FDA OC/ORA Office Building

Silver Spring, MD

Technical Assignment 2

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Structural Option

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Executive Summary

The pro-con structural study of alternate floor systems looks at a typical bay of the FDA OC/ ORA Office Building. The two-way flat slab existing system is studied and then compared to three alternate systems. Each system uses the typical bay of 19.685' x 29.528', except the one-way slab system that requires a change in the bay size. The existing structure is a 9.5 inch thick two way flat slab with 7.09 inch drop panels at the interior columns. The direct design method was used to design the reinforcement along the frames that were chosen to be studied. Punching shear and wide beam shear were also checked at the drop panels and the column locations, but loading did not exceed nominal capacity.

The three alternate system that were studied included

1. Composite Steel Beams on Composite Deck
2. Hollow Core Precast Planks on Steel Beams
3. One-way Concrete Slab

The composite steel framing was designed using the AISC Steel Construction Manual and United Steel Deck Design Manual. The design was composed of the 2" LOK-Floor Metal Deck with a 4.5" slab, W14x22 beams, and W16x26 girders. The 4' x 10" Hollow Core Precast Panels with a 2" Topping were selected from the NItterhouse Design Catalog. The supporting girders were determined to be W18x50, using the AISC Steel Construction Manual. The One-way concrete slab was designed using the ACI 318-08. A slab thickness of 8", using #6 at 12" O.C. for flexural reinforcement, and #5 at 18" O.C. for temperature reinforcement. The beams supporting the slab were also sized by ACI 318-08; a beam depth of 34" was used with the beam width of 24" to match the column dimension.

The advantages and disadvantages were discussed for each framing systems, and it was determined that the one way slab system was not a feasible option. The one way slab system requires a change to the existing layout to allow for a one way slab to be designed. Also, the depth of the beams will introduce complications for coordinating the mechanical and electrical systems. For the original system the drop panels only occurred at the columns. In the middle of the bay, the depth of the system was only 9.5", but for the one way slab system it was 34". In general, the best alternative floor system considered in this report is the composite steel system. In only increased the depth of the system to only 20.2 inches, and it also made the system much lighter.

Introduction

Starting the fifth phase of the consolidation efforts by the FDA, the OC/ ORA Office building plans to move the Office of Commissioner (OC), Office of Regulatory Affairs (ORA) Office building to the White Oak Campus. On the site of the former US Navy facility at the Federal Research Center- Naval Ordnance Laboratory, the OC/ ORA Office Building sits on the southern end, and forms its shape around the existing buildings.

Forming an S shaped building, the 500,000 S.F. office building was laid out and designed to mirror the existing buildings on the site and to form a unique face of the campus from the main drive off of New Hampshire Ave. Broken up into two buildings with four wings, Building 31 is comprised of Wing A, and Building 32 is comprised of wings B through D (Figure 1)

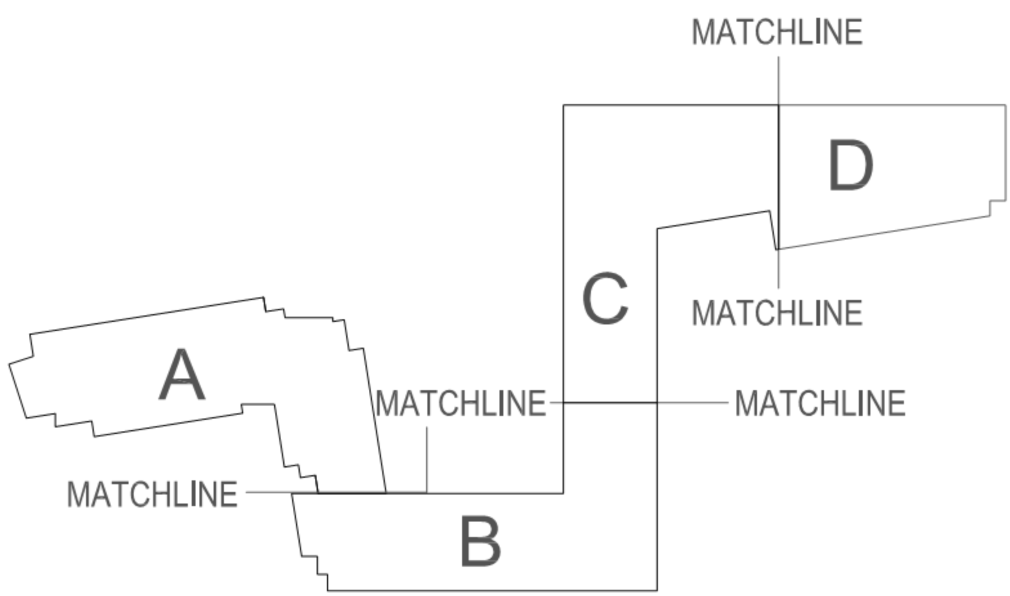


Figure 1: Key Plan

Structural System

Foundation:

The foundation of the building is separated into two categories. Spread footings that bear on undisturbed soil or spread footings that sit on a number of Geopiers. Schnabel Engineering conducted soil test to determine the bearing capacities of the soils. Where 95% compaction could not be met the use of Geopiers or vibropiers was recommended.

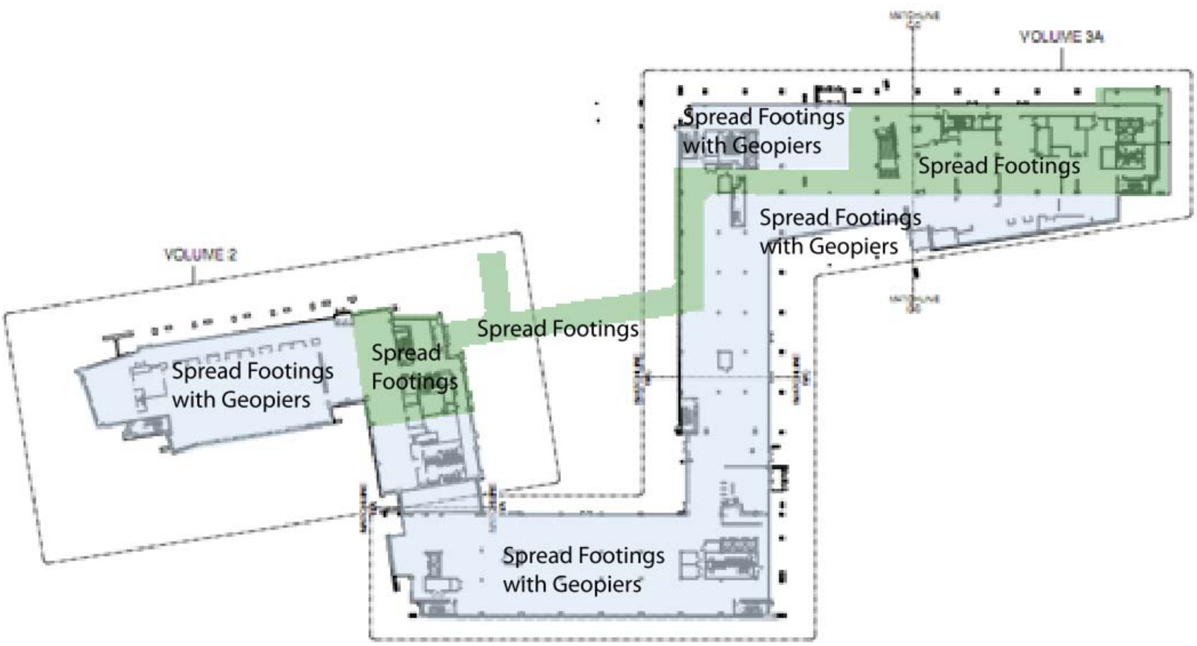


Figure 2: Foundation Key

For non-basement areas of Building 31 (Wing A), the western and central wings (Wings B and C) of Building 32, and the non-basement areas of Wing D, deep existing fill is expected within the majority of the buildings footprint. Geopiers are to be used in these areas to provide adequate bearing capacity (Figure 2). Geopiers use the concept of over consolidation to increase the soils bearing capacity. The 30 inch diameter Geopeirs should reach a depth of at least 10 feet. A detail of the typical spread footing with Geopiers is shown in Figure 3.

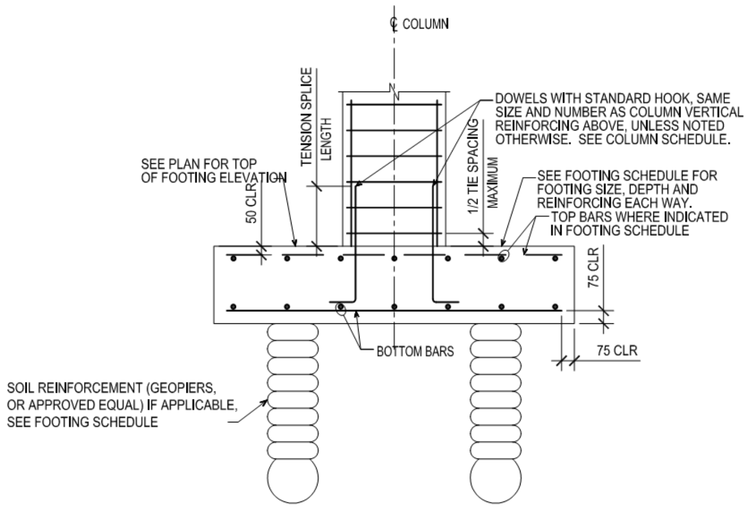


Figure 3: Typical Geopier Foundation Detail

For the basement level of Building 31 (Wing A), the basement level of Wing D of Building 32, and the underground tunnels, the foundations reach a sufficient depth where the bearing capacities on the spread footings are adequate (Figure 2).

Normal weight concrete was designed to be used with all the spread footings of the foundations. With a unit weight of 2350 kg/m^3 (147 pcf), the concrete has a 28 day strength of 28 MPa (4061 psi) concrete. A water to cement ratio of .48 is specified along with only 1% maximum chloride content.

Schnabel Engineering recommended the use minimum safe bearing capacities at the different locations of the foundation system. Where spread footings bear on undisturbed soil a bearing capacity of 192 kPa (4010 psf) was estimated. Beneath the spread footings of Wing A, where Geopiers were used, the estimated bearing capacity is 192 kPa (4010 psf). In the sections of Building 32 where Geopiers were used, a bearing capacity of 287 kPa (5994 psf) was estimated.

Floor System:

Building 31:

Building 31 utilizes a one way slab floor system for the majority of the buildings layout. The typical one way slab construction is an 8.07 inch thick slab with 5.91 inch drop panels, unless noted differently on the drawings. On the first three floors of Wing A there is a large open assembly space, and prevents any typical bay spacing. However, on the fourth floor the typical bay spacing is 21.85' x 26.74' to 19.685' x 19.685'.

Resistance to progressive collapse was designed into the exterior reinforced beams of building 31. Typical progressive collapse beam sizes range from 23.62" x 42.32" to 18.11" x 35.43". The interior beams on Building 31 are reinforced concrete beams with typical sizes of 18.11" x 35.43" to 18.11" x 23.62".

A large assembly pace on the first floor of Wing A is open up through the third floor. On the fourth floor framing level, post tension transfer girders were designed to support the column loads above the fourth floor and transfer the load to the foundation (Figure 4). The post tension transfer girders are 35.43" x 70.89" and have a post tension strand force of 4540 kN.

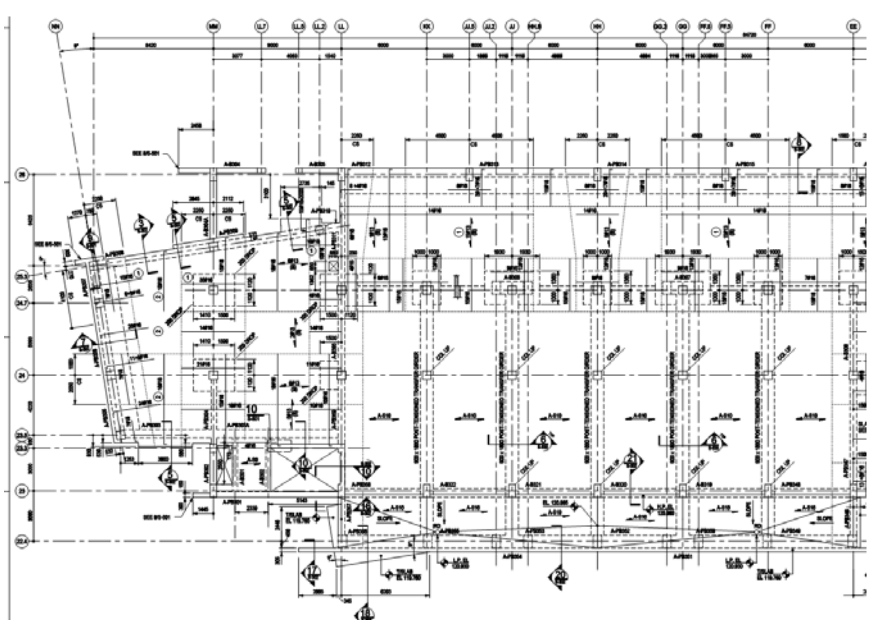


Figure 4: Framing Flan for Post Tension Transfer Girders

An atrium is provided between Wing A and Wing B that is primarily a steel superstructure with lightweight concrete on metal deck (Figure 5). The walkways over the atrium connecting the two wings are cast in place lightweight concrete on steel metal deck. The rib height on the metal deck is 50 mm with an additional 83 mm of concrete above. Supporting the walkway is W360 x 32.9 steel beams that frame into W360 x 32.9 girders with a shear connection. On the Wing A side of the atrium the girders site on an L152x152x9.5 that is attached to the concrete beam in Wing A. On the Wing B side on the atrium, an expansion joint is place, so the girders rest on a sliding connection that is connected to a beam in Wing B (Figure 6 and 7).

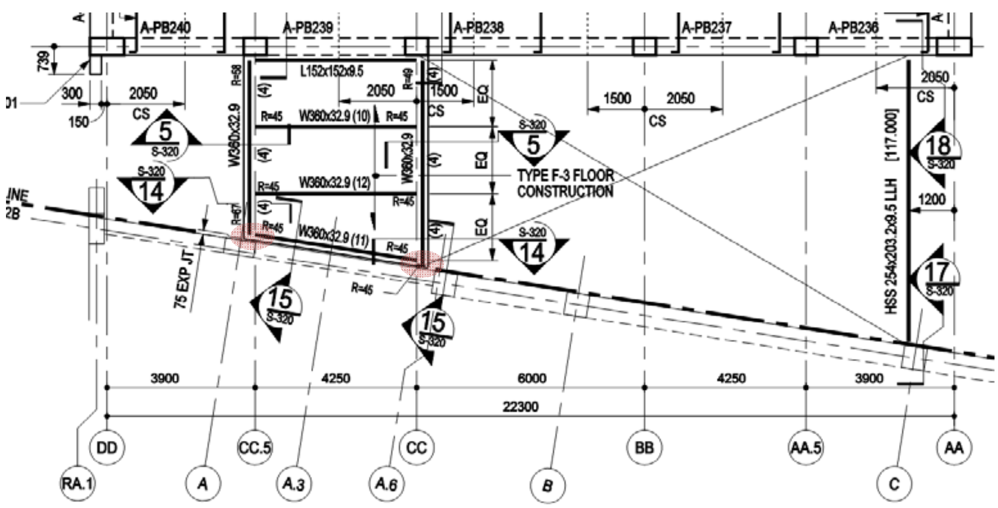


Figure 5: Wing A Atrium

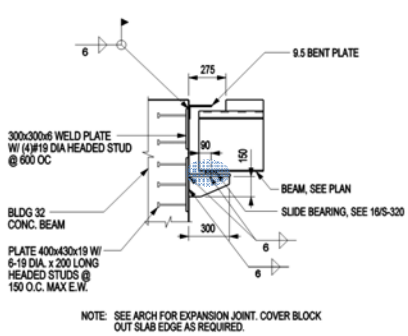


Figure 6: Expansion Joint Detail (Red)

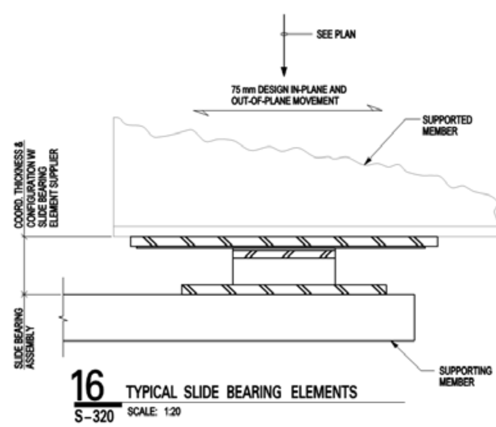


Figure 7: Expansion Joint Detail (Red)

Building 32:

Building 32 utilizes a two way flat slab system for the majority of the building's floor system. A 5.91" thick slab on grade is provided for the ground level and the basement levels of the building. The two-way flat slab is typically 9.449" thick with a 7.09" thick drop panel, unless noted differently on the structural drawings. The typical interior bay spacing for Building 32 is 29.528' x 19.685', and the typical exterior bay spacing of 27.559' x 29.528', figure 8 shows the typical layout of the bays.

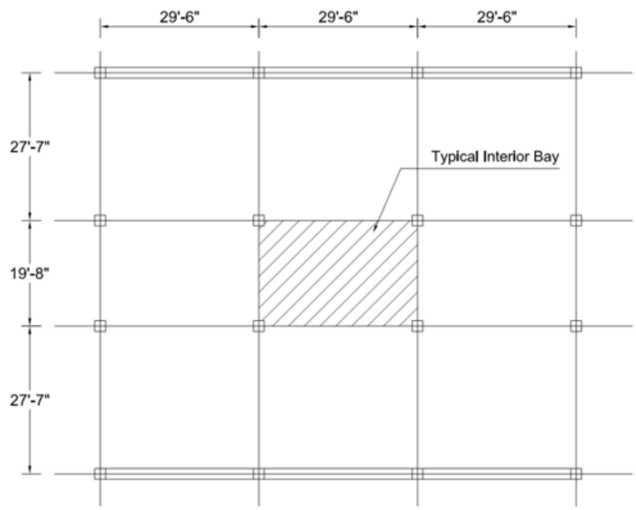


Figure 8: Building 32 Wing B Typical Bay Layout

Resistance to progressive collapse was designed into the exterior reinforced concrete beams of building 32. Typical progressive collapse beam sizes ranging from 23.62" x 40.95" to 15.75" x 40.95".

