TECHNICAL REPORT II

University Academic Center

Eastern USA

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Structural Option

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Executive Summary

The goals of this report are to research and design three alternative floor framing systems for the University Academic Center and compare these systems along with the as-built system to determine overall feasibility. This was accomplished through analysis with current codes and standards, design catalogs, and computer analysis software.

The existing composite flooring system was analyzed in a typical 32'-8"x30' bay located in the central classroom wing, being the most common bay layout in the building. The analysis confirmed the use of a 2VLI18 deck type as specified in Vulcraft deck catalog equating to a 2" 18 gage composite deck with 3-1/4" LWC topping. The use of W18x40 beams with 24 studs and W21x73 girders with 34 studs was also confirmed. Computer analysis was also done in RAM to double check the beam/girder sizes. This system resulted in an overall depth of 26.25", and cost of $20.91 plus cambering estimated at $30 per member.

Alternative 1 was a non-composite system. Calculations resulted in an increase to 3" 20 gage non-composite deck with 4-1/2" LWC topping. Beam and girder sizes also increased to W18x55 and W24x68 respectively. This system resulted in an overall depth of 31.5", cost of $23.03, and a 134% original load applied to the foundations. The non-composite system was considered feasible for future design.

Alternative 2 was a precast concrete hollow core plank flooring system supported by steel beams. Design data for the hollow core planks came from Nitterhouse Concrete Products. To incorporate the planks the layout had to be altered to a 32'x30'. Design resulted in 10"x4' planks 30' in length with a 2" topping. A beam size of W27x94 was used to support the planks. This system resulted in an overall depth of 39", cost of $18.95, and a 199% original load applied to the foundation. The hollow core plank system was considered unsuitable for future investigation.

Alternative 3 was a two-way flat slab floor system with drop panels. Thickness of slab was chosen at 11" based on code minimum slab thickness. Drop panel dimensions were also determined to be 11'x10' based on ACI 318 code limitations and punching shear calculations. The remainder of the design was determined using spSlab. This system resulted in an overall depth of 15", cost of $17.72 and a 297% original load applied to the foundations. A two-way slab system was considered a possibly feasible alternative floor system.

Introduction

Located in the eastern United States, the University Academic Center is a 192,000 square foot building designed to house a library resource center, dining area, 45 classrooms, and over 120 offices. Other key features include a 5-story atrium and multiple roof gardens.

The layout of the building consists of three main sections. The northern 3-story section contains mostly dining and classroom areas. In the center of the building, a 4 story section houses the library and the majority of classrooms, as well as acting as the main entrance. The southern end of the building consists almost entirely of office spaces. On either side of the center section are the vertical circulation cores which also provide access to the roof gardens.

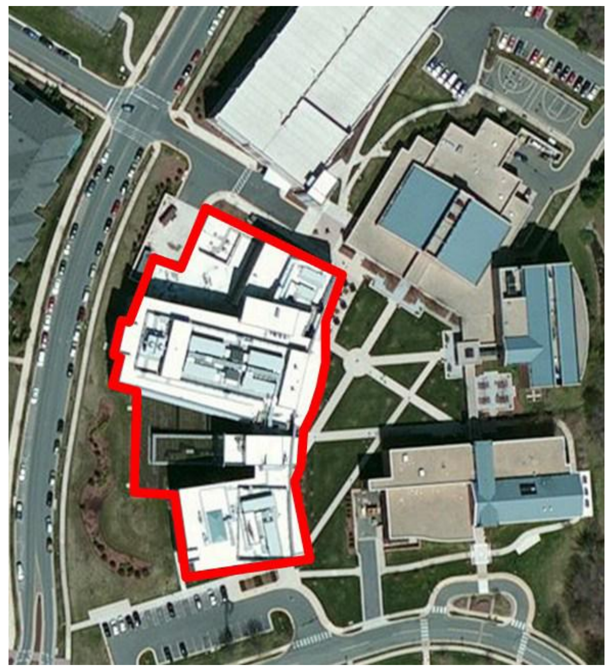


Photo taken from Bing Maps

There are 4 main types of building façade implemented in this building. The 3 and 5-story sections of the building have a brick façade with cast stone bands running horizontally across the brick surface. Glass curtain walls are used in the vertical circulation located on either side of the 4-story section. The 4-story section's façade is mostly metal panels. There is also glazed CMU used to accent the other façade types at various places.

Through the use of multiple energy saving techniques the University Academic Center holds a LEED gold rating. This includes energy efficient HVAC equipment and the use of natural daylighting, as well as shading devices, to help minimize energy consumption. All these features, along with the roof gardens, provide a "green" learning environment. LEED credits were also gained through site design to minimize storm water runoff, use of recyclable and local materials, and the addition of bike racks and on site showering facilities to promote alternative modes of transportation.

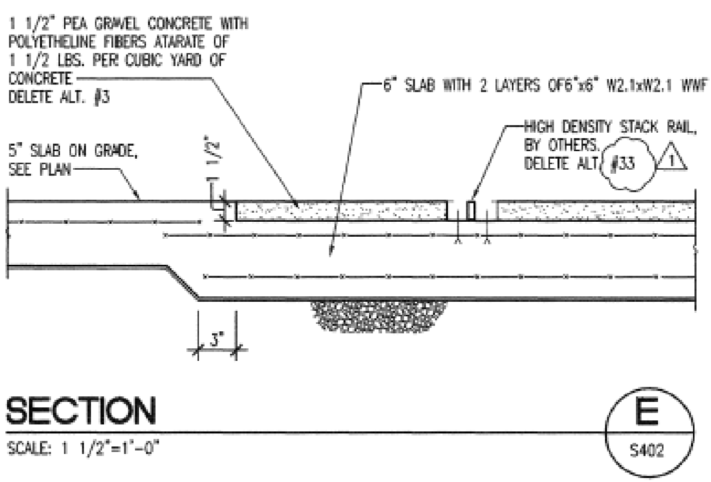
Structural Overview

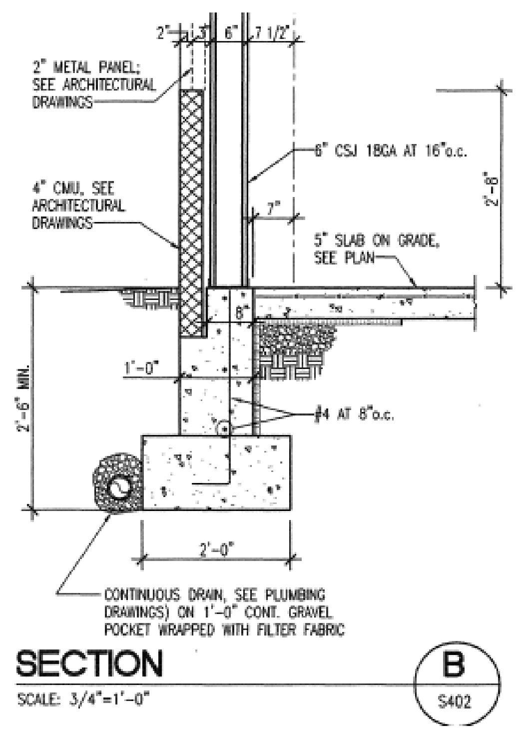
The University Academic Center is a steel framed building with composite metal decking supported by a foundation of spread footings and slab-on-grade. The building resists lateral forces by a combination of braced and moment frames.

Foundation

Based on the 2002 geotechnical report taken, footings for University Academic Center are designed for an allowable bearing capacity of 3000 psf. Footings are placed on undisturbed soil or on structurally compacted fill. The bottoms of exterior footings are 2'-6" below grade.

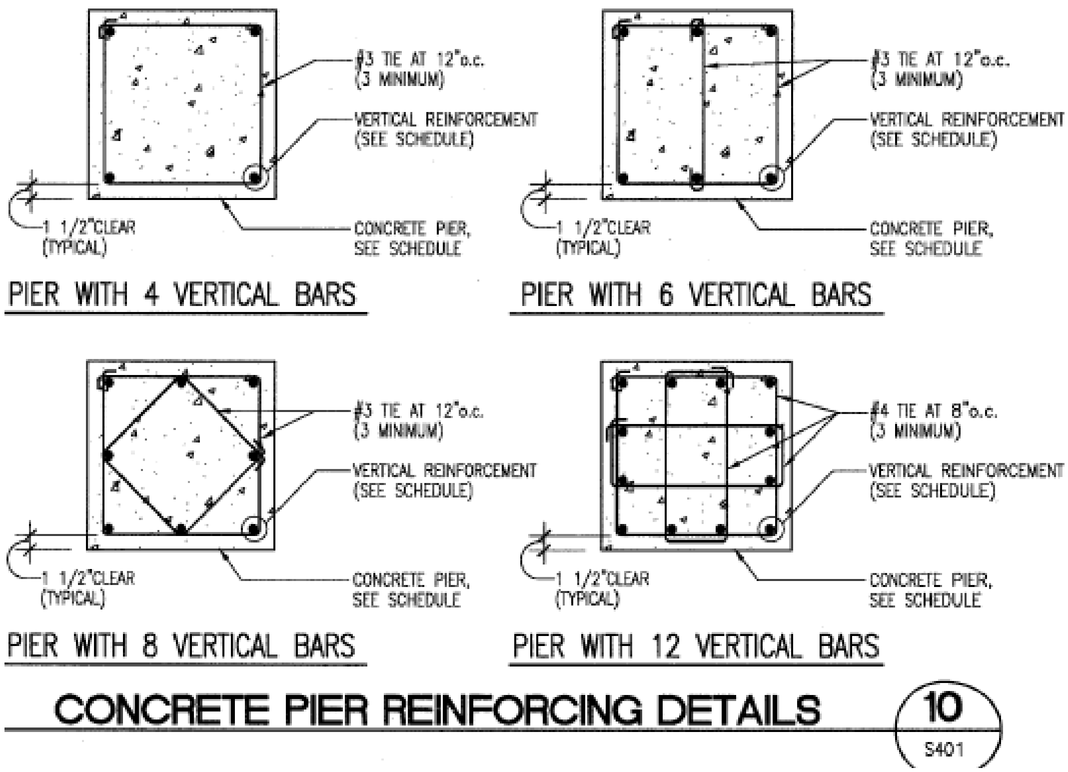
Slab-on-grade sits on a coarse granular fill material compacted to 95% of maximum density as defined by ASTM D1557 modified proctor test. The slab-on-grade is designed as 5" thick concrete reinforced with 6"x6", W1.4xW1.4 WWF. This is the reinforcement for all slab-on-grade except for the area located under the library stacks which is 6" thick concrete reinforced with 2 layers of 6"x6", W2.1xW2.1 WWF.





