Balance

$w = 173.25(21 ft) = 3.63\ klf$

Force needed in tendons

$P = \frac{wl^2}{8a_{net}} = \frac{3.63(21.5)^2}{8(5.5)} = 38.13^k$



$a_{net} = 7 - 1.5 = 5.5$

$\frac{38.13^k}{27^k}$ → Force specified by designer per 0.5" strand

Need 2 strands/tendons

$d = \frac{1}{2}"$

$A = .153 in^2$

$f_{pj} = 290\ ksi$

$F_{py} = 243\ ksi$

$f_{pe} = 159\ ksi$

$P_{ACTUAL} = 52^k$

$\frac{P_{ACTUAL}}{A} = \frac{52^k}{21 \times 12 \times 7} = 29.4\ psi$

Appendix B

E. LANDIS | TECH II | ALT. 1

PRECAST HOLLOW CORE PLANKS ON STEEL BEAMS

LIVE LOAD

100psf = OFFICE + PARTITIONS

DEAD LOAD

5psf = SDL

BAY 21 x 21.5 ft PLANKS RUNNING PARALLEL TO 21ft DIR.

DUE TO SPAN AND LOADING A
NITTERHOUSE 8" x 4'-0" w/ 2" TOPPING IS TO BE TRIED

2hr FIRE RATING

PLANK LOADING:

$w = 1.2(0) + 1.6(100+5) = 168psf < 271psf$ ✓

SDL IS TREATED AS LL AS TO COORDINATE WITH NITTERHOUSE'S STRENGTH ANALYSIS

PLANK WEIGHT: 61.25psf
2" TOPPING: 25psf

6 - 1/2" Ø LOAD ALLOWED @ 21ft = 271psf

GIRDER DESIGN: PERPENDICULAR TO PLANKS

LL = 100psf LL REDUCES TO 75psf (SEE STEEL DECK & BM)

DL = 61.25psf + 25psf => PLANK & TOPPING
5psf => SDL
5psf => BEAM ALLOWANCE

$$w = \left[1.2(61.25 + 25 + 5 + 5) + 1.6(75)\right](21) = 4.9\ klf$$

$$M_u = \frac{wl^2}{8} = \frac{(4.9)(21.5)^2}{8} = 283\ k \cdot ft$$

$L_b = 0$

W21x55 => $\phi M_n = 314\ k \cdot ft$

W21x62 => $\phi M_n = 359\ k \cdot ft$

E. LANDIS TECH II ALT. 1

$V_u = \frac{4.9(21.5)}{2} = 53^k$

$W21\times55 \Rightarrow \phi V_n = 234^k$

$W21\times62 \Rightarrow \phi V_n = 252^k$

$$\Delta_{DL} = \frac{5(2.02)(21.5)^4}{384(29000)(1140)}(1728) = .29'' \quad W21\times55$$

$$= .25'' \quad W21\times62$$

$w_{DL} = 21(96.25) = 2.02 \text{ klf}$

$$\Delta_{LL} = \frac{5(1.575)(21.5)^4}{384(29000)(1140)}(1728) = .22'' \quad W21\times55$$

$$= .19 \quad W21\times62$$

$w_{LL} = 21(75) = 1.575 \text{ klf}$

$l/240 = 21.5(12)/240 = 1.075'' \checkmark \text{OK} > .29''$

$l/360 = 21.5(12)/360 = .72'' \checkmark \text{OK} > .22''$

$\boxed{\text{USE } W21\times55}$

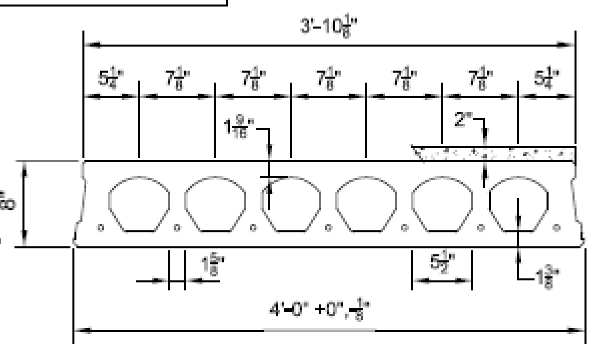
Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 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 = 92.3 k-ft at 60% jacking force
   - 6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   - 7-1/2"Ø, 270K = 147.8 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)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

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.

