

## Technical Assignment | 2

---

### Fraser Centre State College, Pennsylvania



### Technical Report II

Tyler Strange  
Structural Option  
AE Consultant: Dr. Thomas Boothby  
October 27<sup>th</sup>, 2010

---

## Technical Assignment | 2

---

### Table of Contents

I.	Executive Summary.....	3
II.	Introduction.....	4
III.	Structural Systems .....	5
IV.	Design Criteria .....	7
V.	Analysis of Floor Systems .....	9
	Existing Floor System: Flat Plate Two-Way Slab.....	9
	Alternative #1: One-Way Slab.....	11
	Alternative #2: Hollow Core Planks on Steel Beams .....	13
	Alternative #3: Composite Deck with Composite Steel Beams.....	15
VI.	System Comparison .....	17
VII.	Conclusion .....	17
VIII.	Appendix A: Existing System: Two-Way Slab.....	19
IX.	Appendix B: Alternative #1: One Way Slab .....	23
X.	Appendix C: Alternative #2 Hollow Core Plank on Steel Beams.....	26
XI.	Appendix D: Alternative #3: Composite Deck on Composite Beams.....	27

## Technical Assignment | 2

---

### Executive Summary

The pro-con structural study of alternative floor systems for the Fraser Centre looks at a typical frame through a residential floor. The typical frame has a tributary width of 40 feet and is 69 feet long. The existing two-way flat slab was studied and then compared to three alternative systems. The existing structure is a 12 inch thick slab. The direct design method was used to design the reinforcement along the chosen frames. The three alternative systems that were studied are one-way reinforced concrete slab, hollow core precast planks on steel beams, and composite deck and composite steel beams.

The one-way reinforced concrete slab was designed using ACI 310-08. The design yielded a 13" thick slab with #6 bars at 12" for flexure as well as shrinkage and temperature. The beams were also designed with ACI 310-08 with a depth of 30" and width of 24". The Nitterhouse Design Catalog and AISC Steel Construction Manual were used to design the hollow core precast planks and supporting steel beams. The catalog and manual resulted in 16"x4' Hollow Core Precast Panels with a 2" topping and a W18x76 girder. The composite system consists of a 4 ½" thick slab on 18 gage 3" VLI deck with W18x35 beam and W21x132 girder. This system was designed with the AISC Steel Construction Manual and the Vulcraft Design Catalog.

The advantages and disadvantages were discussed for each system and it was determined that the one-way slab and hollow core precast panels systems were not viable alternatives. They increased the depth of the system by 18" and 21" respectively. The one-way slab also increased the system weight 10 psf while the precast panels decreased the weight by 60 psf. The composite slab and girder system was determined to be the best alternative due the system weight being reduced by 75 psf and the total system depth only being increased by 13.5 inches.

## Technical Assignment | 2

### Introduction

The Fraser Centre is a mixed-use, high-rise development located in downtown State College, Pennsylvania (See Fig. 1). The site will encompass an entire block on the corner of Beaver Avenue and Fraser Street, at an approximate elevation of 1100 feet above sea level. The development was designed by Wallace, Roberts, and Todd LLC, to be the only building in State College to have an all glass and aluminum façade. The structure was engineered by David Chou and Associates, Inc.; the MEP was engineered by AKF Engineers; and the theater was engineered by JKR Partners, LLC.



Figure 1: Site view of Fraser Centre (blue) bounded by Fraser St., Calder Way, Miller Alley, and Beaver Ave. Photo courtesy of Bing Maps.

Fraser Centre is an eleven story multi-use building. The first floor is exclusively parking; with 94 parking spaces. Residential parking takes up the majority of the second floor along with the theater lobby and 3 retail spaces. The entire third floor is occupied by the ten-auditorium movie theatre. The mechanical equipment is located on the fourth floor, or mechanical floor. At the fourth floor the building foot print reduces from roughly 270ft x 165ft to 190ft x 76ft. Floors five through eleven are all residential levels; floor five consists of nine units, levels six through ten all have eight units, and three penthouse suites makes up the penthouse or eleventh floor.

The structural system of Fraser Centre is reinforced concrete. The gravity load resisting system consists of concrete columns, shear walls, and two-way slabs. The lateral system is composed of reinforced concrete shear walls located throughout the entire building.

## Technical Assignment | 2

---

### Structural Systems

#### Gravity System

Columns are designed with 5000 psi concrete for the columns below the sixth level and 4000 psi concrete will be used for columns above the sixth level. Figure 2 in the Appendix shows the column locations and the column size and reinforcement can be found in Figure 3a through 3g. Column sizes vary from 18"x24" and 16"x32" to 24"x72" and 36"x60" and there are also 24" diameter columns.

Beams on level 2 garage vary in width from 10" to 36" with 18" being the most common and a depth between 24" and 111", 30" is the most common depth. The theater level beams vary from 12" to 72" and 20" to 48" in width and depth respectively. Beams vary in depth from 24" to 40" and 16" to 48" on the mechanical floor. 12"x 78" and 48"x30" is the range of beams on the roof. All beams are made with 4000 psi concrete.

The parking garage has 9" slabs on grade reinforced with 13#5 bars on top and a bottom grid of #4 bars at 12" each way. 4000psi concrete will be used for the slab on grade. 18#5 top bars and a grid of #5 bottom bars at 12" reinforce the 14" concrete slab of the theatre level. In addition to #7 bottom bars at 9" East-West and #5 bottom bars North-South in the 16" slab, the mechanical floor also has a 12'-6"x7' transfer girder with 40 #11 bottom bars and 20 #11 top bars. The residential levels and penthouse (5 through 11) as well as the roof have 12" slabs reinforced with a grid of #5 bars at 14" east-west and 12" north-south. All of the structural slabs will have 5000 psi concrete and a typical span of 40 feet. Steel beams are used for the projection of the mezzanine floor, and they vary from W8x10 to W12x22.

#### Lateral System

Concrete shear walls will be used in Fraser Centre to resist lateral loads. Shear walls are composed of 5000 psi concrete and reinforced with #5 horizontal bars and #6 vertical bars. Shear walls are located along column lines 3, 4, 5, 6, and 7 as shown in Figures 2 and 3. The theatre level has 14" shear walls and 16" walls are typical of the parking levels and the residential levels.