Drawings provided by Skanska

The columns in the University Academic Center sit on piers ranging in size depending on loading and connection type. The columns are embedded 8" in concrete then anchored to a base plate which sits on the pier. These piers are a minimum of 8" ranging to a maximum depth of 3'-9". The piers come in 4 types: 4, 6, 8, and 12 vertical bar piers. Footings also range in size under the columns with a maximum 19'x19' under a single column.

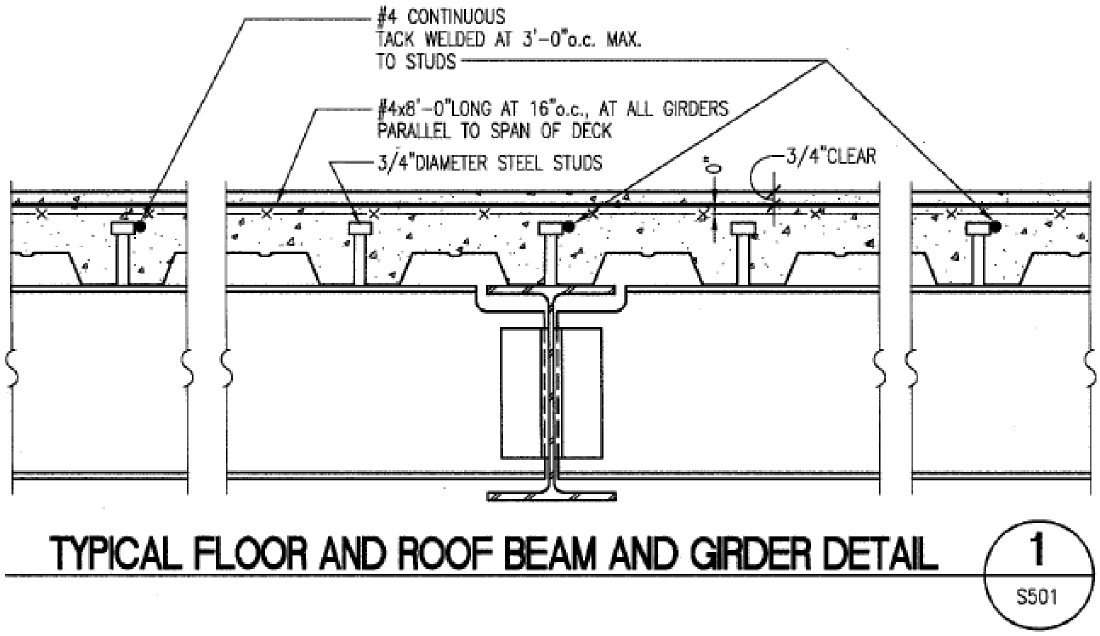


Drawings provided by Skanska

Floor and Roof Systems

The University Academic Center utilizes a composite metal deck flooring system. This includes 2" composite 20 gage deck with ribs 12" o.c. and 1.5" type B, wide rib 20 gage deck. All metal deck is designed to be continuous over 3 spans. Floor system also includes shear studs and lightweight concrete topping varying based on location and loading.

Roofing systems also varies due to some areas like the roof gardens and mechanical spaces of greater loading. Decking for roofs includes both 2" composite 18 gage deck with ribs 12" o.c. and 1.5" type B, wide rib 20 gage deck, covered by a built up roof and rigid insulation.



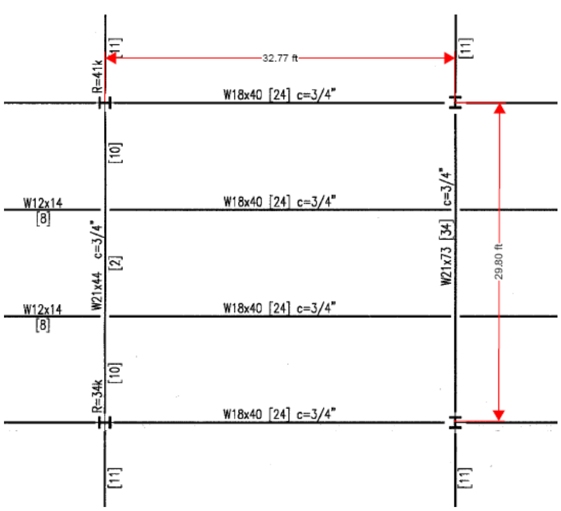
Drawings provided by Skanska

Framing System

The framing system for the University Academic Center includes C-shapes, HSS members, and Wide Flange members with the majority being W-shapes. Gridlines are set at multiple angles with bay sizes varying throughout the building. Areas with consistent framing between floors are located in the classroom wing in the central section of the building and the office spaces on the south side.

Lateral System

The lateral system for this building includes braced frames of varying heights and types located throughout the building. To the right is a plan view of University Academic Center with the 15 lateral braced frames shown in blue.



Drawings provided by Skanska

Resources:

As Designed Codes:

- 2000 ICC International Building Code
- 2000 ICC International Mechanical Code
- 2000 ICC International Plumbing Code
- 2000 ICC International Fuel-Gas Code
- 2000 ICC International Fire Code
- 2000 ICC International Energy Conservation Code
- 2000 NFPA Life Safety Code
- 2000 Americans with Disabilities Act – Accessibility Code
- 1999 National Electrical Code

Thesis Calculations:

- American Society of Civil Engineers (ASCE) 7-10
- AISC Steel Construction Manual, 14th Edition
- ACI 318-11
- Nitterhouse load tables
- Vulcraft deck catalog
- spSlab
- RAM
- RS Means Costworks data

Design Loads

Dead Loads

Dead loads are estimated based off material weights found in the AISC Steel Construction Manual since no values were given on drawings except for weights of rooftop units which range from 8,000-45,000 lbs. Deck weight is compared to similar weights in Vulcraft catalog based on topping thickness and deck type.

Dead Loads	
Description	Load (psf)
Framing	10
Superimposed DL	10
MEP	5
Composite Deck	
3.25” LCW topping	42
4.75” LCW topping	50
5” NWC topping	70
Roof Garden	80
Façade	
Brick	40
Glass	10
Metal Panel	15

Live loads

Live load values were given on the drawings. These values are shown along with the values given in ASCE7-10 in the table below. Where values are not given in one source the value from the other source was used in calculations. Likewise, when differing values are present the larger of the two was used in thesis calculations.

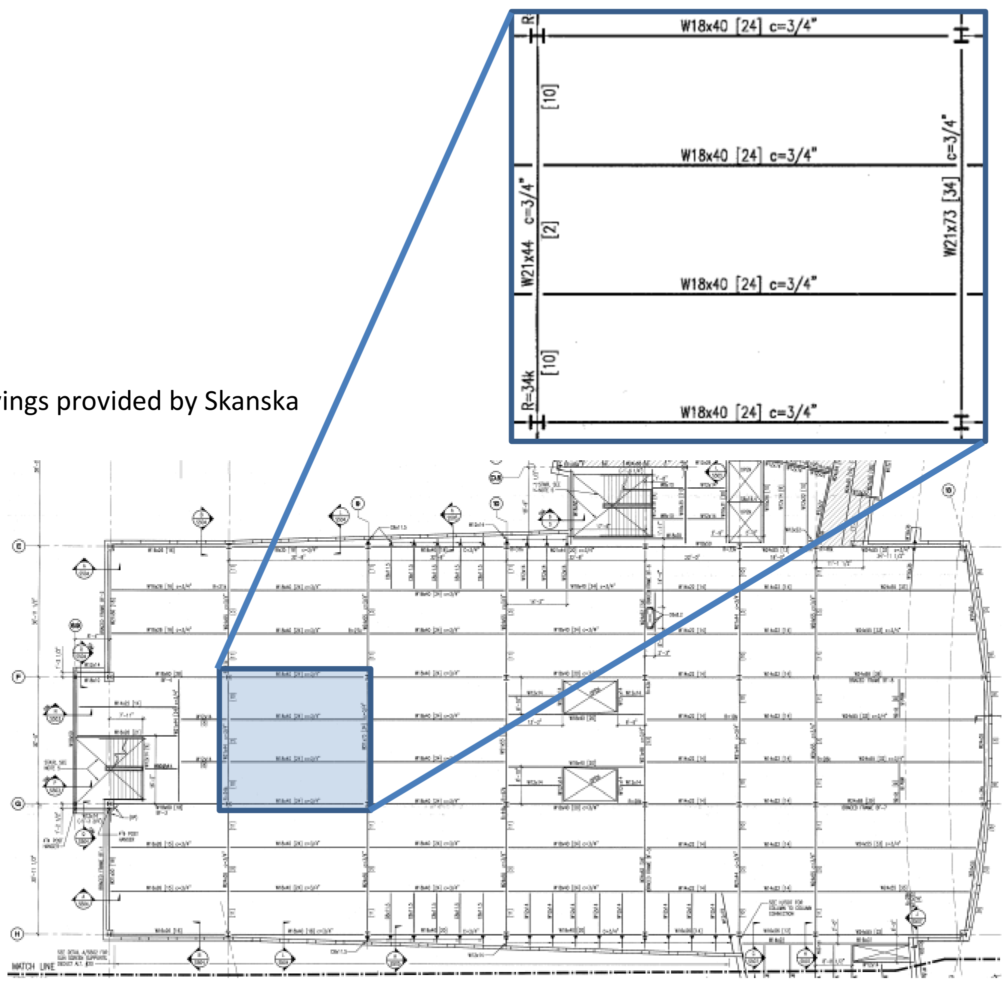
Live Loads		
Description	Designed Load (psf)	ASCE 7-10 Load (psf)
Slab on grade	100	100
Library slab on grade	150	150
Storage	125	125
Offices	50 + 20 (partition allowance)	50 + 15 (partition allowance)
Classrooms	50 + 20 (partition allowance)	50 + 15 (partition allowance)
Corridors (elevated floors)	80	80
Lobbies	100	100
Recreational areas	100	100
Mechanical/Electrical	125	N/A
Stairs	100	100
Chiller room	150 + equipment	N/A
Boiler room	200 + equipment	N/A
Roof	30	20
Roof Garden	N/A	100

Alternative Floor Systems:

Many factors go into a designers choice of building materials and layouts such as weight, cost, floor to floor heights, spans, deflections, and foundation considerations to name a few. When comparing alternative flooring systems these factors were weighed to determine feasibility. This report will compare the existing system and 3 alternative systems which include:

- Composite deck on composite steel beams and girders
- Non-composite deck on steel beams and girders
- Precast concrete hollowcore planks on steel beams
- Two-way slab with drop panels

Below is the 32'-8"x30' typical bay chosen for comparison purposes.