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

11/03/08

8SF2.0T

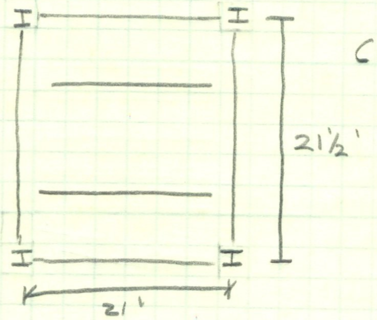
Appendix C

E. LANDIS | TECH II | ALT. 2

STEEL DECK w/ STEEL BEAMS & GIRDERS

LIVE LOAD
PARTITIONS = 20psf
OFFICE = 80psf

DEAD LOAD
SDL = 5psf



CHOOSE: BEAM SPACING OF 7'-3"
VULCRAFT DECK 3C22 6" CONC.
CHECK: CONST. CLEAR SPANS
3-SPAN = 10'-1" ≥ 7'-3' ✓
WEIGHT = 56psf

DECK LOADING

W = 20 + 80 + 5 + 56 = 161psf < 165psf ALLOWED 3-SPANS OF 7'-6" ✓

FOR 3C22 3SPAN @ 7'-6" $l/240$ = 214psf > 100psf ✓ OK FOR LL DEFL.

BEAM DESIGN

LIVE LOAD: 100psf $K_{LL} A_T \le 400$ SF ∴ LL REDUCTION NOT APPL

DEAD LOAD = 5psf SDL
5psf BEAM ALLOWANCE
56psf DECK/SLAB

$w = 1.2(7.25\,ft)(66\,psf) + 1.6(7.25\,ft)(100\,psf) = 1734\,plf$

$M_u = \frac{wl^2}{8} = \frac{(1734\,plf)(21\,ft)^2}{8} = 95.6\ k\cdot ft$

CONTINUOUSLY SUPPORTED $L_b = 0$

W12×22 $\phi M_n = 110\ k\cdot ft$ → BETTER DEFL CONTROL

W10×22 $\phi M_n = 97.5\ k\cdot ft$ → HIGHER CEILING HIEGHT

$V_u = 21(1734)/2 = 18.2^k$

W12×22 $\phi V_n = 95.9^k$
W10×22 $\phi V_n = 73.4^k$ BOTH O.K.

E. LANDIS | TECH II | ALT. 2

DEFLECTION CHECK

$$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_x} = \frac{5(.725\,klf)(21)^4}{384(29{,}000)(156)}(1728) = .663'' \text{ FOR W12x22}$$

$$= .927'' \text{ FOR W10x22}$$

$w_{LL} = 7.25(100) = 725\,plf$

I_x W12x22 = 156 in^2 I_x W10x22 = 118 in^2

$l/360 = 21(12)/360 = .70$ MUST USE W12x22

CHECK WEIGHT ESTIMATE = $\frac{22}{7.25} = 3.0\,psf < 5\,psf$ ✓

GIRDER DESIGN

LIVE LOAD = 100 psf $K_{LL}A_T = 2(21 \times 21.5) = 903\,SF > 400$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{903}}\right) = 75\,psf > 0.5L_o \checkmark$$

DEAD LOAD = 5psf SDL
5psf MEMBER ALLOWANCE
56 psf SLAB/DECK

$W = 1.2(21ft)(66psf) + 1.6(21ft)(75psf) = 4183\,plf$

$$M_n = \frac{(4183plf)(21.5ft)^2}{8} = 242\,K\cdot ft$$

$L_b = 0$

W18x35 $\phi M_n = 249\,k\cdot ft$

W16x40 $\phi M_n = 274\,k\cdot ft$

$V_n = (21.5)(4183)/2 = 45\,k$

W18x35 $\phi V_n = 159^k$

W16x40 $\phi V_n = 146^k$

E. LANDIS | TECH II | ALT. 2

DEFLECTION CHECK

$$\Delta_{LL} = \frac{5(1.575\text{ klf})(21.5)^4}{384(29000)(510)}(1728) = .512'' \text{ For } W18\times35$$

$$= .504'' \text{ For } W16\times40$$

$w_{LL} = 21(75\text{psf}) = 1575\text{ klf}$

I_x W18×35 = 510 in^3 I_x W16×40 = 518 in^3

$$l/360 = 21.5(12)/360 = .716''$$

SELF WEIGHT CHECK $\frac{40}{21'} = 2.0 \leq 5$ ✓ OK USE W16×40 FOR MORE STORY CLEARANCE

ADDITIONAL DEFLECTION CHECKS

BEAM

$$\Delta_{DL} = \frac{5(.479)(21)^4}{384(29{,}000)(156)}(1728) = .463'' \text{ W12×22}$$