## Technical Assignment 2

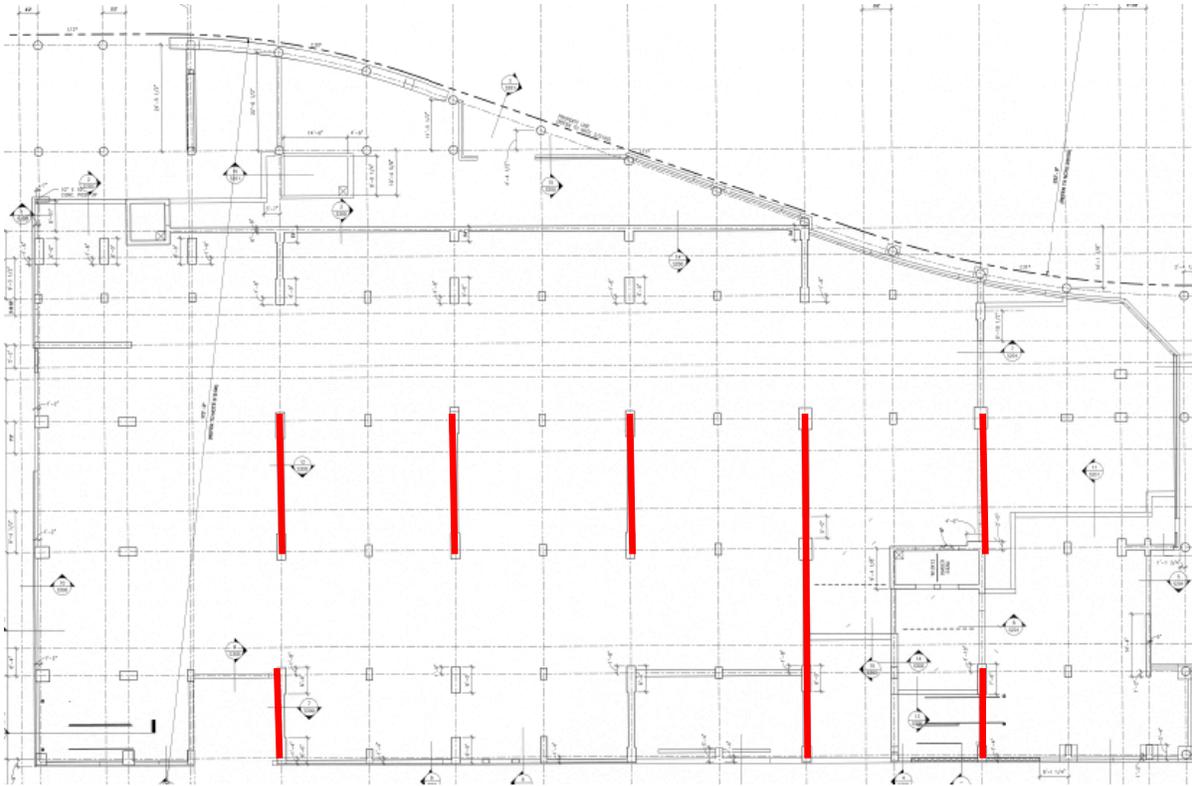


Figure 2: First Floor Shear Wall Plan

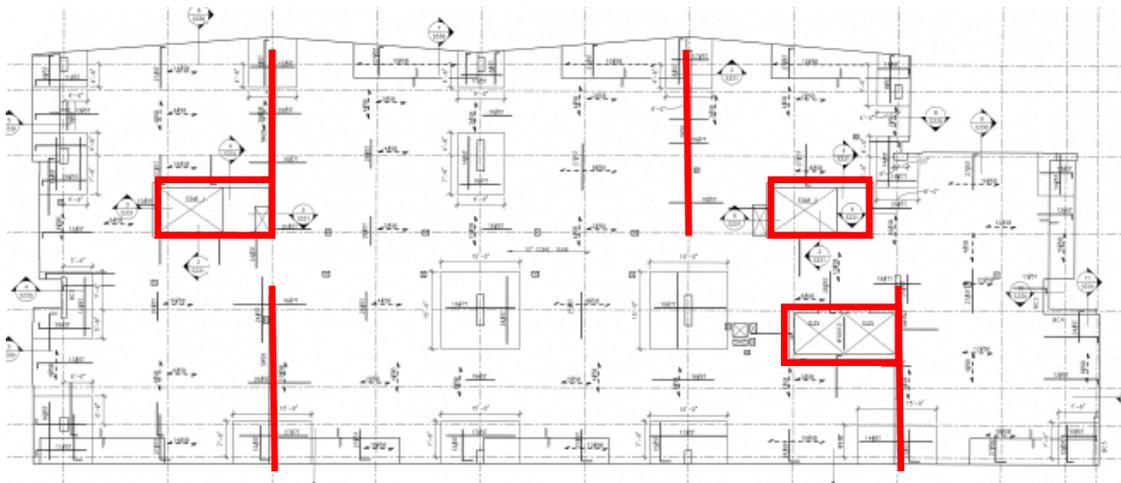


Figure 3: Typical Residential Floor Shear Wall Plan

## Technical Assignment | 2

### Design Criteria

The following data is provided to illustrate the general design criteria for Fraser Centre.

#### Codes & Design Standards

<b>Applied to Original Design</b>
<b>International Building Code IBC 2006</b>
<b>American Concrete Institute Building Code ACI 318-05</b>
<b>American Institute of Steel Connection AISC, 9<sup>th</sup> Edition</b>
<b>Steel Deck Institute SDI Specification</b>
<b>Building Code Requirements for Masonry Structures ACI 530-05</b>
<b>American Society for Civil Engineers ASCE 7-05</b>

<b>Substituted for Analysis</b>
<b>International Building Code IBC 2006</b>
<b>American Concrete Institute Building Code ACI 318-08</b>
<b>American Institute of Steel Connection AISC, 13<sup>th</sup> Edition</b>
<b>American Society for Civil Engineers ASCE 7-10</b>

Table 1: Codes and Standards used for Original Design and Analysis.

## Technical Assignment | 2

### Material Strength Requirements

Material	Strength Requirement
Cast –In-Place Concrete:	
Footings	4 ksi NWC
Basement and Bearing Walls	4 ksi NWC
Shear Walls and Columns	5 ksi NWC
Grade Beams and Slab on Grade	4 ksi NWC
Structural Slab	5 ksi NWC
Reinforcement	ASTM A615, Grade 60
Structural Steel:	
Steel Shapes	ASTM A992
Structural Tubes	ASTM A500
Plates	ASTM A36

Table 2: Material Strength Requirements per drawing S001

### Dead and Live Loads

Area	Design Live Load (psf)
Roof/Ground Snow (from drawing S001)	Min 40
Mechanical	125
Rooms	40
Stairs/Public Rooms/Corridors/ Balconies	100
Theater	60
Retail Sales	100
Light Storage	125

	Design Super-Imposed Dead Load (psf)
Roofing	10
Partitions	20
4" Hollow Non-Bearing Block	30 (/sf of wall)
8" Hollow Non-Bearing Block	55 (/sf of wall)
Brick Veneer	40 (/sf of wall)

Table 3: Design Live and Super-Imposed Dead Loads per drawing S001

## Technical Assignment | 2

---

### Analysis of Floor Systems

A typical frame through a residential floor was used in the analysis of all four floor systems. The frames are spaced 40 feet apart and are 69 feet long.

#### Existing Floor System: Flat Plate Two-Way Slab

##### Material Properties:

Concrete: 12" Normal Weight Concrete Slab

$f'_c=5000$  psi

##### Loading:

Dead: 150 psf

Live Load: 40 psf

Superimposed Dead Load: 20 psf

##### Description:

The two-way reinforced flat slab system is a 12" normal weight concrete slab, figure 4 shows the layout of the existing floor system. The typical bottom reinforcement across the entire bay is #5 at 12.2 inches on center, and the top reinforcement varies over the column strips and middle strips.

A typical two bay strip through a residential floor was used to analyze the existing floor system. The Direct Design Method prescribed by the ACI 318-08 was used to design the two-way flat slab floor system. The bay was split into two frames, Frame A and Frame B noted in Appendix A. The slab was checked for flexural, shear, and minimum thickness. The slab thickness of 12 inches is below the minimum requirement of 14 inches. 12 inches was used in analysis because the existing floor is 12 inches instead of 14 inches, not in accordance with ACI 318-08 Table 9.5 c. Punching shear was also checked at the columns, but did not exceed the limits. All supporting calculations for this analysis can be found in Appendix A.