Atriums are provided between Wings B and C, and between wings C and D. The floor system for the atriums is a cast in place lightweight concrete on metal deck. The rib height on the metal deck is 1.97" with an additional 2.52" of concrete above. Supporting the walkways are W150 x 30 steel beams that frame into W610 x 217 girders with a shear connections. Expansion joints at the Intersections of the wings are provided and sliding connections are required at the atrium walkways.

Columns

Typical reinforced concrete columns were designed for the FDA OC/ ORA Office Building. Designed as the primary gravity system, the typical sizes of the columns are 600mm x 600mm, 900mm x 600mm, and 600 mm diameter. Various types of columns are provided ranging from square columns, rectangular columns and circular columns (Figure 9). The concrete for the columns is a normal weight concrete with 28 day strength of 28 MPa (4061 psi). The slab and the beams are monolithic with the columns forming a continuous system.

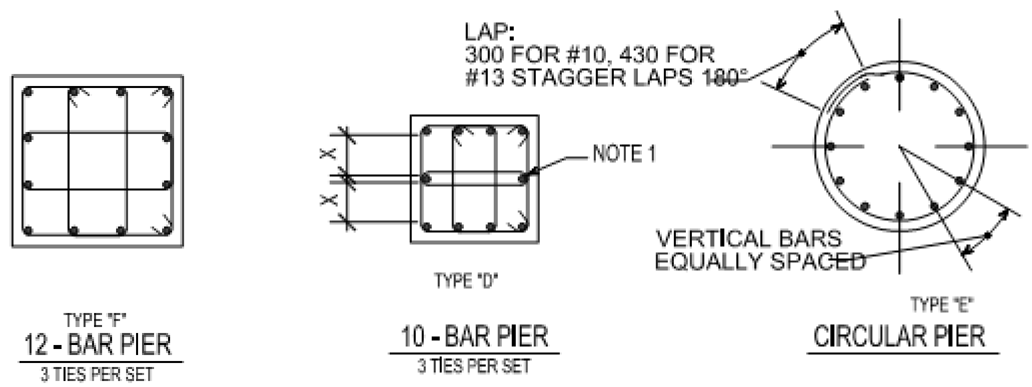


Figure 9: Typical Column Details

Lateral System

Ordinary reinforced concrete shear walls were design for the primary lateral resisting system. The typical shear wall has #16 at 300mm (#5 at 11.82 inches) for both vertical and horizontal reinforcement with 13 #16 (13 #5) for the end zone reinforcement and #13 ties at 300mm (#5 ties at 11.81 inches) for the vertical reinforcement (Figure 10 and 11).

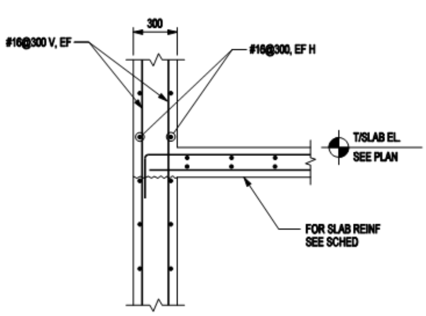


Figure 10: Shear Wall Detail

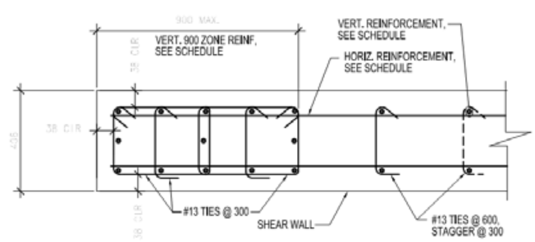


Figure 11: Shear Wall End Zone

Shear walls are provided around each elevator core and the stair shaft of Wing A. Wings B through D provide shear walls around each elevator core; Figures 16 through 19 shows the location of the shears walls in each wing, shown in red. At the intersection of each wing, in the atriums, slide bearing connections are provided at the expansion joints, shown in blue. These connections allow each wing's lateral systems to act independently of the other wing.

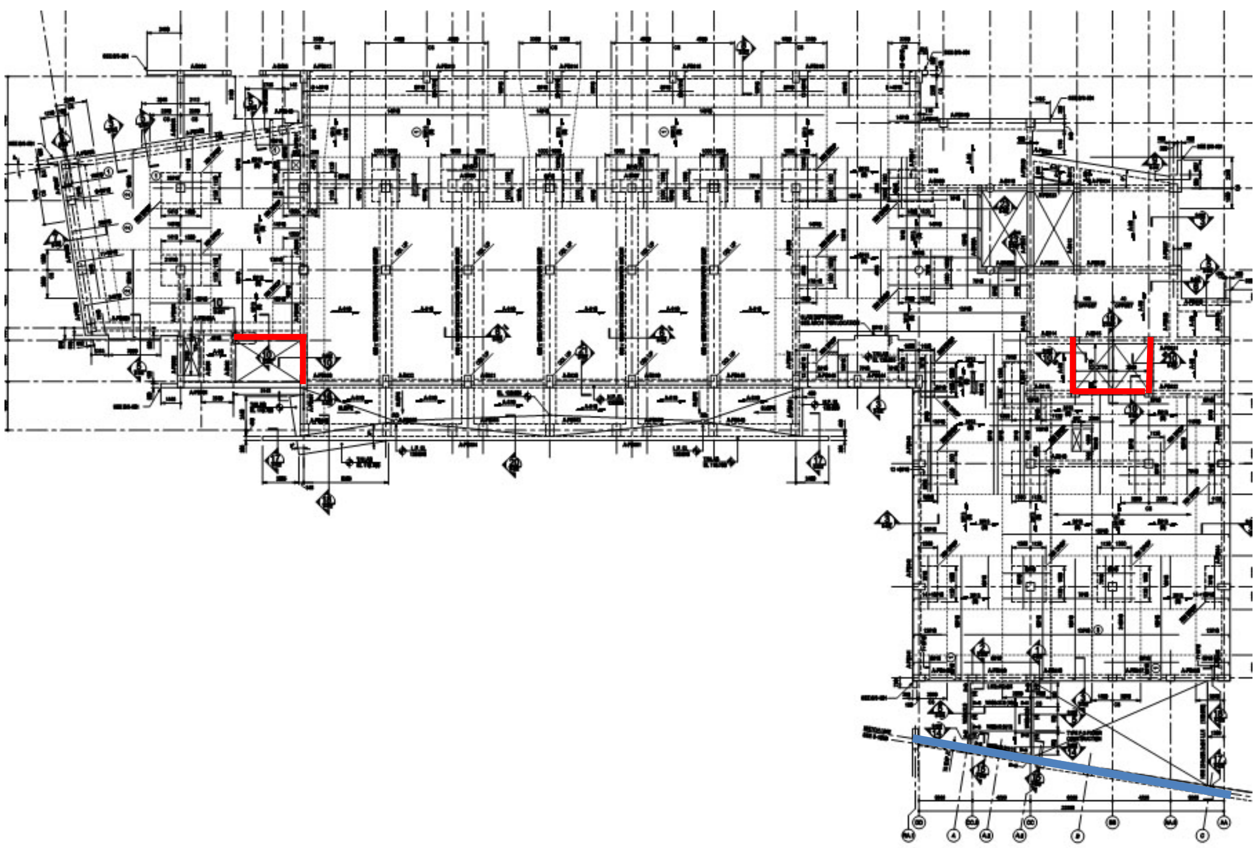


Figure 12: Shears Walls of Wing A

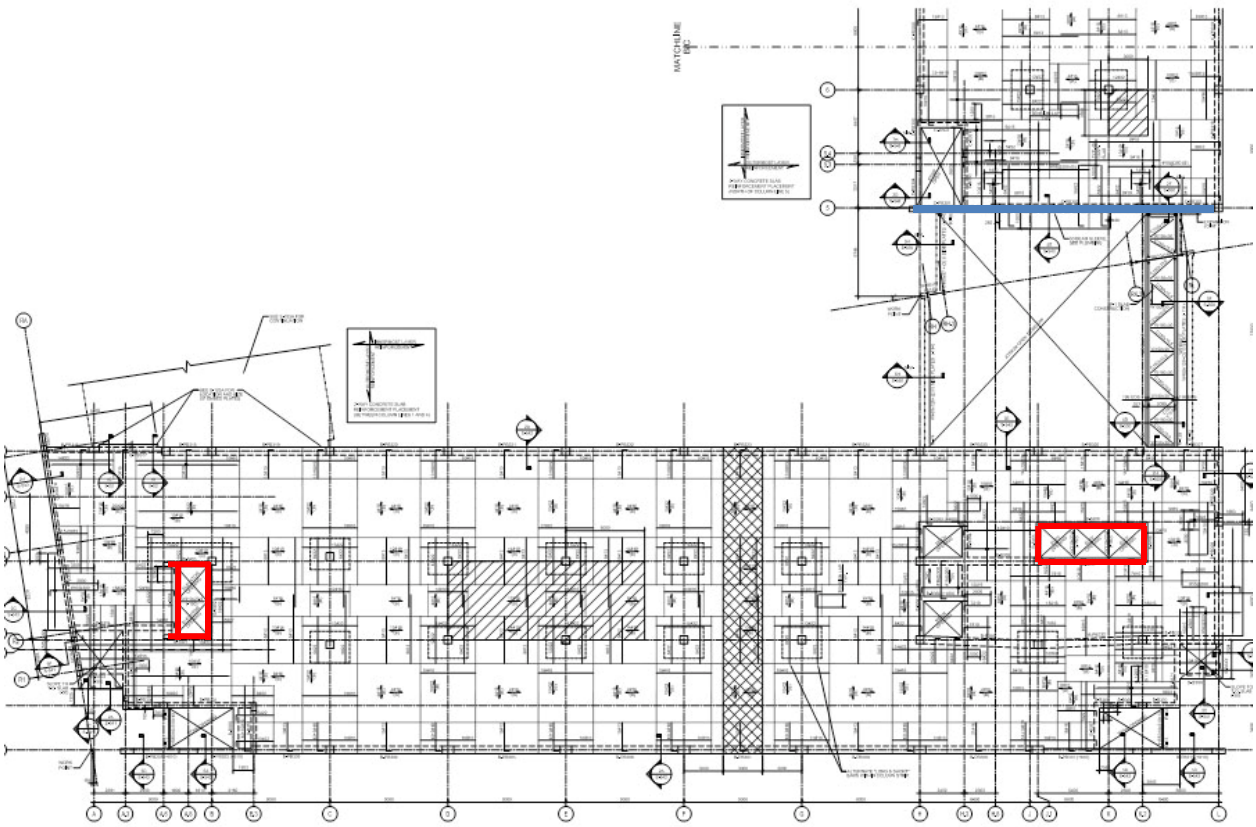


Figure 13: Shear Walls of Wing B

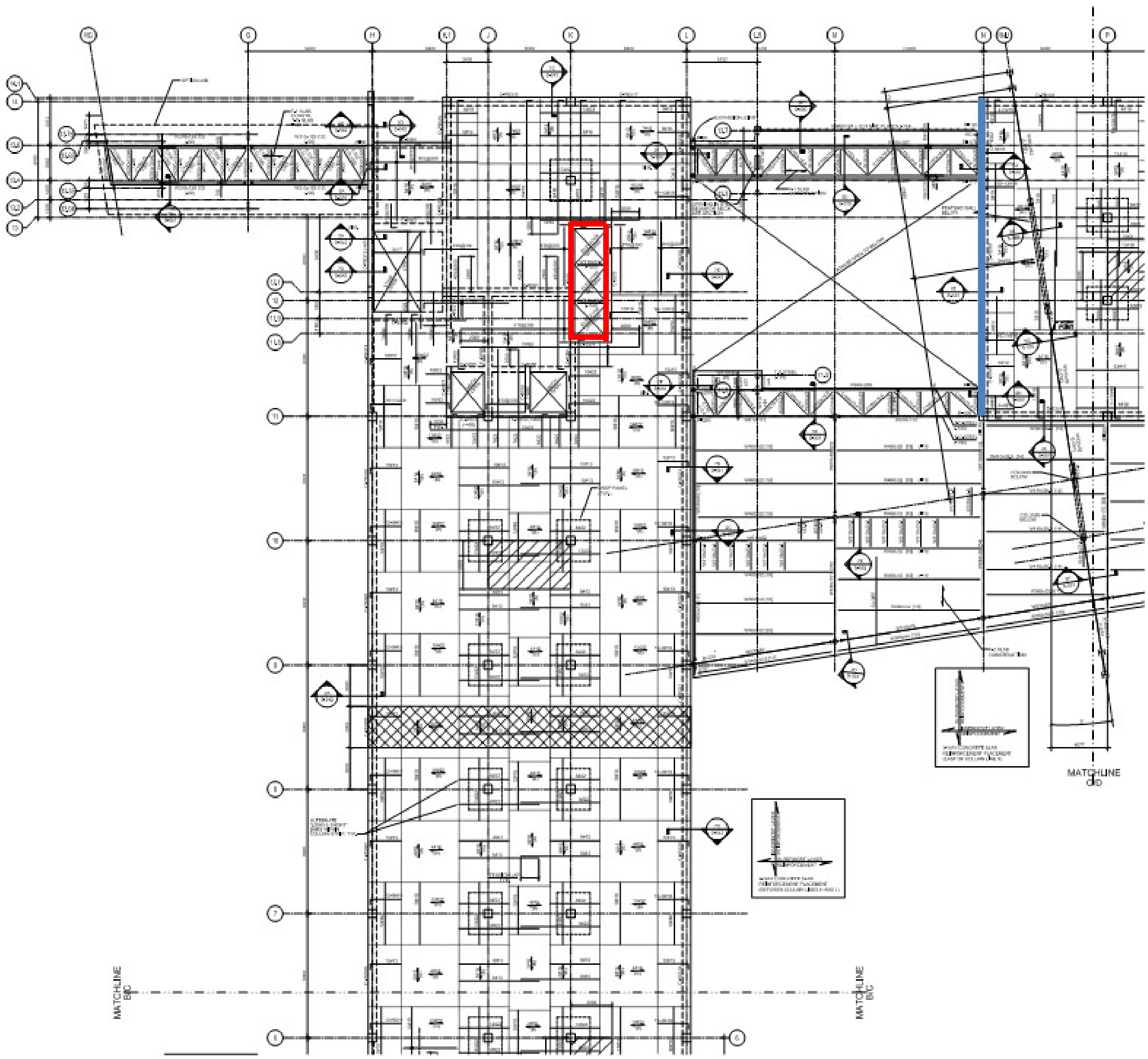


Figure 14: Shear Walls of Wing C

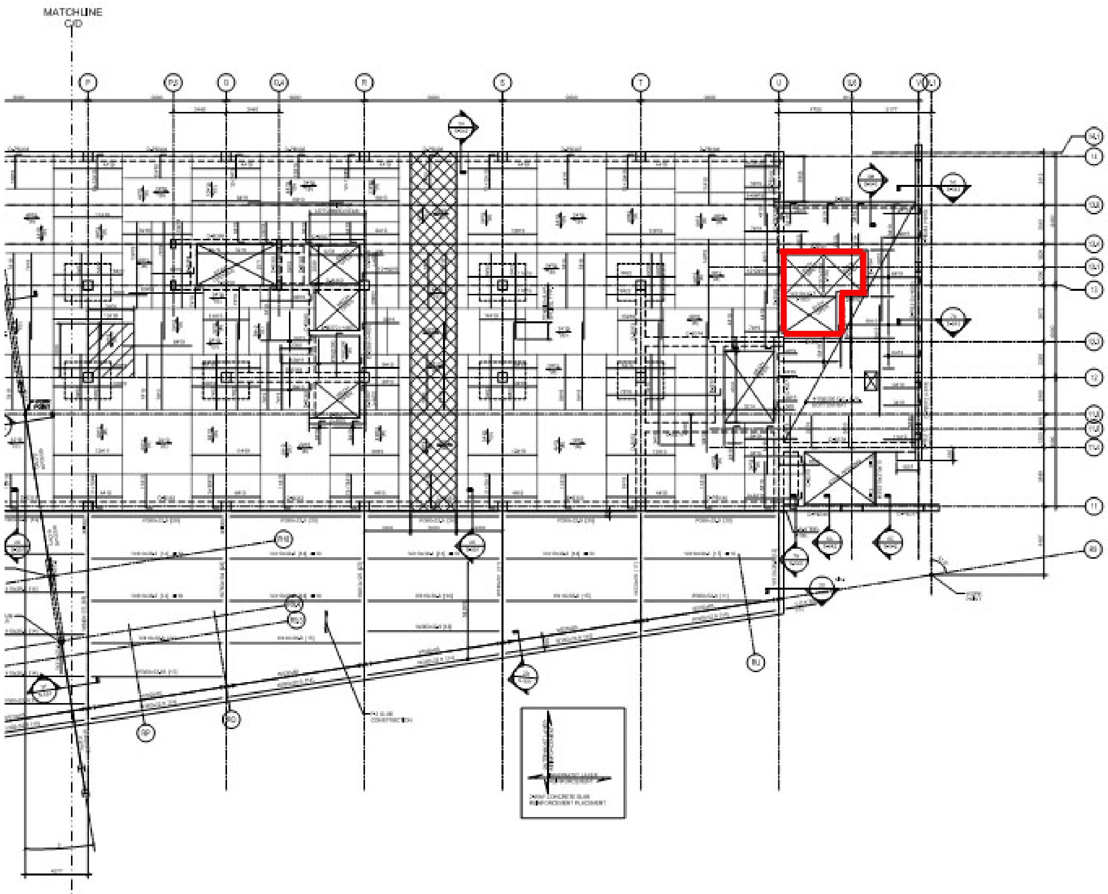


Figure 15: Shear Walls of Wing D

Load Paths

Gravity Load Resisting System:

Reinforced Concrete columns make up the primary gravity load resisting system. The live load, self weight and superimposed dead load that sits on the floor system is transferred to the reinforced concrete beams. Reinforced concrete columns pick up the loads from the beams and the load is transferred to the buildings foundations. In Wing A reinforced concrete columns bear on a post tension transfer girder. There the load is transferred from the columns into the transfer girder. Surrounding columns that the transfer girders bear on transfer the load from the girders into the columns. Columns then transfer the load into the foundation of the building.

Resistance to progressive collapse has been designed for the office building. Design considerations that are involved with this design are removing an exterior column, and the floor system above and the adjacent columns are designed to carry the additional load.

Lateral Load Resisting System:

Reinforced concrete shear walls are the primary lateral load resisting system. Lateral force due to wind is transmitted against the curtain wall of the building. Rigid floor system picks up each story shear at each level and transmits the lateral force to the shear walls located around each elevator core. Shear walls are design to resist the moment from the lateral load. The resisting moment forces are transmitted through the shear walls onto large spread footings.

Each wing acts independently with respect to the others wings. This is primarily due to the large expansion joints provided between each wing, along with the slide bearing connections design at the atriums connections.