Drawings provided by Skanska

Composite Floor System

Advantages

Composite systems use the concrete in the deck to take compression loads while the steel takes the tension loads, this combined effort allows for smaller member sizes reducing weight and depth as well as deflections. The efficiency of composite systems also allow for increased bay sizes maximizing space useage. As with all steel systems, the composite system is lighter than a concrete system allowing for smaller foundations. Composite systems also have the option of cambering members, which is utilized in the University Academic Center, in order to further reduce deflections.

Disadvantages

A composite system can be costly with a higher level of difficulty in construction than other systems. This is because all the things that make composite more efficient cost money to imploy such as cambering beams and installing shear studs. All steel systems including composite must also be fireproofed and are typically hidden from view with a drop ceiling.

Analysis

The current composite system was analyzed using both hand calculations and RAM software to verify that member sizes pass all requirements, this can be found in Appendix A. This system resulted in an overall depth of 26.25" and a self weight of 48.23 psf. The system used cambering to keep deflections within limits but this also adds cost, about $30 per beam according to Erine Criste's article in STRUCTURE magazine, making the real cost per square foot higher than the one calculated from RS Means Costwork of $20.91. University Academic Center uses a drop ceiling to hide the composite system which must be fireproofed also adding cost.

Non-composite Floor System

Advantages

A non-composite system is a cheaper, faster, and less labor intensive alternative to the composite system. There are no shear studs to weld to members and therefore less opportunity for mistakes during installation. Non-composite systems share the weight advantages of a composite system as well as the reduction of space usage.

Disadvantages

Non-composite systems share some composite system disadvantages including the need for fireproofing and drop ceilings. They also are more prone to deflection and vibration issues due to their relatively flexible frames. The main disadvantage of a non-composite system is the reduced amount of strength when compared to composite systems. The inability to maximize the concrete's strength through composite action results in concrete simply acting as a load for the steel to bear. This results in larger and heavier members than those used in composite systems.

Analysis

The non-composite design uses the same bay layout as the existing composite system making comparisons more precise. Since a non-composite system does not use the concrete in the deck to take loads in flexure member sizes increased from W18x40 and W21x73 to W18x55 and W24x68. This increase in member size also increases overall depth to 31.5" and self weight to 64.58 psf.

The non-composite system is slightly worse than the existing system in weight, depth, and deflection but when the lower price of $23.03 per square foot and the ease of construction are factored in, this system may be considered feasible for redesign.

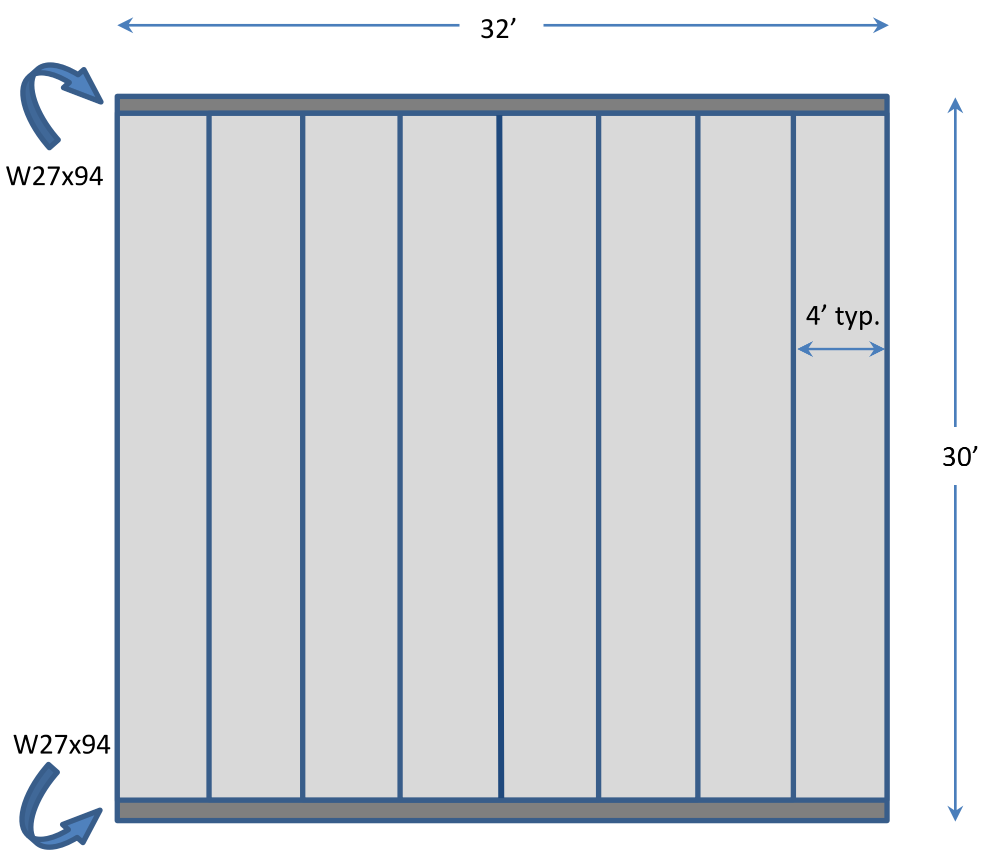
Precast Hollowcore Plank System

Advantages

Hollowcore planks provide a high level of consistancy as far as strength since they are pre-made. They also eliminate the waiting time to cure and allow for less time spent during construction and a easier construction. Hollowcore planks can eliminate the need for drop ceilings and, if coordinated properly, allow for MEP equipment to be installed in the voids in the planks keeping them out of sight. Hollowcore planks are also capable of spanning large distances increasing bay sizes and usable floor space.

Disadvantages

Although the planks themselves do not require fireproofing the steel beams supporting them will. These beams will also increase system depth and most likely utilize drop ceilings making system depth a major disadvanage to a hollowcore plank design. The pre-made planks also result in limited options reguarding layouts making this system more favored to rectilinear designs than complex geometries.



Analysis

Hollowcore planks were chosen from Nitterhouse Concrete Products. Based on their load tables and the loading in a typical classroom bay, a 10" plank with 2" topping for fire protection was chosen. Because of the modular size of 4' wide planks the bay size was slightly changed to 32'x30'. To support the planks a W27x94 girder was needed. This would put the overall depth at 39", the largest of the 4 systems, and the self weight at 95.94 psf, most likely requiring some sort of foundation redesign. Using RS Means Costworks data the price of a hollowcore system was estimated at $18.95 per square foot.

Hollowcore planks are cheaper, easier, and faster to install than the existing system. They also provide the option to incorporate MEP systems within the floor, but the added weight and depth of this system along with its inflexible layout options make hollowcore planks an undesirable option for the University Academic Center.

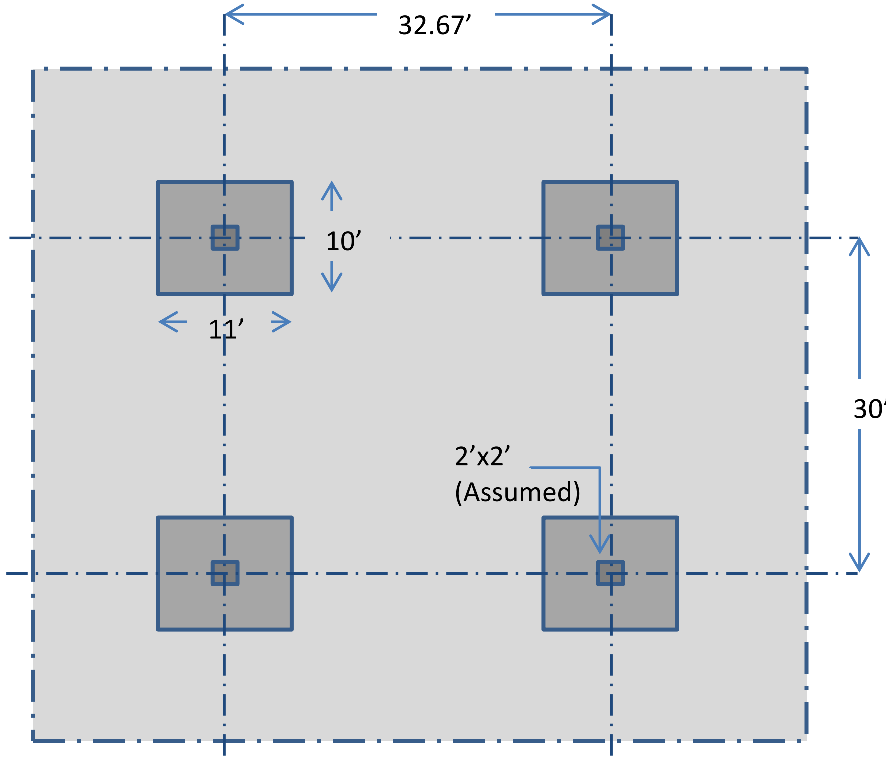
Two-way Slab with Drop Panels

Advantages

Using a concrete system like a two-way slab offers many advantages over steel systems. Cocrete is a cheap, continuous type of construction. It also acts as a natural fireproofing provided efficient clear cover and can be finished possibly eliminating the addition of drop ceilings. Two-way slabs with drop panels offer a unique advantage in that overall depth is significantly reduced allowing ample room for MEP needs and providing more options for floor heights. The added mass also reduces deflection and vibration concerns.

Disadvantages

One major disadvantage to a two-way slab is the added weight putting a larger stress on the foundation system most likely resulting in a redesign to handle the new loads. Other disadvantages include the time required to cure concrete, the cost of formwork, and the placing of rebar.



Analysis

Design of the two-way slab with drop panels was done using spSlab, however first minimum slab depth and drop panel dimensions were calculated by hand using ACI 318-11 limits. It was determined that a slab thickness of 11” should be used and drop panel dimensions of 11’x10’ with a thickness of 15” were needed to resist punching shear. Results from spSlab showed deflections far lower than all other systems averaging around 0.1 inch. Reinforcement, deflection, and layout diagrams can be found along with initial calculations in Appendix D. This system came out to be the heaviest by far at 143.23 psf self weight, putting almost 300% loading on the foundation. Price was estimated at $17.72 per square foot using the closest assembly in RS Means Costworks.

The two-way slab system has the most weight requiring a foundation redesign, as well as difficult and lengthy construction compared to the other systems. However, two-way slabs offer many advantages the steel systems do not including the lowest overall depth, best deflection/vibration control, and built in fireproofing with the possibility of eliminating drop ceilings. This along with the lowest estimated price, makes a two-way slab with drop panels a feasible alternative flooring system for the University Academic Center.