$$.648'' \text{ W10×22}$$

$w_{DL} = 7.25(66) = .479$ klf

$$l/240 = 21(12)/240 = 1.05'' > .463 \text{ W12×22 OK ✓}$$

GIRDER

$w_{DL} = 21(66) = 1.386$ klf

$\Delta_{DL} = .45''$ W18×35

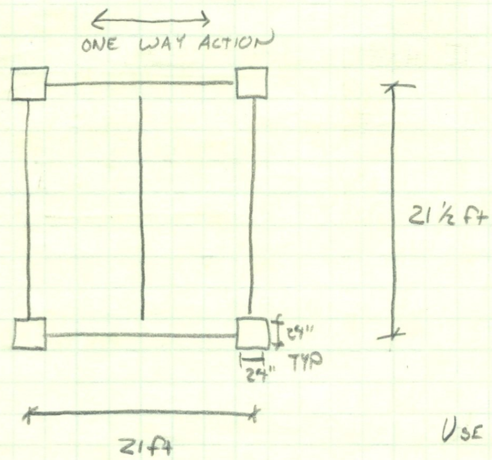
.44 W16×40

$$l/240 = 21.5(12)/240 = 1.075'' > .44'' \text{ W16×40 OK ✓}$$

Appendix D

E. LANDIS | TECH II | ALT. 3

ONE-WAY REINFORCED CONCRETE SLAB



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BEAMS @ 10'9" SPACING

SLAB THICKNESS

TABLE 9.5a ACI-318.11
CONTINUOUS BOTH ENDS ⇒ ONE WAY

$t = 10.5(12)/28 = 4.5'' \rightarrow \underline{5''}$

USE #4 BARS w/ 3/4" CLEAR

$d = 5'' - .75'' - 0.5/2 = 4''$

$d_b\ \#4 = 0.5''$

DEAD LOAD

$150 \times 5/12 = 62.5\text{psf}$

$SDL = 5\text{psf}$

LIVE LOAD

OFFICE + PARTITIONS = 100psf

$w = 1.2(62.5) + 1.6(100) = 235\text{psf}$

ASSUMING SLAB IS TENSION CONTROLLED $\phi = 0.9$

* REQUIREMENTS IN 8.3.3 ACI 318-11 ARE MET

$$-M_u = w_u l_n^2/11 = \frac{(235\text{psf})(1\text{ ft})(9.83)^2}{11} = \underline{2.0\ k\cdot ft/ft}$$

$l_n = 10.5' - 10''/12 = 9.83ft$ * ASSUMING BMS ARE 10" WIDE

* ASSUME FLEXURAL REINFORCEMENT ARM $0.95d = jd$

$$A_s \geq \frac{M_u}{\phi f_y(d - a/2)} \cong \frac{M_u}{\phi f_y(jd)} = \frac{2.0\ k\cdot ft/ft\ (12)}{0.9(60)(0.95)(4)} = .117\ in^2/ft$$

E. LANDIS | TECH II | ALT. 3

$b = 12\text{ IN}$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{.117(60)}{0.85(4)(12)} = .172\text{ IN}$$

IT IS EVIDENT THAT THIS WILL BE A TENSION CONTROLLED SECTION DUE TO "a" BEING VERY SMALL AND

$$c = a/\beta_1 = \frac{.172}{.85} = .202\text{ IN} < \tfrac{3}{8}d \quad \therefore \phi = 0.9$$

$$A_s \geq \frac{M_u}{\phi f_y (d - a/2)} = \frac{2.0\text{ k}\cdot\text{ft/ft}\,(12)}{0.9(60)(4 - .172/2)} = .114\text{ IN}^2/\text{ft}$$

$$\rho = \frac{A_s/\text{ft}}{bd} = \frac{.114}{12(4)} = 0.0023 < .114 \quad \checkmark\text{OK}$$

SHEAR CHECK

$$V_u = \frac{w_u l_n}{2} = \frac{235(1\text{ ft})(9.83)}{2} = 1155\text{ lb/ft}$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1.0)\sqrt{4000}\,(12)(4) = 6071\text{ lb/ft}$$

$$\phi V_c = 0.75(6071) = 4553\text{ lb/f} > 1155\text{ lb/ft} \quad \checkmark\text{OK}$$