Codes and References

Design Codes:

National Model Code:

GSA Facilities Standards for the Public Building Service

International Building Code 2003

Structural Standards:

GSA Facilities Standards for the Public Building Service

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

Design Codes:

AISC-ASD, Specifications for Structural Steel Buildings – Allowable Stress Design

ACE 318-02, Building code Requirements for Structural Concrete

Design Codes (Used for this Thesis)

National Model Code:

GSA Facilities Standards for the Public Building Service - 2005

2006 International Building Code

Structural Standards

GSA Facilities Standards for the Public Building Service – 2005

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Standards:

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Design of Buildings to Resist Progressive Collapse 2005, Unified Facilities Criteria

Gravity Loads

The primary design guide lines for the FDA OC/ORA Office Building are the GSA Facilities Standards for the Public Service-2005, and the ASCE 7-02. The GSA outlines general requirements for the required live load for office interiors and the telecom room. The GSA Facilities Standards for the Public Building Service requires the designer to implement progressive collapse design into the structural design.

The latest version of design codes is being used for the analysis of the buildings gravity and lateral systems. When comparing to the designed loads and the ASCE 7-05 required loads, only one major difference appeared. ASCE 7-05 requires a load of 100 psf for special purpose roofs, specifically green roofs. Comparing to the designed load of 31.33 psf, one possible reason for the significant difference is the dead load; the structural engineer added a green roof dead load.

Live Loads					
	Design		GSA 05	ASCE 7-05	
Location	kPa	psf	psf	psf	
Office	3.8	79.36	80	50	(Partitions)
Typical Roof	1.5	31.33		20	
Public Lobbies	4.8	100.25		100	
Mech Room	7.3	152.46		150	(Assumed)
Telecom Room	12	250.63	250	150	
Pedestrian Bridge	4.8	100.25		60	
Balconies	4.8	100.25		100	
High Density Filing	12	250.63		250	(Assumed)
Green Roof	1.5	31.33		100	

Figure 16: Live Loads

Dead Loads		
	psf	
Superimposed Dead Load (MEP, Ceiling)	15	(Assumed)
Roofing System	40	(Assumed)
Mechanical Unit	150	(Assumed)
Exteior Curtain Wall	30	(Assumed)
Atrium Cutrain Wall	20	(Assumed)
Mechanical Pentouse Walls	20	(Assumed)

Figure 17: Dead Loads

SNOW LOADS (S)					ASCE 7-05 Ref.
Ground Snow Load	p_g =	25	psf		Figure 7-1
Exposure Factor	C_e =	1		Terrain Category B	Table 7-2
Thermal Factor	C_t =	1			Table 7-3
Importance Factor	I =	1		Occupance Category II	Table 7-3
	p_f =	17.5	psf	p_f = .7*Ce*Ct*I*pg	Eq. 7-1
	p_{fmin} =	20	psf	$p_{fmin} = p_g$*I	Section 7.3
	p_f =	**20**	**psf**		
Snow Drift					
Snow Density	γ =	30	pcf		Eq. 7-3
	h =	14.66	ft		
	h_d =	0.67	ft		
	h_c =	13.99	ft		
Snow Surcharge	**S_d =**	**52.5**	**psf**		Section 7.7.1

Figure 18: Snow Loads

Analysis of Floor Systems

For this report, the typical interior bay of Wing B is analyzed for the existing floor system and three alternative floor systems, typical framing plans of Wing B are provided in Appendix A. Figure 19 shows the layout of the typical interior bay and the surrounding bays. The design of each system is provided in the Appendices B through E. Assumptions in the design of the floor systems included that loading was uniform over the bay, and this requirement was only valid for some of the typical bays. The loads used in this thesis were obtained from the GSA Facilities Standards for the Public Building Service and ASCE 7-05.

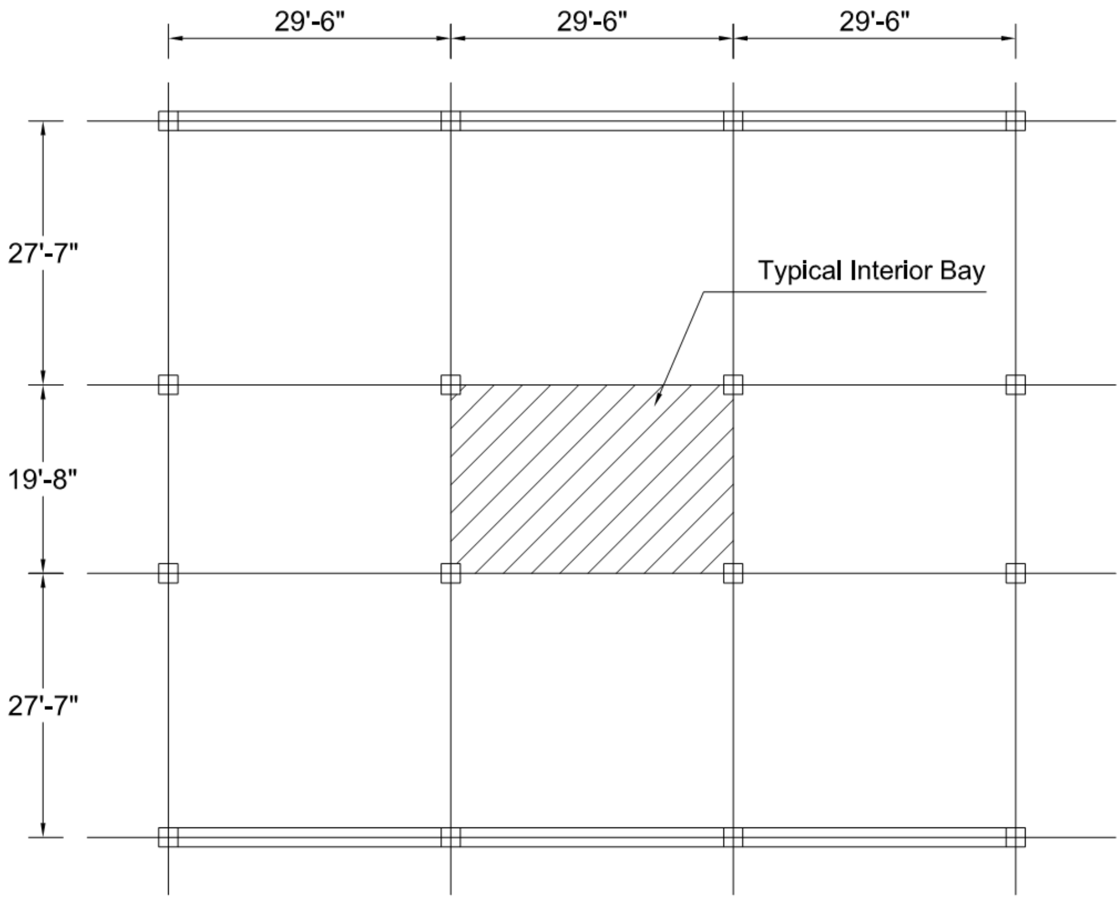


Figure 19: Wing B Typical Interior Bay

Existing System: Flat Plate Two-Way Flat Slab with Drop Panels

Material Properties

Concrete:	9 ½” Normal Weight Concrete, with 7.09” Drop Panels
	23.62” x 23.62” Columns
	f’c= 4000 psi
Reinforcement:	Fy= 60,000 psi

Loading

Dead (Self weight):	129.9 psf
Live (Partitions):	80 psf
SDL:	15 psf

Description

The two-way reinforced flat slab system is a 9.5” normal weight concrete slab with 7.09” drop panels at the interior columns, figure 20 shows the layout of the existing floor system. The typical bottom reinforcement across the entire bay is #4 at 11.81 inches on center, and the top reinforcement varies over the column strips and middle strips.

A typical interior bay on the second 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 Figure 21. The slab was checked for flexural, shear, and minimum thickness. The slab thickness of 9.5 inches exceeded the minimum requirement of 9.19 inches, in accordance with ACI 318-08 Table 9.5 c. Punching shear and wide beam shear were also checked at the drop panels and the columns, but did not exceed the limits. All supporting calculations for this analysis can be found in Appendix B.

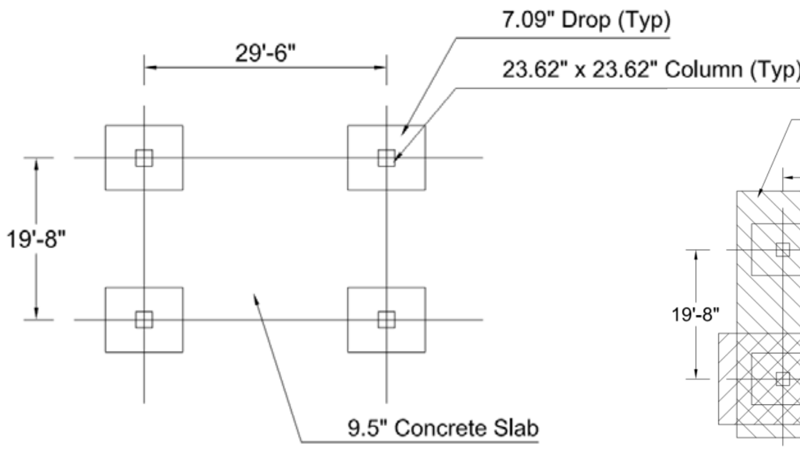


Figure 20: Two-way Slab Layout

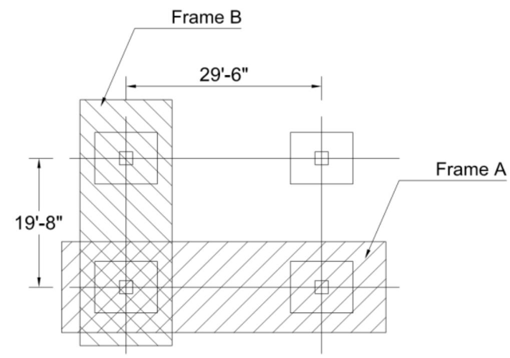


Figure 21: Frame Layout

Advantages

A two-way flat slab system provides a large floor to ceiling height, also allowing more space between the ceiling and the bottom of the slab for mechanical and electrical equipment. No interior beams were used to support the slab; therefore more space could be coordinated with the mechanical and electrical disciplines. Additional fireproofing is not required for the concrete system because it is built into the clear cover of the steel. The current system is already designed to meet the requirements for resistance to progressive collapse.

Disadvantage

Two-way flat slab design requires an aspect ratio of less than 2. The center bays of Wing B do meet this requirement. Near the ends of the building, however, the bay sizes are not typical and do not meet the aspect ratio. Construction time for placing concrete is long because of the forming and shoring of the concrete.

Alternative #1: Composite Deck with Composite Steel Beams

Material Properties

Concrete:	4 ½" Normal Weight Concrete Slab on Metal Deck f'c= 3000 psi
Decking:	18 Gage Metal Deck with 2" LOK-Floor (USD)
Steel:	A992 W-Shapes Beams: W14x22 Girders: W16x26

Loading

Dead (Self weight):	46.4 psf
Live (Partitions):	80 psf
SDL:	15psf

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 interior bay on the second floor was used to design the composite steel systems, (see figure 22 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 C contains the supporting calculations for the design of the composite steel system.

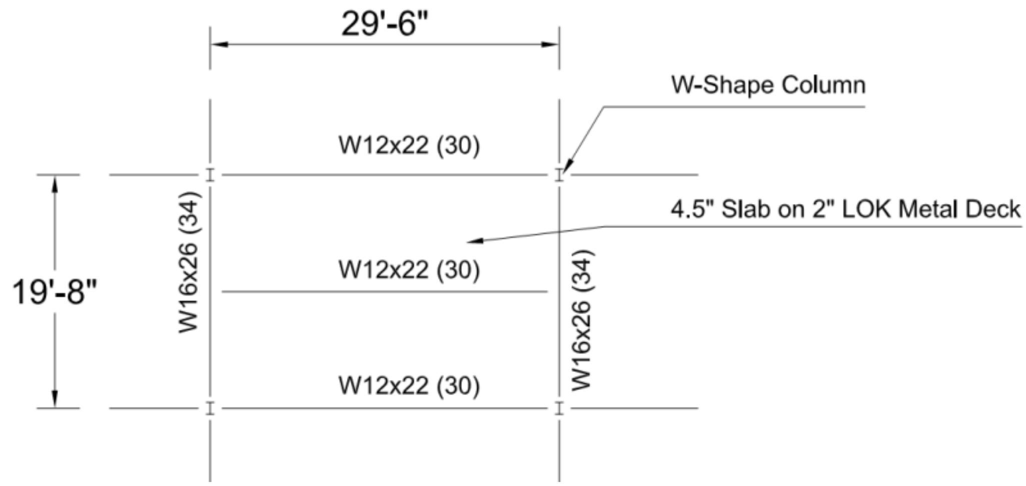


Figure 22: Composite Steel Frame Layout

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. The exterior bays of the floor system will need extra design for the resistance of progressive collapse.

Alternative # 2: Hollow Core Planks on Steel Beams

Material Properties

Concrete:	Hollow Core Planks (Nitterhouse)
	10” x 4’ Hollow Core Plank with 2” Topping
	6-1/2” Strand Pattern
	f’c= 6000 psi
Steel:	A992 W-Shape
	Girder: W18 x 50
	Beam: W12 x 26

Loading

Dead (Self Weight):	93 psf
Live (Partitions):	80 psf
SDL:	15 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 10” x 4’ hollow core plank is sufficient to support the loads across the 30 foot span. Either a minor adjustment to the column layout or a custom made plank will be needed to allow for the 19.685 foot span. A typical interior bay on the second floor with span lengths of 19.685’ x 29.528’ was used to design the floor system, (see Figure 23 for the layout of this system). The effect of moving the columns will be small to the space because the original design included 2’ x 2’ columns. 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.

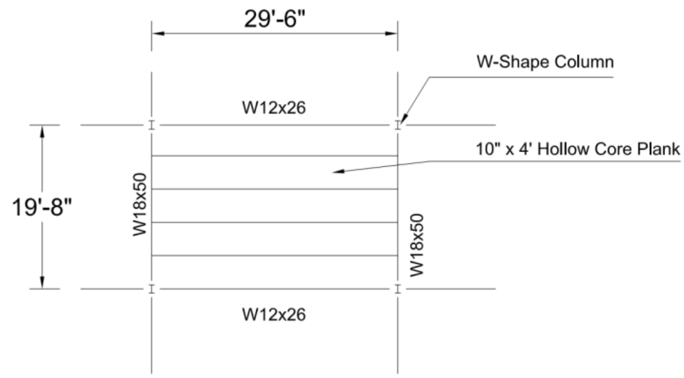


Figure 23: Hollow Core Planks on Steel Beam Layout

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 increased 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. On the site with existing structures above and below ground, the change in the building's dimension could impact these elements on the site. With the increase in the depth of the steel members and a 10" plank, the deeper floor system can cause conflicts with the mechanical and electrical systems. The hollow core planks are designed to achieve a fire rating of 2 hours; however, the steel beams will require spray-on fireproofing. The exterior bays of the floor system will need extra design for the resistance of progressive collapse.

Girder Slab System

The Girder Slab system was the initial direction for this alternate system, after doing research on the Girder slab system (Figure 24 is a detail of the Girder Slab System), it was determined not to be a feasible option for the building. The typical bay spacing and high service loads exceeded the limits of the standard members produced by Girder Slab. Also, the

Girder Slab system requires the use of 8” hollow core planks and the design plank was actually 10”. For the girder slab system to be a feasible option, a custom W-shape member with an angle as a seated connection would need to be designed (Figure 25 shows a detail of this connection). For the purpose of this report, the hollow core planks were designed to rest on top of the beams, (Figure 26 shows a detail of this connection). If this system is chosen for further research, the planning of a system similar to the Girder Slab system will be done.

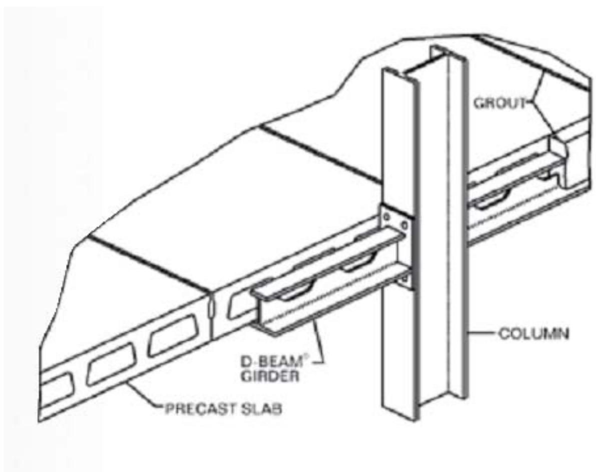


Figure 24: Girder Slab System

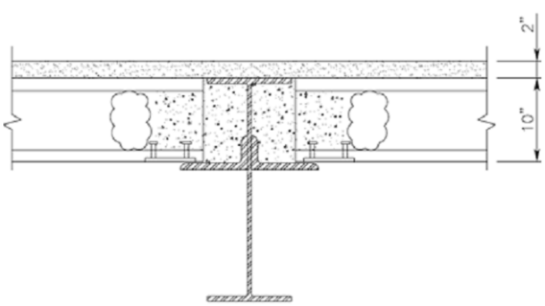


Figure 25: Alternate to Girder Slab System

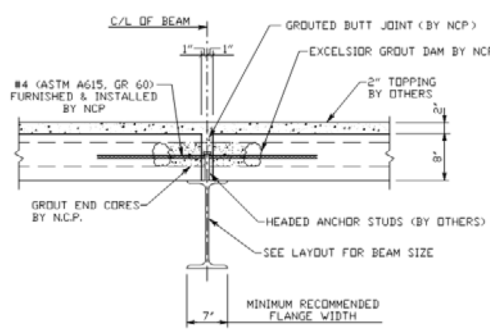


Figure 26: Hollow Core Planks Bearing on Steel Beam

Alternative #3: One Way Slab

Material Properties

Concrete:	8" Normal Weight Concrete Slab 24" x 24" Columns f'c= 4000 psi
Reinforcement:	Fy= 60,000 pis

Loading

Dead (Self weight):	208.2 psf
Live (Partitions):	80 psf
SDL:	15 psf

Description

The one-way slab system designed for the interior bay was an 8" concrete slab that spanning 19.685' direction. A girder spans between the columns, 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 aspect ratio of the bays was less than 2.0. This meant that the column arranged needed to be changed to increase the aspect ratio. The 29.528' span was increased to 40' to increase the aspect ratio; (Figure 27 shows the layout of the floor system). This solution is not the only solution available to allow for the designing of one way slab, but only this solution was examined for this technical report. The impact on the architectural layout and foundation system need to be considered before this system can be implemented for the entire building.