## Technical Assignment 2

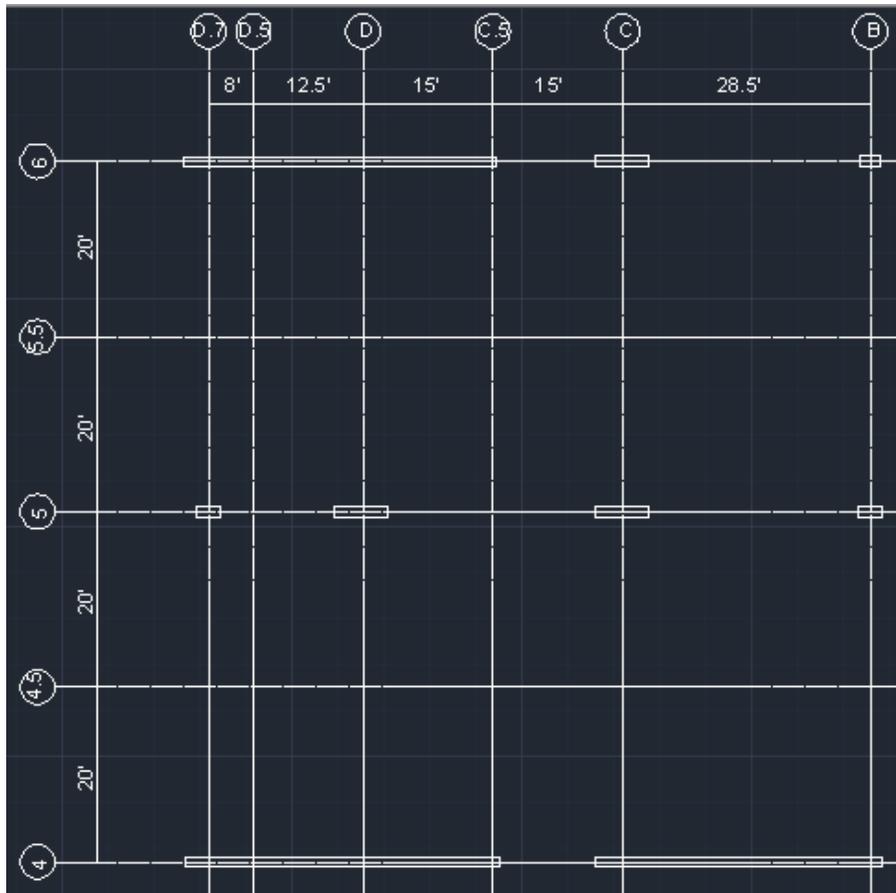


Figure 4: Existing Floor System Layout

### Advantages:

Two-way flat plates seldom require additional fireproofing. This cuts down on construction and lead time. Construction of a two-way flat plate requires simple formwork and simple construction techniques. Concrete and steel reinforcing bars are widely available which cuts down on lead time.

### Disadvantages:

This system requires an aspect ratio less 2.0 which is barely met on the north side of the building. Two-way flat plates are not intended for long spans or live loads in excess of 50 psf. The long spans present at Fraser Centre yielded a thick two-way plate that was full of reinforcing bars. To make the two-way slab a better system for Fraser Centre a shorter span length would be beneficial. The large depth provided by the two-way slab would hamper the installation of the buildings mechanical and electrical systems.

## Technical Assignment | 2

---

### Alternative #1: One-Way Slab

#### Material Properties:

Concrete: 13" Normal Weight Concrete Slab

$f'_c=5000$  psi

Reinforcement:  $F_y=60000$  psi

#### Loading:

Dead: 162.5 psf

Live Load: 40 psf

Superimposed Dead Load: 20 psf

#### Description:

The one-way slab system designed for the interior bay was a 13" concrete slab that spans a maximum of 30'. A girder spans between the columns and shear walls, allowing the slab to frame into the girder, and the load is transferred to the columns. ACI 318-08 requires the aspect ratio for the bay to be larger than 2.0 for the designing of a one-way slab. The smallest aspect ratio of the bays was 2.67. Figure 5 shows the layout of the floor system. The impact on the architectural layout and foundation system need to be considered before this system can be implemented for the entire building.

The 13" slab was designed to have #6 at 12" O.C. for flexural steel which also satisfies the shrinkage and temperature steel requirement. The main girder that spans along the 40' direction was designed to support the one way slab, with a beam size of 30" deep x 24" wide. All supporting calculations for this analysis can be found in Appendix B.

## Technical Assignment 2

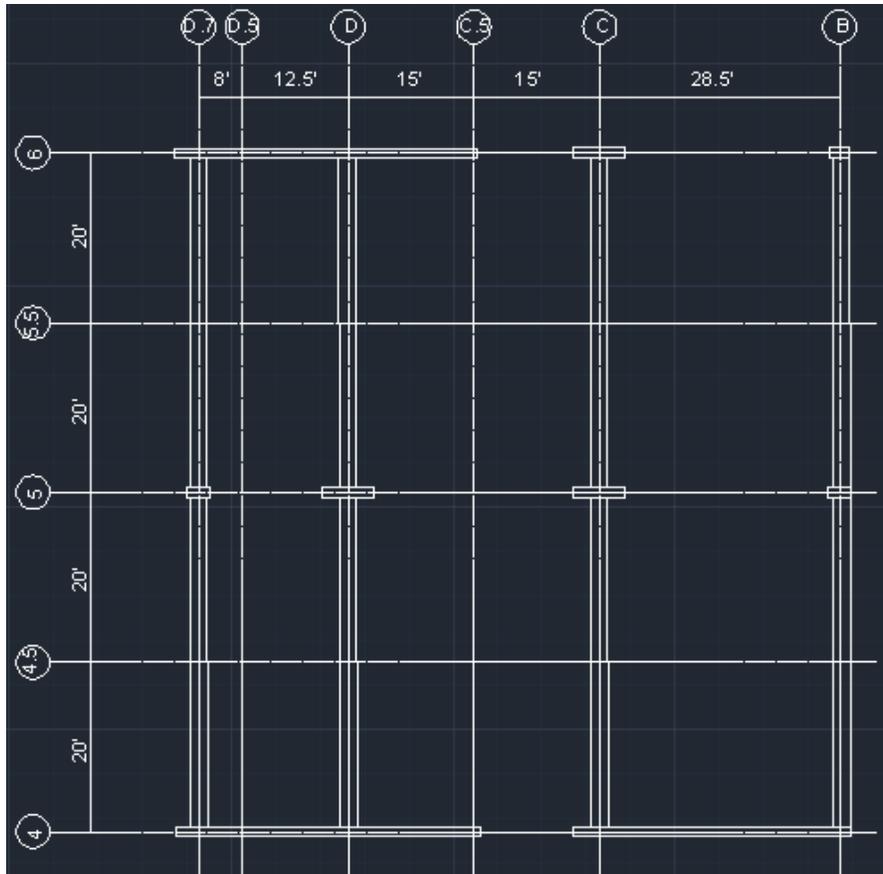


Figure 5: Layout of One-Way Slab System

### Advantages:

The one-way slab does not have any additional advantages over the existing system that are evident. The column/shear wall layout is unchanged. In addition there is no need for fire protection; the 13" slab is more than adequate for a 2 hour rating.

### Disadvantages:

There are some disadvantages to the one-way slab. The foundation system will need to be rechecked due to the slightly thicker slab and additional girder weight. The girder will also cause conflicts with the mechanical spaces and either decrease the floor to ceiling height or, more likely, increase the overall building height.