System Comparison

The table below shows a comparison of the 3 alternative systems compared to the existing composite system. Positive aspects when compared to the original system are shown in green while negative aspects are shown in red.

Floor System Summary				
Floor System	Composite	Non-composite	Precast Concrete Hollowcore Planks	Two-way Slab w/ Drop Panels
Bay size	32’-8”x30’	32’-8”x30’	32’x30’	32’-8”x30’
Slab Thickness **Overall Depth**	5.25” 26.25”	7.5” 31.5”	10”(+2” topping) 39”	11” 15”
Self Weight	48.23 psf	64.58 psf	95.94 psf	143.23 psf
Cost (per square foot)	$20.91 + camber (~$30/beam)	$23.03	$18.95	$17.72
Fire Rating **Additional Fire Protection**	2 Hr Yes (Structural Steel)	2 Hr Yes (Structural Steel)	2 Hr Yes (Structural Steel)	2 Hr No
Deflection/ Vibration	Poor	Poor	Poor	Good
Foundation Impact	None	Minimal (134% load)	Major (199% load + locations change)	Major (297% load)
Constructability	Average	Simple	Simple	Difficult
Feasability	Yes	Yes	No	Yes

Conclusion

Based on the study of the existing system and the advantages and disadvantages offered by each of the 3 alternative systems it has been concluded that all systems except for the precatst hollowcore planks can be considered for future study.

The existing composite system offered efficiency in its use of space and strength. A non-composite system, while not as strong as a composite system, required increased member sizes, but offered a cheaper and easier construction to keep it a feasible option. The two-way slab system offered a completely different type of appeal being the most efficient as far as price, depth, and stiffness were concerned. However it was the heaviest system causing foundation concerns. It still had enough positive aspects to make it worth further investigation.

The only system that was considered too impracticle was the precast hollowcore plank system. This system still offered long spans and was reasonably priced, but this system has both the undesirable depth of a steel system with the weight of a concrete one. This along with the limitations in layout due to the modular sized planks make a hollowcore system an unviable flooring option for University Academic Center.

Appendix A: Composite Calculations and RAM results

Existing : Composite | Tech 2 | Alexander Altemose

Loads

LL = 50 psf + 15 (partitions) = 65 psf

DL = 10 psf (framing)
+ 10 psf (super imposed)
+ 5 psf (MEP)
+ 42 psf (deck w/ LWC)
67 psf

Span = 10'

Existing Members

- 3 1/4" LWC topping 2" 18 ga composite deck (floor)
- W 18×40 [24] c = 3/4" (beam)
- W24×55 c = 3/4" (girder)

* 2-hr fire rating required *

Deck Check:

- 3 1/4" LWC topping
- 2" 18 gage composite deck
- 3 span = 10'
- Live load = 65 psf

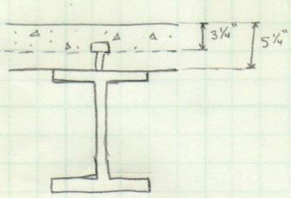
* From Vulcraft deck catalog

⟹ $\boxed{\text{2VLI18 deck type}}$

superimposed LL @ 10' = 205 psf > 65 psf ✓ OK
Max unshored clear span = 12'-7" > 10' ✓ OK
fire rating for unprotected deck = 2 hr ✓ OK

Beam Check:

$\boxed{\text{W18×40 [24] c = 3/4"}}$



Properties

f'c = 3 ksi
fy = 50 ksi

Span = 32'-8"
Spacing = 10'
⊥ deck
Studs = 3/4" dia, weak, 1 per rib

Beam dimensions:

$A = 11.8\ in^2$	$d = 17.9\ in$
$I = 612\ in^4$	$t_f = 0.525\ in$
$S = 68.4\ in^3$	$b_f = 6.02\ in$

Existing: Composite | Tech 2 | Alexander Altemose

Beam Check: (cont'd)

LL Reduction $\rightarrow$ $A_T = 10'(32.67') = 326.7\ ft^2$ $\rightarrow$ $653.4\ ft^2 > 400\ ft^2 \rightarrow$ Reduce
$K_{LL} = 2$

$$L = 65\left(0.25 + \frac{15}{\sqrt{653.4}}\right) = 54.4\ psf$$

$$w_u = 1.2(67) + 1.6(54.4) = 167.44\ psf$$

$$w_u = \frac{167.44(10')}{1000\ lb} = 1.67\ k/ft$$

$$M_u = \frac{1.67(32.67)^2}{8} = \boxed{223\ k\text{-}ft}$$

$$b_{eff} = \min \begin{cases} \frac{32.67'}{4} = 8.1675' \text{ or } 8'\text{-}2'' \leftarrow \text{controls} \\ 10' \end{cases}$$

PNA Location:

$$V_c' = 0.85(3\ ksi)(8.1675')(12'')(3.25'') = 812.2\ k$$

$$V_s' = 11.8(50) = 590\ k$$

$$V_q' = \Sigma Q_n = 24(17.2) = 412.8\ k \quad \leftarrow \text{controls} \therefore \text{partially composite}$$

$$a = \frac{412.8}{(.85)(3)(8.1675)(12)} = 1.65'' < 2'' \text{ deck } \checkmark \text{ok}$$

$$A_{s-c} = \frac{590 - 412.8}{2(50)} = 1.772\ in^2$$

$$x = \frac{A_{s-c}}{b_f} = \frac{1.772}{6.02} = 0.294\ in < t_f = 0.525\ in \rightarrow \text{PNA in flange}$$

$$M_n = 590\left(\frac{17.9}{2}\right) + 412.8\left(6.02 - \frac{1.65}{2}\right) - 2(50)(6.02)(0.294)\left(\frac{0.294}{2}\right) = \frac{7396.9}{12} = 616.6\ k\text{-}ft$$

$$\phi M_n = 0.9(616.6) = \boxed{554.92\ k\text{-}ft} > 223\ k\text{-}ft = M_u \quad \checkmark \text{ok}$$

Existing: Composite | Tech 2 | Alexander Altemose

Beam Check: (cont'd)

Deflections: $\Delta = \frac{5wL^4}{384EI_{LB}}$

$w_D = \frac{57(10)}{1000} = 0.57\ k/ft + 0.04\ k/ft$ (beam wt) $= 0.61\ k/ft$

$w_L = \frac{54.4(10)}{1000} = 0.544\ k/ft$

$w_{tot} = 1.2(0.61) + 1.6(0.544) = 1.602\ k/ft$

$$I_{LB} = I + A_s\left(\bar{y} - \frac{d}{2}\right)^2 + \frac{\Sigma Q_n}{f_y}\left(d + y_2 - \bar{y}\right)^2$$

$$\bar{y} = \frac{A_s\left(\frac{d}{2}\right) + \frac{\Sigma Q_n}{f_y}(d + y_2)}{A_s + \frac{\Sigma Q_n}{f_y}} = \frac{11.8\left(\frac{17.9}{2}\right) + \frac{412.8}{50}\left(17.9 + 5.25 - \frac{1.65}{2}\right)}{11.8 + \frac{412.8}{50}} = 14.46\text{ in}$$

$$I_{LB} = 612 + 11.8\left(14.46 - \frac{17.9}{2}\right)^2 + \frac{412.8}{50}\left(17.9 + 5.25 - \frac{1.65}{2} - 14.46\right)^2 = 1481\text{ in}^4$$

$$\Delta_{DL} = \frac{5(0.544)(32.67)^4(1728)}{384(29000)(1481)} = \boxed{0.325\text{ in}}$$

$$\Delta_{LL} = \frac{5(0.65)(32.67)^4(1728)}{384(29000)(1481)} = \boxed{0.388\text{ in}} < \Delta_{LL\ allow} = \frac{L}{360} = \frac{32.67(12)}{360} = 1.089\text{ in} \checkmark_{OK}$$

$$\Delta_{TL} = \frac{5(1.602)(32.67)^4(1728)}{384(29000)(1481)} = \boxed{0.956\text{ in}} < \Delta_{TL\ allow} = \frac{L}{240} = \frac{32.67(12)}{240} = 1.634\text{ in} \checkmark_{OK}$$

Shored vs Unshored

$\phi M_p = 294$ k-ft

$w_u = 1.2(0.61) + 1.6\frac{(20)(10)}{1000} = 1.052\ k/ft$

$M_u = \frac{1.052(32.67)^2}{8} = 140.4$ k-ft < 294 k-ft $= \phi M_p$ ✓ Unshored OK

$\boxed{W18\times40\ [24]\ c = 3/4"\ \text{is OK}}$

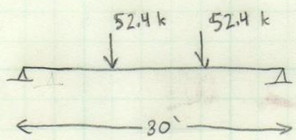
Existing : Composite | Tech 2 | Alexander Altemose

Girder Check :

$W_{TOT} = 1.602 \text{ k/ft}$

$\boxed{W21\times73 \ [34] \ c = 3/4''}$

$P = 1.602\,(32.67') = 52.4 \text{ k}$



Beam Properties :

$A = 21.5 \text{ in}^2 \quad d = 21.2 \text{ in}$

$I = 1600 \text{ in}^4 \quad t_f = 0.740 \text{ in}$

$S = 151 \text{ in}^4 \quad b_f = 8.30 \text{ in}$

$M_U = P_a = 52.4(10') + \frac{0.073(30)^2}{8} = \boxed{532.2 \text{ k}}$

PNA Location :

$b_{eff} = \min \begin{cases} \frac{30'}{4} = 7.5' \leftarrow \text{controls} \\ 32.67' \end{cases}$

$V_c' = 0.85(3)(7.5')(12'')(3.25'') = 745.9 \text{ k}$

$V_s' = 21.5(50) = 1075 \text{ k}$

$V_q' = \Sigma Q_n = 34(17.2) = 584.8 \text{ k} \longleftarrow$ controls $\therefore$ partially composite

$a = \frac{584.8}{0.85(3)(7.5)(12)} = 2.55 \text{ in}$

$A_{s-c} = \frac{1075 - 584.8}{2(50)} = 4.902 \text{ in}^2$

$x = \frac{4.902}{8.30} = 0.591 \text{ in} < t_f = 0.740 \text{ in} \longrightarrow$ PNA in flange

$M_n = 1075\left(\frac{21.5}{2}\right) + 584.8\left(8.3 - \frac{2.55}{2}\right) - 2(50)(8.3)(0.591)\left(\frac{0.591}{2}\right) = \frac{15519.5}{12} = 1293.3 \text{ k-ft}$

$\phi M_n = 0.9(1293.3) = \boxed{1164 \text{ k-ft}} > 532.2 \text{ k-ft} = M_U \quad \checkmark \text{OK}$