The 8" slab was designed to have #6 at 12" O.C. for flexural steel, spanning the 19.685' direction, and #5 at 18" O.C. were provided for temperature steel. The beam spanning between the columns in the 19.685' direction, theoretically, does not see load from the slab, but it was designed using tributary area to allow for stability in the building frame. The main girder that spans along the 40' direction was designed to support the one way slab, with a beam size of 34" x 24", and with the 24" dimension matching the 24" columns size.

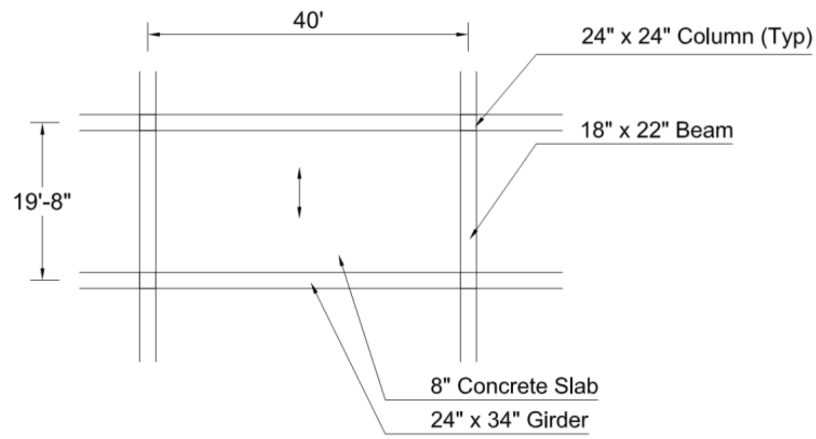


Figure 27: One-way Slab Layout

Advantages

There were no noticeable advantages for the one-way slab system as it was designed. More advantages may be available once other frame layouts are considered and the effects on the architectural spaces are considered.

Disadvantages

There are several disadvantages that are encountered with the design of the one way floor system. Changing the column layout will have a large impact on the architectural spaces. The increased weight of the floor system will require the foundation system to be rechecked. The increase span will pose a possible problem with the exterior beams that are designed to resist progressive collapse. The deeper beam sections will cause conflicts with the mechanical space and will either increase the building height or decrease the floor to ceiling height.

System Comparison

Floor System Comparison of a Typical Bay				
	Floor Systems			
	Existing Two-way Flat Slab	Composite Steel	Precast Hollow Core Planks on Steel Beams	Concrete One-Way Slab
System Weight (psf)	130	46.4	93	208
Slab depth (in)	9.5	4.5	10	8
Total depth (in)	16.59	20.2	28	34
Additional Fire Proffing	No	Yes	Yes	No
Fire Rating	2	2	2	2
Material (cost/S.F.)	9.15	17.60	9.05	12.70
Labor (cost/S.F.)	9.20	5.95	4.41	13.80
Total (cost/S.F.)	18.35	23.55	13.46	26.50
Foundation Impact	None	None	None	Yes
Architectural Impact	None	Some	Some	Yes
Constructability	Moderate	Easy	Easy	Moderate
Vibration Concerns	Minimal	Some	Minimal	Minimal
Alternative	N/A	Yes	Yes	Yes
Additional Study	N/A	Yes	Yes	Yes

Figure 28: Comparison of Floor Systems

Conclusion

The typical interior bay on the second floor of Wing B was used to analyze the existing system, and to design three alternate systems.

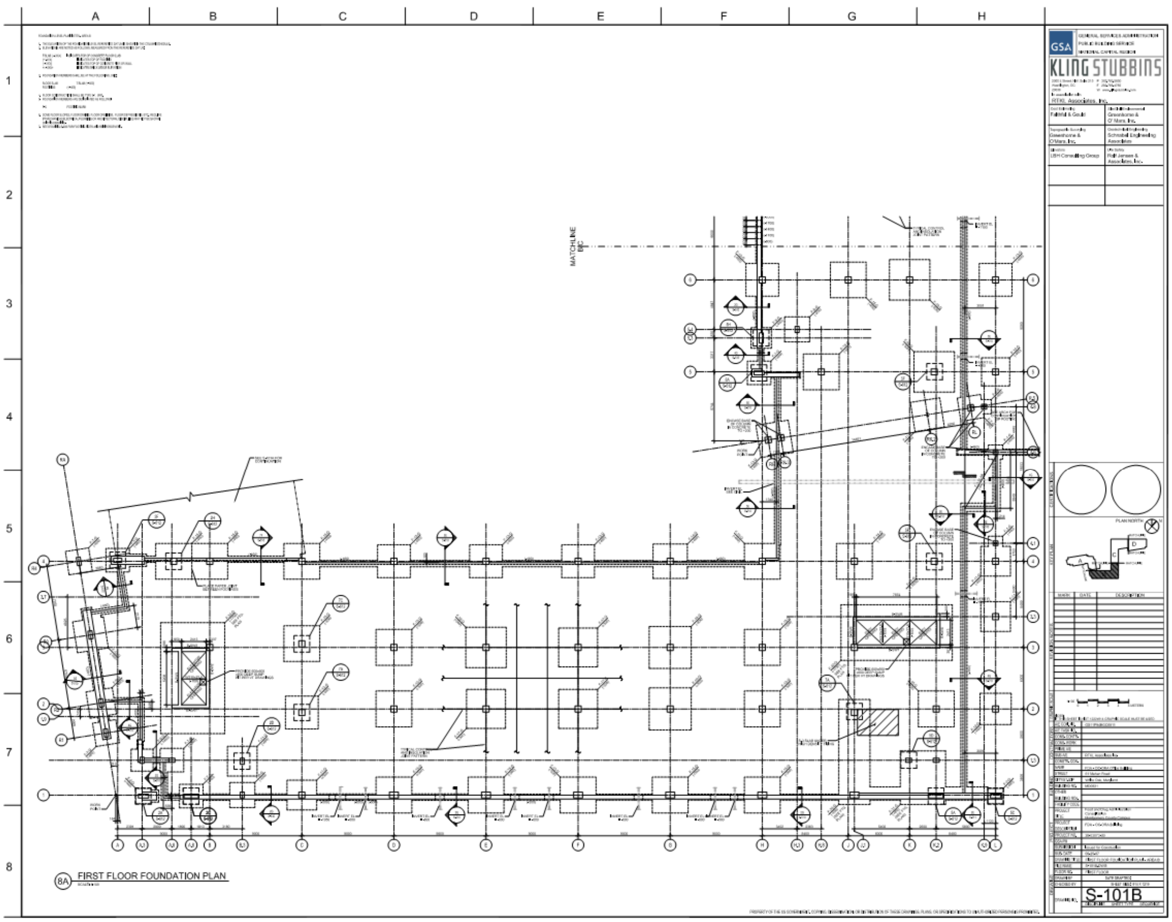
1. Two-way Flat Slab System (Existing)
2. Composite Steel Beams on Composite Deck
3. Hollow Core Precast Planks on Steel Beams
4. One-way Concrete Slab

The composite steel framing was designed using the AISC Steel Construction Manual and United Steel Deck Design Manual. The design was composed of the 2" LOK-Floor Metal Deck with a 4.5" slab, W14x22 beams, and W16x26 girders. The 4' x 10" hollow core precast panels with a 2" topping were selected from the Nitterhouse Design Catalog. The supporting girders were determined to be W18x50, using the AISC Steel Construction Manual. The one-way concrete slab was designed using the ACI 318-08. A slab thickness of 8", using #6 at 12" O.C. for flexural reinforcement, and #5 at 18" O.C. for temperature reinforcement was designed for the one-slab. The beams supporting the slab were also sized by ACI 318-08; a beam depth of 34" was used with the beam width of 24" to match the column dimension.

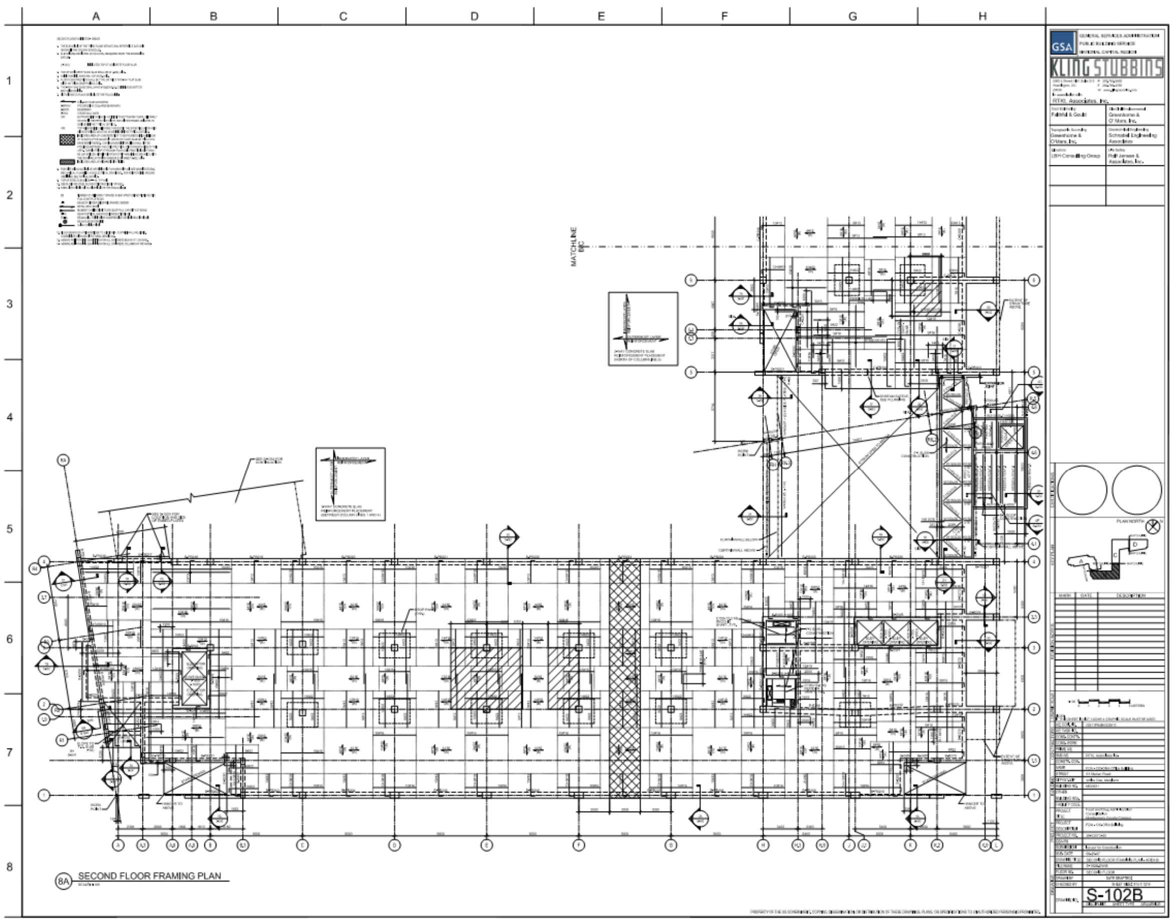
After reviewing the advantages and disadvantages of each system, it was determined that the one-way slab system was not a feasible option for this building. The need to adjust the column layout to permit the use of one way slab design can create conflicts with the architectural spaces, as well as structural and foundation considerations and the design to resist progressive collapse. If the one-way slab were to be considered, alternate methods of laying out the bay would also need to be considered. In addition, the hollow core plank system that was analyzed was not the best option for the framing system; however, the Girder Slab System or equivalent system could eliminate the floor-to-ceiling height conflicts. The composite steel framing system was the best alternative system to the two-way slab that was designed for the building. This is because the composite steel system provided a floor depth that was just a little deeper than the existing system, while at the same time providing a much lighter system.