## Technical Assignment | 2

---

### Alternative #2: Hollow Core Planks on Steel Beams

#### Material Properties:

Concrete: 16"x4' Planks  
 $f'_c=5000$  psi

Steel: A992 W shape

#### Loading:

Dead (Self Weight): 91.75

Live Load: 40 psf

Superimposed Dead Load: 20 psf

#### Description:

Hollow Core Planks are precast members that are pre-stressed to allow for longer spans and higher loads for a concrete system. The hollow core plank was picked using the Nitterhouse Design Catalog, and a 16" x 4' hollow core plank is sufficient to support the loads across the 40 foot span. A typical bay on the residential floors was used to design the floor system, (see Figure 6 for the layout of this system). A W 18x76 girder was used to support the planks between the columns and shear walls. No adjustment to the column layout is needed for this system. The impact on the architectural space in this system should be considered and investigated at a deeper level. Hollow core planks bear directly onto W-shape steel beams, and a 2" topping is poured over the connection between the beam and the hollow core plank to provide a stable connection. All supporting calculations for this analysis can be found in Appendix C.

## Technical Assignment 2

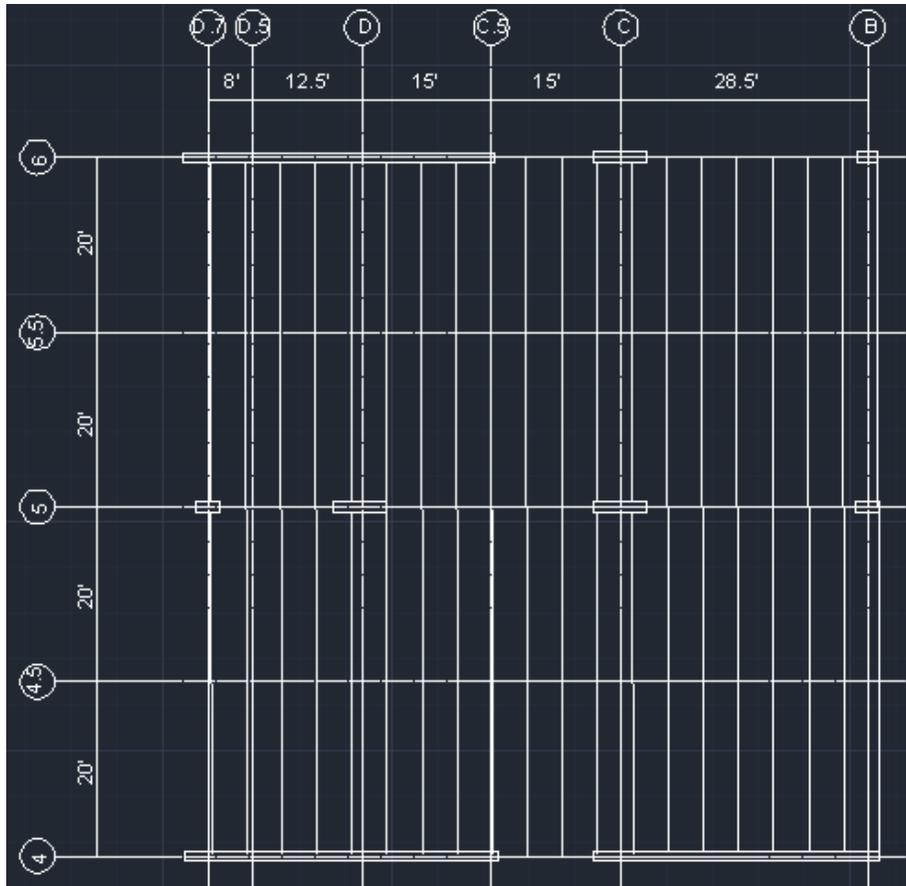


Figure 6: Typical Bay for Hollow Core Plank System

### Advantages:

The hollow core plank system has several benefits. The precast members are constructed in a concrete plant, where curing takes place under controlled conditions. The construction process is improved because the members are up to strength at the time of erection, which allows for possible fast tracking and early occupancy. The products can be constructed year round because curing takes place in the precast plant. The pre-stressed tendons allow for longer spans to be achieved with a relatively low thickness.

### Disadvantages:

The impact on the bay size to account for the 4 foot width of each plank could have an impact on the architectural layout of the building. With the increase in the depth of the steel members and a 16" plank, the deeper floor system can cause conflicts with the mechanical and electrical systems as well as a reduction in floor to ceiling height or an increase in overall building height. The hollow core planks are designed to achieve a fire rating of 2 hours; however, the steel beams will require spray-on fireproofing.

## Technical Assignment | 2

---

### Alternative #3: Composite Deck with Composite Steel Beams

#### Material Properties:

Concrete: 4 ½" Normal Weight Concrete Slab

$f'_c=5000$  psi

Decking: 18 Gage 3" VLI (Vulcraft)

Steel: A992 W Shape

Beams: W18x35

Girders: W30x90

W21x132

#### Loading:

Dead (Self Weight): 75 psf

Live Load: 40 psf

Superimposed Dead Load: 20 psf

#### Description:

The composite steel beam on composite metal deck is a system that combines the strengths of steel in tension and concrete in compression, to provide a very effective system. A typical bay on a residential floor was used to design the composite steel systems, (see figure 7 for the layout). W-shape girders span from column to column with an infill beam framing into the girder. The metal deck that sits on the beam spans perpendicular to the beam. When using metal decking, composite action is easily obtained. However, extra design steps are needed to obtain composite beam action. For a beam to obtain composite action with the slab, shear studs are required along the length of the beam. The shear studs transfer the load from the concrete slab into the beam. Appendix D contains the supporting calculations for the design of the composite steel system.

## Technical Assignment 2

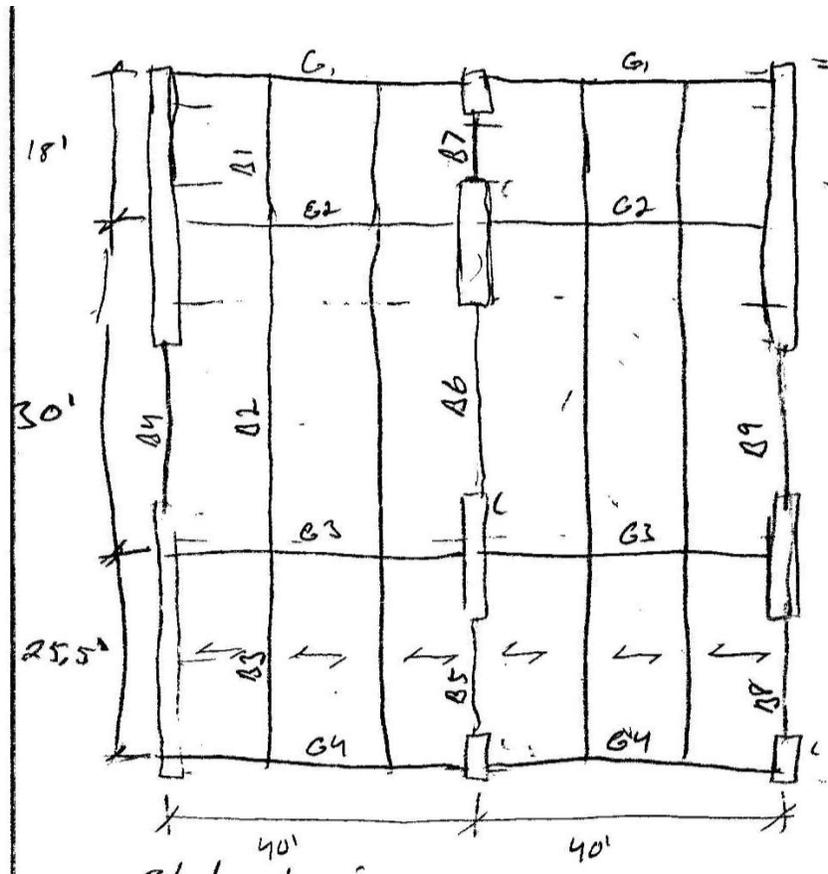


Figure 7: Typical Bay for Composite Slab and Girder System

### Advantages:

A composite metal deck on composite steel system has many advantages. The metal deck provides the necessary formwork to place the concrete, and if the spacing of the beams is appropriate, no shoring is required during construction. The composite system allows the use of smaller steel members and a thinner concrete slab.