Existing : Composite | Tech 2 | Alexander Altemose

Girder Check : (cont'd)

Deflections : $\Delta = \dfrac{PL^3}{28 E I_{LB}} + \dfrac{5 W_{GIRDER} L^4}{384 E I_{LB}}$

$$\bar{y} = \frac{21.5\left(\frac{21.2}{2}\right) + \frac{584.8}{50}\left(21.2 + 8.3 - \frac{2.55}{2}\right)}{21.5 + \frac{584.8}{50}} = 16.81 \text{ in}$$

$$I_{LB} = 1600 + 21.5\left(16.81 - \frac{21.2}{2}\right)^2 + \frac{584.8}{50}\left(21.2 + 8.3 - \frac{2.55}{2} - 16.81\right)^2 = 3953 \text{ in}^4$$

$P_{DL} = w_{DL}(32.67') = 0.61(32.67) = 19.93$ k

$P_{LL} = w_{LL}(32.67') = 0.65(32.67) = 21.24$ k

$P_{TL} = w_{TL}(32.67') = 1.772(32.67) = 57.9$ k

$w_{GIRDER} = 0.073$ k/ft

$$\Delta_{DL} = \frac{19.93(30')^3(1728)}{28(29000)(3953)} + \frac{5(0.073)(30)^4(1728)}{384(29000)(3953)} = \boxed{0.301 \text{ in}}$$

$$\Delta_{LL} = \frac{21.24(30)^3(1728)}{28(29000)(3953)} = \boxed{0.309 \text{ in}} < \Delta_{LL\ ALLOW} = \frac{L}{360} = \frac{30(12)}{360} = 1.00 \text{ in} \quad \checkmark \text{OK}$$

$$\Delta_{TL} = \frac{57.9(30)^3(1728)}{28(29000)(3953)} + \frac{5(0.073)(30)^4(1728)}{384(29000)(3953)} = \boxed{0.853 \text{ in}} < \Delta_{TL\ ALLOW} = \frac{L}{240} = \frac{30(12)}{240} = 1.50 \text{ in} \quad \checkmark \text{OK}$$

Shored vs Unshored

$\phi M_p = 645$ k-ft

$W_U = 1.2(0.61) + 1.6\dfrac{(20)(10)}{1000} = 1.052$ k/ft

$P_U = 1.052 \text{ k/ft}\,(32.67) = 34.37$ k

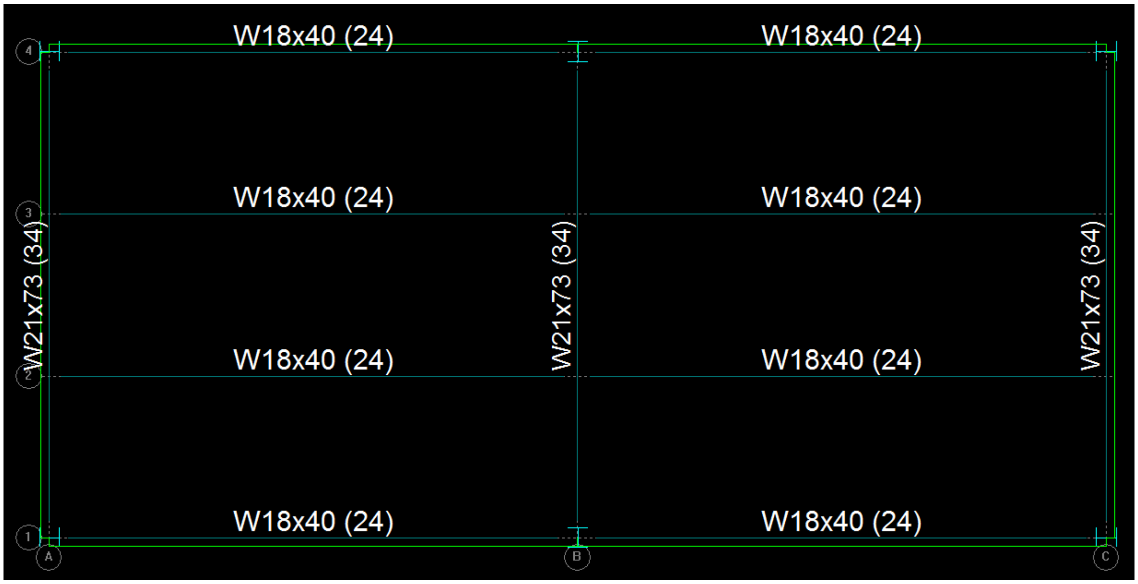
$M_U = P_U a + \dfrac{w_{GIRDER} L^2}{8} = 34.37(10) + \dfrac{0.073(30)^2}{8} =$

$M_U = 351.9 \text{ k-ft} < 645 \text{ k-ft} = \phi M_p$ ✓ Unshored OK

$\boxed{\text{W21×73 [34] } c = 3/4'' \text{ is OK}}$

* Note: combined deflections from beam & girder will be addressed through camber to meet requirements

RAM Results


W18x40 (24)
W18x40 (24)
W18x40 (24)
W18x40 (24)
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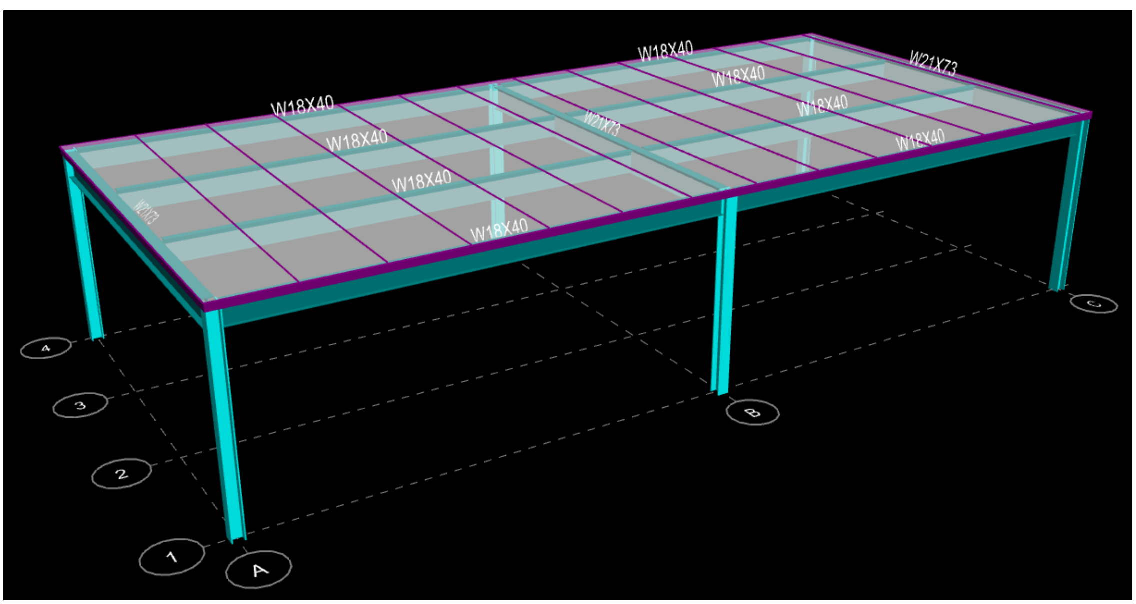
W18x40 (24)
W18x40 (24)
W18x40 (24)
W21x73 (34)
W21x73 (34)
W21x73 (34)
4
3
2
1
A
B
C


W18X40
W18X40
W18X40
W18X40
W18X40
W18X40
W18X40
W18X40
W21X73
W21X73
4
3
2
1
A
B
C

Gravity Beam Design

RAM Steel v14.04.07.00
University Academic Center
DataBase: University Academic Center 10/11/12 16:23:42
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical Bay Beam Number = 7

SPAN INFORMATION (ft): I-End (0.00,20.00) J-End (32.67,20.00)

Beam Size (User Selected) = W18X40 Fy = 50.0 ksi
Total Beam Length (ft) = 32.67

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 98.00	Y bar(in)	=	17.44
Mnf (kip-ft)	= 640.13	Mn (kip-ft)	=	509.16
C (kips)	= 206.76	PNA (in)	=	15.51
Ieff (in4)	= 1370.09	Itr (in4)	=	1892.62
Stud length (in)	= 4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
\# of studs: Max = 64 Partial = 18 Actual = 24
Number of Stud Rows = 1 Percent of Full Composite Action = 35.04

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.433	0.433	0.000	---	NonR	0.000	0.000
	32.666	0.433	0.433	0.000			0.000	0.000
2	0.000	0.250	0.000	0.500	16.3%	Red	0.150	0.200
	32.666	0.250	0.000	0.500			0.150	0.200
3	0.000	0.040	0.040	0.000	---	NonR	0.000	0.000
	32.666	0.040	0.040	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 29.04 kips 1.00Vn = 169.15 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	118.5	16.3	0.0	1.00	0.90	294.00
	Init DL	1.4DL	88.4	16.3	---	---		
	Max +	1.2DL+1.6LL	237.1	16.3	---	---	0.90	458.24
Controlling		1.2DL+1.6LL	237.1	16.3	---	---	0.90	458.24

REACTIONS (kips):

	Left	Right
Initial reaction	11.00	11.00
DL reaction	11.82	11.82
Max +LL reaction	9.28	9.28
Max +total reaction (factored)	29.04	29.04

DEFLECTIONS:

Initial load (in)	at	16.33 ft =	-0.684	L/D = 573
Live load (in)	at	16.33 ft =	-0.367	L/D = 1069
Post Comp load (in)	at	16.33 ft =	-0.528	L/D = 743
Net Total load (in)	at	16.33 ft =	-1.211	L/D = 324

Gravity Beam Design

RAM Steel v14.04.07.00
University Academic Center
DataBase: University Academic Center — 10/11/12 16:23:42
Building Code: IBC — Steel Code: AISC 360-10 LRFD

Floor Type: Typical Bay **Beam Number = 5**

SPAN INFORMATION (ft): I-End (32.67,0.00) J-End (32.67,30.00)

Beam Size (User Selected) = W21X73 Fy = 50.0 ksi
Total Beam Length (ft) = 30.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 90.00		Y bar(in)	=	17.87
Mnf (kip-ft)	= 1169.57		Mn (kip-ft)	=	1016.19
C (kips)	= 300.60		PNA (in)	=	17.21
Ieff (in4)	= 3024.37		Itr (in4)	=	3843.66
Stud length (in)	= 4.00		Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.7 Rg = 1.00 Rp = 0.75
\# of studs: Full = 126 Partial = 32 Actual = 34
Number of Stud Rows = 1 Percent of Full Composite Action = 27.11