Appendix A: Wing B Framing Plans

Wing B: First Floor Framing Plan

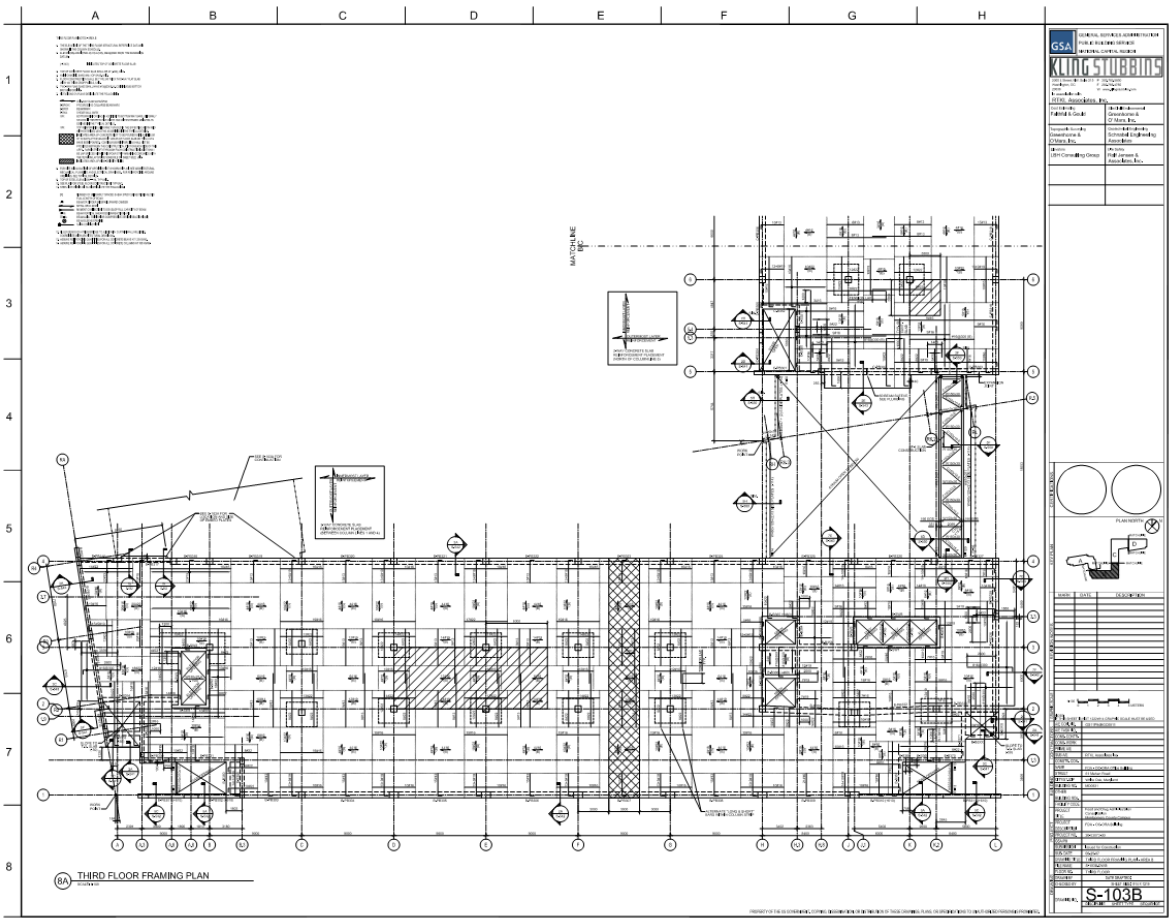


Wing B: Second Floor Framing Plan

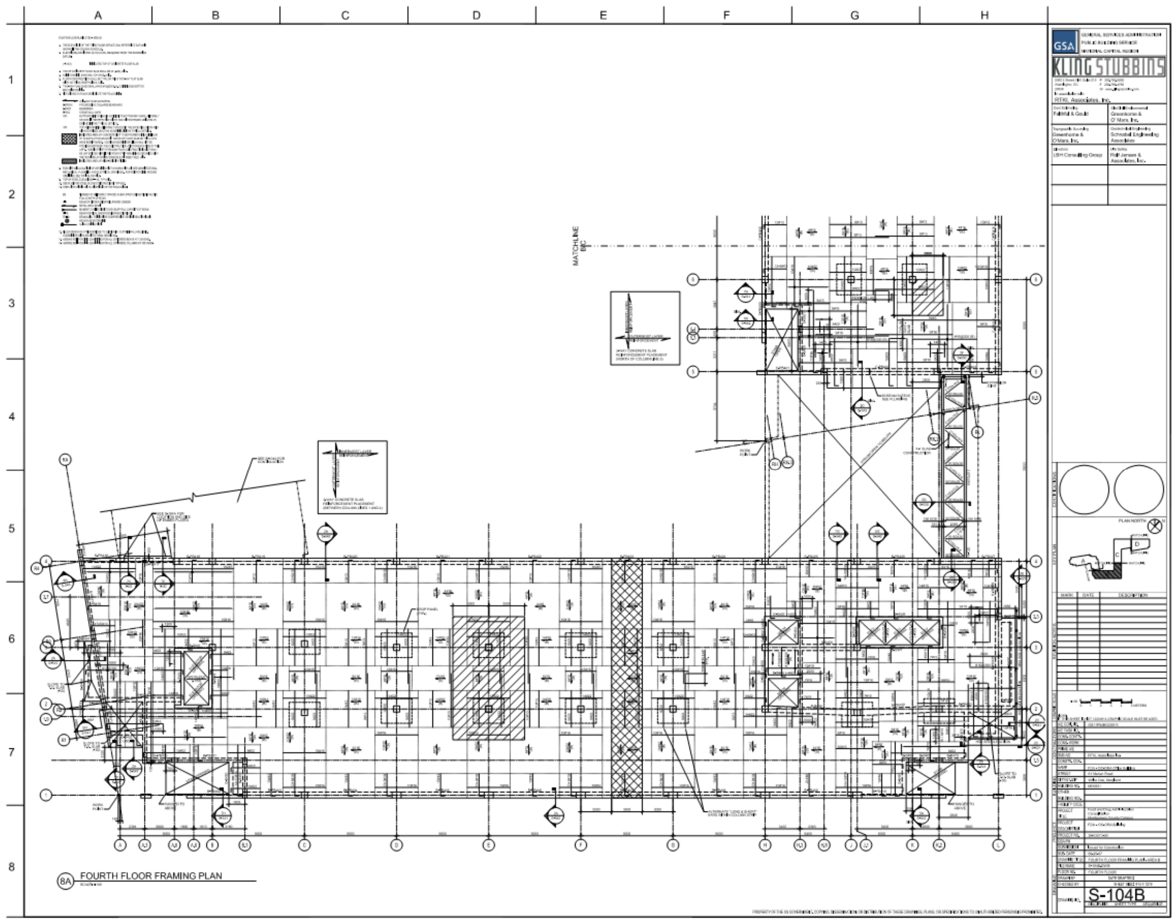


Technical Assignment #2

Wing B: Third Floor Framing Plan

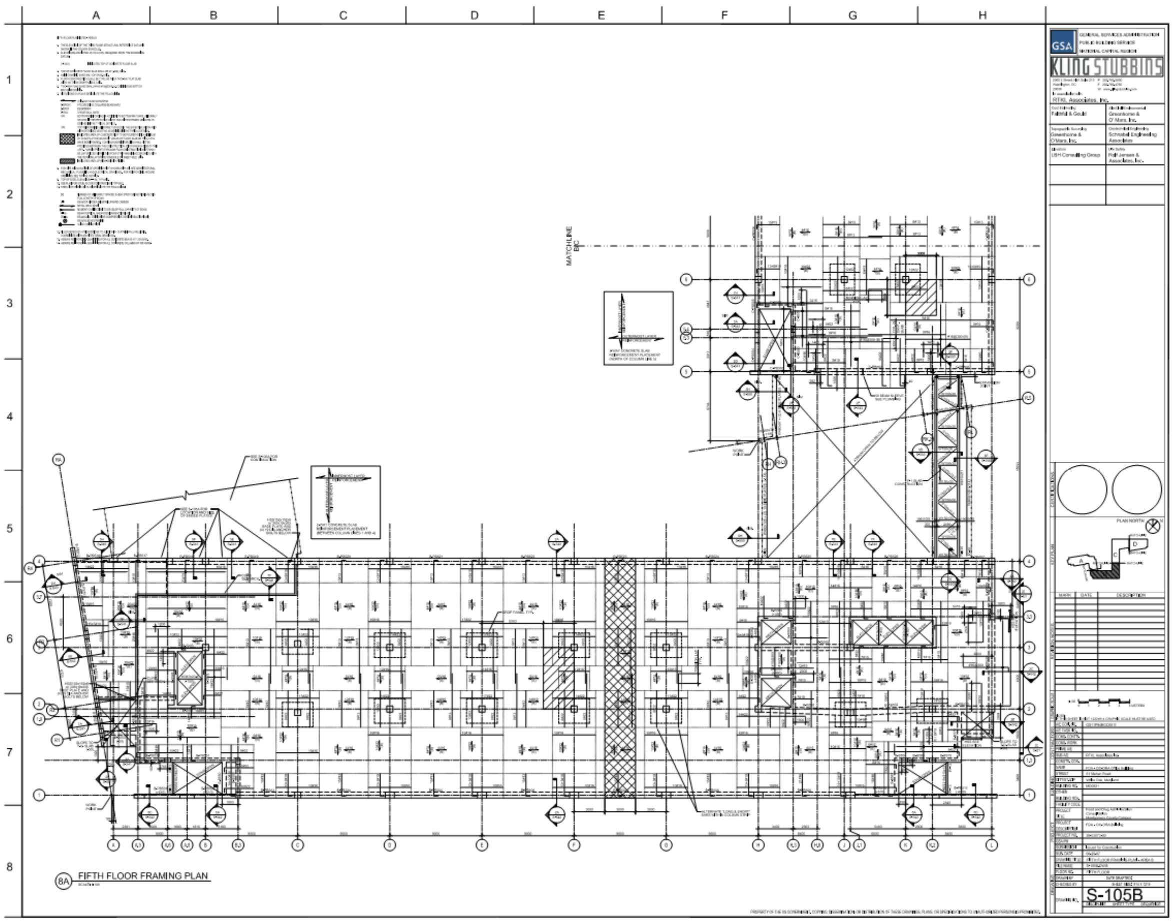


Wing B: Fourth Floor Framing Plans

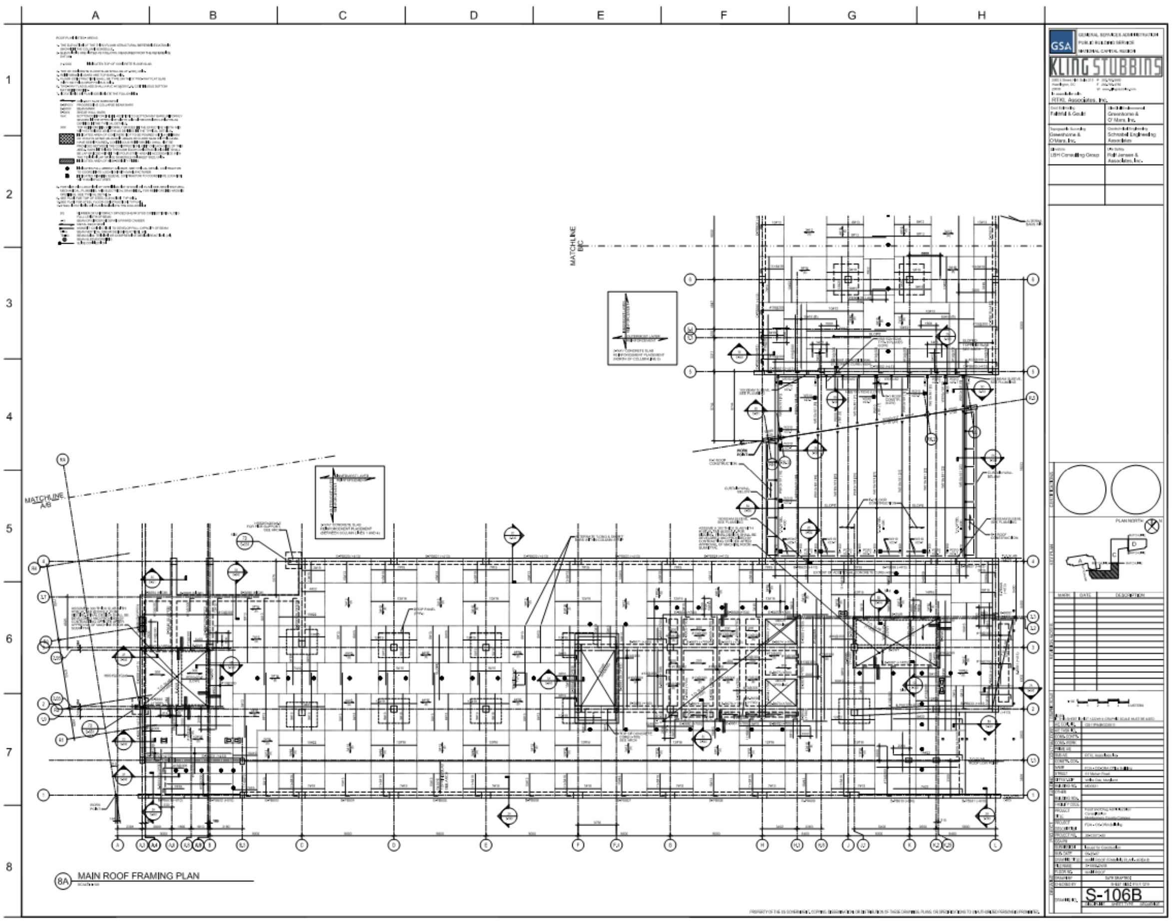


Technical Assignment #2

Wing B: Fifth Floor Framing Plan



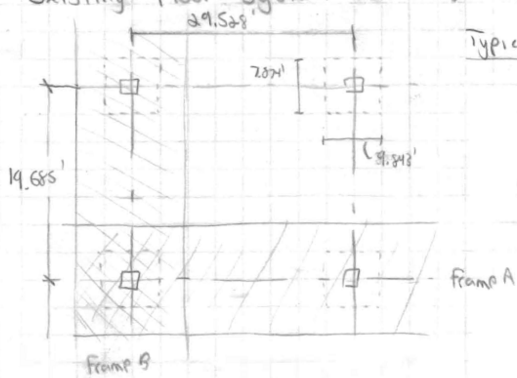
Wing B: Main Roof Framing Plan



Appendix B: Existing System: Two Way Flat Slab with Drop Panels

Existing floor 1

Existing floor System: Two-Way Flat slab with Drop Panels



Typical Interior Bay

$f'_c = 4000$ psi
$f_y = 60{,}000$ psi
LL = 80 psf
SDL = 15 psf
DL = $\frac{9.5}{12}(150)$ = 118.75 psf

Frame A: $l_n = 29.528 - \frac{23.62}{12} = 27.56'$
$S_n = 19.685 - \frac{23.62}{12} = 17.72'$
$l_2 = 19.685'$

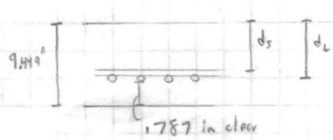
$CS_{width} = .5(19.685) = 9.843'$
$MS_{width} = 9.843'$

Frame B: $l_n = 17.72'$
$S_n = 27.56'$
$l_2 = 29.528'$

$CS_{width} = 9.843'$
$MS_{width} = 19.686'$

As Designed: two way flat slab with Drop panels, no interior beams

- 9.449" thick slab w/ 7.09 in drop panels at int. columns
  - $f'_c = 4000$ psi $f_y = 60{,}000$ psi
- Reinforcing #5 bars



$d_s = 9.449 - .787 - \frac{1}{2}(.625) - .625 = 7.72$ in
$d_L = 7.72 + .625 = 8.35$ in

at Drop: $d_s = (9.449 + 7.09) - .787 - \frac{1}{2}(.625) - .625 = 14.81$ in
$d_L = 14.81 + .625 = 15.44$ in

Bottom reinf: #4 @ 11.81 in o.c. Both directions
Top reinf:

Technical Assignment #2

Existing Floor 2

Direct Design Method - ACI 318-08 chapter 13.6

- Requirements
  - minimum of three continuous spans in each direction - Yes
  - Panels shall be rectangular, with a ratio of longer to shorter span center to center of supports not exceed 2 - Yes
  - Successive span lengths center to center of supports in each direction shall not differ by more than one third the longer span - Yes
  - Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines. Yes
  - All loads shall be due to gravity loads only and uniformly distributed over an entire panel. The unfactored live load shall not exceed two times the unfactored dead load. Yes

- Factored Load

  $1.2(118.75+15)+1.6(80) = 288.5$ psf

- Frame A 23.62" x 23.62" collumns D2, D3, C2, C3

  - $M_o = \frac{1}{8} w l_2 l_n^2 = \frac{1}{8}(.2885)(19.685)(29.528 - 23.62/12)^2 = 538.87$ ft-k
  - $\alpha = 0$ b/c no int beams
  - Min thickness of slab w/o interior beams [Table 9.5.3]

    For deflection control w/ drop panels

    $$l_n/36 = \frac{(29.528 - \frac{23.62}{12})12}{36} = 9.19" < 9.449" \text{ ok}$$

    * do not need to check deflection

  - Distribute moments m^- and m^+

    For interior frames

    $m^- = .65 M_o = .65(538.9)$ ft-k $= 350.27$ ft-k

    $m^+ = .36 M_o = .36(538.9) = 188.61$ ft-k

  - Distribute of M^- to C.S. and M.S. by ACI 318-08 sect. 13.6.4.1

    $l_2/l_1 = \frac{19.685}{29.528} = .67$, $\alpha = 0$

    75% of M^- to CS $= .75(350.27) = 262.7$ ft-k

    25% of M^- to MS $= .25(350.27) = 87.57$ ft-k

  - Distribute of M^+ to C.S. and M.S. by ACI 318-08 sect. 13.6.4.4

    $l_2/l_1 = .67$, $\alpha = 0$

    60% of M^+ to CS $= .60(188.61) = 113.17$ ft-k

    40% of M^+ to M.S $= .40(188.61) = 75.44$ ft-k

Existing Floor 3

Frame 13 23.62" × 23.62" collumns C2, C3, D2, D3

- $M_o = \frac{1}{8} w \, l_2 \, l_n^2 = (1/8)(.2885)(29.528)(19.685 - 23.62/12)^2 = 334.24$

- $\alpha = 0$ no int beams
- min thickness of slab w/o int. beams [table 9.5.3]
  for deflection control w/ drop panels.

$$\frac{l_n}{36} = \frac{(19.685 - \frac{23.62}{12})12}{36} = 5.91'' < 9.449'' \text{ ok}$$

* do not need to check deflection

- Distribute moments M^- and M^+

  For interior frames
  $M^- = .65 M_o = .65(334.24) = 217.25^{1k}$
  $M^+ = .35 M_o = .35(334.24) = 116.98^{1k}$

- Distribute moments M^- to C.S. and M.S by ACI 318-08 section

  $\alpha = 0$ 75% of M^- to C.S = .75(217.25) = 162.94 ^{1k}
  25% of M^- to M.S. .25(217.25) = 54.31 ^{1k}

- Distribute of M^+ to C.S and M.S. by ACI

  $\alpha = 0$ 60% of M^+ to C.S = .60(116.98) = 70.19 ^{1k}
  40% of M^+ to M.S = .4(116.98) = 46.79 ^{1k}

Existing Floor 4

Reinforcing Design: Frame A C.S.

Int Span

Item	Description	M^-	M^+
1	M_u (ft-k)	262.7	113.17
2.	CS width w Drop Panel	94.49 in	118.1
3.	Effective Depth	15.44 in	8.35 in
4.	$M_n = M_u/\phi = .9$	$\frac{262.7}{.9} = 291.9$	$\frac{113.17}{.9} = 125.7$
5.	$R = M_n/bd^2$	$\frac{291.9(12000)}{(94.49)(15.44)^2}$ 155.5 psi	$\frac{125.7(12000)}{118.1(8.35)^2}$ 183.2 psi
6.	$\frac{M(12)}{b}$	$\frac{262.7(12)}{94.49} = 33.36 \frac{in\text{-}k}{in}$	$\frac{113.17(12)}{118.1} = 11.5 \frac{in\text{-}k}{in}$
7.	ρ_{req}	.00265	.00314
8.	$As_{req} = \rho bd$	3.87 in²	3.096 in²
9.	$As_{min} = .0018bt$	.0018(94.49)(9.449+7.09) 2.81 in²	.0018(118.17)(9.449) 2.01 in²
10.	$N = \frac{\text{larger of 8,9}}{.31\ (A_{b5})}$	12.48 13	9.99 10
11.	$N_{min} = \frac{\text{width of strip}}{2t}$	$\frac{94.49}{2(16.539)} = 2.86 \Rightarrow 3$	$\frac{118.1}{2(9.449)} = 6.25 \Rightarrow 7$
		13 #5	10 #5

Existing floor 5

Reinforcing Design: Frame A M.S.

Item	Description	M^-	M^+
		87.57 k	75.44 k
1	M_u (ft-k)	118.12	118.12
2	M.S. width	8.35 in	8.35 in
3	Effective depth	$\frac{87.57}{.9}$ = 97.3	$\frac{75.44}{.9}$ = 83.82
4	$M_n = M_u/\phi$: .9		
5	$\frac{M_u(12)}{b}$	$\frac{87.57(12)}{118.12}$ = 8.9 in-k/in	$\frac{75.44(12)}{118.12}$ = 7.66 in-k/in
6	$R = \frac{M_n}{bd^2}$	$\frac{97.3(12000)}{118.12(8.35)^2}$	$\frac{83.82(12000)}{118.12(8.35)^2}$
		141.77 in-k	122.15 in-k
7	ρ_{req}		
	$R = \rho f_y(1 - .59\rho \frac{f_y}{f'_c})$	.00241	.00207
8	$A_{s,req} = \rho bd$	2.38 in²	2.042 in²
9	$A_{s,min} = .0018bt$	.0018(118.12)(9.449)	
		2.01 in²	2.01 in²
10	$N = \frac{A_{s,req}}{.31\ (A_{bar})}$	7.68	6.59
		8	7
11	$N_{min} = \frac{width}{2t}$	$\frac{118.12}{2(9.449)}$	
		6.25	
		7	7
		8 #5	7 #5

Technical Assignment #2

Existing Floor 6

Reinforcing Design: Frame B C.S.

Item	Description	M^-	M^+
1	M_u (ft-k)	162.94	70.19 k
2	C.S. Width w/ Drop Panel	94.49 in	118.1 in
3	Effective depth	14.81 in	7.72 in
4	$M_n = M_u/\phi = .9$	181.04 k	77.99 k
5	$\frac{M(12)}{b}$	$\frac{162.94(12)}{94.49} = 20.69$ ik	7.13 in-k
6	$R = M_n/bd^2$	$\frac{181.04(12000)}{94.49(14.81)^2}$ = 104.82 in-lb	$\frac{77.94(12000)}{118.1(7.72)^2}$ = 132.96 in-lb
7	ρ_{req} $R = \rho f_y(1-.59\rho f_y/f'_c)$	.00177	.00226
8	$A_{s,req} = \rho bd$	2.477 in²	2.06 in²
9	$A_{s,min}$ .0018 bt	.0018(94.49)(9.449 ?) = 2.813 in²	.0018(118.1)(9.449) 2.01 in²
10	$N = \frac{A_s}{A_{\#5} = .31}$	9.07 10	6.65 7
11	$N_{max} = \frac{\text{width strip}}{2t}$	$\frac{94.49}{2(15.59)} = 2.88 \geq 3$	$\frac{118.1}{2(9.449)} = 6.25 \geq 7$
		10 #5	7 #5

Existing Floor 7

Reinforcing Design : Frame B M.S.