### Disadvantages:

A composite beam system does have smaller beams, but the beams are still around 16 inches deep. Obstructions with the mechanical and electrical systems can cause an increase in the space between the ceiling and the bottom of the slab. One of the more expensive parts of the composite steel system is the cost of the connections. A faster construction time is achieved with the composite steel; however there is an increase in labor for the placement of the shear studs. To obtain the proper fire rating for the structural steel, a spray on fireproofing is required.

## Technical Assignment | 2

### System Comparison

Floor System Comparison of Typical Bay				
	Floor Systems			
	Existing Two-way Flat Slab	One-way Slab	Precast Plank on Steel Beams	Composite Slab and Girder
System Weight (psf)	210	222.5	151.75	135
Slab depth (in)	12	13	16	4.5
Total depth (in)	12	30	33	25.5
Extra Fire Proofing Req'd	No	No	Yes	Yes
Fire Rating (hour)	2	2	2	2
Total Cost (\$/SF)	9.50	19.49	11.84	17.60
Foundation Impact	None	None	None	Yes
Architectural Impact	None	Yes	Yes	Yes
Constructability	Moderate	Moderate	Easy	Easy
Vibration Concerns	Minimal	Minimal	Minimal	Some
Possible Alternative	N/A	Yes	Yes	Yes
Additional Study	N/A	Yes	Yes	Yes

## Technical Assignment | 2

---

### Conclusion

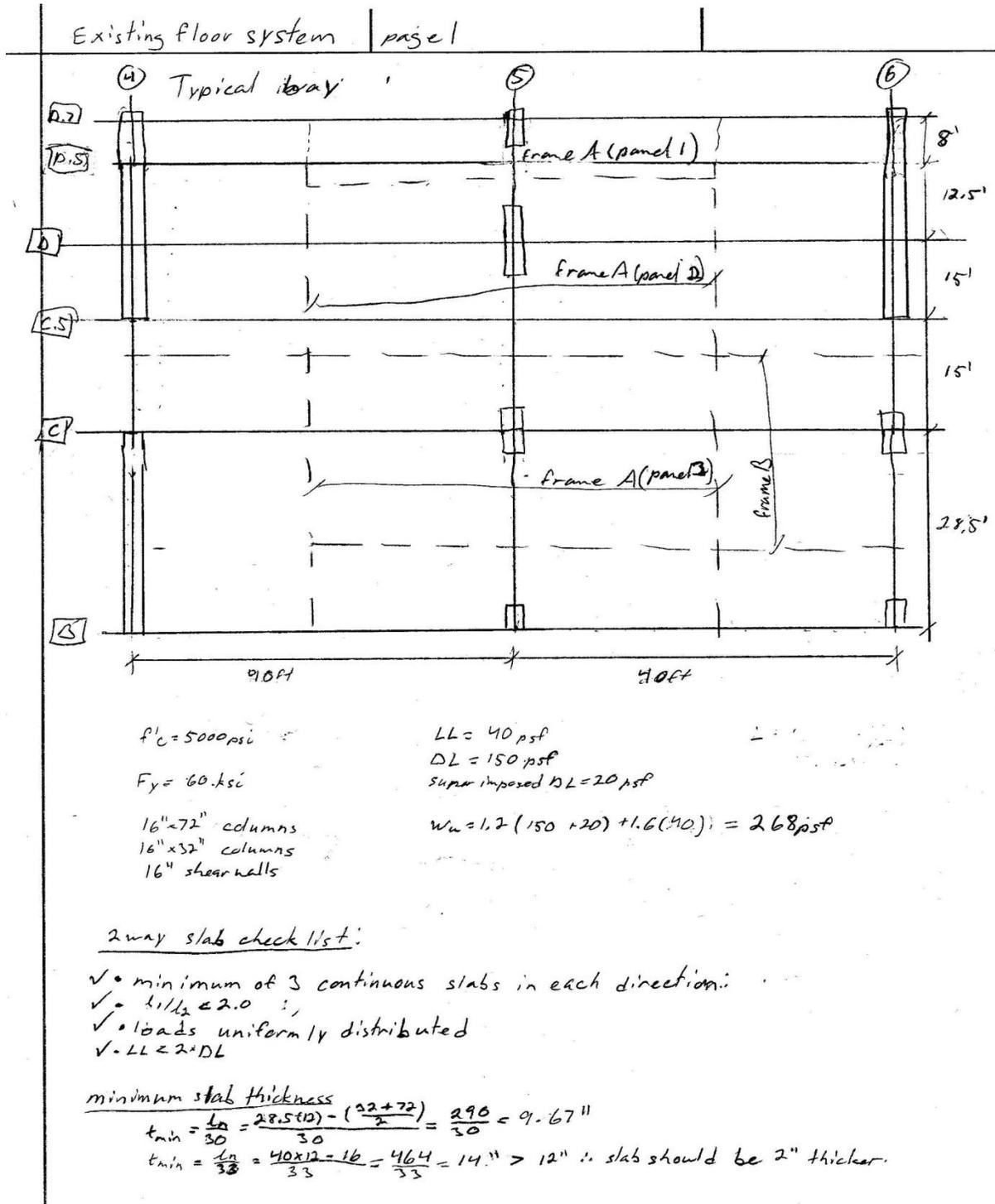
The typical frame has a tributary width of 40 feet and is 69 feet long. The three alternative systems that were studied are one-way reinforced concrete slab, hollow core precast planks on steel beams, and composite deck and composite steel beams.

The one-way reinforced concrete slab was designed using ACI 310-08. The design yielded a 13" thick slab with #6 bars at 12" for flexure as well as shrinkage and temperature. The beams were also designed with ACI 310-08 with a depth of 30" and width of 24". The Nitterhouse Design Catalog and AISC Steel Construction Manual were used to design the hollow core precast planks and supporting steel beams. The catalog and manual resulted in 16"x4' Hollow Core Precast Panels with a 2" topping and a W18x76 girder. The composite system consists of a 4 ½" thick slab on 18 gage 3" VLI deck with W18x35 beam and W21x132 girder. This system was designed with the AISC Steel Construction Manual and the Vulcraft Design Catalog.

After reviewing the advantages and disadvantages of each system and it was determined that the one-way slab and hollow core precast panels systems were not viable alternatives. They increased the depth of the system by 18" and 21" respectively. The one-way slab also increased the system weight 10 psf while the precast panels decreased the weight by 60 psf. The hollow core precast plank was the cheapest of the alternatives at \$11.84/SF and the one-way slab was the most expensive at \$19.49/SF. The composite slab and girder system was determined to be the best alternative due the system weight being reduced by 75 psf and the total system depth only being increased by 13.5 inches. Although it does increase the cost \$5.74/SF more than the precast plank its weight and depth offset its disadvantages.