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
10.000	11.82	7.73	8.17	33.5	0.00	0.00	0.0	0.00	Snow	2.45	3.27
10.000	11.82	7.73	8.17	33.5	0.00	0.00	0.0	0.00	Snow	2.45	3.27
20.000	11.82	7.73	8.17	33.5	0.00	0.00	0.0	0.00	Snow	2.45	3.27
20.000	11.82	7.73	8.17	33.5	0.00	0.00	0.0	0.00	Snow	2.45	3.27

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.073	0.073	0.000	---	NonR	0.000	0.000
	30.000	0.073	0.073	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 54.90 kips 1.00Vn = 289.38 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	300.0	15.0	10.0	1.00	0.90	575.57
	Init DL	1.4DL	228.1	15.0	---	---		
	Max +	1.2DL+1.6LL	545.7	15.0	---	---	0.90	914.57
Controlling		1.2DL+1.6LL	545.7	15.0	---	---	0.90	914.57

REACTIONS (kips):

	Left	Right
Initial reaction	23.10	23.10
DL reaction	24.73	24.73
Max +LL reaction	15.76	15.76

DEFLECTIONS:

Initial load (in)	at	15.00 ft =	-0.581	L/D =	620
Live load (in)	at	15.00 ft =	-0.298	L/D =	1210
Post Comp load (in)	at	15.00 ft =	-0.452	L/D =	797
Net Total load (in)	at	15.00 ft =	-1.033	L/D =	349

Appendix B: Non-composite Calculations

Alternative 1: Non-Composite | Tech 2 | Alexander Altemose

Deck Design:

Loads

LL = 50 psf + 15 psf (partitions) = 65 psf

DL = 10 psf (superimposed DL)
5 psf (MEP)
15 psf

TL = 80 psf

Additional Design Criteria

span = 10 ft

2hr fire rating → topping ≥ 2 ½"

keep LWC as in existing

keep existing beam/girder layout

TRY 3C Conform decking

Reinf. → clear span = 10' → TRY 7.5" thickness, 4.5" topping → 4×4-W2.9×W2.9

allowable load = 134 psf > 80 psf ✓ok

7.5", 4.5" topping → 3 span = 12'-2 > 10' ✓ok → TRY 3C20 weight = 57 psf

3C20 → $\ell/240$ allowable uniform load = 114 psf > LL = 65 psf ✓ok

→ $\ell/180$ allowable uniform load = 152 psf > TL = 80 + 57 = 137 psf ✓ok

USE 3C 20 Non-composite deck 4.5" LWC topping, 7.5" total
w/ 4×4 - W2.9×W2.9 WWF reinforcing

Alternative 1: Non-Composite | Tech 2 | Alexander Altemose

Beam Design:

$DL = 15\ psf + 57\ psf + 5\ psf\ (\text{selfweight}) = 77\ psf$

$LL = 65\ psf \rightarrow LL\ \text{reduction} \rightarrow LL = 54.4\ psf$

$w_u = 1.2\ \frac{(77)(10)}{1000} + 1.6\ \frac{(54.4)(10)}{1000} = 1.794\ k/ft$

$V_u = \frac{w_u L}{2} = \frac{1.794\,(32.67)}{2} = 29.3\ k$

$M_u = \frac{w_u L^2}{8} = \frac{1.794\,(32.67)^2}{8} = 239.3\ k\text{-}ft$

TRY W18×40

$\phi M_{px} = 294\ k\text{-}ft > M_u$ ✓ok

$\phi V_n = 169\ k > V_u$ ✓ok

$I_x = 612\ in^4$

$w_{TL} = 72 + 54.4\ (10) + 40 = 1.304\ k/ft$

$\Delta_{LL} = \frac{5\,(0.544)(32.67)^4(1728)}{384\,(29000)(612)} = 0.786\ in$

$\Delta_{Allow} = \frac{L}{360} = \frac{32.67(10)}{360} = 0.907\ in > 0.786\ in$ ✓ok

$\Delta_{TL} = \frac{5\,(1.304)(32.67)^4(1728)}{384\,(29000)(612)} = 1.883\ in$

$\Delta_{Allow} = \frac{L}{240} = \frac{32.67(10)}{240} = 1.361\ in < 1.883\ in$ ✗ No Good

$I_x\ \text{required} = \frac{5\,(1.304)(32.67)^4(1728)}{384\,(29000)\,(1.361)} = 847\ in^4$

TRY W18×55

$I_x = 890\ in^4$

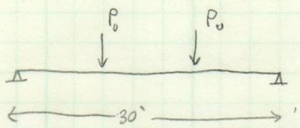
$\Delta_{TL} = \frac{5\,(1.304)(32.67)^4(1728)}{384\,(29000)(890)} = 1.300\ in < 1.361\ in$ ✓ok

$\boxed{\text{USE W18×55 for beams}}$

Alternative 1 : Non-Composite | Tech 2 | Alexander Altemose

Girder Design :

$$w_u = 1.2\left(\frac{72(10)+55}{1000}\right) + 1.6\left(\frac{54.4(10)}{1000}\right) = 1.80\ k/ft$$



$P_u = 1.8(32.67) = 58.8\ k$

$V_u = 58.8\ k$

$M_u = P_u a = 58.8(10) = 588.2\ k\text{-}ft$

TRY W 21×73

$\phi M_{px} = 645\ k\text{-}ft > M_u$ ✓ok

$\phi V_n = 289\ k > V_u$ ✓ok

$I_x = 1600\ in^4$

$$P_{LL} = \frac{54.4(10)(32.67)}{1000} = 17.8\ k$$

$$\Delta_{LL} = \frac{17.8(30)^3(1728)}{28(29000)(1600)} = 0.639\ in$$

$$\Delta_{ALLOW} = \frac{L}{360} = \frac{30(12)}{360} = 1.00\ in > 0.639\ in$$ ✓ok

$$P_{TL} = \left(\frac{72(10)+73}{1000}\right)(32.67) + \frac{54.4(10)(32.67)}{1000} = 43.7\ k$$

$$w_{GIRDER} = \frac{73}{1000} = 0.073\ k/ft$$

$$\Delta_{TL} = \frac{43.7(30)^3(1728)}{28(29000)(1600)} + \frac{5(0.073)(30)^4(1728)}{384(29000)(1600)} = 1.598\ in$$

$$\Delta_{ALLOW} = \frac{L}{240} = \frac{30(12)}{240} = 1.500\ in < 1.598\ in$$ X No Good

TRY W24×68

$I_x = 1830\ in^4$

$$\Delta_{TL} = \frac{43.7(30)^3(1728)}{28(29000)(1830)} + \frac{5(0.068)(30)^4(1728)}{384(29000)(1830)} = 1.395\ in < 1.500\ in$$ ✓ok

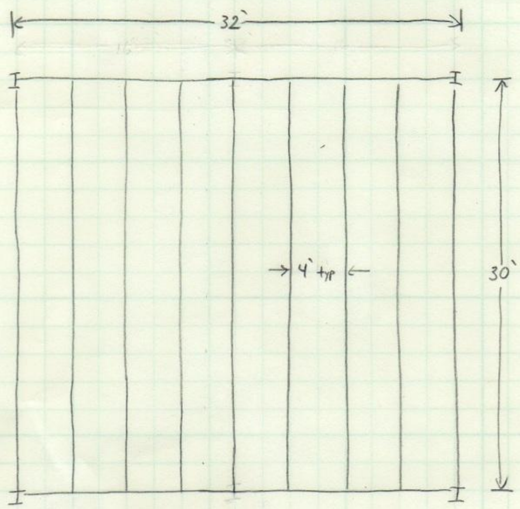
$\boxed{\text{USE W24×68 for girders}}$

Appendix C: Hollowcore Plank Calculations

Alternative 2 : Hollow Core Planks | Tech 2 | Alexander Altemose

Hollow core Precast Planks w/ steel framing

[Nitterhouse Concrete Products]



→ Bay size altered from 32`-8" × 30` to 32` × 30` to fit 4' hollow core planks

Loads :

DL = 20 psf

LL = 65 psf

TRY 8" × 4` Hollow Core Plank w/ 2" topping

→ 2 Hr Fire rating ✓ok

→ wt. = 61.25 psf + 25 psf (topping)

superimposed TL = 1.2(20) + 1.6(65) = 128 psf > 114 psf for 7-½" ϕ ✗ No Good

TRY 10" × 4` Hollow Core Plank w/ 2" topping

→ 2 Hr Fire Rating ✓ok

→ wt. = 68 psf + 25 psf (topping)

Superimposed TL = 1.2 (20) + 1.6 (65) = 128 psf < 162 psf ✓ok

USE 10" × 4` Hollow Core Plank
2" topping
7- ½" ϕ strand pattern

Alternative 2 : Hollow Core Plank | Tech 2 | Alexander Altemose

Beam Design :

$$w_u = 1.2\left(\frac{20+68+25}{1000}\right)(30) + 1.6\left(\frac{65}{1000}\right)(30) = 7.188\ k/ft$$

$$V_u = 7.188(32) = 230\ k$$

$$M_u = \frac{7.188(32)^2}{8} = 920\ k\text{-}ft$$

TRY W27 × 94

$$\phi M_{px} = 1040\ k\text{-}ft > 920\ k\text{-}ft \quad \checkmark \text{ok}$$

$$\phi V_n = 395\ k > 230\ k\text{-}ft \quad \checkmark \text{ok}$$

$$I_x = 3270\ in^4$$

Check deflections :

$$\Delta_{LL} = \frac{5(1.95)(32)^4(1728)}{384(29000)(3270)} = 0.485\ in$$

$$\Delta_{ALLOW} = \frac{L}{360} = \frac{32(12)}{360} = 1.067\ in > 0.485\ in \quad \checkmark \text{ok}$$

$$\Delta_{TL} = \frac{5(5.34)(32)^4(1728)}{384(29000)(3270)} = 1.329\ in$$

$$\Delta_{ALLOW} = \frac{L}{240} = \frac{32(12)}{240} = 1.600\ in > 1.329\ in \quad \checkmark \text{ok}$$

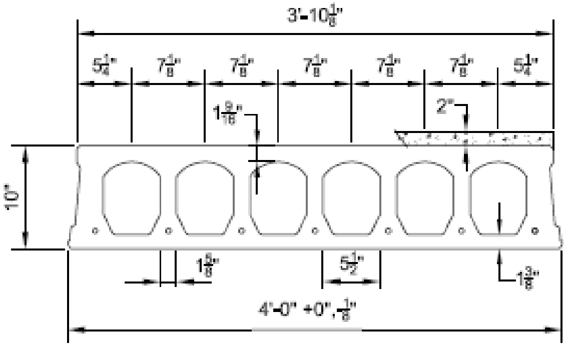
$$\boxed{\text{USE W27} \times \text{94 for beam}}$$