REINFORCEMENT DESIGN

$$A_{s\,min} = 0.0018 \times b \times h = 0.0018 \times 12\text{ IN} \times 5\text{ IN} = 0.108\text{ IN}^2/\text{ft}$$

USE $0.114\text{ IN}^2/\text{ft}$

#4 @ 12" $A_s = .20\text{ IN}^2/\text{ft} \geq .114\text{ IN}^2/\text{ft}$ ✓OK

TEMP/SHRINK REIN.

$$A_s = 0.0018 \times 12 \times 5 = 0.108\text{ IN}^2/\text{ft}$$

MAX SPACING $\leq 5h \leq 18"$

$$\frac{0.108\text{ IN}^2/\text{ft}}{.2\text{ IN}^2(12)} = S^{-1} = 22"$$

USE SPACING OF 18" #4's

TRANSVERSE DIRECTION

E. LANDIS TECH II ALT. 3

BEAM DESIGN

$W_u = 235(10.5\text{ ft}) = 2468\ \text{lb/ft}$

$M_u = \dfrac{2.468(20.33)^2}{11} \times 1.1 = 102\ \text{k.ft}$

$l_n = 21.5\text{ ft} - 14''/12 = 20.33\text{ ft}$ ASSUMING GIRDERS 14" WIDE

1.1 = 10% ALLOWANCE FOR SELFWEIGHT

ESTIMATE SIZE: $bd^2 = 20M_u$ TRY $b = 4/5\,d$

$d = \sqrt[3]{\dfrac{20(102)5}{4}} = 13.66''$

$h = 13.66 + 2.5 = 16.16'' \Rightarrow 17''$ $b = 14'' \geq 10''$ ✓ OK ASSUMPTION FOR SLAB

SELF WEIGHT EFFECTS

$W_{sw} = 150 \times \dfrac{17 \times 14}{144} = 83.3\ \text{plf}$

$W_u = 2468 + 1.2(83.3) = 2568\ \text{plf}$

$M_u = \dfrac{(2.568)(20.33)^2}{11} = 96.49\ \text{k.ft}$

$A_{S\,REQ} \approx \dfrac{M_u}{4d} = \dfrac{96.49}{4(14.5)} = 1.66\ \text{in}^2$

(3) #7'S $= .60(3) = 1.80\ \text{in}^2$

ϕM_n CHECK:

$a = \dfrac{1.80(60)}{0.85(4)14} = 2.27\ \text{in}$ $c = 2.27/0.85 = 2.66$

$M_n = 1.80(60)(14.5 - 2.66/2)/12 = 118.53\ \text{k.ft}$

$\varepsilon_s = \dfrac{0.003}{2.66}(14.5 - 2.66) = .013 > .005$ $\phi = 0.9$

$\phi M_n = 106\ \text{k.ft} > 96.5\ \text{k.ft}$ ✓ OK

E. LANDIS TECH II ALT. 3

USE (3) #7's AT EACH SUPPORT TOP

USE (3) #7's AT MID-SPAN BOTTOM (EASE OF CONSTRUCTIBILITY AND CONSERVATIVE)

SHEAR REINFORCEMENT:

$V_u = (2.568\ klf)(20.33)/2 = 26.10^k$

$\phi V_c = \phi 2\lambda\sqrt{f'c}\ b_w d = .75(2)\sqrt{4000}(14)(14.5) = 19.2^k$

TRY #4 STIRRUPS @ $s \leq d/2 = 14.5/2 = 7.25'' \Rightarrow 7''$

$A_v = 0.4 in^2$

$V_{smin} = A_v f_y d/s = 0.4(60)(14.5)/7 = 49.71^k$

CHECK SPACING ASSUMPTION

$V_s = 49.71 \leq 4\sqrt{f'c}\ b_w d = 4\sqrt{4000}(14)(14.5) = 51^k$ ✓

MIN STEEL REQ.

$$A_{vmin} = \begin{cases} 50 b_w s/f_y = 50(14)(7)/60{,}000 = \underline{.081 in^2} \\ 0.75\sqrt{f'c}\ b_w d/f_y = 0.75\sqrt{4000}(14)(14.5)/60{,}000 = .16 in^2 \end{cases}$$

$.081 in^2 < 0.4 in^2$ ✓OK

USE #4 STIRRUPS @ 7" FULL LENGTH

$\phi V_n = \phi V_c + \phi V_{smin} = 19.2^k + 0.75(51) = 57.45^k > 26.10^k$ ✓OK

E. LANDIS | TECH II | ALT. 3

GIRDER DESIGN

$w_u = 235(21.5) = 5053\ plf$

$M_u = \frac{5.053(19)^2}{11} \times 1.1 = 182.4\ k \cdot ft$

$l_n = 21ft - 2ft = 19ft$

ESTIMATE SIZE: $d = \sqrt[3]{\frac{20(182.4)5}{4}} = 16.5_{IN}$ TRY $b = 4/5\, d$

$h = 16.5 + 2.5 = 19''$ $b = 16'' \geq 14''$ ✓OK FOR ASSUMPTION IN BM CALC.