## Technical Assignment 2

### Appendix A: Existing System: Two-Way Slab



## Technical Assignment 2

Existing

base 2

check punching shear

column 5-5  
 $V_u = 268 [14.25 \times 40 - \frac{16 \times 32}{144}] = 152 \text{ k}$

#7 bars  
 $d = 12 - 1.5 - 0.875 = 9.625$

$b_o = (16 + 9.625) \times 3 = 70.875$

$V_c = 4 \cdot \sqrt{f_c} \cdot b_o \cdot d = 4 \sqrt{5000} (70.875) (9.625)$

$V_c = 809.46 \text{ k}$

$\phi V_c = 0.75 (809.46) = 607.1 \text{ k} > 152 \text{ k} \therefore \text{no drop panel required}$

total factored moment

Frame A panel 2

$l_2 = 40 \text{ ft}$   
 $l_n = 30 - \frac{72}{12} = 24 \text{ ft}$

Frame A panel 1  $l_2 = 40$

$l_n = 20.5 - \frac{72 + 72}{2(12)} = 16.17$

$M_o = \frac{0.268 (40) (24)^2}{8} = 783 \text{ k}$

$M_o = \frac{0.268 (40) (16.17)^2}{8} = 350 \text{ k}$

Frame A panel 3

$l_2 = 40 \text{ ft}$   
 $l_n = 28.5 - \frac{72 + 72}{2(12)} = 24.17 \text{ ft}$

$M_o = \frac{0.268 (40) (24.17)^2}{8} = 615 \text{ k}$

Frame B

$l_2 = \frac{71.5 + 15}{2} = 21.75 \text{ ft}$   $l_n = 40$

$M_o = \frac{0.268 (21.75) (40)^2}{8} = 1166 \text{ k}$

Distribution of P.Mo (no edge beams or interior beams)

Frame A panel 3

$M_{int}^- = 0.70 (615) = 431 \text{ k}$

$M_{ext}^+ = 0.52 (615) = 320 \text{ k}$

$M_{ext}^- = 0.26 (615) = 160 \text{ k}$

panel 2

$M^- = 0.65 (783) = 509 \text{ k}$

$M^+ = 0.35 (783) = 274 \text{ k}$

panel 1

$M_{int}^- = 0.7 (350) = 245 \text{ k}$

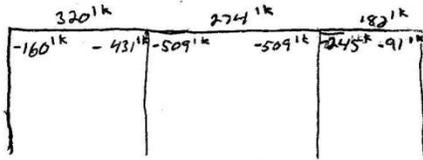
$M_{ext}^+ = 0.52 (350) = 182 \text{ k}$

$M_{ext}^- = 0.26 (350) = 91 \text{ k}$

# Technical Assignment 2

Existing

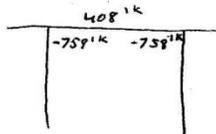
page 3



frame B

$$M^* = 0.55(1166) = 408 \text{ k}$$

$$M = 0.65(1166) = 758 \text{ k}$$



frame A panel 3

a) neg moment @ ext face  $100\%$  to CS =  $160 \text{ k}$   
 $k_2/k_1 = \frac{40}{20.5} = 1.95$   $\alpha_2 = 0$   $\beta_2 = 0$   $0\%$  to MS =  $0 \text{ k}$

b) neg moment @ int face  $75\%$  to CS =  $323 \text{ k}$   
 $25\%$  to MS =  $108 \text{ k}$  c) pos moment  
 $60\%$  to CS =  $192 \text{ k}$   
 $40\%$  to MS =  $178 \text{ k}$

distribution of moments in CS, beam, MS

frame A panel 1

a) neg moment @ ext face

$$k_2/k_1 = \frac{40}{20.5} = 1.95 \quad \beta_2 = 0 \quad \alpha_2 = 0 \Rightarrow 100\% \text{ to CS} = 91 \text{ k}$$

$$0\% \text{ to MS} = 0 \text{ k}$$

b) neg moment @ int face

$$\alpha_2 k_2 / \beta_1 = 0$$

$$75\% \text{ to CS} = 184 \text{ k}$$

$$25\% \text{ to MS} = 61 \text{ k}$$

c) positive moment

$$60\% \text{ to CS} = 109 \text{ k}$$

$$40\% \text{ to MS} = 73 \text{ k}$$

frame A panel 2

a) neg moment @ int face

$$-509 \text{ k} \quad 75\% \text{ to CS} = 382 \text{ k}$$

$$25\% \text{ to MS} = 127 \text{ k}$$

b) pos moment

$$274 \text{ k} \quad 60\% \text{ to CS} = 164 \text{ k}$$

$$40\% \text{ to MS} = 110 \text{ k}$$

frame B

a) neg moment @ int face

$$758 \text{ k} \quad 75\% \text{ to CS} = 569 \text{ k}$$

$$25\% \text{ to MS} = 189 \text{ k}$$

b) pos moment

$$408 \text{ k} \quad 60\% \text{ to CS} = 245 \text{ k}$$

$$40\% \text{ to MS} = 163 \text{ k}$$

## Technical Assignment 2

Existing page 4

frame A: total width: 40ft    CS width: 14.25'    MS width: 25.75'

	-160 <sup>k</sup>	320 <sup>k</sup>	-431 <sup>k</sup>	-509 <sup>k</sup>	274 <sup>k</sup>	-509 <sup>k</sup>	-245 <sup>k</sup>	182 <sup>k</sup>	-91 <sup>k</sup>
total mom	-160 <sup>k</sup>	192 <sup>k</sup>	-323 <sup>k</sup>	-382 <sup>k</sup>	164 <sup>k</sup>	-382 <sup>k</sup>	-184 <sup>k</sup>	109 <sup>k</sup>	-91 <sup>k</sup>
CS mom	0 <sup>k</sup>	128 <sup>k</sup>	-109 <sup>k</sup>	-177 <sup>k</sup>	110 <sup>k</sup>	-127 <sup>k</sup>	-61 <sup>k</sup>	33 <sup>k</sup>	0 <sup>k</sup>
MS mom									

panel 3
panel 2
panel 1

frame B: total width: 21.75    CS width: 10.875ft    MS width: 10.875ft

	-758 <sup>k</sup>	408 <sup>k</sup>	-758 <sup>k</sup>
total mom	-569 <sup>k</sup>	245 <sup>k</sup>	-569 <sup>k</sup>
CS	-159 <sup>k</sup>	163 <sup>k</sup>	-189 <sup>k</sup>
MS			

*sample calc for tables*

CS  $b = 14.25'' = 171''$

$d_{eff} \Rightarrow cover = 0.75$   
 bar diameter = 0.625'' (#5)

$d_{short} = 12 - 0.75 - \frac{0.625}{2} = 10.94''$

$d_{long} = 12 - 0.75 - \frac{3}{2}(0.625) = 10.31''$

$M_n = \frac{160}{0.9} = 178^{k}$

$R = \frac{M_n}{\phi b d^2} \Rightarrow \frac{160}{(171)(10.31)^2} (12 \times 1000)$   
 $= -105.63$

$\rho_{req} \Rightarrow R = \rho F_y (1 - 0.59 \rho F_y / \rho_c)$   
 $\rho = \frac{R}{F_y (1 - 0.59 \rho F_y / \rho_c)} = 0.0029$