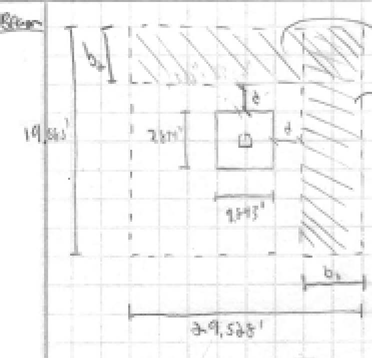
Item	Description	M^-	M^+
1	M_u (ft-k)	54.31	46.79
2.	MS width	236.2 in	236.2 in
3.	Effective depth	7.72 in	7.72 in
4.	$M_n = M_u/\phi = .9$	$\frac{54.31}{.9} = 60.34$ ik	$\frac{46.79}{.9} = 51.99$ ik
5.	$\frac{M(12)}{b}$	$\frac{54.31(12)}{236.2} = 2.76$ in-k	$\frac{46.79(12)}{236.2} = 2.38$ in-k
6.	$R = M_n/bd^2$	$\frac{60.34(12000)}{236.2(7.72)^2} = 51.44$ in-lb	$\frac{51.99(12000)}{236.2(7.72)^2} = 44.32$ in-lb
7.	ρ_{req} $R = \rho f_y(1 - .59\rho f_y/f'_c)$	.00086	.000744
8.	$A_{s\,req} = \rho b d$	1.57 in²	1.36 in²
9.	$A_{s\,min} = .0018bt$	.0018(236.2)(9.449) = 4.02 in² 4.02 in²	4.02 in²
10.	$N = \frac{A_s}{A_{\#5} = .31}$	12.97 => 13	13
11.	$N_{max} = \frac{\text{Width}}{2t} = \frac{236.2}{2(9.449)}$	12.49 => 13 13 #5	13 13 #5

Technical Assignment #2

- Shear checks

Existing Floor 8

wide Beam



Critical Area 1:

$\frac{29.528}{2} - \frac{9.843}{2} - \frac{14.81}{12} = 8.608$ ft

$A_{c1} = 8.608(19.685) = 171\ ft^2$

Critical Area 2:

$\frac{19.685}{2} - \frac{7.874}{2} - \frac{14.81}{12} = 4.67$ ft

$A_{c2} = 4.67(29.528) = 137.94\ ft^2$

$d = 14.81$ in

Load 6 wide beam action

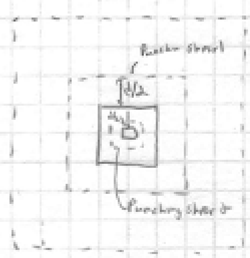
$V_u = w_u \cdot A_{c1}$

$w_u = 1.2(118.75+15) + 1.6(80) = 288.5$ psf $= .2885$ ksf

$V_1 = .2885(171) = 49.3^k$ $\phi V_{n1} = .75(2)\sqrt{4000}(19.685)(12)(7.72)/1000 = 173^k$ ok

$V_{u2} = .2885(137.94) = 39.79^k$ $\phi V_{n2} = .75(2)\sqrt{4000}(29.528)(12)(7.72)/1000 = 259.5^k$ ok

Punching shear



Area of Punching shear 1, 2

$A_{P1} = (7.874 + 7.72/12)(9.843 + 7.72/12) = 89.3\ ft^2$

$A_{P2} = \left(\frac{23.62}{12} + 14.81/12\right)^2 = 10.256$

$A_T = (19.685)(29.528) = 581.26\ ft^2$

Punching shear 1

$V_u = w_u A = .2885(581.26 - 89.3) = 141.93^k$

$V_c = 4\sqrt{f'_c}\, b_o d$

$= 4\sqrt{4000}(38.01)(7.72)(12)/1000 = 890.85^k$

$b_o = 2(7.874 + \frac{7.72}{12}) + 2(9.843 + 7.72/12)$
$= 38.01$ ft

$\beta = \frac{9.843 + 7.72/12}{7.874 + 7.72/12} = 1.23$

$V_c = (2 + 4/\beta)\sqrt{f'_c}\, bod$

$5.25\sqrt{f'_c}\, bod \rightarrow$ D.N.C.

$V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{\frac{38.01(12)}{7.72}} + 2\right)(\sqrt{4000})(38.01)(12)(7.72)/1000 = 596.2^k$

$\phi V_c = .75(596.2) = 447.^k > V_u = 141.9^k$ ok

Existing floor 9

Shear checks cont

Punching shear 2

$V_u = w_u A = .2555(581.26 - 10.256) = 164.7^k$

$b_o = 2(23.62 + 14.81) + 2(23.62 + 14.81)$

$b_o = 153.72$ in

$V_c = 4\sqrt{4000}(153.72)(14.81)/1000 = 575.94^k$

$\beta = 1 \Rightarrow$ square

$V_c = (2 + 4/\beta)\sqrt{f'_c}\, b_o d \rightarrow$ D.W.C.

$6\sqrt{f'_c}\, b_o d$

$V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{\frac{153.72}{14.81}} + 2\right)(\sqrt{4000})(153.72)(14.81)/1000 = 842.9^k$

$\phi V_c = .75(575.94) = 431.95^k > V_u = 164.7^k$ ok

Weight per square foot of two way flat plat system

Slab: $\frac{9.449}{12}(150) = 118.1$ psf $\quad w = 118.1(29.528)(19.685) = 68,646.65$ lbs

Drop Panel: $\frac{7.09}{12}(150) = 88.6$ psf $\quad w = 88.6(7.874)(9.843) = 6,866.84$ lbs

$w_T = \frac{68646.65 + 6866.84}{(29.528)(19.685)} = 129.9$ psf

$w_T = 129.9$ psf

Cost per square foot

Fire Protection

Table 2.3—Minimum cover in concrete floors and roof slabs

Aggregate type	Cover*† for corresponding fire resistance, in.					
	Restrained	Unrestrained				
	4 or less	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
			Nonprestressed			
Siliceous	3/4	3/4	3/4	1	1-1/4	1-5/8
Carbonate	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Semi-lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
			Prestressed			
Siliceous	3/4	1-1/8	1-1/2	1-3/4	2-3/8	2-3/4
Carbonate	3/4	1	1-3/8	1-5/8	2-1/8	2-1/4
Semi-lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4
Lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4

*Shall also meet minimum cover requirements of 2.3.1.
†Measured from concrete surface to surface of longitudinal reinforcement.

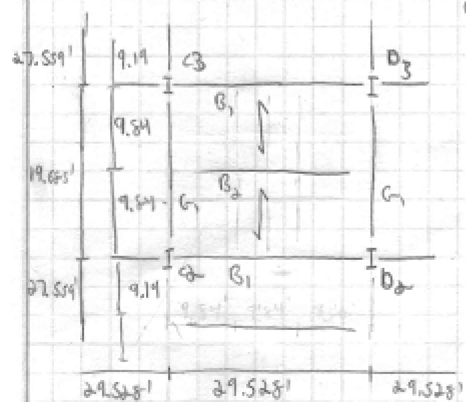
RS Means data was taking for a 25’ x 30’ bay size.

B1010 222 — Cast in Place Flat Slab with Drop Panels

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB & DROP (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	COST PER S.F. INST.	COST PER S.F. TOTAL
1700	15 x 15	40	12	6 - 1-1/2	117	5.35	7.70	13.05
1720	RB1010 -010	75	12	6 - 2-1/2	153	5.50	7.75	13.25
1760		125	14	6 - 3-1/2	205	5.85	7.90	13.75
1780		200	16	6 - 4-1/2	281	6.25	8.10	14.35
1840	15 x 20	40	12	6-1/2 - 2	124	5.70	7.80	13.50
1860	RB1010 -100	75	14	6-1/2 - 4	162	6.05	8	14.05
1880		125	16	6-1/2 - 5	213	6.50	8.20	14.70
1900		200	18	6-1/2 - 6	293	6.80	8.35	15.15
1960	20 x 20	40	12	7 - 3	132	6	7.95	13.95
1980		75	16	7 - 4	168	6.50	8.15	14.65
2000		125	18	7 - 6	221	7.30	8.40	15.70
2100		200	20	8 - 6-1/2	309	7.50	8.55	16.05
2300	20 x 25	40	12	8 - 5	147	6.80	8.25	15.05
2400		75	18	8 - 6-1/2	184	7.50	8.55	16.05
2600		125	20	8 - 8	236	8.35	8.90	17.25
2800		200	22	8-1/2 - 8-1/2	323	8.75	9.15	17.90
3200	25 x 25	40	12	8-1/2 - 5-1/2	154	7.15	8.30	15.45
3400		75	18	8-1/2 - 7	191	7.70	8.60	16.30
4000		125	20	8-1/2 - 8-1/2	243	8.50	8.95	17.45
4400		200	24	9 - 8-1/2	329	8.95	9.15	18.10
5000	25 x 30	40	14	9-1/2 - 7	168	7.85	8.65	16.50
5200		75	18	9-1/2 - 7	203	8.60	9	17.60
5600		125	22	9-1/2 - 8	256	9.15	9.20	18.35
5800		200	24	10 - 10	342	9.80	9.50	19.30

Appendix C: Alternate System 1: Composite Steel Framing

Alternate System 1: Composite steel on metal deck.



Given:
LL: 80 psf
SDL = 15 psf
DL: self weight of members

Assume
Normal weight Concrete
Use: United Steel Deck to determine deck size, and slab thickness

- By United steel Deck determine deck size, and slab thickness

Deck Properties

Use 18 Gage Deck with the following Properties 2" Lok-Floor

$t = .0474$ in — $I = .560\ in^4$ — $R_b = 1080$ lb per 12" width
$w = 2.4$ psf — $S_p = .523\ in^3$ — $\phi V_n = 3150$ lb per 12" width
$A_s = .710\ in^2$ — $S_n = .529\ in^3$ — studs = .57 # studs per foot to obtain full resisting moment

Composite Properties - For 18 gage deck with $4\frac{1}{2}$" slab depth

slab depth = 4.5 in
$\phi M_{nf} = 62.08$ in k
$w = 42$ psf

Max unshored spans
3 span = 12.48 ft

L, uniform Live Service Loads, psf - For 18 gage deck, Normal weight Conc.
For slab depth of 4.5 in, and a span of 9.84' ≈ 10'

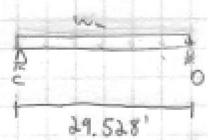
Uniform Live Load (Including self weight) = 225 psf
* Exceeds required loading ∴ ok

- Use: 18 gage 2" Lok-Floor Deck
with 4.5 in Normal weight Concrete slab depth

Alternate system: 2

Alternate System 1: Composite steel on metal deck

Beam Design



Tributary width: 9.84'

Loads: DL: 44.4 psf not including beam, increase by 10% for beam self weight

DL: 48.84 psf
SDL: 15 psf
LL: 80 psf (Includes partitions)

w_u: 1.2(48.84+15) + 1.6(80) = 204.61 psf (1.4D D.N.C.)

w_u: 204.61(9.84)/1000 = 2.01 K/ft

$M_u = \frac{wl^2}{8} = \frac{2.01(29.528)^2}{8} = 219.43$ ft-k

$V_u = \frac{wl}{2} = \frac{2.01(29.528)}{2} = 29.7$ k

Assume a = 1.5 in ∴ $Y_2 = 4.5 - 1.5/2 = 3.75$ in

Try W14X22 → $\phi M_p = 125$ 'K

ϕM_n: 230 'K use Y_2: 3.5 to be conservative

Y_1: 3

ΣQ_n: 241 k

Studs: $\frac{\Sigma Q_n}{q_n} = \frac{241}{17.2} = 14.01 = 15$ studs

across entire beam = 30 studs

Equiv. Weight: 29.528(22) + 54(10 lb/ft) = 949.6 lb = .95 K
assumed

$M_u = \frac{(.95/29.528)(29.528)^2}{8} = 3.5$ ft kips

M_{u_T}: 219.43 + 3.5 = 222.9 ft k < 230 ft kips ok

check assumption

b_{eff}: 9.84 or $\frac{29.528}{4}$ = 7.382

b_{eff}: 7.382(12) = 88.58 in

$a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{241}{.85(3)(88.58)} = 1.07$ in

a = 1.07 < 1.5 in conservative.

Alternate System 1: 3

Alternate Design 1: Beam cont

Check load during Construction

DL = 48.84 psf = 49.4 (9.84) / 1000 = .437 k/ft

Selfweight: 195/29.528 = .032 k/ft

$M_c = \frac{w\ell^2}{8} = \frac{(.437 + .032)\,29.528^2}{8} = 51.12^{\,\text{ik}} < \emptyset M_p = 125^{\,\text{ik}}$ ok

Check deflection

$-\Delta_{cons} = \frac{5w\ell^4}{384EI}$ $w_0 = .044(9.84) = .433$ k/ft

$w_{sw} = .032 + .01 = .032$ k/ft
beam st.

$\Delta_{cons} = \frac{5(.433 + .032)(29.528)^4 (1728)}{384(29000)(199)} = 1.38$ in

limit $\ell/360 = 29.528(12)/360 = .98$ in

$\Delta_{cons} = 1.38 > \ell/360 = .98$ no good

- Add a 1/2" Camber to beam

Live load deflection

$\Delta_L = \frac{5w\ell^4}{384EI} = \frac{5(.08)(9.84)(29.528)^4 (1728)}{384(29000)(496)} = .936$ in

limit $\ell/360 = .98$ in Table 3-20

$\Delta_L = .936 < .98$ in ok

Check shear:

$V_u = 29.7^k$

$\emptyset V_n = 94.8$ by Table 3-2

$V_u = 29.7^k < \emptyset V_n = 94.8^k$ ok

* B_1 has trib width of $(9.84 + 9.19)/2 = 9.52'$
which is less than B_2.
Design for B_2 beam is conservative for B1

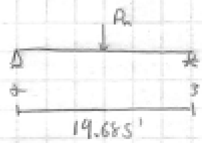
Use W14x22 (30) for B1, B2

d = 13.7 in

Alternate System 1: 4

Alternate System 1: Composite Beam, Composite Deck

Girder Design



Reaction From Beam on one side

$R = V_L = 29.7^k$

$P_u = 2(29.7)^k = 59.4^k$

$P_D =$

$M_u = \frac{P\ell}{4} = \frac{59.4(19.685)}{4} = 292.3$ ft-k

$V_u = \frac{P}{2} = \frac{59.4}{2} = 29.7^k$

Assume $a = 2$ in $\therefore Y_2 = 4.5 - 2/2 = 3.5$ in

Try W16x26

$\phi M_p = 166^{'k}$

$\phi M_n = 301^{'k}$

$\Sigma Q_n = 259^k$

$Y_1 = 3$

By Table 3-21 Parallel Deck 3/4" stud $Q_n = 12.1^k$

$\# \text{ studs} = \frac{\Sigma Q_n}{Q_n} = \frac{259}{12.1} = 16.9 = 17$

Total # studs: 34 studs

Equiv weight: $19.685(26) + 34(10) = 851.8$ lb $= .852$ k

Check assumptions

$b_{eff} = 19.685/4 = 4.92' = 59.1$ in

$a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{259}{.85(3)(59.1)} = 1.92$ in < 2 in ok conservative

Check deflection

$P_D = (.044(9.84)(29.528) + .95)/2 = 6.87^k$

From 1 Beam $P_L = (.08)(9.84)(29.528)/2 = 11.62^k$

Total $P_D = 2(6.87) = 13.73^k$

$P_L = 2(11.62) = 23.24^k$

Alternate system 1: Steel on Metal Deck

Girder Design cont.

Check deflection

Construction Loads

$\Delta_{const} = \frac{P\ell^3}{48EI} = \frac{13.73(19.685)^3(1728)}{48(29000)(301)} = .432 \text{ in}$

Limit $\ell/360 = \frac{19.685(12)}{360} = .656 \text{ in}$

$\Delta_{const} = .432 \text{ in} < .656 \text{ in}$ ok

Live Load Deflection

$\Delta_L = \frac{P\ell^3}{48EI} = \frac{22.24(19.685)^3(1728)}{48(29000)(726)} = .303$

$\Delta_{all} = \ell/360 = .656$

$\Delta_L = .303 < .656$ ok

Use W16x26 (34) For Girders
d = 15.7 in

Equiv weight of Floor system

Slab and metal Deck
42 psf
2.4 psf   total = 44.4 psf

Beams - 3 Beams

$\frac{22 \text{ plf}(29.528)}{29.528(19.685)} = 1.12 \text{ psf}$

Girders - 2 Girders

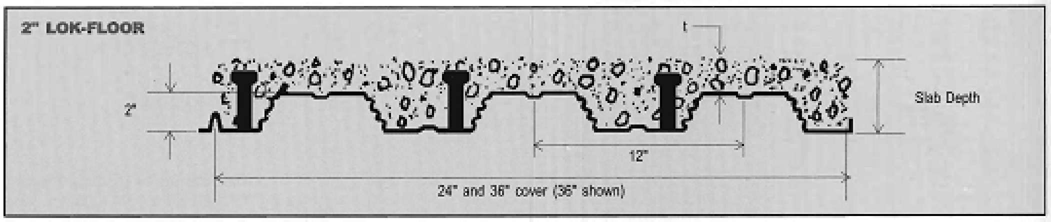
$\frac{26 \text{ plf}(19.685)}{29.528(19.685)} = .88 \text{ psf}$

Total = 44.4 + 1.12 + .88
= 46.4 psf

United Steel Deck Design Manual

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

United Steel Deck, Inc. LRFD



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

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{sf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.^{2} per foot of width. **Vol.** is the volume of concrete in ft.^{3} per ft.^{2} needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.^{2}. S_c is the section modulus of the "cracked" concrete composite slab; in.^{3} per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.^{4} per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{½} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

DECK PROPERTIES

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.36
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.43
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.51
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.57
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3890	0.72

COMPOSITE PROPERTIES

	Slab Depth	ϕM_{sf} in.k	A_c in^2	Vol. ft^3/ft^2	W psf	S_c in^3	I_{av} in^4	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	46.44	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	49.53	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.39	0.029
	5.50	52.61	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	58.78	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	61.87	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	64.95	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	71.12	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	74.21	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	77.29	64.3	0.542	79	2.21	26.0	61.88	7970	4.74	6.22	6.31	0.050
20 gage	4.50	48.60	32.6	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	56.18	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	59.96	40.0	0.354	51	1.60	9.8	44.84	6140	6.32	8.36	8.63	0.029
	5.50	63.75	42.6	0.375	54	1.71	11.3	48.07	6380	6.18	8.18	8.45	0.032
	6.00	71.32	48.0	0.417	60	1.95	14.5	54.63	6880	5.94	7.85	8.11	0.036
	6.25	75.11	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	78.90	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	86.47	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	90.26	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	94.05	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050
19 gage	4.50	55.85	32.6	0.292	42	1.45	6.7	40.69	5850	7.65	9.76	10.08	0.023
	5.00	64.68	37.5	0.333	48	1.71	9.0	47.87	6300	7.26	9.30	9.61	0.027
	5.25	69.10	40.0	0.354	51	1.84	10.4	51.56	6540	7.09	9.09	9.39	0.029
	5.50	73.52	42.6	0.375	54	1.97	11.9	55.30	6780	6.93	8.90	9.19	0.032
	6.00	82.35	48.0	0.417	60	2.24	15.2	62.90	7280	6.65	8.54	8.83	0.036
	6.25	86.77	50.8	0.438	63	2.38	17.1	66.76	7540	6.56	8.38	8.66	0.038
	6.50	91.19	53.6	0.458	66	2.52	19.2	70.65	7800	6.48	8.23	8.50	0.041
	7.00	100.03	59.5	0.500	73	2.80	23.8	78.50	8350	6.32	7.94	8.20	0.045
	7.25	104.44	61.9	0.521	76	2.94	26.3	82.46	8570	6.24	7.81	8.07	0.047
	7.50	108.86	64.3	0.542	79	3.08	29.0	86.45	8790	6.17	7.68	7.94	0.050
18 gage	4.50	62.08	32.6	0.292	42	1.62	7.0	45.34	6080	8.42	10.48	10.83	0.023
	5.00	72.04	37.5	0.333	48	1.90	9.5	53.36	6670	7.98	9.99	10.32	0.027
	5.25	77.02	40.0	0.354	51	2.05	10.9	57.48	6910	7.79	9.77	10.10	0.029
	5.50	82.00	42.6	0.375	54	2.20	12.4	61.66	7150	7.61	9.56	9.88	0.032
	6.00	91.95	48.0	0.417	60	2.50	15.9	70.18	7650	7.30	9.18	9.49	0.036
	6.25	96.93	50.8	0.438	63	2.66	17.9	74.50	7910	7.20	9.01	9.31	0.038
	6.50	101.91	53.6	0.458	66	2.81	20.0	78.85	8170	7.11	8.85	9.14	0.041
	7.00	111.87	59.5	0.500	73	3.13	24.8	87.66	8720	6.93	8.54	8.82	0.045
	7.25	116.85	61.9	0.521	76	3.28	27.4	92.10	8940	6.85	8.40	8.68	0.047
	7.50	121.83	64.3	0.542	79	3.44	30.2	96.57	9160	6.77	8.26	8.54	0.050
16 gage	4.50	62.08	32.6	0.292	42	1.99	7.7	45.34	6080	9.58	11.63	12.02	0.023
	5.00	72.04	37.5	0.333	48	2.35	10.4	53.36	6980	9.08	11.10	11.47	0.027
	5.25	77.02	40.0	0.354	51	2.53	11.9	57.48	7450	8.85	10.85	11.22	0.029
	5.50	82.00	42.6	0.375	54	2.72	13.6	61.66	7940	8.65	10.63	10.98	0.032
	6.00	91.95	48.0	0.417	60	3.10	17.4	70.18	8460	8.29	10.21	10.55	0.036
	6.25	96.93	50.8	0.438	63	3.29	19.5	74.50	8720	8.17	10.02	10.35	0.038
	6.50	101.91	53.6	0.458	66	3.48	21.8	78.85	8980	8.07	9.84	10.17	0.041
	7.00	111.87	59.5	0.500	73	3.88	27.0	87.66	9530	7.86	9.50	9.82	0.045
	7.25	116.85	61.9	0.521	76	4.08	29.8	92.10	9750	7.77	9.35	9.66	0.047
	7.50	121.83	64.3	0.542	79	4.28	32.8	96.57	9970	7.67	9.20	9.50	0.050