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 b_w = 13.13 in.
I_c = 5102 in.^{4} Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in. Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in. Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in. Precast Wt. = 272 PLF
Precast Wt. = 68.00 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 and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 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)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

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

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

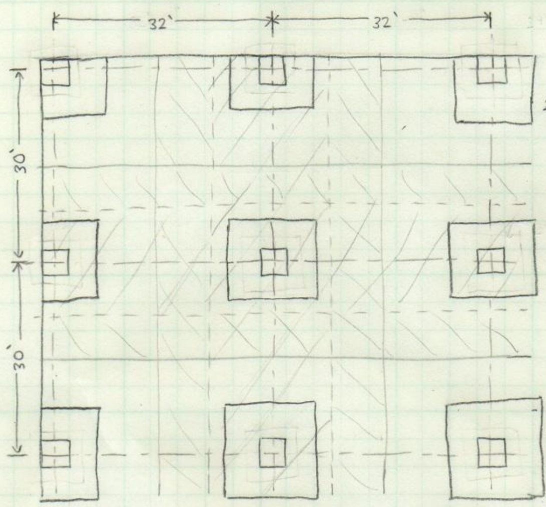
11/03/08

10F2.0T

Appendix D: Two-way Slab Calculations and spSlab Results

Alternative 3: Two-way Slab | Tech 2 | Alexander Altemose

Two-Way Slab w/ drop panels



24"×24" typ. columns (assumed)

Assumptions

- f_y (reinf) = 60 ksi
- f'_c = 3 ksi
- NWC
- 24"×24" columns
- #5 rebar size

Table 9.5(c) Min Thickness of Slab w/o int beams → exterior panels → $f_y = 60{,}000$ psi

$$\frac{l_n}{33} = \frac{(32' - 2')12}{33} = 10.9" \cong \boxed{11"}$$

$$d = 11" - 0.3125 - (0.5 + 0.75) = 9.4375" \cong \boxed{9"}$$

$$cc = 11 - 9 - 0.3125 = 1.6875" > (0.75 + 0.5) = 1.25"$$

for 2hr fire protection

$$\text{Slab wt} = \frac{11"}{12"}(150\text{ pcf}) = 137.5\text{ psf}$$

$$SI_{DL} = 20\text{ psf}$$

$$LL = 65\text{ psf}$$

$$q_o = 1.2(20 + 137.5) + 1.6(65) = 293\text{ psf}$$

$$\boxed{\text{USE Slab Thickness} = 11"}$$

Alternative 3 : Two-way Slab | Tech 2 | Alexander Altemose

Determine Drop panel dimensions thru punching shear

$V_u = 0.293\,(32(30) - 4) = 280.1\text{ k}$

$\phi V_c > V_u \quad , \quad V_c = 4\lambda\sqrt{f'c}\,b_o d$

min dimension $\rightarrow \frac{\ell}{6} = \frac{32'}{6} = 5'\text{-}4'' \; , \; \frac{30'}{6} = 5' \longrightarrow$ TRY $11' \times 10'$ drop panel

$280.1\text{ k} < 0.75\;4\,(1)\sqrt{3000}\,(538'')(8.5'')$

$280.1\text{ k} < 751.4\text{ k}$ ✓OK $\boxed{\text{USE } 11' \times 10' \text{ drop panel}}$

Assume drop panel $h_d = 1.25\,h = 13.75'' \longrightarrow$ TRY $h_d = 14''$

$q_{dp} = 1.2\left(\frac{3''}{12''}\right)(150\text{ pcf}) = 45\text{ psf}$

$V_u = 0.293\,(32(30)-4) + 0.045\,(110-4) = 284.9\text{ k}$

$b_o = (12 + 24)4 = 144$

$\phi V_c = 0.75\,(4)(1)\sqrt{3000}\,(144)(12) = 283.9\text{ k} < 284.9\text{ k}$ ✗ No Good

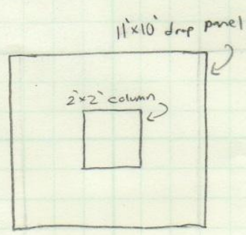
TRY $h_d = 15''$

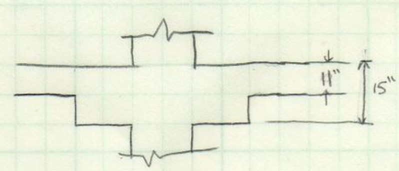
$q_{dP} = 1.2\left(\frac{4''}{12''}\right)(150\text{ pcf}) = 60\text{ psf}$

$V_u = 0.293\,(32(30)-4) + 0.06\,(110-4) = 286.5\text{ k}$ $\qquad b_o = (13 + 24)4 = 148''$

$\phi V_c = 0.75(4)(1)\sqrt{3000}\,(148'')(13'') = 316.1\text{ k} > 286.5\text{ k}$ ✓OK

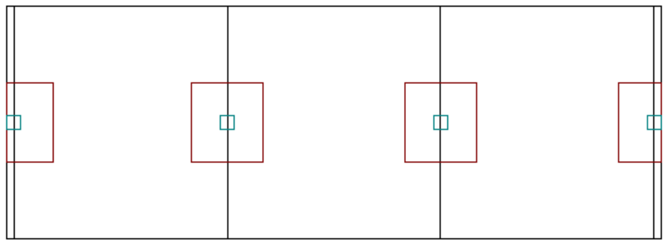
$\boxed{\text{USE } 11' \times 10' \text{ drop panels w/ } h_d = 15''}$

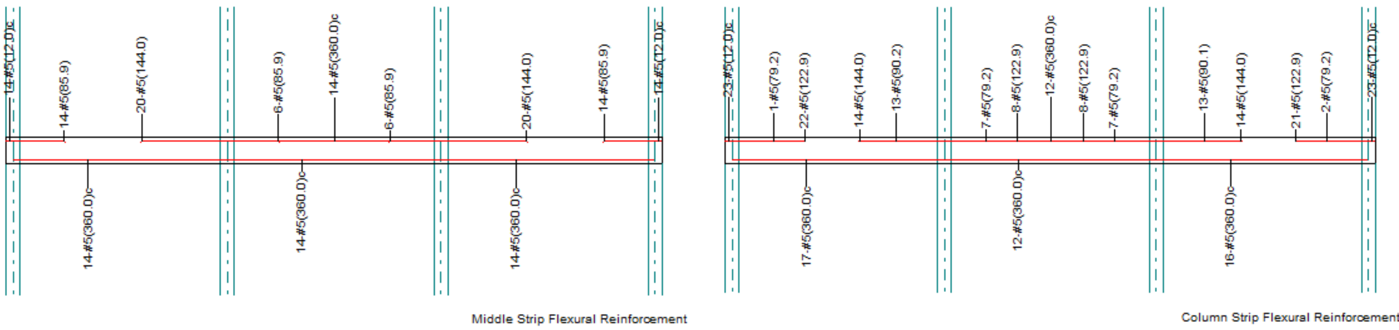


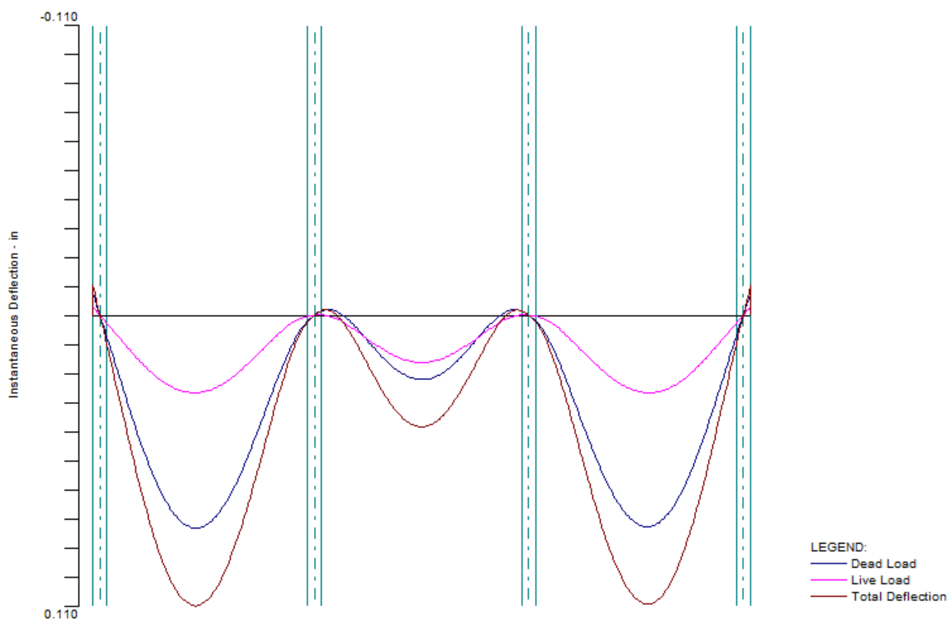


[Knowing dimensions for floor system, design of reinforcement was done in spSlab]