$A_s req'd = \rho b d = 0.0029 (171)(10.31) = 0.511 in^2$

$A_{s min} = 0.002 b t = 0.002 (171)(12) = 4.104 in^2$

$N_{min} = \frac{b}{2t} = \frac{171}{2(12)} = 7.125 \Rightarrow 8$      $N = \frac{A_s}{0.31} = 13.2 \Rightarrow 14$   
45°



## Technical Assignment 2

one way slab

page 2

check reinforcement spacings for crack control

$$s = \begin{cases} 15 \left( \frac{40,000}{f_s} \right) - 2.5 c_c \\ \min 12 \left( \frac{40,000}{f_s} \right) \end{cases} \quad f_s = \frac{2}{3} f_y = 40,000 \text{ psi}$$

$$= \begin{cases} 15 \left( \frac{40,000}{40,000} \right) - 2.5(0.75) = 13.125 \\ \min 12 \left( \frac{40,000}{40,000} \right) = 12'' \text{ @ controls} \end{cases}$$

$s_{max} = 12'' \geq 12''$  ∴ ok

Shrinkage & Temperature reinforcing

$$f_y = 60 \text{ ksi} \Rightarrow p_{min} = 0.0018$$

$$A_s = 0.0018 b h = 0.0018(12)(13) \\ A_s = 0.287 \text{ in}^2/\text{ft}$$

$A_{s,min}$  governed by Shrinkage & Temperature (ACI 10.5.4)

$$s_{max} = \begin{cases} 3h = 39'' \\ \min 18 \text{ in } [18''] \text{ @ governs} \end{cases}$$

$$\text{use } \#6 \text{ @ } 12'' \Rightarrow A_s = 0.44 \text{ in}^2/\text{ft}$$

beam design ACI 318-08 table 9.5a

$$h = L/16 = \frac{40 \times 12}{16} = 30'' \quad L_{beam} = 40'$$

$$b_w = 24'' \Rightarrow 0.15(h)$$

$$W_{beam+slab} = \frac{(30-13) \times (24)}{144} \times 150 = 425 \text{ plf}$$

$$W_D = 0.1425 + 5.475 \\ = 5.9 \text{ klf}$$

$$W_{slab+SDL} = \left[ \left( \frac{1}{2} \right) (150) + 20 \right] (30) = 5.475 \text{ k/ft}$$

$$W_L = 40(30) = 1.2 \text{ klf}$$

$$W_u = 1.2(5.9) + 1.6(1.2) = 9 \text{ klf}$$

$$b_{eff} = \begin{cases} 0.25L = 0.25(40 \times 12) = 120'' \\ b_w + 16(h_f) = 24'' + 16(13) = 232'' \\ \min \frac{1}{2} \text{ clear distance} = \frac{1}{2}(14 \times 12) = 84'' \end{cases}$$

$b_{eff} = 84''$

use #11 bars

$$d = 30 - 0.75 - \frac{1}{2}(1.41) = 28.545''$$

## Technical Assignment 2

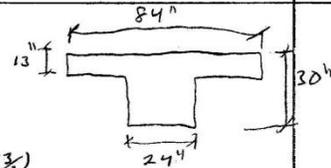
one way slab

page 3

check T-beam behavior

$$M_u = \frac{w_u l_n^2}{11} = \frac{9(39.33)^2}{11} = 1266 \text{ k}$$

$$l_n = 40 - \frac{16}{24} = 39.33'$$



assume  $a = h_f$

$$\phi M_n = \phi 0.85 f'_c b h_f (d - \frac{h_f}{2}) = 0.65(0.85)(5)(84)(13)(28.5 - \frac{13}{2})$$

$$\phi M_n = 66.4 \text{ k}$$

$M_u > \phi M_n$  ∴ treat as T beam

$$\rho_{max} = 0.75 \beta_1 \frac{f'_c}{f_y} \frac{E_s}{E_c + 10,000} = 0.85(0.845) \left(\frac{5}{60}\right) \left(\frac{29,000}{6,000}\right) = 0.0224$$

$$A_s = 0.0224(84)(28.5)$$

$$= 53.63 \text{ in}^2$$

→ use (36) #11 in 2 layers X  
 $A_s = 56.16 \text{ in}^2$

due to spacing requirements  
 use (35) #11 bars in 2 rows

$$b_{min} = 2(0.75) + 7(1.41) + 6(1.5)$$

$$= 20.37 < 24 \text{ in} \text{ ∴ ok}$$

$$A_s = 54.6 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{56.16(60)}{0.85(5)(84)} = 9.44 \text{ in} < 13 \text{ in in slab}$$

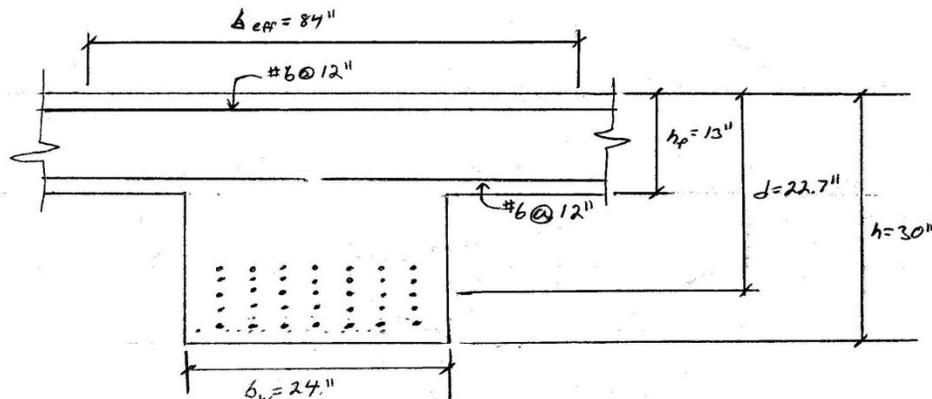
$$d = 30 - 0.75 - 2\left(\frac{1.41}{2}\right) - 2(1.5) = 22.725$$

$$\phi M_n = A_s f_y (d - \frac{a}{2}) = 0.9(54.6)(60) \left(22.725 - \frac{9.44}{2}\right) = 53086 \text{ k}$$

$$\phi M_n = 4424 \text{ k}$$

$$\phi M_n = 4424 \text{ k} > 1266 \text{ k} = M_u \text{ ∴ ok}$$

use 30" x 24" beam w/ 5 layers of (7) #11 bars



## Technical Assignment 2

### Appendix C: Alternative #2 Hollow Core Plank on Steel Beams

Hollow Core Plank | page 1

from Nitterhouse Concrete Products Spec:

16" x 4'-0" Hollow Core Plank w/ 2" topping  $\Rightarrow$  2 hr fire rating  
 7-1/2"  $\phi$   
 For 40' spans - safe super imposed load = 118 psf LL = 40 psf  
DL<sub>SEC</sub> = 70 psf  
 $1.2D + 1.6L = 1.2(20) + 1.6(40) = 88 \text{ psf} < 118 \therefore \text{ok}$

check deflections

$$\Delta = \frac{5(LL + \frac{4}{1000})(\text{span})^4}{384(E)(I)} = \frac{5(40 + \frac{4}{1000})(40)^4(1728)}{384(4030)(15498)} = 0.148" < 1.33" \therefore \text{ok}$$

$$I = 15498 \text{ in}^4 \quad \frac{L}{360} = 1.33"$$

$$E = 57\sqrt{f'_c} = 40200 \text{ ksi}$$

$$f'_c = 5000 \text{ psi}$$

design support girder (5-ED)

$$W_{\text{plank}} = 91.75 \text{ psf}$$

$$W_h = 1.2(91.75 + 20)(40) + 1.6(40)(40) = 7.92 \text{ klf}$$

$$M_h = \frac{7.92(24)^2}{8} = 570.24 \text{ k} \quad L_n = 30' - \frac{7 \times 12}{2} = 27'$$