2" LOK-FLOOR

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

	Slab Depth	φMn in.k	L, Uniform Live Service Loads, psf * 6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
22 gage	4.50	40.27	400	365	310	265	230	200	175	155	135	120	105	95	85
22 gage	5.00	46.44	400	400	360	305	265	230	200	175	155	140	125	110	95
22 gage	5.50	52.61	400	400	400	350	300	260	230	200	175	155	140	125	110
22 gage	6.00	58.78	400	400	400	390	335	295	255	225	200	175	155	140	125
22 gage	6.50	64.95	400	400	400	400	370	325	285	250	220	195	175	155	135
22 gage	7.00	71.12	400	400	400	400	400	355	310	275	240	215	190	170	150
22 gage	7.25	74.21	400	400	400	400	400	370	325	285	250	225	200	175	155
22 gage	7.50	77.29	400	400	400	400	400	385	340	295	260	230	205	185	165
20 gage	4.50	48.60	400	400	380	325	285	245	215	190	170	150	135	120	110
20 gage	5.00	56.18	400	400	400	380	330	285	250	220	195	175	155	140	125
20 gage	5.50	63.75	400	400	400	400	375	325	285	250	225	200	175	160	140
20 gage	6.00	71.32	400	400	400	400	400	365	320	285	250	225	200	180	160
20 gage	6.50	78.90	400	400	400	400	400	400	355	315	280	245	220	195	175
20 gage	7.00	86.47	400	400	400	400	400	400	390	345	305	270	240	215	195
20 gage	7.25	90.26	400	400	400	400	400	400	400	360	320	285	255	225	205
20 gage	7.50	94.05	400	400	400	400	400	400	400	375	330	295	265	235	210
19 gage	4.50	55.85	400	400	400	380	330	290	255	225	200	180	160	145	130
19 gage	5.00	64.68	400	400	400	400	385	335	295	260	230	205	185	165	150
19 gage	5.50	73.52	400	400	400	400	400	380	335	295	265	235	210	190	170
19 gage	6.00	82.35	400	400	400	400	400	400	375	335	295	265	235	215	190
19 gage	6.50	91.19	400	400	400	400	400	400	400	370	330	295	265	235	210
19 gage	7.00	100.03	400	400	400	400	400	400	400	400	360	320	290	260	235
19 gage	7.25	104.44	400	400	400	400	400	400	400	400	375	335	300	270	245
19 gage	7.50	108.86	400	400	400	400	400	400	400	400	395	350	315	280	255
18 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
18 gage	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
18 gage	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
18 gage	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
18 gage	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
18 gage	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
18 gage	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
18 gage	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
16 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
16 gage	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
16 gage	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
16 gage	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
16 gage	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
16 gage	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
16 gage	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
16 gage	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
22 gage	4.50	29.40	305	255	215	185	160	135	120	105	90	80	70	60	50
22 gage	5.00	34.53	360	305	255	220	185	160	140	120	105	95	80	70	65
22 gage	5.50	39.81	400	350	295	255	215	190	165	140	125	110	95	85	75
22 gage	6.00	45.21	400	400	340	290	250	215	185	160	140	125	110	95	85
22 gage	6.50	50.70	400	400	380	325	280	240	210	185	160	140	125	110	95
22 gage	7.00	56.26	400	400	400	360	310	270	235	205	180	155	140	120	105
22 gage	7.25	59.07	400	400	400	380	325	285	245	215	190	165	145	130	115
22 gage	7.50	61.88	400	400	400	400	345	295	260	225	200	175	155	135	120
20 gage	4.50	35.43	375	315	270	230	200	170	150	130	115	100	90	80	70
20 gage	5.00	41.65	400	375	315	270	235	205	175	155	135	120	105	95	85
20 gage	5.50	48.07	400	400	365	315	270	235	205	180	160	140	125	110	95
20 gage	6.00	54.63	400	400	400	360	310	270	235	205	180	160	140	125	110
20 gage	6.50	61.31	400	400	400	400	350	300	265	230	205	180	160	140	125
20 gage	7.00	68.09	400	400	400	400	390	335	295	260	230	200	180	160	140
20 gage	7.25	71.50	400	400	400	400	400	355	310	270	240	210	190	165	150
20 gage	7.50	74.93	400	400	400	400	400	370	325	285	250	225	200	175	155
19 gage	4.50	40.69	400	370	315	270	230	200	175	155	135	120	105	95	85
19 gage	5.00	47.87	400	400	370	315	275	240	210	185	160	145	125	115	100
19 gage	5.50	55.30	400	400	400	365	320	275	240	215	190	165	150	130	120
19 gage	6.00	62.90	400	400	400	400	365	315	275	245	215	190	170	150	135
19 gage	6.50	70.65	400	400	400	400	400	355	310	275	245	215	190	170	155
19 gage	7.00	78.50	400	400	400	400	400	395	350	305	270	240	215	190	170
19 gage	7.25	82.46	400	400	400	400	400	400	365	320	285	255	225	200	180
19 gage	7.50	86.45	400	400	400	400	400	400	385	340	300	265	235	210	190
18 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
18 gage	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
18 gage	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
18 gage	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
18 gage	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
18 gage	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
18 gage	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
18 gage	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220
16 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
16 gage	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
16 gage	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
16 gage	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
16 gage	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
16 gage	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
16 gage	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
16 gage	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220

LRFD

1 STUD/FT.

NO STUDS

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

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

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

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

2" LOK-FLOOR

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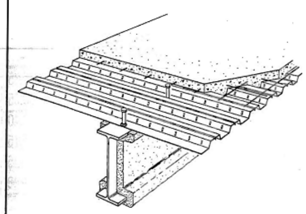
RS Means data was taking for 25' x 30' bays size.

6·54.13

B10 Superstructure

B1010 Floor Construction



Description: Table below lists costs ($/S.F.) for a floor system using composite steel beams with welded shear studs, composite steel deck, and light weight concrete slab reinforced with W.W.F. Price includes sprayed fiber fireproofing on steel beams.

Design and Pricing Assumptions:
Structural steel is A36, high strength bolted.
Composite steel deck varies from 22 gauge to 16 gauge, galvanized.
Shear Studs are 3/4".
W.W.F., 6 x 6 – W1.4 x W1.4 (10 x 10)
Concrete f'c = 3 KSI, lightweight.
Steel trowel finish and cure.
Fireproofing is sprayed fiber (non-asbestos).
Spandrels are assumed the same as interior beams and girders to allow for exterior wall loads and bracing or moment connections.

System Components	QUANTITY	UNIT	COST PER S.F. MAT.	COST PER S.F. INST.	COST PER S.F. TOTAL
SYSTEM B1010 256 2400					
20X25 BAY, 40 PSF S. LOAD, 5-1/2" SLAB, 17-1/2" TOTAL THICKNESS					
Structural steel	4.320	Lb.	7.26	1.73	8.99
Welded shear connectors 3/4" diameter 4-7/8" long	.163	Ea.	.12	.30	.42
Metal decking, non-cellular composite, galv. 3" deep, 22 gauge	1.050	S.F.	3.08	.90	3.98
Sheet metal edge closure form, 12", w/2 bends, 18 ga, galv	.045	L.F.	.26	.10	.36
Welded wire fabric rolls, 6 x 6 - W1.4 x W1.4 (10 x 10), 21 lb/csf	1.000	S.F.	.20	.34	.54
Concrete ready mix, light weight, 3,000 PSI	.333	C.F.	2.58		2.58
Place and vibrate concrete, elevated slab less than 6", pumped	.333	C.F.		.47	.47
Finishing floor, monolithic steel trowel finish for finish floor	1.000	S.F.		.78	.78
Curing with sprayed membrane curing compound	.010	C.S.F.	.06	.08	.14
Shores, erect and strip vertical to 10' high	.020	Ea.		.38	.38
Sprayed mineral fiber/cement for fireproof, 1" thick on beams	.483	S.F.	.28	.43	.71
TOTAL			13.84	5.51	19.35

B1010 256 Composite Beams, Deck & Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	SLAB THICKNESS (IN.)	TOTAL DEPTH (FT.-IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	COST PER S.F. INST.	COST PER S.F. TOTAL
2400	20x25	40	5-1/2	1 - 5-1/2	80	13.85	5.50	19.35
2500	RB1010 -100	75	5-1/2	1 - 9-1/2	115	14.40	5.55	19.95
2750		125	5-1/2	1 - 9-1/2	167	17.70	6.50	24.20
2900		200	6-1/4	1 - 11-1/2	251	19.85	7	26.85
3000	25x25	40	5-1/2	1 - 9-1/2	82	13.70	5.25	18.95
3100		75	5-1/2	1 - 11-1/2	118	15.30	5.35	20.65
3200		125	5-1/2	2 - 2-1/2	169	15.95	5.75	21.70
3300		200	6-1/4	2 - 6-1/4	252	22	6.70	28.70
3400	25x30	40	5-1/2	1 - 11-1/2	83	14	5.20	19.20
3600		75	5-1/2	1 - 11-1/2	119	15.10	5.25	20.35
3900		125	5-1/2	1 - 11-1/2	170	17.60	5.95	23.55
4000		200	6-1/4	2 - 6-1/4	252	22	6.80	28.80
4200	30x30	40	5-1/2	1 - 11-1/2	81	13.95	5.40	19.35
4400		75	5-1/2	2 - 2-1/2	116	15.15	5.60	20.75
4500		125	5-1/2	2 - 5-1/2	168	18.40	6.30	24.70
4700		200	6-1/4	2 - 9-1/4	252	22	7.30	29.30
4900	30x35	40	5-1/2	2 - 2-1/2	82	14.65	5.55	20.20
5100		75	5-1/2	2 - 5-1/2	117	16.05	5.70	21.75
5300		125	5-1/2	2 - 5-1/2	169	19	6.45	25.45
5500		200	6-1/4	2 - 9-1/4	254	22	7.35	29.35
5750	35x35	40	5-1/2	2 - 5-1/2	84	15.75	5.55	21.30
6000		75	5-1/2	2 - 5-1/2	121	18	5.95	23.95

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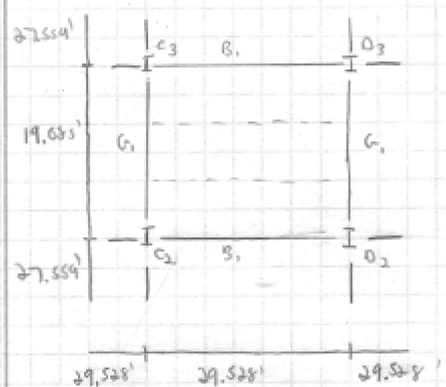
Appendix D: Alternate System 2: Hollow Core Planks on Steel

Alternate System 2: 1

Alternate System 2: Hollow Core Planks on steel Beams



Use 10"x4' Hollow Core Planks w/ 2" topping by Nitterhouse

Precast w_t = 68 psf
topping w_t = 25 psf
SDL = 15 psf
LL = 80 psf

Superimpose service loads
$w = 80 + 15 + 25 = 120$ psf

For 30' span → 6-1/2" strand pattern
safe superimposed service load is 128 psf > 120 psf ok
From PCI 2.05

Plank capacity
$1.2(15+25+68) + 1.6(80) = 257.6$ psf
$257.6(4') = 1030.4$ plf

$M_u = \frac{1.03(29.528)^2}{8} = 112.3$ 'k < 168.1 'k ok

Use 10" x 4' Hollow Core Plank w/ 2" topping
6-1/2" strand pattern

- Design Beams - assume tributary width on both sides to be 2' (1/2 of 4' plank)

B1

DL = $(68+25)4 = 372$ plf
SDL: $(15)4 = 60$ plf
LL: $80(4) = 320$ plf
$W_u = 1.2(372+60) + 1.6(320) = 1030.4$ plf $= 1.03$ klf
L/40 D.N.C.

$M_u = \frac{wl^2}{8} = \frac{1.03(29.528)^2}{8} = 111.2$ 'k

$V_u = \frac{wl}{2} = \frac{1.03(29.528)}{2} = 15.06$ k

Alternate System 2: 2

Alternate system 2 cont. Hollow Core Planks on steel beams:

Beam Design Cont.

$M_u = 111.2$ 1k
$V_u = 15.06$ k

ϕM_n = W12x26 = 140 ft k > 111.2 1k okay
ϕV_n = 84.3 > 15.06 k okay

$D_L = \frac{5 w l^4}{384 E I} = \frac{5(.32)(29.528)^4 1728}{384(29{,}000)(204)} = .925$ in

Limit $= l/360 = \frac{29.528(12)}{360} = .984$

$D_L = .925 < .984$ in ok

* The Beams are primarily for stability but was designed to support the load.

Use W12x26 for Beam B1

Girder Design: G1

DL = (68+25+15)(29.528) = 3189.0 plf
LL = (80)(29.528) = 2362.2 plf

$w_u = 1.2(3189/1000) + 1.6(2362.2/1000) = 7.61$ klf
1.4D D.N.C.

$M_u = \frac{w l^2}{8} = \frac{7.61(19.685)^2}{8} = 368.4$ ft-k

$V_u = \frac{w l}{2} = \frac{7.61(19.685)}{2} = 74.9$ k

* Hollow Core Toping and connection provides full lateral support

W18x50 $\phi M_p = 379$, $\phi V_n = 192$ k
check self weight

$M_u = \frac{1.2(50/1000)(19.685)^2}{8} = 2.91$ ft-k

$M_u = 368.4 + 2.91 = 371.3$ ft-k < 379 ft-k okay

$V_u = 74.9$ k < $\phi V_n = 192$ k ok

Alternate System 2: 3

Alternate system 2 cont: Hollow Core Planks

Check deflections

construction load

$$D_{cons} = \frac{5wl^4}{384EI} = \frac{5(3.189)(19.685)^4\,1728}{384(29{,}000)(800)} = .464 \text{ in}$$

$$\text{Limit } l/360 = \frac{19.685(12)}{360} = .656 \text{ in}$$

$$D_{cons} = .464 \text{ in} < .656 \text{ in} \quad \text{ok}$$

Live Load Deflection

$$D_L = \frac{5wl^4}{384EI} = \frac{5(2.362)(19.685)^4\,1728}{384(29{,}000)(800)} = .344 \text{ in}$$

$$\text{Limit } l/360 = \frac{19.685(12)}{360} = .656 \text{ in}$$

$$D_{LL} = .344 \text{ in} < .656 \text{ in} \quad \underline{\text{ok}}$$

Total Load Deflection

$$D_{TL} = \frac{5wl^4}{384EI} = \frac{5(3.189+2.362)(19.685)^4\,1728}{384(29{,}000)(800)} = .808 \text{ in}$$

$$\text{Limit } l/240 = \frac{19.685(12)}{240} = .984$$

$$D_{TL} = .808 \text{ in} < .984 \text{ in} \quad \underline{\text{ok}}$$

<u>Use W18x50 for Girder: G1</u>

Approximate self weight of floor system

Precast = 68 psf

Topping = 25 psf

(2) Beams $\frac{2(26)(29.528)}{(19.685)(29.528)} = 1.76$ psf

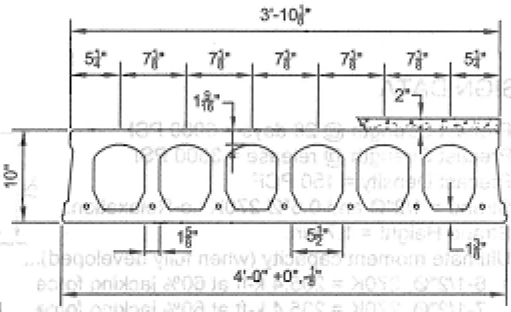
(2) Girders $\frac{2(50)(19.685)}{(19.685)(29.528)} = 3.39$ psf

Total = 68 + 25 + 1.76 + 3.39
= 98.2 psf

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

Technical Assignment #2

RS means data for the hollow core planks was taken for a 30 foot span.