SELF-WEIGHT EFFECTS

$w_{sw} = 150 \times \frac{19 \times 14}{144} = 277\ plf$

$w_u = 5053 + 1.2(277) = 5385\ plf$

$M_u = \frac{(5.385)(19)^2}{11} = 176.7^{k \cdot ft}$

$A_{sREQ} = \frac{176.7}{4(16.5)} = 2.68''$

$(5)\ \#7's = .60(5) = 3.0\ in^2$

ϕM_n CHECK

$a = \frac{3(60)}{0.85(4)(16)} = 3.3\ in$ $c = 3.3/.85 = 3.88\ in$

$\varepsilon = \frac{.003}{3.88}(16.5 - 3.88) = .0097 > .005 \therefore \phi = 0.9$

$\phi M_n = 0.9(60)(3)(16.5 - 3.88/2) / 12 = 196\ k \cdot ft > 176^{k \cdot ft}$ ✓OK

USE (4) #8's AT EACH SUPPORT TOP
USE (4) #8's AT MID SPAN BOTTOM

E. LANDIS | TECH II | ALT. 3

SHEAR REINFORCEMENT

$V_u = 5.385(19)/2 = 51.16^k$

$\phi V_c = 0.75(2)\sqrt{4000}\,(16)(16.5) = 25^k$

TRY #4 STIRRUPS @ $s \leq d/2 = 16.5/2 = 8.25" \Rightarrow 8"$

$A_V = 0.4\,in^2$

$V_{s\,min} = 0.4(60)(16.5)/8" = 49.5^k$

CHECK SPACING ASSUMPTION

$V_s = 49.5^k \leq 4\sqrt{4000}\,(16)(16.5) = 66.7^k$ ✓

MIN STEEL REQ

$$A_{v\,min} = \begin{cases} 50(16)(8)/60{,}000 = .1067\,in^2 \\ .75\sqrt{4000}\,(16)(16.5)/60{,}000 = .208\,in^2 \end{cases}$$

$.1067\,in^2 \;\; 0.4\,in^2$ ✓

$\phi V_n = \phi V_c + \phi V_s = 25^k + 0.75(49.5) = 62^k > 51^k$ ✓ OK

USE #4 STIRRUPS @ 8" FULL LENGTH

DEFLECTION OF GIRDER (WORST CASE)

$f_r = 7.5\sqrt{4000} = 474.3\,psi$

$E_c = 57000\sqrt{4000} = 3.61 \times 10^6\,psi$

$n = \frac{29e^6}{3.61e^6} = 8.03$

$I_g = \frac{16(19)^3}{12} = 9145\,in^4$

$\rho = \frac{A_s}{bd} = \frac{3.0}{16(16.5)} = .0113$

$K = -(.0113)(8.03) + \sqrt{[(.0113)(8.03)]^2 + 2(.0113)(8.03)} = .344$

$kd = .344(16.5) = 5.68''$

$I_{CR} = \frac{16(5.68)^3}{12} + 16(5.68)(5.68/2)^2 + 8.03(3.0)(16.5-5.68)^2$

$= 3797\,in^4$

$\frac{I_g}{I_c} = \frac{9145}{3797} = 2.4$

$M_{CR} = \left[\frac{(474.3)(9145)}{9.95}\right]/12000 = 36.33\,k \cdot ft$

$\bar{y} = \frac{16(19)(19/2) + (8.03-1)(3)(16.5)}{16(19) + (8.03-1)3} = 9.95$

$M_d = \frac{1.2(62.5+5)(21.5)(19)^2}{14} = 45\,k \cdot ft$

$$\frac{M_{cr}}{M_D} = \left(\frac{36.33}{45}\right)^3 = .526$$ CONCRETE CRACKS

$$I_{ed} = .526(9145) + (1-.526)3797 = 6610 \text{ in}^4$$

$$M_L = \frac{5.053(19)^2}{14} - 45 = 85.3 \text{ k.ft}$$
$$= 130.2$$

$$M_{sus} = 45 + 85.3/2 = 87.5 \text{ k.ft}$$ 50% LL sus.