$\Rightarrow$  from Table 3-2 (AISC): W24x62  $\Rightarrow$   $M_p = 574 \text{ k}$

- check deflection

$$D_u = \frac{W}{360} = \frac{24(12)}{360} = 0.8"$$

$$\Delta L = \frac{5WuL^4}{384EI} = \frac{5(0.04)(40)(24)^4}{384(29000)(1850)} = 0.266" < 0.8" \therefore \text{ok}$$

$$\Delta_{TL} = \frac{1}{240} = \frac{24(12)}{240} = 1.2"$$

$$\Delta_{TL} = \frac{5(91.75 + 20 + 40)(40)/1000(24)^4(1728)}{384(29000)(1850)} = 1.01" < 1.2" \therefore \text{ok}$$

- check shear

$$V_u = \frac{WuL}{2} = \frac{7.92(24)}{2} = 95.04 \text{ k} < 306 \text{ k} = \phi V_n \therefore \text{ok}$$

W24x62 works - most economical

W18x76 works - less economical / smaller depth.

W18x76  $\Delta L = 0.31" < 0.8" \therefore \text{ok}$   
 $\Delta_{TL} = 1.18" < 1.2" \therefore \text{ok}$   
 $\phi V_n = 232 \text{ k} > 95 \text{ k} \therefore \text{ok}$

## Technical Assignment 2

### Appendix D: Alternative #3: Composite Deck on Composite Beams

Composite	Page 1	
	<p>LL = 40 psf                  SLL = 20 psf                  DL = self weight of members</p>	
<p><u>slab design</u>                  used VULcraft to determine deck size, &amp; slab thickness</p> <p>12' spans <math>\Rightarrow</math> 3 concrete spans = 4 1/2" topping <math>\Rightarrow</math> 18 gage <math>\Rightarrow</math> SLL = 157 psf  <math>\frac{5 \times 11}{11}</math></p> <p>3 VLI 18 <math>\Rightarrow</math> SLL = 157 <math>\Rightarrow</math> max unshored construction = 12' <math>\geq</math> 10.86', ok                  W = 75 psf</p> <p>uniform line load + selfweight = 40 + 20 + 75 = 135 psf &lt; 157 psf, ok</p>		
<p><u>beam design</u></p> <div style="display: flex; justify-content: space-between;"> <div style="width: 30%;"> </div> <div style="width: 65%;"> <p>tributary width = 73.33'</p> <p><math>M_u = \frac{wL^2}{8} = \frac{2.49 (13.3)^2}{8} = 55.3 \text{ k}</math></p> <p><math>V_u = \frac{wL}{2} = \frac{2.49 (13.3)}{2} = 16.6 \text{ k}</math></p> </div> </div> <div style="margin-top: 10px;"> <p>DL = 75 (1.1) = 82.5 psf                  SLL = 20 psf                  LL = 40 psf</p> <p><math>w_u = 1.2(82.5 + 20) + 1.6(40) = 187 \text{ psf}</math>  <math>W_u = 187 (17.73) = 3.3 \text{ k}</math></p> </div> <p>assume <math>a = 1.5''</math>  <math>Y = 7.5''</math>  <math>Y_2 = 7.5 - \frac{1.5}{2} = 6.75''</math>                  use <math>Y_2 = 6.5''</math> (take conservative)</p> <p>try 10 x 12 <math>\Rightarrow \phi M_n = 47.3 \text{ k}</math>  <math>Y_1 = 0.1053'' \Rightarrow \phi M_n = 131 \text{ k}</math>  <math>\Sigma Q_n = 135 \text{ k}</math>  <math>\# \text{ studs} = \frac{\Sigma Q_n}{q_n} = \frac{135}{17.2} = 7.8 \Rightarrow 8 \text{ studs} \Rightarrow 16 \text{ studs across entire beam}</math>                  equiv. weight: <math>20(1.2) + 16(10 \frac{1}{2}) = 0.5216</math>  <math>M_u = \frac{0.52 (30)^2}{8} = 1.95</math>  <math>55.3 \text{ k} + 2.0 \text{ k} = 57.3 \text{ k} &lt; 131 \text{ k}</math>, ok</p>		

## Technical Assignment 2

composite

page 2

check assumption

$$b_{eff} = \begin{cases} 15.33' \\ 30/4 = 7.5' \end{cases} \quad b_{eff} = 7.5(12) = 90''$$

$$a = \frac{\Sigma Q_n}{0.85 F_c b_{eff}} = \frac{135}{0.85(5)(90)} = 0.35' < 1.5'' \therefore \text{conservative}$$

check load during construction

$$DL = \frac{87.5(15.33)}{1000} + 0.5^2/30 = 1.12 \text{ k/ft}$$

$$M_c = \frac{1.12(30)^2}{8} = 126 \text{ k} > 126 \text{ k} \Rightarrow \text{use } W12 \times 26 + M_p = 140 > 126 \therefore \text{ok}$$

check deflection

$$\Delta_{cons} = \frac{5wL^4}{384EI} = \frac{5(1.12)(30)^4}{384(29000)(804)} = 3.5''$$

$$\Delta_{init} = \frac{1}{360} = \frac{30(12)}{360} = 1 < 2.4 \therefore \text{no good} \Rightarrow I_{req'd} = 510 + \text{comber}$$
  
 use  $W18 \times 35 + M_p = 249 \quad I_x = 510 \Rightarrow \Delta_{LL} = 1.38'' > 1.0 \therefore \text{add } 1/2'' \text{ comber to beam}$

live load deflection

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(0.04)(15.33)(30)^4}{384(29000)(510)} = 0.657 < 1.0 = \frac{1}{360} \therefore \text{ok}$$

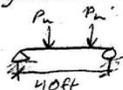
check shear

$$V_u = 16.6 \text{ k} \quad V_n = 26.6 < 159 = \phi V_n \therefore \text{ok}$$
  

$$\phi V_n = 159 \text{ k}$$

beams have same tributary widths but B2 has largest span therefore B2 design is conservative for all beams.

girder design



$$P_u = 16.6 + 14.11 = 30.71$$

$$M_u = \frac{P L}{3} = \frac{30.71(40)}{3} = 409.5 \text{ k}$$

$$V_u = P = 30.71 \text{ k}$$

AISC table 3-10

$$\therefore \text{unbraced length} = 13.33 \text{ ft} \Rightarrow \text{use } W21 \times 62 \quad \phi M_n = 414 \text{ k} > 409.5 \text{ k} \therefore \text{ok}$$
  

$$\phi V_n = 252 \text{ k} > 30.71 \text{ k} \therefore \text{ok}$$

check deflection

$$\Delta_{LL} = \frac{P_u L^3}{48EI} = \frac{30.71(40)^3(1728)}{48(29000)(1330)} = 3.14'' > 1.33 \therefore \text{not ok}$$

most economical  $W30 \times 90 \Rightarrow I = 3610 \Rightarrow \Delta_{LL} = 1.16'' < 1.33 \therefore \text{ok}$   
 more architectural (smaller depth)  $W21 \times 132 \Rightarrow I = 3220 \Rightarrow \Delta_{LL} = 1.20'' < 1.33 \therefore \text{ok}$