B10 Superstructure

B1010 Floor Construction

B1010 229 — Precast Plank with No Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
1700	45	40	12	70	110	9.15	1.88	11.03

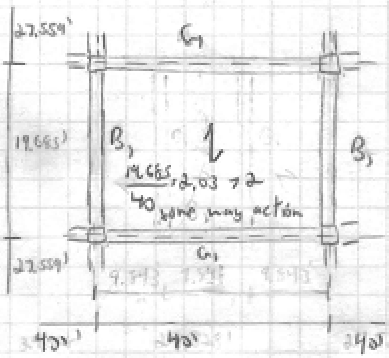
B1010 230 — Precast Plank with 2″ Concrete Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2000	10	40	6	75	115	7.25	5.20	12.45
2100		75	8	75	150	8.35	4.74	13.09
2200		100	8	75	175	8.35	4.74	13.09
2500	15	40	8	75	115	8.35	4.74	13.09
2600		75	8	75	150	8.35	4.74	13.09
2700		100	8	75	175	8.35	4.74	13.09
2800	20	40	8	75	115	8.35	4.74	13.09
2900		75	8	75	150	8.35	4.74	13.09
3000		100	8	75	175	8.35	4.74	13.09
3100	25	40	8	75	115	8.35	4.74	13.09
3200		75	8	75	150	8.35	4.74	13.09
3300		100	10	80	180	9.05	4.41	13.46
3400	30	40	10	80	120	9.05	4.41	13.46
3500		75	10	80	155	9.05	4.41	13.46
3600		100	10	80	180	9.05	4.41	13.46
3700	35	40	12	95	135	9.50	4.15	13.65
3800		75	12	95	170	9.50	4.15	13.65
3900		100	14	95	195	10.15	3.94	14.09
4000	40	40	12	95	135	9.50	4.15	13.65
4500		75	14	95	170	10.15	3.94	14.09
5000	45	40	14	95	135	10.15	3.94	14.09

Appendix E: Alternate System 3: One Way Slab

Alternate System 3 : 1

Alternate System 3: one way slab



Given:
LL: 80 psf
SDL: 15 psf
DL: self weight of members

Assume: Normal weight concrete
$f'_c = 4000$ psi
$f_y = 60{,}000$ psi
24" x 24" column sizes

* Aspect ratio is actually less than 2, so one way action is not there, need to relocate columns to increase aspect (Increase 29.528 to 40' span)

- Minimum thickness of one-way slabs by Table 9.5a
  Both ends continuous

$$h_{min} = \frac{l_n}{28} = \frac{(19.685-2)(12)}{28} = 7.58 \text{ in}$$

use $h = 8$ in for detailing
$d = 8 - .75 = 7.25$

* Do not need to check deflection

- Factored loads
  DL = 12/12 (150) = 150 psf
  SDL = 15 psf
  LL = 80 psf

  $1.2(150+15) + 1.6(80) = \underline{326 \text{ psf}}$
  $1.4(150+15) = 231$ psf

- Design Moment

$$M_u = \frac{w l_n^2}{8} = \frac{326(19.685-2)^2}{8(1000)} = 12.74 \text{ ft-k}$$

- Estimate A_s

$$A_s = \frac{M_u}{4d} = \frac{12.74}{4(7.25)} = .441 \text{ in}^2 \text{ per foot strip}$$

provide $\underline{\#6 \text{ @ } 12" \text{ o.c for effective } .44 \text{ in}^2 \text{ per foot}}$

- Check $\phi M_n > M_u$ $\quad d = 8 - .75 - .75/2 = 6.875$ in
  Assume $f_s > f_y$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.441(60)}{.85(4)(12)} = .647 \qquad c = \frac{a}{\beta_1} = \frac{.647}{.85} = .761$$

$$\varepsilon_t = \frac{.003}{.761}(6.875 - .761) = .024 > .005 \rightarrow \phi = .9$$

$$\phi M_n = \phi A_s f_y (d - a/2) = .9(.441)(60)(6.875 - .647/2)/12 = 12.97 \text{ ft-k} > 12.74 \text{ ft-k ok}$$

- Temperature & shrinkage Reinf

$$A_t = .0018\,bh = .0018(12)(8) = .1728 \text{ in}^2/\text{ft} \rightarrow \underline{\#5 \text{ @ } 18" \text{ o.c. prov. } A_{s,eff} = .207 \text{ in}^2/\text{ft}}$$

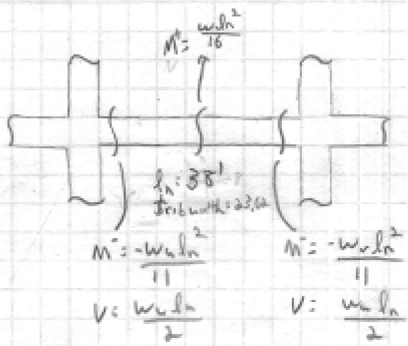
- crack control

$$s \le 15 - 2.5c_c = 15 - 2.5(.75) = 13\tfrac{1}{8} \text{ in} \Rightarrow s = 12" \text{ ok}$$
$$\le 12 \qquad = 12"$$

Alternate System 3: 2

Alternate Floor System 3: One Way Slab

Design Moment, Shear, Deflection for G1



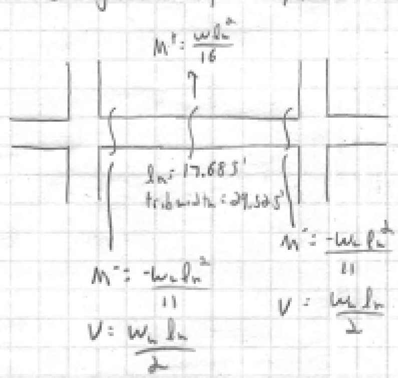
$M_u^- = -\frac{[326(23.62)](38)^2}{11(1000)} = -1010.8$ 'k

$M_u^+ = \frac{[326(23.62)](38)^2}{16(1000)} = 694.9$ 'k

$V_u = \frac{326(23.62)(38)}{2(1000)} = 146.3$ k

$h_{min} = \frac{l_n}{21} = \frac{38(12)}{21} = 21.7$ in

Design Moment, Shear, Deflection for B1



$M_u^- = \frac{[326(29.528)](17.685)^2}{11(1000)} = 273.7$ 'k

$M_u^+ = \frac{326(29.528)(17.685)^2}{16(1000)} = 188.2$ 'k

$V_u = \frac{326(29.528)(17.685)}{2(1000)} = 85.12$ k

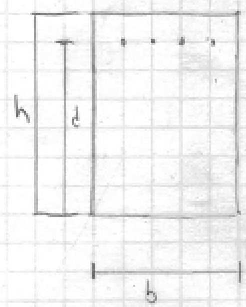
$h_{min} = \frac{l_n}{21} = \frac{17.685(12)}{21} = 10.1$ in

*B1 does not actually see any load due to the one way action however it was design to carry load, but is smaller than G1, so only impacts cost of construction

Alternate System 3: 3

Alternate Floor System 3: One Way Slab

Design G1: Negative Moment (Each End Typ) ; $M_u = 1010.8$ 'k



self weight

$M_u = 1010.8$ 'k $(1.05) = 1061.4$ 'k

Assume $b = 24$" to match column size

$\hookrightarrow bd^2 = 20 M_u$

$d = \sqrt{\frac{20(1061.4)}{24}} = 29.74$ in

$h = d + 2.5 = 32.2 + 2.5 = 32.25$ use 34 in

$h = 34$ in > 21.7 in for deflection control ok

$d = 34 - 2.5 = 31.5$ in

- Check self weight
  - $w_{sw} = \frac{34(24)}{144}(150) = 850$ plf

  $w_u = 326(23.62) + 850(1.2) = 8720$ plf $= 8.71$ klf

  $M_u = \frac{8.7(38)^2}{11} = 1142.1$ 'k

- Steel req'd:

  $A_s = \frac{M_u}{4d} = \frac{1142.1}{4(31.5)} = 9.06$ in²

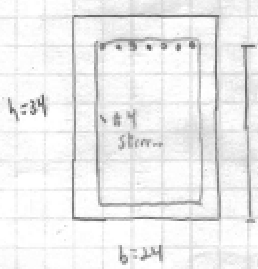
  use 8 #10 $A_s = 10.16$ in²

  $\rho = \frac{10.16}{24(34)} = .012 < 0.015$

- Design and detailing

  $d = 34 - 1.5 - 4/8 - 1/2(1.27) = 31.37$ in

  Bar spacing: $24 - 2(1.5) - 2(.5) > 8(1.27) + 7(4/8)$ $20 > 14.5$ ok



$A_{s,min} \geq \max \left| \begin{array}{l} \frac{3\sqrt{4000}}{60{,}000}(24)(31.37) = 2.38 \text{ in}^2 \quad \text{ok} \\ \frac{200}{60000}(24)(31.37) = 2.51 \text{ in}^2 \end{array} \right.$

$\rho_{max} = .85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + .004} = .85(.85)(4/60)\frac{.003}{.003+.004} = .0206$

$\rho_{max} > \rho_{pr} = .0206 > .012$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{10.16(60000)}{.85(4000)(24)} = 7.47$ in

$c = a/\beta_1 = 7.47/.85 = 8.79$ in

Check $\varepsilon_s > \varepsilon_y$ $\varepsilon_s = \frac{.003}{8.79}(31.37 - 8.79) = .008$

$\varepsilon_s > \varepsilon_y = .00207$ ok

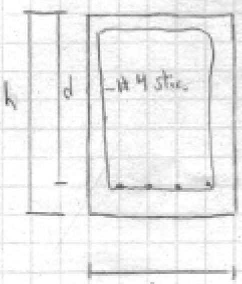
$\varepsilon_s > \varepsilon_t = .005$ ok $\rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - a/2) = .9(10.16)(60)(31.37 - 7.47/2)/12 = 1263.5$ 'k > 1061.4 'k ok

Alternate System 3: 4

Alternate Floor System 3: One Way slab

Design G1 : Positive Moment : M_u: 695 ^{1k}



$h: 34"$, $d: 31.37"$, $b: 24"$

$M_u: 695^{1k}$

$M_{sw}: \frac{1.2(859/1000)(38)^2}{16} = 92.1^{1k}$

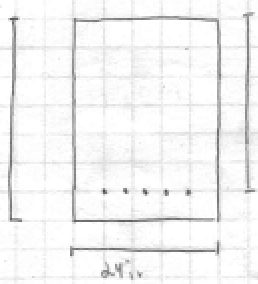
$M_u: 694.9 + 92.1 = 787^{1k}$

- req'd steel: $A_s: \frac{M_u}{4d} : \frac{787}{4(31.37)}$, $6.27\ in^2$

use 8 #8 $A_s: 6.32\ in^2$

$\rho: .0084$

- Design and Detailing



d: 31.87

(8) #8 → $A_s: 6.32\ in^2$

Bar spacing

$24 - 2(1.5) - 2(.5) > 8(1) + 7(4/3)$

$20 > 17.3$ in ok

$A_{s_{min}}$ ≥ | $2.38\ in^2$ ok
$2.51\ in^2$

$\rho_{max}: .0206 > .008$ ok

$a: \frac{A_s f_y}{.85 f'_c b} : \frac{6.32(60)}{.85(4)(24)} = 4.65$ in $c: a/\beta_1\ \frac{4.65}{.85} : 5.47$ in

check $\varepsilon_s > \varepsilon_y$ $\varepsilon_s: \frac{.003}{5.47}(31.37 - 5.47) = .014$

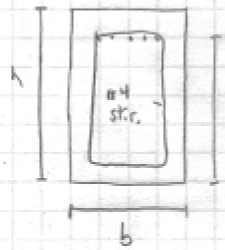
$\varepsilon_s > \varepsilon_y : .00207$ ok
$\varepsilon_s > \varepsilon_t : .004$ ok

$\phi = .9$ $\varepsilon_s > .005$

$\phi M_n: \phi A_s f_y (d - a/2): .9(6.32)(60)(31.37 - \frac{4.65}{2}) / 12 : 826^{1k} > 787^{1k}$ ok

Alternate System 3: 5

Alternate Design 3: One Way Slab
Design B1: Negative Moment (Each End Typ): M_u: 273.7 ^{1k}



self wgt

M_u : 273.7 (1.15) = 314.8 ^{1k}
Assume b = 18"
bd^2 = 20M_u
$d = \sqrt{\frac{20(314.8)}{18}}$ = 18.7 in

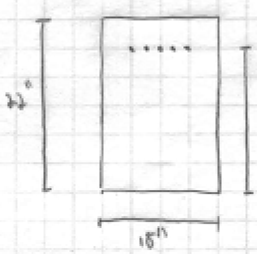
h = 18.7 + 2.5 = 21.2 in = 22 in
d = 22 - 2.5 = 19.5 in

-Check self weight
- w_{sw} = $\frac{22(18)}{144}$(150) = 412.5 plf

w_u = 326(29.508) + 412.5(1.2) = 10121.1 = 10.12 klf
M_u = 10.12(17.885)^2/11 = 287.7 ^{1k} < 314.8 ^{1k}

-Steel reqd
$A_s = \frac{M_u}{4d} = \frac{287.7}{4(19.5)}$ = 3.69 in^2 use 5 #8 A_s = 3.95 in^2
$\rho = \frac{3.95}{18(19.5)}$ = .011

-Design and Detailing



(5) #8 - A_s = 3.95 in^2
d = 22 - 1.5 - 4/8 - ½(8/8) = 19.5 in

Spacing: 18 - 2(1.5) - 2(.5) > 5(1) + 4(4/3)
14 > 10.33 ok

$A_{s\,min}$ > $\frac{3\sqrt{4000}}{60,000}$(18)(19.5) = 1.11 in^2 ok
$\frac{200}{60000}$(18)(19.5) = 1.17 in^2

a = $\frac{A_s f_y}{.85 f'_c b} = \frac{3.95(60)}{.85(4)(18)}$ = 3.87 in   c = $a/\beta_1 = \frac{3.87}{.85}$
c = 4.55 in

$\rho_{max} = .85\beta_1\frac{f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + .004} = .85(.85)(4/60)\frac{.003}{.003+.004}$ = .0206
$\rho_{max} > \rho_{pn}$: ok

check $\varepsilon_s > \varepsilon_y$ $\varepsilon_s = \frac{.003}{4.55}(19.5 - 4.55)$ = .0098 $\varepsilon_s > \varepsilon_y$ = .00207 ok
$\varepsilon_s > \varepsilon_t$ = .004 ok

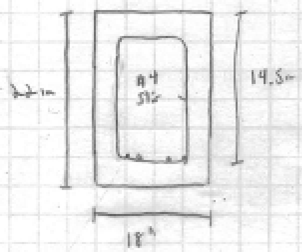
$\varepsilon_s > \varepsilon_t$ = .005 ⇒ ø = .9

$\phi M_n = \phi A_s f_y(d - a/2)$ = .9(3.95)(60)(19.5 - 3.87/2)/12 = 312.2 ^{1k} > 287.7 ^{1k} ok

Alternate system 3: 6

Alternate Design 3: one way slab

Design B1: Positive Moment : $M_u = 188.2^{'k}$



$h = 22\text{ in},\ d = 19.5\text{ in},\ b = 18''$

$M_u = 188.2^{'k}$

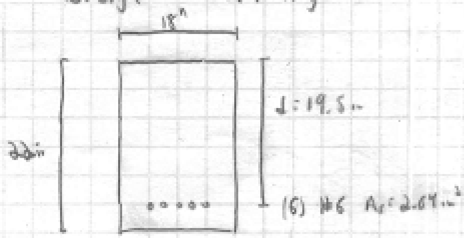
$M_{sw} = \frac{1.2(412.5)/1000\,(17.685)^2}{16} = 9.68^{'k}$

$M_u = 188.2 + 9.68 = 197.9^{'k}$

- req'd steel : $A_s = \frac{M_u}{4d} = \frac{197.9}{4(19.5)} = 2.54\text{ in}^2$

use 6 #6 $A_s = 2.64\text{ in}^2$

- Design and Detailing



Bar Spacing

$18 - 2(1.5) - 2(.5) > 6(6/8) + 5(4/3)$

$14\text{ in} > 11.17\text{ in}$ ok

$A_{s_{min}} \geq$ 3.11 in² ok

1.17 in²

$\rho = \frac{2.04}{18(19.5)} = .0075$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{2.64(60)}{.85(4)(18)} = 2.59\text{ in}$ $\quad c = a/\beta_1 = \frac{2.59}{.85} = 3.04\text{ in}$

check $\varepsilon_s > \varepsilon_y$ $\quad \varepsilon_s = \frac{.003}{3.04}(19.5 - 3.04) = .016$ $\quad \varepsilon_s > \varepsilon_y = .00207$ ok

$\varepsilon_s > \varepsilon_t = .004$ ok

$\varepsilon_s > .005 \Rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - a/2) = .9(2.64)(60)(19.5 - 2.59/2)/12 = 216.3^{'k} > 197.9^{'k}$ ok

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Equiv weight of system

slab = 8/12 150 = 100 psf

$B_1 = \frac{2(860)(38)}{2(38)(19.685)} = 86.4$ psf

$B_2 = \frac{2(412.5)(19.685)}{2(38)(19.685)} = 21.87$ psf

$w_t = 100 + 86.4 + 21.87 = \underline{208.2\text{ psf}}$

* No shear reinforcement was designed
* Alt Beam Spacing could be consided to improve system.

Minimum cover in concrete floors for fire protection

Table 2.3—Minimum cover in concrete floors and roof slabs

Aggregate type	Cover*† for corresponding fire resistance, in.					
	Restrained	Unrestrained				
	4 or less	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Nonprestressed						
Siliceous	3/4	3/4	3/4	1	1-1/4	1-5/8
Carbonate	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Semi-lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Prestressed						
Siliceous	3/4	1-1/8	1-1/2	1-3/4	2-3/8	2-3/4
Carbonate	3/4	1	1-3/8	1-5/8	2-1/8	2-1/4
Semi-lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4
Lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4

*Shall also meet minimum cover requirements of 2.3.1.
†Measured from concrete surface to surface of longitudinal reinforcement.

RS Means Data for a one-way slab system. It was assumed that the 40' span would control the cost, and cost per square foot was taken from the 35' x 40' bay size.

B10 Superstructure

B1010 Floor Construction

B1010 219	Cast in Place Beam & Slab, One Way							
	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB THICKNESS (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
7000	30x30	40	14	7-1/2	150	7.70	10.65	18.35
7100		75	18	7-1/2	191	8.65	11.20	19.85
7300		125	20	7-1/2	245	9.30	11.95	21.25
7400		200	24	7-1/2	328	10.55	13.25	23.80
7500	30x35	40	16	8	158	8.10	11	19.10
7600		75	18	8	196	8.80	11.35	20.15
7700		125	22	8	254	10.05	12.60	22.65
7800		200	26	8	332	11.15	13.05	24.20
8000	35x35	40	16	9	169	9.20	11.35	20.55
8200		75	20	9	213	10.05	12.35	22.40
8400		125	24	9	272	11.25	12.85	24.10
8600		200	26	9	355	12.55	13.75	26.30
9000	35x40	40	18	9	174	9.45	11.55	21
9300		75	22	9	214	10.35	12.45	22.80
9400		125	26	9	273	11.40	12.95	24.35
9600		200	30	9	355	12.70	13.80	26.50