$$\frac{M_{cr}}{M_{sus}} = \left(\frac{36.33}{87.5}\right)^3 = .0715$$

$$(I_e)_{sus} = .0715(9145) + (1-.0715)(3797) = 4179 \text{ in}^4$$

$$\frac{M_{cr}}{M_{d+l}} = \left(\frac{36.33}{130.2}\right)^3 = .0217$$

$$(I_e)_{d+l} = .0217(9145) + (1-.0217)(3797) = 3913 \text{ in}^4$$

SHORT TERM

$$(\Delta_i)_d = \frac{(1)(5/48)(45)(21)^2}{(3610)(6610)}(1728) = .156''$$

$$(\Delta_i)_{l+d} = \frac{(5/48)(130)(21)^2}{3610(3913}(1728) = .765''$$

$$(\Delta)_l = .765' - .156'' = .609''$$

$$(12)(21)/240 = 1.05 > .156 \checkmark \text{OK}$$

$$12(21)/360 = .7 > .609'' \checkmark \text{OK}$$

Appendix E: R.S. Means Assemblies

Two Way Post Tensioned Slab

Assembly B10102234600 **Based on National Average Costs**

Flat plate, concrete, 8.5" slab, 24" column, 20'x20' bay, 125 PSF superimposed load, 231 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.97500	S.F.	1.11	5.51	6.62
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, i...	0.03000	SFCA	0.02	0.19	0.21
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.72000	Lb.	1.52	1.17	2.69
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.70800	C.F.	2.95	0.00	2.95
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.70800	C.F.	0.00	0.91	0.91
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$5.70	$8.73	$14.43

R.S. Means does not include pricing for post tensioning in their assemblies. Above is the closest estimated that was found. Post tensioned systems are on average $2 more than regularly reinforced concrete. Thus, $2 will be added to my one-way slab beam pricing.

Total = $19.51

Prestressed Hollow Core planks on Steel Beams

Assembly B10102303000 **Based on National Average Costs**

Precast concrete plank, 2" topping, 8" total thickness, 20' span, 100 PSF superimposed load, 175 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.10000	L.F.	0.02	0.41	0.43
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.71	0.00	0.71
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.17000	C.F.	0.00	0.26	0.26
Concrete finishing, floors, basic finishing for unspecified flatwork, bull float, manual fl...	1.00000	S.F.	0.00	1.13	1.13
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Precast slab, roof/floor members, grouted, solid, 6" thick, prestressed	1.00000	S.F.	7.15	2.88	10.03
Total			$8.10	$5.13	$13.23

This is the closest estimate available in R.S. Means assemblies. The 20 ft. span is very close to the 21 ft. used in this analysis and the loading is slightly underestimated. Thus, being a good estimate for this report.

Non-composite Steel Deck with Steel Beams and Girders

Assembly B10102581050 **Based on National Average Costs**

Floor, metal deck, 22 ga, 1.5" deep, concrete slab, 6' span, 4" deep, 150 PSF superimposed load, 189 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.05000	L.F.	0.01	0.21	0.22
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.00900	C.Y.	1.01	0.00	1.01
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.00900	C.Y.	0.00	0.29	0.29
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Metal roof decking, steel, open type B wide rib, galvanized, over 500 Sq, 1-1/2" D, 22...	1.05000	S.F.	1.45	0.62	2.07
Total			$2.72	$2.47	$5.19

The assembly is underestimating concrete depth and span. An increase of 20% is added to the total cost to offset this estimation.

Total = $6.22

One-way Reinforced Concrete Slab

Assembly B10102194400 **Based on National Average Costs**

Cast-in-place concrete beam and slab, 5" slab, one way, 16" column, 20'x20' bay, 125 PSF superimposed load, 206 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.15900	SFCA	0.14	1.63	1.77
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.33300	SFCA	0.36	2.80	3.16
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.85800	S.F.	0.98	4.85	5.83
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.69000	Lb.	1.51	1.16	2.66
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.53700	C.F.	2.23	0.00	2.23
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.53700	C.F.	0.00	0.82	0.82
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$5.30	$12.21	$17.51

This is the closest estimated that R.S. Means had to offer. The 20’x20’ bay size is very close to the analyzed bay size. The loading is underestimated.