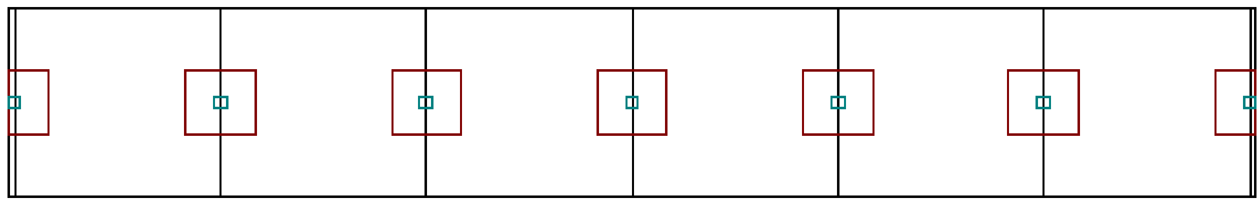
N-S Analysis with spSlab

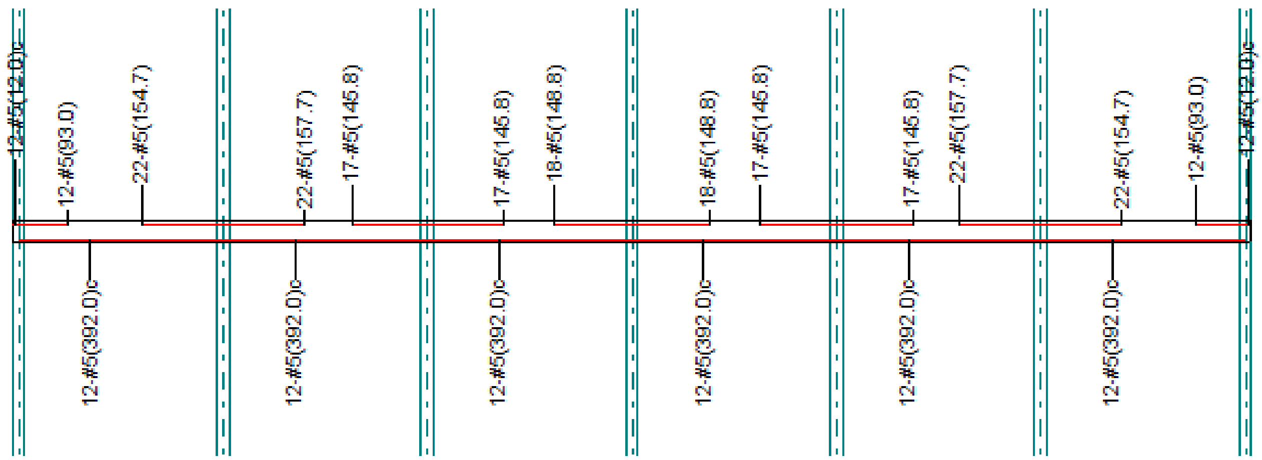




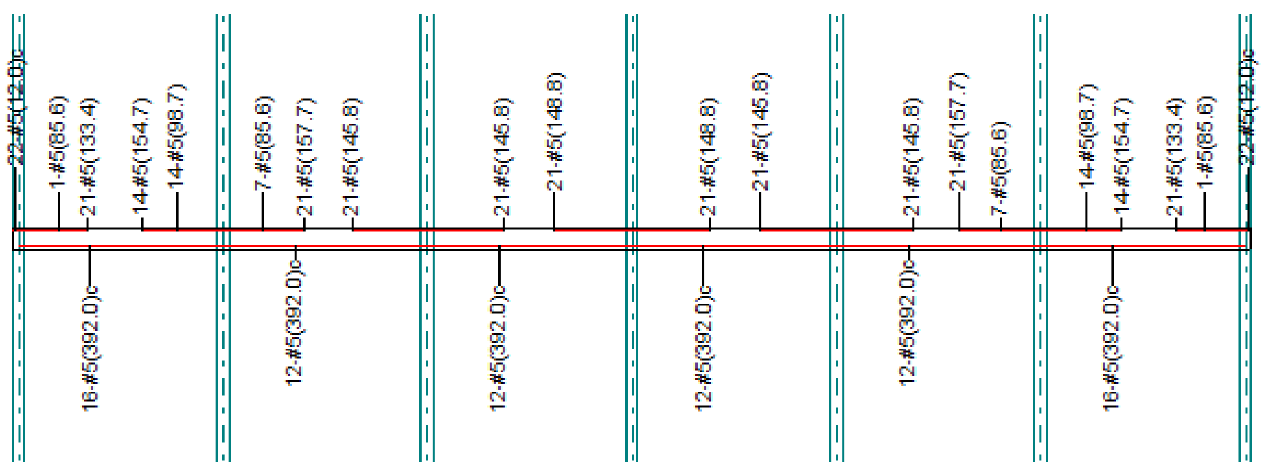


E-W Analysis in spSlab

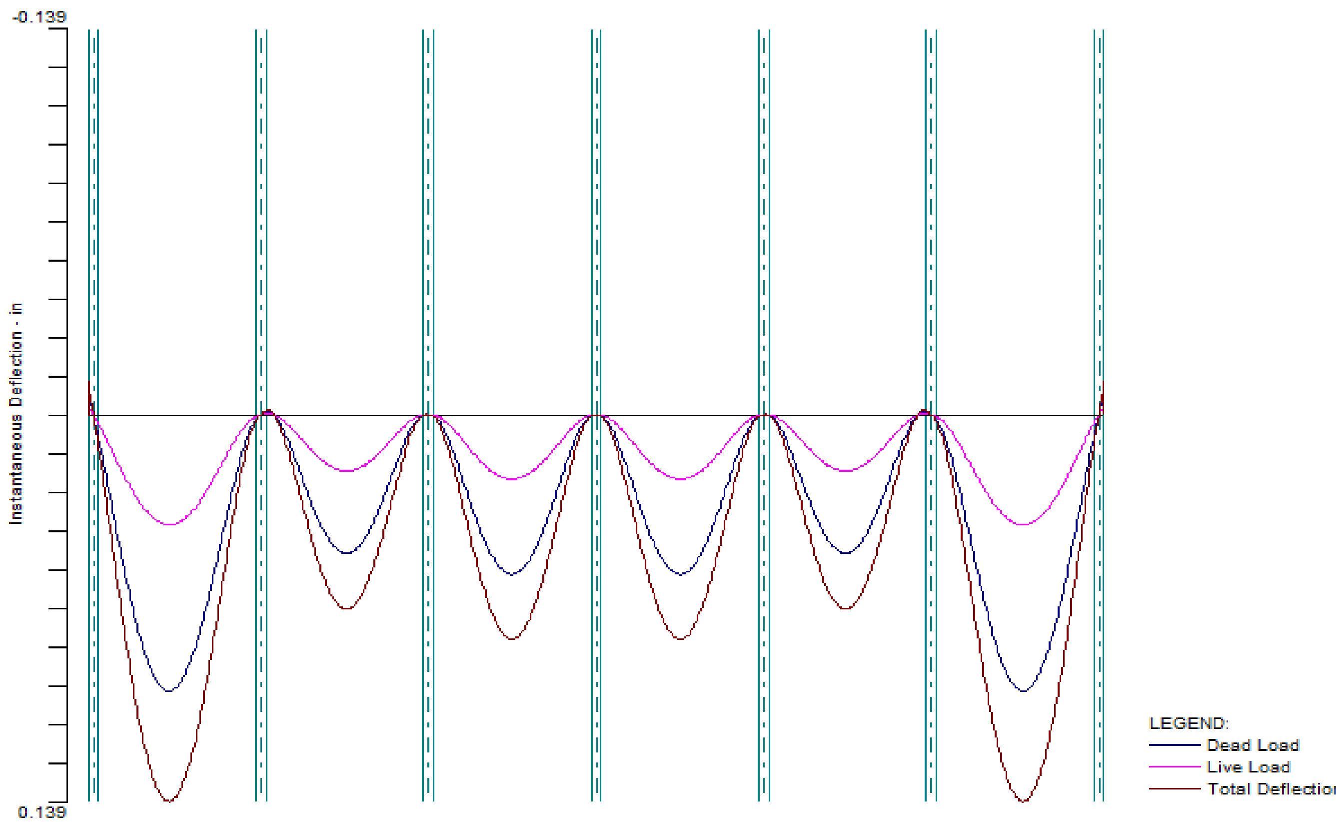




Middle Strip Flexural Reinforcement



Column Strip Flexural Reinforcement


-0.139
0.139
Instantaneous Deflection - in
LEGEND:
Dead Load
Live Load
Total Deflection

Appendix E:

Comparison Calculations and RS Means Costwork Assemblies

Assembly B10102564400 **Based on National Average Costs**

Floor, composite metal deck, shear connectors, 5.5" slab, 30'x30' bay, 26.5" total depth, 75 PSF superimposed load, 116 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Shores, vertical members, to 10' high, includes erect and strip by hand	0.01100	Ea.	0.00	0.22	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...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.33300	C.F.	0.00	0.51	0.51
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggre...	0.33300	C.F.	2.41	0.00	2.41
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
Weld shear connector, 3/4" dia x 4-7/8" L	0.12600	Ea.	0.09	0.25	0.35
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	4.91200	Lb.	6.88	2.11	8.99
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 20 gauge	1.05000	S.F.	2.32	1.04	3.36
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.03300	L.F.	0.13	0.08	0.21
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.58000	S.F.	0.34	0.57	0.91
Total			$12.40	$6.09	$18.49

Estimate for Composite and Non-composite systems

Assembly B10102303500 **Based on National Average Costs**

Precast concrete plank, 2" topping, 10" total thickness, 30' span, 75 PSF superimposed load, 155 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, hollow, 8" thick, prestressed	1.00000	S.F.	7.85	2.52	10.37
Total			$8.80	$4.77	$13.57

Estimate for Precast Hollowcore Plank system

Assembly B10102226600 **Based on National Average Costs**

Flat slab, concrete, with drop panels, 10.5" slab/7.5" panel, 18" column, 30'x30' bay, 75 PSF superimposed load, 217 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.03500	SFCA	0.03	0.36	0.39
C.I.P. concrete forms, elevated slab, flat slab with drop panels, to 15' high, 4 use, incl...	0.99700	S.F.	1.28	5.83	7.11
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	4.08800	Lb.	2.29	1.76	4.05
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.94400	C.F.	3.93	0.00	3.93
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.94400	C.F.	0.00	1.22	1.22
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			$7.60	$10.12	$17.72

Estimate for Two-way slab with drop panels

System Comparison | Tech 2 | Alexander Altemose

Composite

self-weight $= 42\,psf + \frac{40\,plf}{10'} + \frac{73\,plf}{32.67'} = 48.23\,psf$

Foundation Impact = None

cost on table

revised cost for self weight

cost = \$18.49 − 8.99 + 11.41

cost = \$20.91 + camber cost (~\$30 per beam)

Non-Composite

self-weight $= 57\,psf + \frac{55\,plf}{10'} + \frac{68\,plf}{32.67'} = 64.58\,psf$

Foundation Impact $= \frac{64.58}{48.23}(100) = 134\%$ load

cost on table

revised cost for self weight

cost = \$18.49 − 8.99 + 13.88 − 0.35

shear studs cost on table

cost = \$23.03

Hollow core Plank

self-weight $= (68+25)psf + \frac{94\,plf}{32'} = 95.94\,psf$

cost on Table

revised cost for on self weight

cost = \$13.57 + 5.38

cost = \$18.95

Foundation Impact $= \frac{95.94}{48.23}(100) = 199\%$ load

(and location of footings will change)

Two-way Slab w/ drop panels

self-weight $= \frac{11''}{12''}(150\,pcf) + \frac{4''}{12''}(150\,pcf)\left(\frac{11' \times 10'}{32' \times 30'}\right) = 143.23\,psf$

Foundation Impact $= \frac{143.23}{48.23}(100) = 297\%$ load

cost = \$17.72 ← cost on table